THE HOOPSTAD STABILIZED KALAHARI SAND LTPP EXPERIMENT AFTER 55 YEARS

VOLUME 1 : FINAL REPORT

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Cover photo: General view towards Bultfontein of Section F (sand plus 5 % PBFC base) and others towards Section K in December 2016 after 55 years and about 1.5M E80 per lane.

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Abstract

The Hoopstad long-term road pavement performance (LTPP) experiment was constructed in 1962 as part of the road P 21/3 on Route R700 between Hoopstad and Bultfontein in the Free State Province of South Africa.

The purpose of the experiment was to evaluate the performance as base course of a fine-grained, nonplastic, A-2-4(0), aeolian, Kalahari-type sand stabilized with various amounts of ordinary and portland blast furnace cement, lignosulfonate, tar and bitumen in comparison with “crusher-run” graded crushed stone and neat, unstabilized sand as control sections.

The performance of the neat sand section in comparison with the adjacent two cement stabilized sections was reported in a previous AFCAP report. However, the traffic limit can now be extended to 0.3M E80.

In this report the performance of the stabilized sections is reported in comparison with the neat sand and crusher run sections.

Whilst there is a dearth of performance- and traffic- related information over the years, the fact remains that in June 2017, after 55 years and some 1.5M E80/lane, all the sections were still there and carrying traffic, none had been rehabilitated, and none appears to have ever exhibited structural failures.

In December 2016 all the sections except the crusher-run were in a terminal condition with respect to the 20% distress limit for a Category C road in regard to cracking of the surfacing, and/or edge breaking and edge patching. However, cracking was largely confined to the old, brittle surfacing, and not due to the underlying layers, and there was little rutting, no shear failures and only one pothole. The main distress was extensive and severe edge breaking extending into the outer wheelpaths in places as the road was too narrow for the large six- and seven-wheel trucks currently using it.

It is concluded that similar sand can be used with 3 – 5% of cement as base course for a Category C or D low volume road designed to carry up to 1.0 E80 / lane over 20 years provided it is compacted to at least 97% MAASHO on a good support, is protected from surface carbonation during construction, is well-drained and well-sealed with at least the equivalent of a double seal which is well-maintained. The seal must also be sufficiently wide to accommodate the traffic expected.

Similar sand can also be used under similar conditions if treated with 4% bitumen emulsion or tar.

Such neat or weakly cemented designs using Kalahari sand in all layers offer tremendous potential for the construction of relatively inexpensive, all-weather, sealed low volume roads in the vast area of arid and semiarid southern Africa in which similar sands and a scarcity of gravel and rock occur. This experiment – the oldest known in southern Africa – has proven that such designs can carry traffic for over fifty years.

It is recommended that the sand should be further characterised by means of triaxial and suction testing in order to provide a more complete understanding of its behaviour and to provide input into modern pavement design.

The cement-treated sections have performed well in spite of being totally carbonated and have not exhibited any distress other than cracking, edge breaking and rare potholes.

The whole road is currently undergoing rehabilitation and this has resulted in the destruction of the experimental sections.

Key words

Cement, bitumen, lignosulfonate, Kalahari, sand

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Acronyms, Units and Currencies

AASHO	American Association of State Highway Officials (now AASHTO)
AASHTO	American Association of State Highway and Transportation Officials
AFCAP	Africa Community Access Partnership
ASTM	American Society for Testing and Materials (now ASTM International)
BS	British Standard
BSI	British Standards Institute
CSIR	Council for Scientific and Industrial Research (South Africa)
DCP	Dynamic cone penetrometer
DoT	Department of Transport (South Africa)
Down Touch	Down Touch Investments (Main rehabilitation contractor for the Hoopstad - Bultfontein road)
FN	Frank Netterberg
Freetrans	Free State Provincial Department of Police, Roads and Transport
FWD	Falling weight deflectometer
Geoplan	Geoplan Laboratories (Pty) Ltd (Site and central laboratories)
LTPP	Long term pavement performance
PPC	Pretoria Portland Cement Co
Propercon	Proper Consulting Engineers (Rehabilitation Engineers for the Hoopstad - Bultfontein road)
RECAP	Research for Community Access Partnership
SABS	South African Bureau of Standards
Saiccor	South African Industrial Cellulose Corporation
SANS	South African National Standard
Sappi	South African Pulp and Paper Industries
SRT	Specialised Road Technologies
TMH	Technical Methods for Highways (South Africa)
TRH	Technical Recommendations for Highways (South Africa)
UKAid	United Kingdom Aid (Department for International Development, UK)

Contents (Volume 1)

Annexes (Volume 2)

Page no.

LIST OF TABLES **Page no.**

List of figures

List of photographs (Annex C) **Page no.**

Page no.

List of equations

$CBR \approx 3\,000\ DN^{-1,46}$	for DN > 10 (Netterberg 2015a, Netterberg and Elsmere 2015)	
$CBR = 410\ DN^{-1,27}$	for DN > 2 (Kleyn 1984)	
$CBR = 354\ DN^{-0,69}$	for DN > 3 (Sampson and Netterberg 1990)	
$CBR = 154\ DN^{-0,511}$	for DN > 1,5 (soaked ; Paige-Green et al 2015)	
$Ep = Lp + \surd A_L$	(TRH 12 :1997)	
$n = 0,044\ (BN_{100})^{1,24}$	(Kleyn and Savage 1982)	
$RoC_{200} = 20\,000 / (D_o - D_2)$	m	Simplified by Author from TRH 12 : 1997 for FWD RoC
$Suction = 6,5 + \log (2 - \log RH)$	pF	(RRL 1952)
$Suction = 0,098\ \text{antilog}\ pF$	kPa	(Author)
$Suction = 9,8 \times 10^{-5} \times 10^{pF}$	MPa	(Author)
$UCS \approx 6,5\ CBR$	kPa	(TRH 13 : 1986 : NITRR 1986)
$UCS = 2\,900\ DN^{-1.08}$	kPa	Kleyn (1984)

Notes

1. The equation for the FWD RoC in Horak (2008) is incorrect, but that in TRH 12 : 1997 is correct.

Notation and methods

Unless otherwise stated, the following notation and methods were used in this investigation and/or occur in the references:

- Bar linear shrinkage (LS) : TMH 1 : 1979 (National Institute for Transport and Road Research (NITRR) 1979)
- Base layer index (BLI) : TRH 12 : 1997, in µm
- California bearing ratio (CBR): TMH 1 : 1986 (NITRR 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
- Carbonation and presence or absence of cement using 0,5 % phenolphthalein solution and dilute hydrochloric acid (HCl) (Netterberg 1984) except that 1,2 N HCl was used instead of the previously recommended 5N 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)
- DCP tests on CBR specimens: Average DN through specimen after CBR test, with annular weight in place as used by EG Kleyn (1984), (2013 pers. comm.) and Sampson and Netterberg (1990), with the cone zero at the bottom of the CBR indentation and with heave measurements which were usually zero during testing (Netterberg method in Annex I)
- Deflection, maximum surface (Do, Ymax) using a Benkelman beam in mm, or a FWD in µm
- Double DCP test for collapsing sand: Comparison between a “dry” DCP test at in-situ water content and an adjacent “wet” DCP test carried out after wetting the hole made by the dry DCP with about 20 l of water. (Netterberg method in Annex J)
- Dynamic cone penetrometer (DCP) : TMH 6 : 1984 (NITRR 1984), with the cone zero at the top of the seal
- Extent of patching (*Ep*) = length as % of section or lane + √Area in m^2 reported as %. (TRH 12: 1992)
- Falling weight deflectometer (FWD), in this case a Dynatest 8002
- Field moisture equivalent (FME) : AASHTO T93-86 (1936) (AASHTO 1998a)
- Fineness index (FI_{075}, FI075) : P075 x 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 + P 425 + P075)] / 100 (SANS 3001- PR5 : 2009)
- Initial cement consumption : SANS 3001–GR57:2014
- Interpretation of distress : TRH 12 : 1997
- Laboratory test methods in general : TMH 1 : 1986 (NITRR 1986)
- Lower layer index (LLI) : TRH 12 : 1997, in µm
- MESA, ME80 : Million equivalent standard 80 kN axles
- Middle layer index MLI) : TRH 12 : 1997, in µm
- MISA : Million actual standard 80 kN axles
- Particle angularity : ASTM C 1252 – 93 (ASTM 1995)
- Particle size distribution (“grading”) :
    - Sieve analysis: TMH 1 : 1986 Method A-1(a)
- Paste EC (electrical conductivity) and pH : TMH 1 : 1986 method A 21 and CSIR Method CA 21 or Netterberg equivalent in Annex H, unless otherwise stated.
- P425, P075, etc: cumulative percentage passing 425, 075µm, sieves etc
- Road positions:

    - LS Left shoulder
    - LO Left outer wheelpath
    - LM Left midlane
    - LI Left inner wheelpath
    - CL Centreline
    - RI Right inner wheelpath
    - RM Right midlane
    - RO Right outer wheelpath
    - RS Right shoulder
- SSG: Selected subgrade
- RoC : Radius of curvature of the deflection bowl, using either a Dehlen curvature meter (RoC) or RoC_{200} using a FWD as stated (see Annex F), in m
- R425, R075, etc: cumulative percentage retained on 425, 075µm, sieves etc
- Sand equivalent (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)
    - 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)
- Visual assessment of the road: TMH 9 : 1992 (Committee of State Road Authorities (CSRA) 1992)

The terminology used is widely accepted internationally, but most terms are defined herein or in TRH4 : 1996 (COLTO 1996) and/or TRH12 : 1997 (COLTO 1997).

1. 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 (**Figure 1**). (This map only shows the distribution of more or less continuous and thick sands. Thinner, less continuous areas of sand occur well outside these boundaries, of which the Hoopstad-Bultfontein area is one.)

Moreover, what good gravels there were near existing alignments have either already been used or are rapidly being depleted (Paige-Green, 2007).

In their untreated state such sands have traditionally been regarded as suitable for use only as selected subgrade and fill (e.g. Sanral 2013) and by 1960 had rarely – if ever – been used as subbase or base course even when stabilized. Since then they have been used as stabilized with cement as base course in Botswana, South Africa and Zambia, and as unstabilized subbase and a test section of unstabilized sand base in Botswana (Botswana Roads Dept (BRD) 2010, Netterberg (2015a). However, the long-term durability of such cement stabilization is in doubt (Netterberg 1987, 1991; Paige-Green et al 1990) and is a major reason for the present investigation.

Long-term pavement performance (LTPP) experimental sections were constructed by the then Orange Free State Roads Department together with the then National Institute for Road Research (NITRR) of the South African CSIR on the P21/3 Hoopstad-Bultfontein road (now part of route R700) in the “sandveld” of the western Free State in 1962 in order to evaluate the use of such sands as base course when stabilized with cement, lignosulfonate (sulphite lye), bitumen, and road tar, using sections of unstabilized sand and “crusher-run” graded crushed stone as control sections, all on a 3 % portland blast furnace cement (PBFC) stabilized subbase (Gregg 1963).

All of these sections apparently performed well and a fairly standard design using similar sand treated with 5% PBFC on a 3% PBFC stabilized subbase was apparently subsequently adopted for the rest of the road as well as for others in the area constructed in the 1960s and 1970s.

As part of an AFCAP (Africa Community Access Programme) project on the increased use of such sands in low volume roads (InfraAfrica et al 2014) under the auspices of the Association of Southern African National Roads Authorities (ASANRA), the author was contracted to locate and investigate the performance of the unstabilized sand section in relation to some of the others. This work has already been reported as an AFCAP report (Netterberg 2015b) and by Netterberg and Elsmere (2015).

In this report the performance of the stabilized sand sections is reported in comparison with the neat and crusher-run sections and only the most important or new information provided concerning aspects such as climate, soils, drainage, etc., already dealt with in more detail in the previous report.

2. Location, Layout and As-Built Data

2.1 Location, layout and as-built data

The location, layout and as-built test results compiled by the author mostly from Gregg (1963) are shown in **Figure 2 and Tables 1** and **2**.

The approximate location of both the first (A – E) and second (F – K) set of sections is also shown on the Google Earth satellite image in **Annex A** which also includes a road log of the sections and salient points.

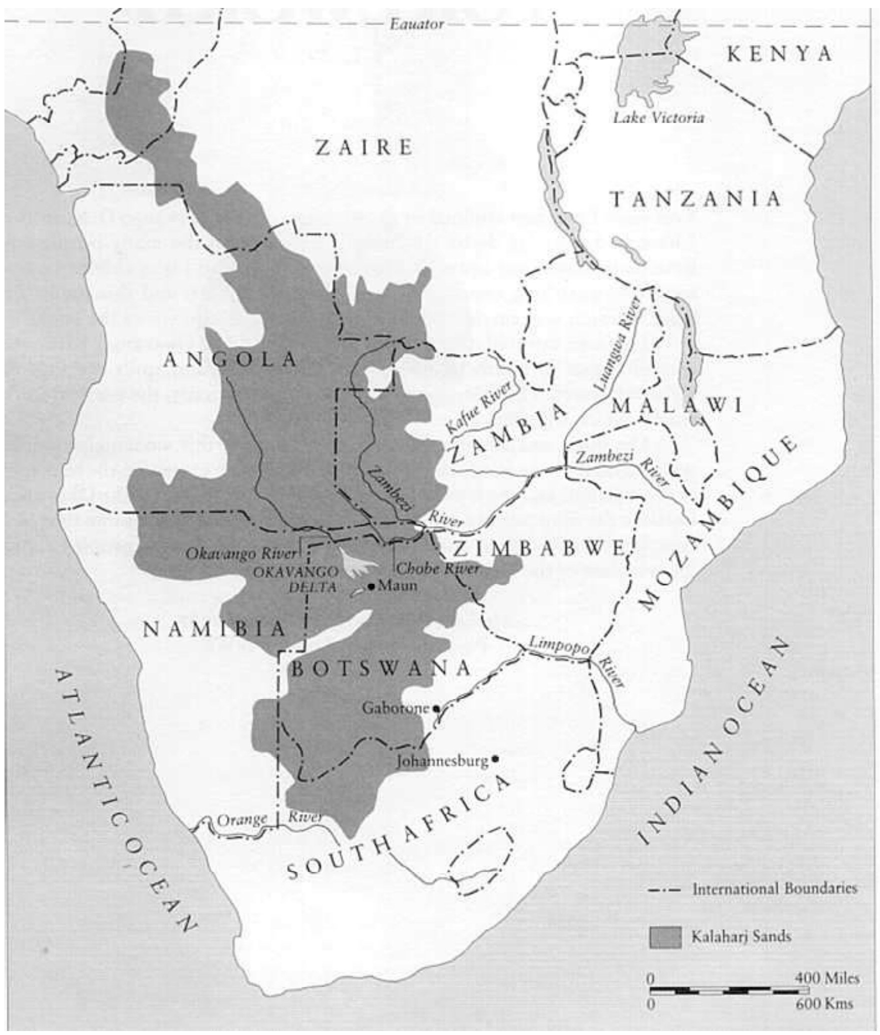

Figure 1. Distribution of Kalahari Sands
(Main 1987, in Botswana Roads Department 2010)

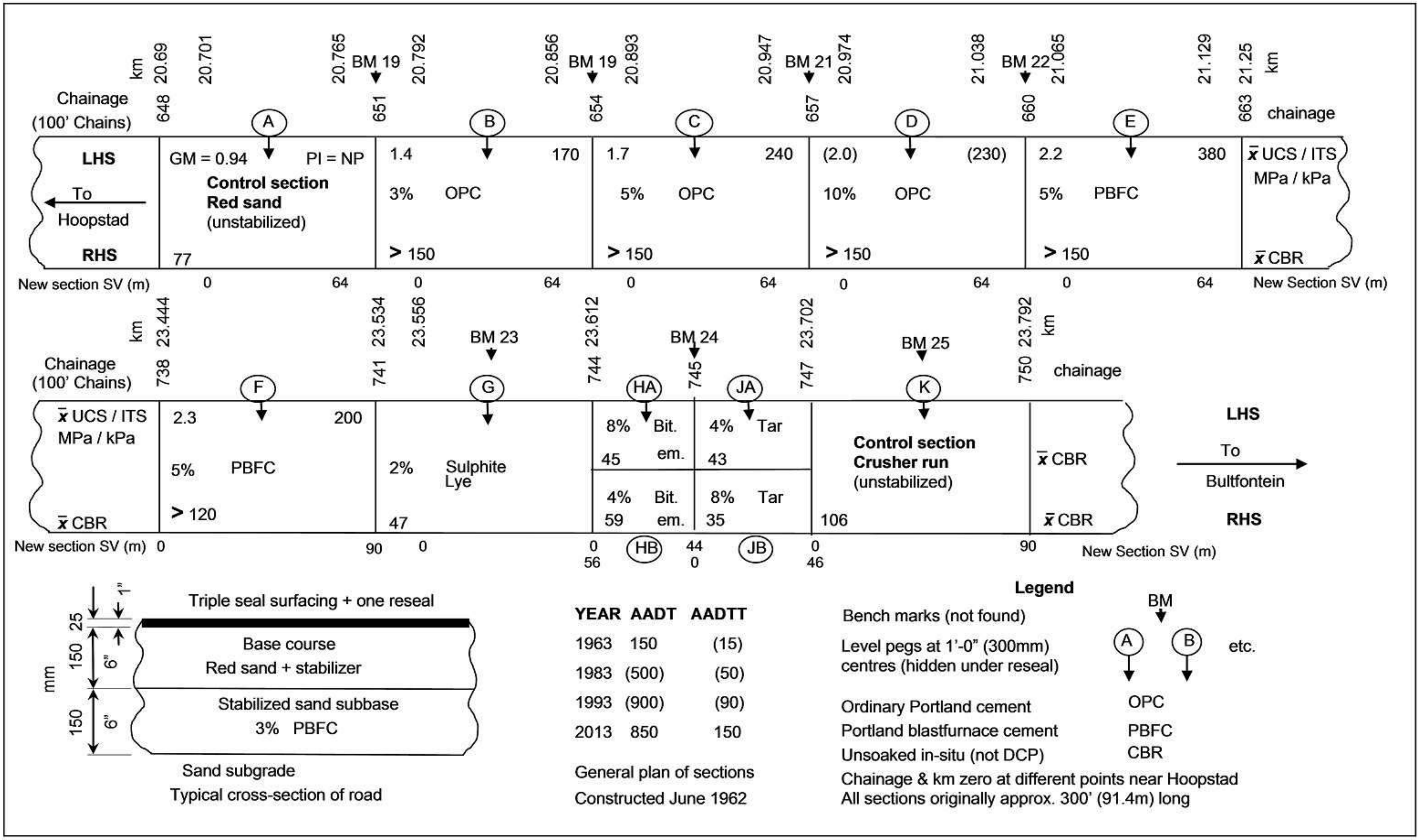

Figure 2 Kalahari sand stabilization experiment on Hoopstad-Bultfontein road showing mean as-built base course strengths, traffic history, and new section SVs with their corresponding current log kms

Table 1 Location, layout and as-built test results on the Hoopstad sand base course experiments constructed in June 1962 [1]

←TO HOOPSTAD					Surfacing : 6.1 m wide 25 mm triple seal: 2.0 m wide neat sand shoulders								TO BULTFONTEIN →
Section	Units	A	B	C	D	E	F	G	HA (LHS)	HB (RHS)	JA (LHS)	JB (RHS)	K
Road stake value	ch.	648-651	651-654	654-657	657-660	660-663	738-741	741-744	744-745.5	744-745.5	745.5-747	747.5-747	747-750
150 mm Base (Sand)		Neat (control)	3 % OPC	5% OPC	10% OPC	5% PBFC	5% PBFC	2% Sulphite lye	8% Bitumen emulsion	4% Bitumen emulsion	4% Tar	8 % Tar	Crusher-run (control)
Compaction	kg/m^3	-	1741	1701	1733	1733	-	1854	1832	1933	1846	1826	2222
MDD (MAASHO)	kg/m^3	1896	1882 [13]	1896 [13]	1920 [13]	(1896 [13])	(1896 [13])	1890 [13]	1907	1905	1964	2016	-
Compaction	%	-	93	90	90	91	-	98	96	101	94	91	-
OWC (MAASHO)	%	9.6	10.7 [13]	10.8 [13]	10.9 [13]	(10.8) [13]	(10.8) [13]	7.6 [13]	7.6	8.8	7.5	5.3	-
In-situ CBR	%												
Unsoaked [2]	%	*(n =11)*	*(n = 2)*	-	-	-	*n* = 6	*n* =12	*n*= 12	*n* = 11	n = 13	*n* = 15	*(n = 18)*
Max.	%	122	>333	-	-	-	>143	77	63	> 100	53	43	> 140
80 % ile	%	101	-	-	-	-							132
Mean ($\bar{x}$)	%	81.0	>153	(>150)	(>150)	(>150)	> 120	47	45	59	43	35	109.0
20 % ile	%	61	-	-	-	-							86
Min.	%	58	153	-	-	-	92	34	27	50	31	26	66
SD (*s*)	%	22,7	-	-	-	-							26.2
Soaked [3]		*(n = 6)*	*(n = 3)*	-	-	-	-	-	-	-	-	-	-
Mean ($\bar{x}$)	%	29	122	-	-	-	-	-		-	-	-	-
Mean UCS [4]													
Dried [5]	MPa	1.25	2.20	1.80	6.30	3.60	1.45	3.50	1.40	1.65	1.40	1.40	-
Cured [6]	MPa	0.14	1.40	2 .00	1.65	2.20	2.35	[14]	-	-	-	-	-
Mean ITS [4]													
Dried [5]	kPa	-	55	345	550	690	220	-	-	-	-	-	-
Cured [6]	kPa	-	165	230	240	380	200	-	-	-	-	-	-
Stabilizer content													
Cores ($\bar{x}$) [7]	%	-	4.1	6.5	6.1	6.5	5.4	-	8.2	5.8	3.5	4.8	-
After spreading, ($\bar{x}$) [8]	%	-	3.6	5.6	9.7	5.7	-	-	-	-	-	-	-
transverse (COV) [8]	%		38	29	41	28	-	-	-	-	-	-	-
After mixing, ($\bar{x}$) [9]	%		3.8	5.2	6.6	5.6	-	-	-	-	-	-	-
transverse (COV) [9]	%	-	26	25	20	25	-	-	-	-	-	-	-
After mixing, ($\bar{x}$) [9]	%	-	3.3	5.2	6.4	5.3	-	-	-	-	-	-	-
vertical (COV) [9]	%		48	71	37	49	-	-	-	-	-	-	
Mean deflection [10, 11]	mm	0.19	0.15	0.24	0.26	0.31	0.27	0.32	0.27	0.24	0.22	0.15	0.19
Mean deflection [10,11,12]	mm	0.25	-	-	-	-	-	-	-	-	-	-	0.30
Mean ROC [11, 12]	m	110	-	-	-	-	-	-	-	-	-	-	180

New Section SV (m)		0 ↓ 64	0 64	0 64	0 64	0 64 ↓	0 ↓ 90	0	0 56	44 0	44 0	46 0	0 46 90 ↓
Current log km @ SV shown		20.701				21.129	23.444						23.792
Lat. (S)		28° :00': 01.0197"				28° :00':14.1122"	28° :01': 24.3575"						28° :01':34.9765"
Long. (E)		25° :58': 57.7712"				25° :59': 02.8553"	25° :59': 30.1466"						25° :59':34.2714"
Height (m)		1284.2				1284.7	1288.0						1289.4

NOTES

[1] As-builts compiled mostly from Gregg (1963) with statistics calculated by author and estimates bracketed. Ch. = chainage (100 ft). In-situ CBRs, cores and deflections on Feb. 1963 after 8 months. [2] Water contents 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 and sulphite lye- sand compacted in lab. at Proctor effort (neat sand MDD 1 856 kg/m^3; OMC (11,0 %) ; others on cores from road (density, relative compaction, COV and *n* not stated, but COV said to be very high, *n* apparently ≈ 5 for each mean shown) [5] After 7 days of exposure in the laboratory (water contents 0.5 – 1.7 %) [6] After 7 days in a humid room (water contents 14,8 – 17.2 % for Sections B – F) [7] On above cores (COV and *n* not stated) [8] After spreading (n = 22) [9] After mixing ($n \approx 45$) [10] Benkelman beam [11] 62 kN Axle load, 480 kPa tyre pressure [12] (Dehlen 1962); radius of curvature (ROC) by Dehlen curvature meter [13] **Proctor** [14] Not reported, but said to be less than similar cement-treated specimens after soaking

The pavement was a three-layer design consisting of a triple seal on base, subbase and selected subgrade (SSG).

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

Both ordinary portland cement (OPC) and the then new portland blast cement (PBFC) were used. Both of these were supplied by the Pretoria Portland Cement (PPC) Co. (Gregg, 2013, pers. comm.)

The OPC would have complied with SABS 471 : 1959 and the PBFC with SABS 626 : 1961. It is understood that the PBFC blend was approximately 1:1 OPC and milled, granulated blastfurnace slag, approximately equivalent to a modern SANS 50197 CEM III A.

A Terolas bituminous emulsion with an 80/100 pen base was used. This was a 60 % anionic stable grade using a Vinsol rosin-based emulsifier, equivalent to a modern SANS 4001-BT3 SS60, and supplied by Colas Ltd (K Louw, 2017, pers. comm.).

The tar used was a cutback, high temperature coke-oven type of 30/35 EVT, supplied by ISCOR and presumably compliant with the SABS 748 specification of the time.

Further details of these two binders were provided by Gregg (1963).

For health and safety reasons tar is no longer available in South Africa and the SABS specification has been withdrawn. However, it is understood that road tar is still made and used in Zimbabwe.

Sulphite lye, currently usually known as lignosulfonate, is a by-product of the wood pulp industry usually used as a dust palliative for unsealed roads. However, it has been used as a stabilizer for base courses for sealed roads in the United States and in the Sahara (Fossberg 1966) and elsewhere (Jones and Mitchley 2001). The product was supplied in powder form by the South African Industrial Cellulose Corporation (SAICCOR, now Sappi-Saiccor) from their factory at Umkomaas in Kwazulu-Natal. The powder form had a pH of 3.2, was hygroscopic and self-hardening, but was highly soluble in water, and its dispersant properties were known to increase the maximum dry density (MDD), reduce the optimum water content (OWC) required for compaction, liquid limit (LL) and plastic limit (PL), but to have little effect on the plasticity index (PI), and to retard drying-out and water absorption (Fossberg 1966).

A mechanical spreader was used to spread the aerated cement fed from bulk tankers. The coefficients of variation (COV) of the transverse cement content after spreading were 28-41%, with only 38 and 41 % achieved on Section B and D respectively (**Table 1)** The longitudinal spreading was not measured.

The cement was ripped in with the grader tines and mixed with double disc harrows, grader blading and spring tooth harrows, with water added during mixing. Determination of the cement contents after mixing (**Table 1**) showed that the transverse mixing was adequate (COVs of 20 – 26%), but that the vertical mixing was poor (COVs of 37 – 71%), with Section C being the worst (71%), and all in excess of the maximum of 30% that would be permitted today.

The average cement contents after mixing (average of both transverse and vertical, $n \approx 90$ per section) compared to these found later on cores (*n* not stated) were as follows:

Section B (3% nominal OPC)	:	3.5% ; cores 4.1%
Section C (5% nominal OPC)	:	5.2% ; cores 6.5%
Section D (10% nominal OPC)	:	6.5% ; cores 6.1%
Section E (5% nominal PBFC)	:	5.5% ; cores 6.5%
Section F (5% nominal PBFC)	:	– ; cores 5.4%

Although 9.7% cement had been spread only about 6% was actually found in Section D. According to Gregg (1963) this was due to insufficient cement having been spread.

In short, both the average cement content and the mixing efficiency on most of the sections would be unacceptable by modern standards. This was also remarked on by Gregg (1963) who was particularly concerned about the poor vertical mixing and that insufficient cement was present in the lower half of the layers.

Compaction of the neat sand and the cement-treated sections was carried out with a 50 ton pneumatic roller followed by a “flat” roller.

The sulphite lye powder was mixed with water to a concentration of approximately 30% and then sprayed onto the section. A Ringhoffer R132 pulvimixer, disc harrows and grader were used for mixing, and compaction was with 50 and 20 ton pneumatic rollers and flat rollers.

Mixing of the bitumen and tar was carried out using a Millars Type EE twin shaft paddle mixer of 270 kg capacity except for the 8% tar section which was mixed in place with the pulvimixer. The mixed weigh-batched material was conveyed by tip truck to the road where it was distributed by raking, and compacted in two 75-mm layers with 20 and 50 ton pneumatic rollers.

The 8% tar was sprayed from an Etnyre distributor in several applications with in-between mixing with the pulvimixer. Compaction was carried out in one 150-mm layer with the 20 and 50 ton pneumatic roller.

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. Sand replacement density tests were used.

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 content, unconfined compressive (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 (1952).

2.2 Material properties

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

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 author	
Grading modulus (GM) = 0,94	
Dust ratio [2] = 0,09	
Uniformity coefficient (Cu) ≈ 2,5	
Coefficient of curvature (Cc) ≈ 1,6	
Classification :	
AASHTO M 145-91 (1995): A3/borderline A-2-4(0)	
Unified (ASTM D2487-11): SP–SM (poorly graded sand with silt)	
COLTO (1998): potential G7 at best (no CBR)	

NOTES
[1] Figures bracketed estimated by author
[2] P074 / P420

The compaction characteristics were as follows:

- Maximum dry density (MDD) (kg/m^3) : 1 896 (MAASHO); 1 856 (Proctor)
- Optimum water (moisture) content (OWC) (%) : 9,6 (MAASHO); 11,0 (Proctor)

Although no laboratory CBR was reported, the untreated (i.e. neat) sand had an unconfined compressive strength (UCS) at Proctor compaction of 140 kPa after 7 days of curing in a humid room and 1 200 kPa after 7 days of open curing (static compaction in 102 x 51 mm cylindrical moulds).

Undrained triaxial tests with pore pressure measurements (Gregg 1960) carried out on a similar windblown, reddish brown, fine, silty sand (also A-2-4(0) and SM, but with a percentage passing 075 µm (P075) of 20%, a P002 of 7% and a GM of 0,80) from the same area yielded the following apparent cohesions c' and angles of shearing resistance φ':

- c' of 35 kPa and φ' of 33 ° at a dry density of 1 888 kg/m^3 and a water content of about 10,5 %; and
- c' of 78 kPa and φ' of 34 ° at a dry density of 1 840 kg/m^3 and a water content of about 5%.

No pore pressures developed during these tests even though the specimens at a water content of 10,5 % were probably saturated.

Laboratory testing of 102 mm high by 51 mm diameter specimens at Proctor compaction with varying percentages of sulphite lye yielded UCSs after 7 days of open curing which increased with additive content from 2.4 MPa with 1%, through an interpolated 3.5 MPa with 2%, to 4.6 MPa with 3% No results were reported after humid room curing.

Similar testing after 7 days of humid room curing with varying amounts of OPC yielded interpolated UCSs of 620 kPa with 3%, 970 kPa with 5% and 1.8 MPa with 10%.

The sections were completed in June 1962.

The relative compaction of the neat sand and crusher-run section was not reported.

At least until about November 1963 all the sections had performed satisfactorily and no failures had occurred and it was concluded that the unstabilized 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 by the then National Institute for Road Research (NIRR) of the South African CSIR showed the maximum settlement on any section to be only 1.3 mm. These measurements as well as visual observations were apparently continued up to about 1974. In spite of a search at the CSIR by the CSIR surveyor who carried out the later monitoring (Mr A Bam, now retired), the records could not be found. However, according to Mr Bam (2012 pers. comm.) no distress had occurred up to that time.

2.3 Discussion

Although some of the compaction characteristics were reported as Proctor (indicated by **Note [13]** in **Table 1**), the MDDs are similar to the MAASHO MDDs obtained in the present study. The lower MAASHO OWCs now found indicate that they were indeed Proctor, but probably compacted without the 4-hour delay now in use for cement treated materials.

The compaction of all of the sections for which results were reported except Section G (2% sulphite lye) with 98% Proctor (?) and HB (4% emulsion) with 101% MAASHO was poor, and indeed practically all of them if the reference density was actually Proctor and not the usual MAASHO.

The high degree of compaction obtained with sulphite lye together with the Proctor MDD of 1890 kg/m^3 and OWC of 7.6% against the Proctor MDD of 1 856 kg/m^3 and OWC of 11.0% of the neat sand **(Table 1, Note 4)** indicates that it acted as a compaction aid.

Although the neat sand base was nonplastic, the borderline AASHTO A-2-4(0) classification, the high mean in-situ CBR of 81, the presence of a significant UCS OF 1.2 MPa after partial drying – in the COLTO (1998) C3 range for cemented materials – and the presence of cohesion shown in a triaxial test all indicated that this sand was not the usual cohesionless AASHTO A-3 sand, but had sufficient strength for untreated base course as indicated by Gregg (1963). Whilst the low mean, soaked, in-situ CBR of 29 indicated the necessity of avoiding saturation, the non-development of pore pressure during the apparently saturated triaxial test indicated good internal drainage. This property is probably an important factor in the good performance of this sand base.

In comparison, the sand treated with 2% sulphite lye only yielded a mean, unsoaked, insitu CBR of 47, but a partially dried laboratory UCS of 3.5 MPa, similar to the 3.6 MPa yielded with moist-cured 6.5% PBFC.

It is not unknown for similar, reddish-brown, Kalahari-type, fine, sands to exhibit significant dry strengths. Results on four nonplastic (NP) A-2-4 (0) Namibian sands in the author's possession with P075 of 11 – 18%, sand equivalents (SE) of 20 – 27 and soaked MAASHO CBRs of 32 – 40 yielded dry MAASHO UCSs of 320 – 1 060 kPa.

The sand used was fairly typical of the A-3 to A-2-4 (0) "Kalahari" sands occurring over a vast area of the interior of southern Africa shown in **Figure 1** and reviewed by the Botswana Roads Department (2010), Paige-Green et al (2011, 2015) and InfraAfrica et al (2014).

If all other factors were equal, the UCSs obtained on the cylindrical specimens used would probably have been about 80% of the equivalent in current South African practice using CBR-size specimens.

If the lower 62 kN 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 (RoCs) for an 80 kN axle load of about 0.3 – 0.4 mm for Sections A, G and K and 90 and 150 m for Sections A and K respectively, were all within the sound range of <0.6 mm and > 80 m for a modern, untreated base on a treated subbase for a modern Category C road according to the criteria in TRH 12 : 1997 (Committee of Land Transport Officials (COLTO) 1997); by which criteria even Section A would be expected to have a structural capacity in excess of 5M E80.

With corrected deflections of 0.2 – 0.4 mm most of the cement-treated base (CTB) sections were also within the sound range of <0.4 mm for a CTB although Section E at 0.4 mm was marginal to the warning range of 0.4 – 0.8 mm. With corrected deflections of 0.2 – 0.4 mm, the bitumen- and tar-treated base sections were also within the sound range of <0.5 mm for a BTB.

In this respect, experience with Kalahari sands in Zimbabwe showed them to be exceptionally uniform and an excellent subgrade material with a high compacted strength, yielding significantly lower deflections than normal and a lower 20%-ile design CBR of 19 (Van der Merwe 1970). The significant contribution of such subgrades/roadbeds to the exceptionally good performance of roads on them in Botswana has also more recently been noted by Paige-Green and Overby (2010).

A search at the Free State Provincial Roads Department (Freetrans) by the current head of the central laboratory, Ms D Elsmere, failed to find any further as-built records of the experiment other than that in Gregg (1963) or the rest of the road or any monitoring or performance reports, and it is understood that they were discarded many years ago. However, several long-retired persons were located who helped to provide some of the missing information during telephonic and/or personal discussions. These included the NIRR engineer in charge of the experiment, Mr JS Gregg (in Hermanus), the control technician for the area, Mr AC Nothling (in Kroonstad), the later head of the central laboratory, Mr BM Herbst (in Bloemfontein), the later materials engineer, Mr N van der Walt, (in Pretoria), and the former NIRR surveyor, Mr A Bam (in Kathu).

The cement stabilized sections were cured for 3 - 4 days by means of an initial spray of water followed by a sand cover, after which they were primed with a tar primer and sealed the following day (AC Nothling, 2015, pers. comm.). The general policy at the time was to prime and seal as soon as possible in order to hold the moisture in (BM Herbst, 2014, pers. comm.).

This practice should have prevented carbonation-induced damage to the upper base which has led to the premature distress experienced on some other roads with lightly cemented bases.

Details of the triple seal surfacing were not reported. However, the usual triple seal of the time would typically have consisted of 19, 10 and 7 mm stone layers and would probably have been designed similarly to the methods in NIRR (1971). According to Mr Nothling an ordinary 13 + 7 mm double seal with a fog spray was used over the rest of the road.

The original seal was probably 20 ft (6.1 m) in width and has been reduced slightly to its current average width of about 6.0 m by the severe and extensive edge breaking caused by the wide and heavy trucks now using it.

3. Previous Evaluation in 2013

The detailed pavement evaluation of Sections A – C carried out during 2013 has already been described (Netterberg 2015a, Netterberg and Elsmere 2015) and will not be repeated in detail here.

In outline, the general pavement condition was as follows:

- Section A (neat sand). Poor, because of edge breaking, patching and cracking
- Section B (3% OPC) : Fair, mostly because of cracking
- Section C (5% OPC) : Fair, mostly because of cracking

According to the criteria in TRH 12 : 1992 all three sections were in a severe condition with respect to crocodile cracking (≥ 25% of length), a warning to severe condition with respect to block plus transverse cracking (≥ 50% of length) and Section A with respect to edge patching, although it was only of Degree 2.

However, in all cases shear failures, longitudinal cracking and pumping were absent, rutting was minimal, patching was almost entirely confined to the edge breaking, which was severe and extensive due to the large trucks currently travelling on the edges of the narrow seal.

Further details, as well as of the DCP and light falling weight deflectometer (LWD) surveys and laboratory testing, traffic history climate and weather, geology and soils are provided in the 2015 report.

Only some of this will be repeated in this report for convenience.

A conservative neat sand base course material and construction specification for a road to carrying up to about 0.1M E80 over 20 years was derived empirically from this information.

This specification was deliberately limited to this low traffic level chiefly because of the absence of traffic counts over the first 20 years and the possibility of improvement due to slow remoulding under traffic. However, the structural condition and DCP survey indicated that this could probably be doubled or trebled.

Both the two cement-treated bases B and C and the cement-treated subbases under all three bases were found to be totally carbonated according to the Netterberg (1984) field phenolphthalein and acid tests.

4. Pavement Evaluations in 2016 and 2017

4.1 Site Work

During the week of the 05 December 2016 a detailed pavement evaluation of Sections F - K was carried out comprising visual evaluation according to TMH 9 : 1992 (Committee of State Road

Authorities (CSRA) 1992), measurements of degree and extent of cracking, patching, and edge breaking, rut depths, deflections using a falling weight deflectometer (FWD) on all 12 sections, DCP tests to 800 mm or refusal, profiling (mostly to a depth of about 500mm), phenolphthalein and acid tests, in-situ density tests of the base course, and sampling for moisture (water) content, indicators, CBR and other tests according to a programme similar to that in **Annex B**.

Photographs of Sections A, B and C taken both in 2013 and 2016 and of the other sections taken in 2017 are shown in **Annex C**. Prior to their rehabilitation in 2017 excellent views of all of these sections could also be obtained by calling up the Google Earth image in **Annex A** and then making use of the accompanying Google road survey. However, the earlier surveys may still be available.

The FWD (falling weight deflectometer) measurements were carried out on Monday 05 December 2016 by the Specialist Road Technologies (SRT) team under Mr J Mathetsa, who provided their own traffic control.

The Freetrans team under Ms D Elsmere measured the rut depths and most of the DCP profiles and Ms Elsmere and Dr Netterberg carried out the visuals.

The Geoplan Laboratories (Geoplan) site laboratory team under Mr KI Moenyane carried out the profiling, sampling and densities and assisted with some of the DCPs. The routine laboratory testing was also to be carried out by them.

Traffic control was mostly provided by the Contractor Down Touch Investments under the supervision of the Site Agent, Mr G Forsyth, with additional signage from Freetrans.

The Resident Engineers Mr W Cordier and Mr C Roos of Proper Consulting Engineers (Propercon) were both very helpful.

Most of the time was spent on Sections F – K as Sections A – E were evaluated in October 2013 and mostly only an updating of the visuals and photography was carried out on them.

However, a large bulk sample of the neat sand from Section A was taken for stabilization design and other tests.

The evaluation was designed by Dr Netterberg and all work supervised by him.

As was the case with Sections A – E, all the benchmarks were missing, any level pegs were concealed under the reseal, and the exact equivalent current log kms were unknown.

As before, the sections were therefore located approximately by examination of the base course exposed in the edge breaks (and by digging into the edges where necessary) and by spraying them with dilute hydrochloric acid.

The carbonated PBFC-treated base (Section F) was found to effervesce strongly with HCl, the crusher-run (Section K) weakly, and the sulphite-lye (lignosulfonate) (Section G) and the bitumen (Sections HA and HB)- and tar (Sections JA and JB)- treated bases not at all.

There was no Section I.

In order to allow for this uncertainty and possible contamination between sections during construction a distance of at least 10 m was therefore allowed at the start and end of the longer sections and 5 m on Sections H and J as a safety factor.

Attempts to locate the steel pipe benchmarks near the fence with a Velleman CS100 metal detector (similar to an electromagnetic mine detector) were a failure. Although, if still existing, they could probably have been located with a magnetometer or more powerful device and/or pitting or trenching, this was considered unnecessary.

However, the presumed lines of concealed steel level pegs at the correct spacing were successfully located by this instrument on Section K at section SV 43.3 m, approximately as shown on the plan in the middle of Section K (i.e. at SV 45.7 m and on Section A at 26.2 m, in comparison with 32 m assumed. Whilst simulations in Pretoria with a rebar detector were also successful the Velleman device was more convenient to use.

Although time did not permit trials on all of the sections, it was concluded that this appeared to be a viable technique for use under such circumstances and gave added confirmation that the sections had been sufficiently accurately located.

An updated location plan showing the lengths selected for each section is shown on **Figure 2.**

The start and end of each section was marked on the road, e.g. as 0 and 91 for Section K, and positions at 5 m and 10 m intervals marked out for the rut depth and FWD measurements, respectively.

A copy of the field visual assessment sheet for each section is shown in **Annex D,** the processed rut depths in **Annex E**, and the processed deflection data in **Annex F**.

Unfortunately, most of the initial field and laboratory testing as well as the repeat sampling and testing carried out by the site laboratory had to be rejected.

This required a third round of sampling and laboratory testing, of which the latter was carried out at the Geoplan central laboratory at their own expense.

This third round of field work was initially supervised by Dr Netterberg during 07-09 June 2017 and some of the rest by Mr M van der Westhuizen, an engineer consultant to Geoplan. This work included a second round of DCP tests because all the December water contents taken for the Dec. 2016 survey (**Annex G**) had to be rejected.

Details of the revised field and laboratory testing programme are provided in **Annex B**. This was essentially that envisaged in December 2016, but revised taking into account what could be used of the previous work and what could not be completed then.

The DCP field data and the results of the laboratory work by Geoplan are shown in **Annex H**. The WinDCP 5.1 computer -processed DCP results are shown in **Annex G.**

The road has been evaluated largely as a TRH4 : 1996 Category C, lightly trafficked rural or strategic road intended for a moderate level of service, a medium risk, an 80% design reliability, and 0.03 - 3 M E80/lane over a period of 10 – 20 years.

However, where feasible the data such as deflections and rut depths have also been processed to provide 90%-and 50%- iles appropriate for Category B and D roads, respectively.

5. Visual Assessment and Rut Depths

A TMH 9 : 1992-type visual inspection of both lanes of each section was carried out on foot, supplemented by a sketch plan of the cracking and patching, and deviations under a 2,0 m-long straight edge measured with a 20 mm-wide wedge.

Photographs of each section and some of the pits dug are shown in **Annex C** and the results of most of this work in **Annexes D and E,** and summarised in **Tables 3, 4 and 5.**

The term 'deviations' has been used in **Tables 3 – 5** to include the drop at (but still on) the edge of the seal in order to demonstrate the shape of the road. This was not a rut, but mostly a reflection of the camber.

Only the deviations in the outer and inner wheelpaths were regarded as true traffic-associated ruts.

The extents shown in the tables may differ from the best visually estimated averages shown on the field sheets in **Annex D** because they have been adjusted according to the lengths and areas actually measured later.

Details and photographs of the different types of distress and their degree (D) and extent (E) are provided in TMH 9, but for convenience will be briefly described here.

The only significant **types** of distress seen were block (B), crocodile (C) and transverse (T) cracking and edge breaking (accompanied by arcuate to crocodile cracking). Rutting was minimal and potholing rare except where it was associated with the edge breaking.

The five **degrees** of visually assessed distress are briefly as follows:

Degree	**Severity**
1:	Slight
2:	Between slight and warning
3:	Warning: notable with respect of possible consequences, e.g. open cracks (3 mm) with slight spalling, deformation or secondary cracking
4:	Between warning and severe
5:	Severe: extreme, urgent attention required, e.g. open cracks (> 3 mm) with spalling, deformation or secondary cracking or open cracks (> 10 mm) with no secondary effects

In terms of rutting, Degree 1 is difficult to discern (< 5 mm), Degree 3 first discernible by eye (10 - 15 mm), and Degree 5 Severe, dangerous (> 30 mm).

Rut depths of 10 mm and 20 mm are regarded in TRH 4: 1996 as warning and terminal levels respectively for all categories of road.

The five degrees of **extent** are defined in terms of the percentage of the length of the section (or in this case also the lane) (left or right) affected briefly as follows:

Extent	**Description**
1:	Isolated, not representative of section (< 5%)
2:	Between isolated and scattered (5 – 10%)
3:	Scattered over most of length (> 10 – 20%)
4:	Between scattered and extensive (>20 – 50%)
5:	Extensive over most of length (> 50%)

Table 3 Summary of visual evaluation and rut depths of Sections A – D in December 2016 [1]

Parameter		Section A (neat)				Section B (3 % OPC)				Section C (5 % OPC)				Section D (10% OPC)			
Pavement condition [2]		**Poor**				**Fair**				**Fair**				**Fair (best of A-E) and CTBs**			
Cracking [3]	**Block**	D5 / E4 (80%)				D4 / E5 (100%)				D5 / E5 (100%)				D4 / E5 (100%)			
	Croc:																
	LOWP	D4 / D5 up to 1,0 m from edge (40%)				C4 / E5 (60%)				D4 / E3 (20%)				D4 / E2 (10 %)			
	LIWP	D5 / E5				D5 / E5 to centreline (100%) [10]				40 m D4 / E5 to centreline (60%) [10]				-			
	RIWP	D5 / E5				D4 - 5/E4 to centreline (50%) [10]				40 m D4 / E5 to centreline (60%) [10]				-			
	ROWP	D4-D5/E4 up to 1.0m from edge(50%)				T4 - CC3/E5 (90%)				-				-			
Patching [4]	**Full-width**	5 m (near culvert) = 30 m^2 E_p = 11				-				-				-			
	Edge to:																
	LOWP	(D3 / E5:40 m x 1.0 m geotextile and emulsion) ; 8 m x 1.0 m = 8 m^2 E_p = 15				D2 / E3: 17 m x 0.5 m = 9 m^2 E_p = 30				D1 / E2: 15 m x 0.5 m = 8 m^2 E_p = 26				- -			
	ROWP	D3 / E5: 62 m 0 – 0.5 – 1.5 m = 33m^2 E_P = 103				D2 / E1: 1 m x 0.5 m = 0.5 m^2 E_P = 2				D1 / E1: 2 m x 0.5 m = 1 m^2 E_P = 4				D1/E1: 2 m x 0.5 m = 1 m^2 E_P = 4			
Edge breaking [5]		D3 / E5L; D4 / E5R [9]				D4 / E5L; D3 / E5R				D5 / E4L; D5 / E5R				D4 / E4L; D3 / E5R			
Deviations ("ruts") [6]		90% - ile	80% -ile	50% - ile	Patches	90% - ile	80% -ile	50% - ile	Patches	90% - ile	80% -ile	50% - ile	Patches	90% - ile	80% -ile	50% - ile	Patches
		mm	mm	mm	No.	mm	mm	mm	No.	mm	mm	mm	No.	mm	mm	mm	No.
LHS Bultfontein-bound	Edge [7]	12	10	6	1	5	4	2	3	7	6	3	2	15	12	8	0
	OWP [7]	11	10	7	0	9	8	5	2	12	10	7	1	8	6	4	0
	OWPH	4	3	2	0	9	7	4	2	-	-	-	-	-	-	-	0
	IWP	15	13	10	1	10	9	7	0	13	11	9	0	12	11	8	0
	IWPH	6	5	4	1	7	6	4	0	-	-	-	-	-	-	-	0
	CLL	35	30	20	0	13	10	6	0	20	17	12	0	9	7	3	0
RHS Hoopstad -bound	CLR	17	14	8	0	16	13	9	0	21	17	11	0	13	11	7	0
	IWPH	4	3	1	1	5	5	4	0	-	-	-	-	-	-	-	0
	IWP	9	8	6	0	14	13	10	0	12	11	8	0	8	7	5	0
	OWPH	8	7	4	2	7	6	4	0	-	-	-	-	-	-	-	0
	OWP	12	10	7	2	7	6	4	0	8	7	5	0	6	5	3	0
	Edge [8]	23	19	13	4	5	4	2	1	8	6	3	2	5	4	2	1

NOTES

[1] According to TMH 9 : 1992. Initially as in Oct. 2013, updated to Dec. 2016 except for rut depths. Only the central 64 m length of each section was assessed

[2] General pavement condition according to visuals only. Open culvert on Section A at 51 m

[3] Mean spacing : block-hexagonal (B): 1 m; crocodile (C) : 150 mm; transverse (T) mostly short (≤ 1 m) on Section C edges only. Degree (D); extent (E) as % of length: 1: (<5 %) ; 2 : 5 - 10% ; 3 : >10 - 20% ; 4 : 20 - 50% ; 5 : >50% [4} Mostly edge patching 0.2-0.5 m wide, seldom extending to OWP or midlane except on Sections A and B. Geotextile + emulsion regarded as surfacing patch. E_p = extent of patching

[5] Edge breaking usually accompanied by D5 cracking in outer 300 mm (150 mm on Section D) on 100 mm spacing

[6] Deviations under 2,0 m straight edge (n = 14) measured with 20 mm wide wedge at approx. 5 m intervals including on patches (number shown). Seal edge, OWP, IWP, CLL CLR : straight edge at rest. OWPH, IWPH: straight edge held down at seal edge and centreline respectively. In Oct. 2015, not remeasured in 2016

[7] On section A only including nine deviations on geotextile plus emulsion from 0 to 40 m, but no separate patches in OWP [8] On section A only including four deviations (13-26 mm) on patches (80 % ile : 16 mm without patches; n = 10) [9] Section A filled edge breaks – assumed potholes D5 / E1 R; 1 x D3 / E1 LM [10] D3 / E5 pumping in both lanes in both inner wheelpaths and on centreline in 2016 only

Table 4 Summary of visual evaluation and rut depths of Sections E – HA in December 2016 [1]

Parameter		Section E (5% PBFC)	Section F (5% PBFC)	Section G (sulphite lye)	LHS: Section HA (8% bitumen emul.)
Pavement Condition [2]		**Poor (worst of A - E)**	**Fair**	**Fair**	**Poor**
Cracking [3]	**Block**	D5 / E5 (100%)	Transverse D4 / E5 (100%)	Transverse D4 / E5 (100%)	D5 / E5; Transverse D4 / E5 (80%)
	Croc:				
	LOWP	D5 / E5	D4 / E1	D4 / E2 (20%)	D4 / E1 (10%)
	LIWP	D5 / E3	D4 / E5 (100%)	D4 / E3 (40%) [7]	D4 / E5 (70%) [8]
	RIWP	-	D4 / E5 (60%)	D4 / E2 (20%)	-
	ROWP	D5 / E1	D4 / E3 (20%)	D4/E2 (10%)	-
Patching [4]	**Full-width**	-	(2 mm x 1 m = 2 m^2 on LI-CL-RI) E_p= 4	-	-
	Edge to:				
	LOWP	D3 /E4: 30 m x 0,5-1,5 m = 15 m^2 E_p= 51	D1 / E3: 18 m x 0.3 m = 6 m^2 E_p= 25	D1 / E4: 45 m x 0.3-1 m = 30 m^2 E_p= 95	D2 / E2 (10%) [9] E_p= 6
	ROWP	D2 / E3: 7 m x 0,5-1.5 m = 6 m^2 E_p= 13	D1 / E3: 13 m x 0.2-1.5 m = 7 m^2 E_p= 19	D1 / E4: 31 m x 0.5-1.5 m = 30 m^2 E_p= 67	- -
Edge breaking [5]		D5 / E5	D3 / E3	D3 / E5	D2 / E5

Deviations ("ruts") [6]		E 90% -ile	E 80% -ile	E 50% -ile	E Patches	F 90% - ile	F 80% -ile	F 50% -ile	F Patches	G 90% -ile	G 80% -ile	G 50% - ile	G Patches	HA 90% -ile	HA 80% -ile	HA 50% -ile	HA Patches [5]
		mm	mm	mm	No.	mm	mm	mm	No.	mm	mm	mm	No.	mm	mm	mm	No.
LHS Bultfontein-bound	Edge	18	15	8	5	9	7	5	5	14	11	6	9	12	10	6	1
	OWP	9	8	5	0	10	9	7	0	16	14	10	0	8	8	6	0
	Inset(m)	-	-	-	-	0.8	0.7	0.7	-	1.0	0.9	0.7	-	1.0	0.9	0.7	0
	IWP	14	12	10	0	12	11	9	0	15	13	10	0	11	10	8	0
	Offset(m)	-	-	-	-	0.5	0.4	0.4	-	0.3	0.3	0.3	-	0.5	0.4	0.3	0
	CLL	16	13	8	0	10	9	5	0	20	17	11	0	19	16	11	0
RHS Hoopstad -bound	CLR	16	14	9	0	11	9	5	0	9	7	4	0	-	-	-	-
	IWP	13	12	9	0	13	11	8	0	9	8	6	0	-	-	-	-
	Offset(m)	-	-	-	-	0.4	0.4	0.3	-	0.4	0.4	0.3	-	-	-	-	-
	Inset (m)	-	-	-	-	0.8	0.7	0.5	-	0.6	0.5	0.4	-	-	-	-	-
	OWP	9	7	4	0	10	8	5	1	11	10	7	0	-	-	-	-
	Edge]	17	14	9	6	11	9	6	4	12	10	6	6	-	-	-	-

NOTES

[1] According to TMH 9 : 1992. For Section E initially as in Oct. 2013, updated to Dec. 2016 except for rut depths. Only the central length of each section was assessed as follows: E 64 m; F 80 m; G 50 m; HA 40 m

[2] General pavement condition according to visuals only. Blocked culvert on Section F at 22 m

[3] Mean spacing : block-hexagonal : 1-2 m; crocodile (C) : 150 mm; transverse (T) mostly full-width, 1-2 m spacing, often with secondary C cracking in wheelpaths, especially LIWP. Degree (D); extent (E) as % of length: 1: (<5 %) ; 2 : 5 - 10% ; 3 : >10 - 20% ; 4 : 20 - 50% ; 5 : >50%

[4] Mostly edge patching 0.2-0.5 m wide, seldom extending into OWP on midlane. "Full-width" patches on Section F only along and either side of centreline (CL). E_p= extent of patching

[5] Edge breaking usually accompanied by D5 cracking in outer 300 mm on 100 mm spacing

[6] Deviations under 2,0 m straight edge (n = 14 on Section E, 15 on F; 11 on G; 8 on HA) measured with a 20 mm wide wedge at approx. 5 m intervals including on patches (number shown). All with straight edge at rest. Inset = distance from seal edge to position of max. OWP rut depth. Offset = distance from CL to position of max. IWP rut depth

[7] Possible pumping [8] With D4 pumping (10 m = 25% of length) [9] Emulsion treatment only (no geotextile) 0 – 26 m x 0.5 m LHS edge on Section HA, regarded as surfacing rejuvenation, not patching

Table 5 Summary of visual evaluation and rut depths of Sections HB – K in December 2016 [1]

Parameter		RHS: Section HB (4% bitumen emul.)				LHS: Section JA (4% tar)				RHS : Section JB (8% tar)				Section K (crusher-run)			
Pavement Conditions [2]		**Fair**				**Poor**				**Fair**				**Very good**			
Cracking [3]	**Block**	D5 / E3; Transverse D4 / E5 (60%)				D5 / E5;Transverse D5 / E5 (80%)				D4 / E5; Transverse D4 / E5 (60%)				-			
	Croc:																
	LOWP	-				D4/E2 (10%)				-				-			
	LIWP	-				D4 / E5 (70%) [7]				-				-			
	RIWP	D4 / E3 (20%)				-				D3 / D2 (10%)				-			
	ROWP	D3 / E1 (5%)				-				D3 / D1 (5%)				-			
Patching [4]	**Full-width**	-				-				-				-			
	Edge to :																
	LOWP	- -				D2 / E4: 10 m x 0.5 m = 51 m^2 E_p= 26				- -				D1 / E1: 2 m x 0.2 m = 0,5 m^2 E_p= 3			
	ROWP	D2 / E2: 8 m x 0,3 m = 4 m^2 E_p= 22				- -				D3 / E1: 1 pothole (E_p= 3)				- -			
Edge breaking [5]		D2 / E5				D2 / E5				D2 / E5				D1 / E5			
Deviations ("ruts") [6]		90% -ile	80% -ile	50% -ile	Patches	90% - ile	80% -ile	50% -ile	Patches [4]	90% -ile	80% -ile	50% - ile	Patches	90% -ile	80% -ile	50% -ile	Patches
		mm	mm	mm	No.	mm	mm	mm	No.	mm	mm	mm	No.	mm	mm	mm	No.
LHS Bultfontein-bound	Edge	-	-	-	-	10	9	5	4	-	-	-	-	8	7	4	0
	OWP	-	-	-	-	11	9	7	-	-	-	-	-	6	6	4	0
	Inset(m)	-	-	-	-	0.8	0.7	0.6	-	-	-	-	-	0.8	0.7	0.6	0
	IWP	-	-	-	-	12	10	8	-	-	-	-	-	8	7	5	0
	Offset(m)	-	-	-	-	0.5	0.4	0.4	-	-	-	-	-	0.5	0.4	0.3	0
	CLL	-	-	-	-	5	4	3	-	-	-	-	-	8	7	4	0
RHS Hoopstad -bound	CLR	16	14	9	0	-	-	-	-	5	4	3	0	10	8	5	0
	IWP	13	11	7	0	-	-	-	-	9	8	5	0	8	7	5	0
	Offset(m)	0.4	0.3	0.3	0	-	-	-	-	0.7	0.6	0.5	0	0.5	0.4	0.3	0
	Inset (m)	0.8	0.7	0.5	0	-	-	-	-	0.6	0.5	0.4	0	0.6	0.5	0.4	0
	OWP	12	9	6	0	-	-	-	-	13	11	7	0	5	4	3	0
	Edge]	13	10	5	3	-	-	-	-	13	10	6	0	7	6	5	0

NOTES

[1] Only the central length of each section was assessed according to TMH 9 : 1992 as follows: HB : 40 m; JA : 40 m; JB : 40 m; K : 80 m

[2} General pavement condition according to visuals only

[3] Mean spacing : block-hexagonal : 1-2 m; crocodile (C) : 150 mm; transverse (T) mostly full-width, 1-2 m spacing, often with secondary C cracking, especially in LIWP. Degree (D); extent (E) as % of length : 1: (<5 %) ; 2 : 5 - 10% ; 3 : >10 - 20% ; 4 : 20 - 50% ; 5 : >50%

[4] Mostly edge patching 0,2-0,5 m wide, seldom extending into OWP. E_p= extent of patching

[5] Edge breaking usually accompanied by 25 cracking in outer 300 mm on 100 mm spacing

[6] Deviations under 2,0 m straight edge (n = 14 on Sections HB, JA & JB; 15 on K measured with a 20 mm wide wedge at approx. 5 m intervals including on patches (number shown). All with straight edge at rest. Inset = distance from seal edge to position of max. OWP rut depth. Offset = distance in m from centreline (CLL or CLR to position of max. IWP rut depth

[7] Possible pumping

Although TMH 9: 1992 did not provide any percentage definitions, those above have been adopted here because 5, 10, 20 and 50% represent the respective lengths of Category A (freeways and major rural), B (rural), C (lightly trafficked) and D (rural access) road allowed to perform "unsatisfactorily" (TRH 12 : 1997) at the end of its design life (TRH 4: 1996, TRH 12 : 1997). Thus, for example, a Category C road such as this would be deemed to be in a terminal condition **with respect to cracking** if it had say Degree 5 crocodile cracking of Extent 4 (coded D5/E4).

Performance criteria are provided in TRH 12 : 1997 for the four road Categories A, B, C and D in terms of the extent of crocodile, longitudinal and "other" cracking, patching ravelling, smoothing, riding quality, rut depth, skid resistance, deflections, and DCP measurements (DSN_{800}).

Combinations of cracking "other" than crocodile (e.g. block and transverse) are added together.

In the case of **patching** both the length and the area of the patching are combined into a single index (TRH 12: 1997), here coined E_p:

$E_p = Lp + \sqrt{A_L}$

where Lp = % of unit length patched

A_L = area of patches in m^2

These observations showed ravelling and smoothing to be insignificant, riding quality and skid resistance to be adequate, shear failures, potholing and excessive rutting to be absent or insignificant, and the only problems to be extensive Degree 4 – 5 block to crocodile cracking and edge breaking on Sections F – J (as well as A – E) due to the large trucks travelling close to the edge of the narrow 6,0 m-wide seal, which varied in width from about 5.8 to 6.0 m. In many cases these cracks were only 10 – 15 mm deep and did not reach the base course through the 30 mm-thick surfacing. Sounding with a chain drag indicated that the surfacing had debonded from the base only in areas of crocodile cracking in the wheelpaths and along the edges.

The visual observations are then combined to give an indication of the overall condition of the pavement :

Condition	**Description**
Very good :	Very few or no defects, all < D3 (warning)
Good :	Few defects, mostly < D3
Fair :	Few defects, but seldom D5 (severe). Extent only local (E1) if severe (excluding surfacing defects)
Poor :	General occurrence (E3) of particularly structural D3 – D5 defects
Very poor :	Many defects, mostly D5, of general (E3) to intensive (E5) extent

Recommendations are then given with respect to the type of maintenance (e.g. routine, reseal or rehabilitation) required and its priority (A: Urgent, B: Within 6 months and C: When convenient). These are shown on the visual assessment forms in **Annex D** and are not discussed in detail here.

The general pavement condition of all of these sections except Section K was provisionally only visually rated as poor or fair on this account.

As expected, the crusher-run Section K was the best, with practically no distress and was provisionally rated as very good. The 10% OPC sand section was the second best.

A visual inspection of Sections A – E showed only minimal deterioration since 2013, mostly in the form of pumping on Section B and C and further edge breaking.

These observations showed the general pavement condition of all the sections **inside the edge breaks** to be sound and indicated that, if the edge breaks were patched, the road widened (or the width of the trucks limited) and the cracks sealed, all the sections would continue to carry traffic for several years yet.

In short, all these pavement designs were structurally sound and any future roads to such designs must simply be sufficiently wide to accommodate the traffic expected.

The results of the rut depth survey are shown in **Annex E** and summarised for both lanes in **Tables 3 – 6.**

Because the road had been built with a camber, on Sections A and B rut depths were initially measured both with the straight edge lying free as usual and also with the end held down on the edge of the seal or the centreline as appropriate.

In the case of the rest of the sections only the standard "free" method was used. However, deviations under the straight edge both at the edge and the centreline of the seal were recorded in order to give a better picture of the cross-section.

From Section F to K the inset of the maximum outer wheelpath rut depth from the edge of the seal and the offset of the inner wheelpath from the centreline was also measured.

Although only distress with a severity rating (degree) of 3 – 5 is used to determine the condition of the pavement, because this is a research project all degrees have been recorded.

The following general rules apply to both visual measurements such as rut depth (shown as an example), deflection, roughness, etc., for which criteria are provided in TRH 12: 1997

Present condition	**Extent of Degree 3 – 5 distress**	**Rut depth (mm)**
• Sound (i.e. adequate for design traffic)	< X	< 10
• Warning (adequate for minimum but not maximum design traffic)	≥ X - < Y	≥ 10 - < 20
• Severe (inadequate for minimum design traffic) :	≥ Y	≥ 20

`

6. Deflection Survey

The falling weight deflectometer (FWD) survey using a Dynatest 8002 instrument was carried out in the left outer wheelpath at approximately 10 m intervals over Sections A to G and K, and in the right outer wheelpath on Sections HB and JB, which were only in the right lane. Deflections were also taken in three positions in the midlane in order to estimate the effect of a sealed shoulder.

Due to the relatively narrow seal the outer wheelpath measurements could only be taken at about 0,9 m from the edge, i.e. up to about 0,4 m inside the position of the deepest rut. This means that the results may be slightly optimistic.

Measurements taken both at the usual standard load of 40 kN as well as at 50 kN as now required by Sanral are shown in **Annex F** and the 40 kN results summarised in **Tables 6 – 9** and, more completely, together with the June 2017 DCP results from **Annex G** for Sections A and F – K, in **Tables 10 – 13** to be shown later. No attempt has been made to calculate moduli from the deflections.

The condition rating has been based mostly on the base layer index (BLI) which correlates best with the surfacing and base course.

The middle layer index (MLI) correlates mostly with the subbase.

The lower layer index (LLI) correlates mostly with the selected, any fill, and the roadbed.

The FWD radius of curvature (RoC) at 200 mm correlates with the results of the Dehlen curvature meter. However, it is regarded as less reliable than the BLI due to the closeness of the geophone at only 200 mm from the edge of the loading plate (Horak 2008). This opinion is borne out by the serious overestimates of capacity shown later in **Tables 10 – 13**.

The maximum surface deflection (Ymax or D_O) provides an assessment of the condition of the whole pavement.

According to TRH 12 : 1997 criteria the survey showed that the upper layers of all the sections were in a very flexible behaviour state condition except for the bases of Sections D (10% OPC), G (2% sulphite lye) HB (4% emulsion) and K (crusher-run) and, marginally, F (5% PBFC), which were all in a flexible state.

The lower layers of all the sections were in either a stiff or very stiff state and in either a marginally warning/sound or sound structural condition.

However, according to the criteria for the maximum surface deflection (Ymax) all the sections except K and, marginally, HB were in a severe structural condition.

As all the bases were either in very flexible or flexible states estimates of structural capacity were initially made assuming a flexible base on all.

Estimates appropriate to a bitumen-bound base were also made for the bitumen- and tar-stabilized sections and for a cemented base only to Sections D (10% OPC) and F (5% PBFC), which were the only ones exhibiting a DCP UCS of at least 750 kPa which was assumed to represent the equivalent field UCS of a COLTO : 1998 C4 material (laboratory minimum 97 % MAASHO UCS of 500 kPa).

The results most relevant to a Category C road (i.e. at an 80- or 20- percentile as appropriate) are **embolded**.

Although in the development of the criteria shown in TRH 12 the Ymax and BLI were found to correlate best with capacity (P Joubert 1994 pers. comm.), as is usual, the lowest estimate of **residual** structural capacity using the Ymax, Base Layer, Middle Layer and Lower Layer Indices was conservatively taken as the best FWD-based estimate.

Whilst these estimates are presented here, they must be used with “extreme care” and only “as a rough first indicator of pavement condition” (TRH 12 : 1997). Horak (2008) went further and stated that they could be embarrassingly inaccurate and that the parameters should rather be used to enhance the behaviour state and condition. This is also the current view of Prof G Jordaan (2017 pers. comm.). As this is a research project both approaches are used here.

Table 6 Summary of outer wheelpath December 2016 deflection and 2013 rut depth and DCP test results on Sections A – C, and estimates of residual capacity

SECTION			**Section A (Neat sand)**						**Section B (Sand + 3% OPC)**						**Section C (Sand + 5% OPC)**					
Layer	Test	Units	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)	80%- -ile	50% -ile	20%- ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)
						[1]	[2]	[1]				[1]	[2]	[1]				[1]	[2]	[1]
	Deflection (L)																			
Pavement	Y max	μm	**900**	822	723	V. flex.	Severe	0.15	**840**	800	760	V. flex.	Severe	0.2	**886**	837	789	V. flex.	Severe	0.15
	RoC	m	84	74	**66**	-	Warn.	-	70	66	62	-	Warn.	-	77	70	**64**	-	Warn.	-
[3]	BLI	μm	**523**	482	436	V. flex.	Severe	0.2	**546**	520	495	V. flex.	Severe	0.15	**528**	499	471	V. flex.	Severe	0.2
	MLI	μm	**254**	233	211	V. flex.	Severe	0.3	**226**	196	167	V. flex.	Severe	0.4	**282**	252	222	V. flex.	Severe	0.2
	LLI	μm	**54**	43	28	Stiff	Warn.	2.5	**38**	32	26	V. Stiff	Sound	3.0	**43**	37	32	Stiff	Sound	4.0
	Capacity [8]	ME80	**0.15**	-	-	-	-	0.15	**0.15**	-	-	-	-	**0.15**	**0.15**	-	-	-	-	**0.15**
							[1]	[15]					[1]	[15]					[1]	[15]
Pavement [4, 5]	**Rut Depth** (L)	mm	**10**	7	-	-	Warn.	**1.0**	**8**	5	-	-	Sound	**1.5**	**10**	7	-		Warn.	**1.0**
	DCP (L + R)																			
	DN	mm/b	7.0	6.3	5.6				5.7	4.5	2.6				5.4	4.2	3.5			
	CBR [9]	%	46	41	34				120	67	45				88	67	50			
	CBR/UCS [10]	% / kPa	>100	>100	>100				-	550	-				-	620	-			
150 mm	FWC [11]	%	-	3.5	-				-	5.2	-				-	7.3	-			
Base [6]	OWC [12]	%	-	7.5	-				-	-	-				-	-	-			
	FWC/OWC	-		0.47					-	-	-				-	-	-			
	DN	mm/b	6.9	5.6	5.3				5.5	4.0	2.7				5.2	4.1	3.1			
Redefined	CBR [9]	%	49	46	35				118	79	47				88	67	50			
upper	CBR [10]	% / kPa	>100	>100	>100				-	650	-				-	630	-			
layer [7]	Thickness	mm	358	275	171				595	236	123				334	290	195			
Pavement						**Mean**		[16]				**Mean**		[16]				**Mean**		[16]
Balance No. (A), Category [13]		%.mm	2478	1902	1707	ABD	80%	80%	2149	1893	1605	ABI	80%	80%	2596	2193	1391	ABD	80%	80%
Balance No. (B) [13]		mm	5	1	-1				11	-0.5	-3				16	11	5			
DSN 800 (Including seal) [13]		Blows	205	196	185		Warn.		280	255	190		Warn.		187	175	168		Warn.	
Structural capacity [13]		MISA	3.7	3.1	**2.6**			1.3	11	7.9	**2.8**			1.7	2.7	2.1	1.8			0.9
DSN 800 (Excluding seal) [14]		Blows	181	175	170		Warn.		260	237	175		Sound		174	152	148		Warn.	
Structural capacity [14]		MISA	2.4	2.1	**1.9**			**0.9**	8.5	6.1	**2.1**			**1.2**	2.1	1.3	**1.2**			**0.6**

NOTES

[1] Behaviour state, estimated structural capacity and performance criteria for rut depth and DSN 800 for granular base according to TRH 12 : 1997 [2] Structural condition rating according to Horak (2008) for granular base [3] Deflections on 05 Dec 2016 in LHS outer wheelpath (LO) (n= 6 or 7) at 40 kN and 565 kPa. RoC at 200 mm [4] Rut depths in left outer wheelpath measured on 08 Oct. 2013, n usually about 14 [5] DCPs in January and October/November 2013 with both lanes combined, n = 3 in LO and 3 in RHS (RO) outer wheelpath on Sections A and B, 3 in LO and 2 in RO on Section C. Processed by P Paige-Green using EasyDCP program (J Lea, pers. comm.) using Kleyn granular base model and Cm = 30 (moderate (OWC) moisture regime). Mean UCS calculated from DN by author [6] Surfacing (± 25 mm) removed by inspection of penetration curve during processing (i.e. zero taken at top of base) [7] Surfacing removed as above and uniform layers redefined by computer. Water contents of Section A subbase 8,0; 8,3 %; selected 5,2; 5,3 %, fill 7,8 % [8] Minimum 80%-ile structural capacity assuming granular base [9] Mean CBR of layer from mean Kleyn (1984) relationship: CBR = 410 DN $^{-1,27}$ for DN > 2 in program [10] CBR for uncemented sand from mean sand relationship (Netterberg 2015a : CBR = 3 000 DN $^{-1,46}$ for DN > 10). UCS for cemented materials = 2 900 DN $^{-1.08}$ (Kleyn 1984) [11] Base field water content (n = 4 on A, 2 on B, 3 on C) [12] Mean base OWC = 7,5 % (n = 4) [13] Including surfacing (i.e. zero taken at top of surfacing.) Balance category is for mean (50 % ile) result: AB = averagely balanced, P = poorly, D = deep, I = inverted structure. [14] 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) [15] Assuming linear increase in rut depth to 20 mm from existing rut depth and 1.0M E80 in 2013 [16] Assuming linear increase in rut depth to 20 mm from existing depth [17] **Relevant results for Cat. C road embolded.** [18] Estimated residual structural capacity beyond range of DCP method bracketed

Table 7 Summary of outer wheelpath December 2016 deflection results on all sections, October 2013 rut depths on Sections D and E and December 2016 on F, and 2013 DCPs on Sections D and E and December 2016 on Section F, and estimates of residual capacity

SECTION			Section D (10% OPC)						Section E (5 %PBFC)						Section F (5% PBFC)					
Layer	Test	Units	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap ME80 (80% -ile)	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap ME80 (80% -ile)	80%- -ile	50% -ile	20% ile	State (80% -ile)	Con-dition (80% -ile)	Cap ME80 (80% -ile)
						[1]	[2]	[1]				[1]	[2]	[1]				[1]	[2]	[1]
	Deflection (L)																			
Pavement	Y max	µm	**808**	731	654	V. flex.	Severe	0.25	**836**	784	732	V. flex.	Severe	0.2	**834**	744	656	V. flex.	Severe	0.2
	RoC	m	115	93	**72**	-	Warn.	-	79	72	**64**	-	Warn.	-	105	88	**71**	-	Warn.	-
[3]	BLI	µm	**453**	377	301	Flex.	Severe	0.3	**514**	461	408	V flex.	Severe	0.3	**497**	414	332	Flex.	Severe	0.2
	MLI	µm	**257**	237	217	V. flex.	Severe	0.3	**244**	225	206	V flex.	Severe	0.3	256	227	199	V. flex.	Severe	0.3
	LLI	µm	**63**	55	46	Stiff	Warn.	2.0	**56**	43	29	Stiff	Warn.	2.5	**62**	49	36	Stiff	Warn.	2.0
	Capacity [8]	ME80	**0.25**	-	-	-	-	**0.25**	**0.2**	-	-	-	-	**0.2**	**0.2**	-	-	-	-	**0.2**
							[1]	[15]					[1]	[15]					[1]	[15]
Pavement [4, 5}	**Rut Depth** (L)	mm	**6**	**4**	-	-	Sound	**2.3**	**8**	5	-	-	Sound	**1.5**	**9**	7	-	-	Sound	**1.2**
								-												
	DCP (L + R)																			
	DN	mm/b	3.1	2.5	1.9				6.4	6.0	5.6				3.1	2.7	2.3			
	CBR [9]	%		128						42						117				
	CBR/UCS [10]	% / kPa		1070						400						990				
150 mm	FWC [11]	%		5.7						8.6						-				
Base [6]	OWC [12]	-		-						-						-				
	FWC/OWC	-		-						-						-				
	DN	mm/b	5.9	5.1	4.3				5.7	5.1	4.5				3.9	3.1	2.3			
Redefined	CBR [9]	%		52						5.2						147				
upper	CBR [10]	% / kPa		490						490						850				
layer [7]	Thickness	mm		375						472						1.63				
Pavement						**Mean**		[16]				**Mean**		[16]				**Mean**		[16]
Balance No. (A), Category [13]		%.mm		3385		PBD	**Mean**	**Mean**		1503		ABD	**Mean**	**Mean**		2273		ABI	**Mean**	**Mean**
Balance No. (B) [13]		mm		16						6						-1				
DSN 800 (Including seal) [13]		Blows		245			Sound			214			Sound			327			Sound	
Structural capacity [13]		MISA		6.9				5.5		4.3				3.2		(20)				(13)
DSN 800 (Excluding seal) [14]		Blows		222			Sound			200			Sound			325			Sound	
Structural capacity [14]		MISA		4.9				3.9		3.4				2.6		(20)				(13)

NOTES

[1] Behaviour state, estimated structural capacity and performance criteria for rut depth and DSN_{800} for granular base according to TRH 12 : 1997 [2] Structural condition rating according to Horak (2008) for granular base [3] Deflections on 05 Dec 2016 in left outer wheelpath (n = 6 or 7) at 40 kN and 565 kPa. RoC at 200 mm [4] Rut depths in left outer wheelpath on Sections D and E measured on 08 Oct. 2013, and F on 06 Dec. 2016, n usually about 14 [5] Sections D and E DCPs in January and October/November 2013 with both lanes combined, n = 4 on LHS + 3 on RHS on Section D and 4 on LHS + 3 on RHS on E, Section F done in Dec 2016 n = 4 on **LHS only**. Processed by J Briedenhann and author using WINDCP 5.1. All using Kleyn granular base model and Cm = 30 (moderate (OWC) moisture regime) [6] Surfacing (± 25 mm) removed by inspection of penetration curve during processing (i.e. zero taken at top of base) [7] Surfacing removed as above and uniform layers redefined by computer. Water contents of subbase unknown [8] Minimum 80%-ile structural capacity [9] Mean CBR of layer from mean Kleyn (1984) relationship: $CBR = 410\ DN^{-1.27}$ for DN > 2 in program [10] CBR for uncemented sand from mean sand relationship (Netterberg 2015a : $CBR = 3\ 000\ DN^{-1.46}$ for DN > 10). UCS for cemented materials = $2\ 900\ DN^{-1.08}$ (Kleyn 1984) [11] Base field water content (n = 3 on D (4.4 – 7.3%), 2 on E (8.2 and 8.9%), F unknown [12] Base OWC unknown [13] Including surfacing (i.e. zero taken at top of surfacing). Balance category is for mean (50 % ile) result: AB = averagely balanced, P = poorly, D = deep, I = inverted structure [14] Excluding surfacing (i.e. zero taken at top of base). [15] Assuming linear increase in rut depth to 20 mm from existing rut depth and 1.0M E80 in 2013 [16] Assuming linear increase in rut depth to 20 mm from existing depth **[17] Relevant results for Cat. C road embolded** [18] Estimated residual structural capacity beyond range of DCP method bracketed

Table 8 Summary of outer wheelpath December 2016 deflection, rut depth and DCP test results on Sections G, HA and HB, and estimates of residual capacity

SECTION			Section G (2% sulphite lye)						LHS: Section HA (8% bitumen)						RHS: Section HB (4% bitumen)					
Layer	Test	Units	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)
						[1]	[2]	[1]				[1]	[2]	[1]				[1]	[2]	[1]
Pavement	**Deflection** (L or R)																			
	Y max	µm	**790**	753	716	V. flex.	Severe	0.25	**788**	732	676	V. flex	Severe	0.25	**738**	731	723	Flex.	Warn.	0.3
	RoC	m	100	89	**78**	-	Warn.	-	108	87	**65**	-	Warn.	-	115	101	**87**	-	Warn.	-
[3]	BLI	µm	**452**	418	384	Flex.	Severe	0.3	**542**	411	280	V. flex	Severe	0.15	**364**	355	346	Flex.	Warn.	0.5
	MLI	µm	**243**	219	194	V. flex.	Severe	0.3	**270**	213	156	V. flex	Severe	0.25	**299**	276	253	V. flex.	Severe	0.2
	LLI	µm	**54**	47	41	Stiff	Warn.	2.5	**50**	45	41	Stiff	Warn.	3.0	**52**	35	17	Stiff	Warn.	3.0
	Capacity [8]	ME80	**0.25**	-	-	-	-	**0.25**	**0.15**	-	-	-	-	**0.15**	**0.2**	-	-	-	-	0.2
							[1]	[15]					[1]	[15]					[1]	[15]
Pavement [4, 5]	**Rut Depth** (L or R)	mm	**14**	10	-	-	Warn.	**0.4**	**8**	6	-		Sound	**1.5**	**9**	6	-	-	Sound	**1.2**
	DCP (L or R)																			
150 mm Base [6]	DN	mm/b	6.6	5.8	5.0				5.5	4.8	4.1				5.5	4.6	3.7			
	CBR [9]	%		44						56						59				
	CBR/UCS [10]	% / kPa		>100						(520)						(540)				
	FWC [11]	%		-						-						-				
	OWC [12]	%		-						-						-				
	FWC/OWC	-		-						-						-				
Redefined upper layer [7]	DN	mm/b	5.8	4.9	4.0				5.0	4.3	3.6				4.8	4.3	3.8			
	CBR [9]	%		55						63						65				
	CBR [10]	% / kPa		>100						(580)						(590)				
	Thickness	mm		376						567						567				
Pavement						**Mean**		[16]				**Mean**		[16]				**Mean**		[16]
Balance No. (A), Category [13]		%.mm		2603		ABI		**Mean**		2069		ABI		**Mean**		2502		ABI		**Mean**
Balance No. (B) [13]		mm		-10						-6						-7				
DSN 800 (Including seal) [13]		Blows		263			Sound			239			Sound			249		Sound		
Structural capacity [13]		MISA		8.9				4.4		6.3				4.4		7.3				5.1
DSN 800 (Excluding seal) [14]		Blows		249			Sound			220			Sound			234		Sound		
Structural capacity [14]		MISA		7.4				3.7		4.7				3.3		6.0				4.2

NOTES

[1] Behaviour state, estimated structural capacity and performance criteria for rut depth and DSN 800 for granular base according to TRH 12 : 1997 [2] Structural condition rating according to Horak (2008) for granular base [3] Deflections on 05 Dec 2016 in left (RHS on HB) outer wheelpath (n = 5 on Section G, 3 on HA and 2 on HB) at 40 kN and 565 kPa. RoC at 200 mm [4] Rut depths in LHS outer wheelpath (RHS on HB) measured on 08 - 09 Dec. 2016, n usually about 14 [5] DCPs in December 2016, n = 3 in left outer wheelpath on Sections G and HA, and 3 in right on Section HB. Processed by J Briedenhann and author using Kleyn granular base model and Cm = 30 (moderate (OWC) moisture regime) [6] Surfacing (± 25 mm) removed by inspection of penetration curve during processing (i.e. zero taken at top of base) [7] Surfacing removed as above and uniform layers redefined by computer. Water contents of subbase unknown [8] Minimum 80%-ile structural capacity [9] Mean CBR of layer from mean Kleyn (1984) relationship: CBR = 410 DN $^{-1.27}$ for DN > 2 in program [10] CBR for uncemented sand from mean sand relationship (Netterberg 2015a : CBR = 3 000 DN $^{-1.46}$ for DN > 10). UCS for cemented materials = 2 900 DN $^{-1.08}$ (Kleyn 1984). UCS for bitumen tar-treated sands bracketed. Balance category is for mean (50 % ile) result: AB = averagely balanced, P = poorly, D = deep, I = inverted structure [11] Base field water content unknown [12] Base OWC unknown [13] Including surfacing (i.e. zero taken at top of surfacing) [14] Excluding surfacing (i.e. zero taken at top of base) 15] Assuming linear increase in rut depth to 20 mm from existing rut depth and 1.0M E80 in 2013 [16] Assuming linear increase in rut depth to 20 mm from existing depth **[17] Relevant results for Cat. C road embolded** [18] Estimated residual structural capacity beyond range of DCP method bracketed

Table 9 Summary of outer wheelpath December 2016 deflection, rut depth and DCP test results on Sections JA, JB and K, and estimates of residual capacity

SECTION			**LHS: Section JA (4 % tar)**						**RHS : Section JB (8 % tar)**						**Section K (crusher-run)**					
Layer	Test	Units	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)
						[1]	[2]	[1]				[1]	[2]	[1]				[1]	[2]	[1]
	Deflection (L or R)																			
Pavement	Y max	µm	**816**	761	706	V. flex	Severe	0.2	**800**	668	537	V. flex	Severe	0.3	**629**	559	489	Flex.	Warn.	0.5
	RoC	m	96	81	**65**	-	Warn.	-	144	103	**62**	-	Warn.	-	151	126	**102**	-	Sound	-
	BLI	µm	**536**	429	323	V. flex	Severe	0.2	**565**	369	174	V. flex	Severe	0.15	**330**	283	231	Flex.	Warn.	0.7
[1]	MLI	µm	**305**	236	167	V. flex	Severe	0.2	**239**	182	126	V. flex	Severe	0.3	**190**	165	140	Flex.	Warn.	0.6
	LLI	µm	**53**	40	28	Stiff	Warn.	3.0	**57**	56	55	Stiff	Warn.	3.0	**54**	49	44	Stiff	Warn.	3.0
	Capacity [8]	ME80	**0.2**	-	-	-	-	**0.2**	**0.15**	-	-	-	-	**0.15**	**0.5**	-	-	-	-	**0.5**
							[1]	[15]					[1]	[15]					[1]	[1]
Pavement	**Rut Depth** (L or R)	mm	**9**	7	-	-	Sound	**1.2**	**11**	7	-	-	Warn.	**0.8**	**6**	4	-	-	Sound	**2.3**
[4, 5]	**DCP** (L or R)																			
	DN	mm/b	5.4	4.4	3.4				4.2	3.5	2.8				1.5	1.3	1.1			
	CBR [9]	%		63						85						250				
150 mm	CBR/UCS [10]	% / kPa		(570)						(750)						-				
Base [6]	FWC [11]	%		-						-						-				
	OWC [12]	%		-						-						-				
	FWC/OWC	-		-						-						-				
	DN	mm/b	4.4	3.6	2.8				4.0	3.4	2.8				2.8	2.6	2.4			
Redefined	CBR [9]	%		81						88						125				
upper	CBR [10]	% / kPa		(715)						(770)						1050				
layer [7]	Thickness	mm		528						743						609				
Pavement						**Mean**		[16]				**Mean**		[16]				**Mean**		[16]
Balance No. (A), Category [13]		%.mm		2840		ABI	**Mean**	**Mean**		4105		PBD	**Mean**	**Mean**		4151		PBD	**Mean**	**Mean**
Balance No. (B) [13]		mm		-15						0						2				
DSN 800 (Including seal) [13]		Blows		400			Sound			320			Sound			560			Sound	
Structural capacity [13]		MISA		(38)				(25)		(18)				(12)		(120)				(100)
DSN 800 (Excluding seal) [14]		Blows		375			Sound			286			Sound			528			Sound	
Structural capacity [14]		MISA		(30)				(20)		12				7.8		(100)				(80)

NOTES

[1] Behaviour state, estimated structural capacity and performance criteria for rut depth and DSN 800 for granular base according to TRH 12 : 1997 [2] Structural condition rating according to Horak (2008) for granular base [3] Deflections on 05 Dec 2016 in left (RHS on JB) outer wheelpath (n = 3 on Sections JA and JB and 8 on K) at 40 kN and 565 kPa. RoC at 200 mm [4] Rut depths in the LHS outer wheelpath (RHS on JB) measured on 08 - 09 Dec. 2016, n usually about 14 [5] DCPs done Dec. 2016, n = 3 in left (RHS on JB) outer wheelpath on Sections JA and 2 on K. Processed by J Briedenhann and author using Kleyn granular base model and Cm = 30 (moderate (OWC) moisture regime) [6] Surfacing (± 25mm) removed by inspection of penetration curve during processing (i.e. zero taken at top of base) [7] Surfacing removed as above and uniform layers redefined by computer. Water contents of subbase unknown [8] Minimum 80%-ile structural capacity [9] Mean CBR of layer from mean Kleyn (1984) relationship: $CBR = 410\ DN^{-1{,}27}$ for DN > 2 in program [10] CBR for uncemented sand from mean sand relationship (Netterberg 2015a) : $CBR = 3\ 000\ DN^{-1{,}46}$ for DN > 10). UCS for cemented materials = $2\ 900\ DN^{-1.08}$ (Kleyn 1984). UCS for tar-treated sands bracketed. Balance category is for mean (50 % ile) result: AB = averagely balanced, P = poorly, D = deep, I = inverted structure [11] Base field water content unknown [12] Base OWC unknown [13] Including surfacing (i.e. zero taken at top of surfacing) [14] 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) [15] Assuming linear increase in rut depth to 20 mm from existing rut depth and 1.0M E80 in 2013 [16] Assuming linear increase in rut depth to 20 mm from existing depth **[17] Relevant results for Cat. C road embolded** [18] Estimated residual structural capacity beyond range of DCP method bracketed

The results of a light falling weight deflectometer (LWD) survey in the left outer wheelpath of Sections A, B and C in April 2014, after the rainy season are shown in Table 8 of Netterberg (2015a).

Only Ymax was measured, which was essentially the same on all three sections and which was about two-thirds of the full-size FWD measurements made in December 2016 during the start of the rainy season.

7. DCP Surveys

The results of the 2013 survey of Sections A – C from Netterberg (2015) using EasyDCP (J Lea 2013, pers. comm) and the December 2016 survey processed using the CSIR WinDCP 5.1 Version 10002 (1986-2012) program are shown in **Annex G** and summarised in **Tables 6-9**. This program was used in preference to AFCAP WinDCP as a UCS and CBR in addition to DN were required, as well as the capability for analysis in terms of both the granular and cemented base models, and also for compatibility with the previous work. However, it only calculates means for all results and 80 percentiles for DN. Anything else has to be done manually and it has only been done here for the 20 percentiles of DN.

Typed copies of the June 2017 field sheets as supplied by Geoplan are shown in **Annex H**, the processed results in **Annex G** and the road results summarised in **Tables 10–13**. In this work only the left-hand outer wheelpath was evaluated except in the case of Sections HB and JB which were only on the right-hand side, in which case the right-hand outer wheelpath was tested.

In the case of **Tables 6–9** the results for the outer wheelpaths in both lanes were combined in order to have as many results as possible.

In the case of **Tables 10–13** only the results for the left-hand outer wheelpath have been used, except in the case of Section E where the two wheelpaths had to be combined for enough results, and Sections HB and JB. This usually also yielded more conservative results than when both lanes were combined. The 2013 results for Sections B and C were also re-analysed with WinDCP 5.1.

A TRH: 1996 Category C road, a moderate moisture regime (i.e. approximately at OWC) and, for comparison, a medium traffic (0.2 – 0.8 MISA) design were used as input parameters.

The program was run first using the Kleyn granular base model and then, where appropriate, the De Beer lightly cemented base model.

The granular base model can be used on pavements with both granular and lightly cemented bases (Kleyn and Savage 1982, COLTO 1997), although it tends to overpredict the structural capacity of cemented pavements – sometimes seriously (De Beer et al 1989). In this case the deflections, DCP and visual observations in trial pits all indicated that most of the cemented bases had reverted to an equivalent granular state and only Sections D (10% OPC), F (5% PBFC) and JB (8% tar) had an average, insitu UCS exceeding 750 kPa).

In the June 2017 survey the four outer wheelpath DCP points on each section (with only three on Sections H and J) were selected in order to cover a range of rut depth and as far as possible to coincide with the maximum rut depth at that stake value as this inset was usually about 0.5 – 0.7 m from the edge. Whilst six or eight points would have been preferred this was all that could be achieved in the time available.

This inset is unusually small, presumably due to the narrowness of the seal in comparison with the large trucks currently using the road.

Table 10 Summary of left outer wheelpath December 2016 deflection, October 2013 rut depth and June 2017 DCP test results on Section A and 2013 DCP results on Sections B – C, and estimates of residual capacity

SECTION			Section A (Neat sand)						Section B (Sand + 3% OPC)						Section C (Sand + 5% OPC)					
Layer	Test	Units	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)	80%- -ile	50% -ile	20%-ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)
						[1]	[2]	[1]				[1]	[2]	[1]				[1]	[2]	[1]
	Deflection (LO)																			
Pavement	Y max	µm	**900**	822	723	V. flex.	Severe	**0.15**	840	800	760	V. flex.	Severe	0.2	**886**	837	789	V. flex.	Severe	**0.15**
	RoC	m	84	74	66	-	Warn.	5	70	66	62	-	Warn.	4	77	70	64	-	Warn.	4
[3]	BLI	µm	523	482	436	V. flex.	Severe	0.2	**546**	520	495	V. flex.	Severe	**0.15**	528	499	471	V. flex.	Severe	0.2
	MLI	µm	254	233	211	V. flex.	Severe	0.3	226	196	167	V. flex.	Severe	0.4	282	252	222	V. flex.	Severe	0.2
	LLI	µm	54	43	28	Stiff	Warn.	2.5	38	32	26	V. Stiff	Sound	3.0	43	37	32	Stiff	Sound	4.0
	Str. cap. gran. [8]	ME80	**0.15**	0.2	-	-	-	**0.15**	**0.15**	0.2	-	-	-	**0.15**	**0.15**	0.2	-	-	-	**0.15**
	Str. cap. stab. [8]	ME80	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
							[1]	[20]					[1]	[20]					[1]	[20]
Pavement [4. 5]	**Rut Depth** (LO)	mm	**10**	7	-	-	Warn.	-	**8**	5	-	-	Sound	-	**10**	7	-	-	Warn.	-
	Struct. capacity	ME80	1.0	1.9	-	-	-	**1.0**	1.5	3.0	-	-	-	**1.5**	1.0	1.9	-	-	-	**1.0**
	DCP (LO)																			
150 mm Base [6]	DN	mm/b	7.8	6.8	5.8				5.6	5.2	4.8				6.0	5.4	4.8			
	CBR [9]	%		36						51						48	50			
	CBR/UCS [10]	% / kPa		>100						475						455				
	FWC [11]	%		10.9						5.2						7.3				
	OWC [12]	%		5.4						10.7						10.8				
	FWC/OWC	-		2.0						0.49						0.68				
Redefined upper layer [7]	DN	mm/b	7.0	6.0	5.0				5.4	4.9	4.4				5.9	5.4	4.9			
	CBR [9]	%		42						55						48	50			
	CBR/UCS [10]	% / kPa		>100						511						452				
	Thickness	mm		327						207						423	195			
Pavement						**Mean**	[1]	[21]				**Mean**	[1]	[21]				**Mean**	[1]	[21]
Balance No. (A), Category	[13]	%.mm		2172		ABD	**Mean**	**Mean**		1432		ABI	**Mean**	**Mean**		1561		ABD	**Mean**	**Mean**
Balance No. (B)	[13]	mm		0						-6						5				
DSN $_{800}$ (Incl. seal)	[14]	Blows		205			Sound			248			Sound			172			Warn.	
Struct. cap. (granular, incl.)	[14]	MISA		3.7				2.4		7.2				5.4		2.0				1.3
DSN $_{800}$ (Excl. seal)	[15]	Blows		189			Warn.			242			Sound			160			Warn.	
Struct. cap. (granular, excl.)	[15]	MISA		2.8				1.8		6.2				4.7		1.6				1.0
Struct. cap. (cemented, incl.)	[16]	MISA		0.2				0.13		0.4				0.3		0.8				0.5
BN_{100} (Incl. seal)	[17]	%		16						14						19				
Load equiv. expt (incl.)	[18]	n		1.4						1.2						1.7				
BN_{100} (Excl. seal)	[19]	%		8						9						13				
Load equiv. expt (excl.)	[18]	n		0.6						0.7						1.1				

NOTES TO TABLE 10

[1] Behaviour state, estimated structural capacity and performance criteria for rut depth and DSN 800 for granular base according to TRH 12 : 1997
[2] Structural condition deflection rating according to Horak (2008) for granular base
[3] Deflections on 05 Dec 2016 by SRT in left outer wheelpath (LO) (*n* = 6 or 7) at 40 kN and 565 kPa. RoC at 200 mm. 80%-ile Parameter predicting lowest capacity embolded
[4] Rut depths in left outer wheelpath in Oct. 2013, *n* usually about 14
[5] DCPs on Section A in June 2017 by Geoplan (*n* = 4); Sections B and C in Oct. 2013 by Freetrans (*n* = **3**). Processed by J Briedenhann and author using WinDCP 5.1 Version 10002 assuming medium traffic, Category C road, moderate, i.e. optimum (OWC) moisture condition and granular base unless otherwise stated
[6] Surfacing (25 mm) removed after inspection of penetration curve during processing (i.e. zero taken at top of base)
[7] Uniform layer below surfacing as redefined by computer. Field water content of Section A subbase 11.2%, selected subgrade (SSG) 11.9%
[8] Minimum 80- and 50%- ile structural capacities shown assuming granular (gran.) for all sections, and stabilized (stab.), i.e. bitumen treated base for Sections H and J and cement treated base where base UCS ≥ 750 kPa. The critical parameter which predicts the lowest capacity is **embolded.** In all cases the RoC capacity estimate shown is for granular base using the 20 %-ile value
[9] CBR of layer from mean Kleyn (1984) relationship: $CBR = 410\ DN^{-1,27}$ for DN > 2 in program
[10] CBR for uncemented sand from mean sand relationship (Netterberg 2015a) : $CBR = 3\ 000\ DN^{-1,46}$ for DN > 10) or UCS for cemented materials = $2\ 900\ DN^{-1.08}$ (Kleyn 1984) in program
[11] Base field water content (*n* = 3 (9.1 -12.8%) on A in June 2017, 2 on B, 3 on C in 2013)
[12] Mean Section A MAASHO OWC = 7.5% in 2013 (*n* = 4). Sections B and C from as-builts in Table 1 (Proctor?)
[13] Balance is for **mean** (50 % -ile) including surfacing: AB = averagely balanced, P = poorly, D = deep, I = inverted, structure.
[14] **Including surfacing** (i.e. zero taken at top of surfacing as in TMH6). Structural capacity to **additional** 20 mm rut depth : $MISA = Cm \times (DSN_{800})^{3.5} \times 10^{-9}$ in program from Kleyn (1984) assuming Cm = 30, i.e. ≈ OWC
[15] **Excluding surfacing** (i.e. zero taken at top of base as in Kleyn model). MISA calculated as in [14] above
[16] De Beer cemented model **including surfacing**. Structural capacity to **additional** 20 mm rut depth, including seal as in model
[17] BN_{100} **including** surfacing recalculated manually: $BN_{100} = DSN_{100} / DSN_{800} \times 100$
[18] Mean load equivalency exponent $n = 0,044\ (BN_{100})^{1.24}$ (Kleyn and Savage 1982)
[19] BN_{100} **excluding** surfacing calculated manually: $BN_{100} = (DSN_{125} – DSN_{25}) / (DSN_{825} – DSN_{25}) \times 100$
[20] Assuming linear increase in rut depth to 20 mm from existing rut depth and 1.0M E80 in 2013
[21] Assuming linear increase in rut depth to 20 mm from existing rut depth and DCP prediction to **additional** 20 mm. Mean or 80 %-ile as indicated
[22] **Relevant results for Category C road embolded**, where available
[23] Estimated residual structural capacity beyond range of DCP method bracketed
[24] Laboratory tests on Section A by Geoplan, on Section B and C by Freetrans

Table 11 Summary of left outer wheelpath December 2016 deflection results on all sections, 2013 rut depth and DCP test results on Sections D and E and December 2016 rut depth and June 2017 DCP results on Section F, and estimates of residual capacity

SECTION			Section D (10% OPC)						Section E (5 %PBFC)						Section F (5% PBFC)					
Layer	Test	Units	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)	80%- -ile	50% -ile	20%il e	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)
						[1]	[2]	[1]				[1]	[2]	[1]				[1]	[2]	[1]
	Deflection (LO)																			
	Y max	µm	**808**	731	654	V. flex.	Severe	**0.25**	**836**	784	732	V. flex.	Severe	**0.2**	**834**	744	656	V. flex.	Severe	**0.2**
	RoC	m	115	93	72	-	Warn.	6	79	72	64	-	Warn.	4	105	88	71	-	Warn.	6
Pavement	BLI	µm	453	377	301	Flex.	Severe	0.3	514	461	408	V flex.	Severe	0.3	**497**	414	332	Flex.	Severe	**0.2**
	MLI	µm	257	237	217	V. flex.	Severe	0.3	244	225	206	V flex.	Severe	0.3	256	227	199	V. flex.	Severe	0.3
[3]	LLI	µm	63	55	46	Stiff	Warn.	2.0	56	43	29	Stiff	Warn.	2.5	62	49	36	Stiff	Warn.	2.0
	Str. cap. gran [8]	ME80	**0.25**	0.3	-	-	-	**0.25**	**0.2**	0.3	-	-	-	**0.2**	**0.2**	0.3	-	-	-	**0.2**
	Str. cap. stab.[8]	ME80	**0.15**	0.15	-	-	-	**0.15**	-	-	-	-	-	-	**0.15**	0.2	-	-	-	**0.15**
							[1]	[20]					[1]	[20]					[1]	[20]
Pavement	**Rut Depth** (LO)	mm	**6**	4	-	-	Sound	**-**	**8**	5	-	-	Sound		**9**	7	-	-	Sound	**-**
[4, 5}	Struct. capacity	ME80	2.3	4.0	-	-	-	**2.3**	1.5	3.0	-	-	-	**1.5**	1.2	1.9	-	-	-	**2.2**
	DCP (LO)																			
	DN	mm/b	3.3	2.7	2.1				6.4	6.0	5.6				2.8	2.3	1.8			
	CBR [9]	%		116						42						147				
	CBR/UCS [10]	% / kPa		984						404						1210				
150 mm	FWC [11]	%		5.7						8.6						6.7				
Base [6]	OWC [12]	%		10.9						10.8						10.8				
	FWC/OWC	-		0.52						0.80						0.62				
	DN	mm/b	6.1	5.1	4.1				5.7	5.1	4.5				4.2	3.6	3.0			
Redefined	CBR [9]	%		52						52						81				
upper	CBR/UCS [10]	% / kPa		487						487						715				
layer [7]	Thickness	mm		671						471						695				
Pavement						**Mean**	[1]	[21]				**Mean**	[1]	[21]				**Mean**	[1]	[21]
Balance No. (A), Category [13]		%.mm		4069		PBD	**Mean**	**Mean**		1503		ABI	**Mean**	**Mean**		4680		PBD	**Mean**	**Mean**
Balance No. (B) [13]		mm		13						-6						11				
DSN $_{800}$ (Incl. seal [14]		Blows		244			Sound			214			Sound			314			Sound	
Struct. cap. (granular, incl.) [14]		MISA		6.8				5.4		4.3				3.2		(16)				10
DSN $_{800}$ (Excl. seal) [15]		Blows		233			Sound			207			Sound			279			Sound	
Struct. cap. (granular, excl.) [15]		MISA		5.8				4.6		3.8				2.9		11				7.2
Struct. cap. (cemented, incl.) [16]		MISA		5.7				4.6		0.2				0.15		12				7.8
BN_{100} (Incl. seal) [17]		%		28						13						32				
Load equiv. expt (incl.) [18]		n		2.7						1.1						3.2				
BN_{100} (Excl. seal) [19]		%		30						9						23				
Load equiv. expt (excl.) [18]		n		3.0						0.7						2.1				

NOTES TO TABLE 11

[1] Behaviour state, estimated structural capacity and performance criteria for rut depth and DSN 800 for granular base according to TRH 12 : 1997

[2] Structural condition deflection rating according to Horak (2008) for granular base

[3] Deflections on 05 Dec 2016 by SRT in left outer wheelpath (LO) (*n* = 6 or 7 per section) at 40 kN and 565 kPa. RoC at 200 mm. 80%-ile Parameter predicting lowest capacity embolded

[4] Rut depths in left outer wheelpath in Oct. 2013 on Sections D and E and Dec. 2016 on F, *n* usually about 14

[5] DCPs on Sections D (*n* = 4 in LO) and E (*n* = 2 in LO, 2 in RO combined) in Oct./Nov. 2013 by Freetrans, and Section F (*n* = 4 in LO) in June 2017 by Geoplan. Processed using WinDCP 5.1 Version 10002 by J Briedenhann and author assuming medium traffic, Category C road, moderate, i.e. optimum (OWC) moisture condition and granular base unless otherwise stated

[6] Surfacing (25 mm) removed after inspection of penetration curve during processing (i.e. zero taken at top of base)

[7] Uniform layer below surfacing as redefined by computer. Field water content of Section F subbase 6.8%, selected subgrade (SSG) 6.7%, others not tested

[8] Minimum 80- and 50%- ile structural capacities shown assuming granular (gran.) for all sections, and stabilized (stab.), i.e. bitumen treated base for Sections H and J and cement treated base where base UCS ≥ 750 kPa. The critical parameter which predicts the lowest capacity is **embolded.** In all cases the RoC capacity estimate shown is for granular base using the 20 %-ile value

[9] CBR of layer from mean Kleyn (1984) relationship: $CBR = 410\ DN^{-1,27}$ for DN > 2 in program

[10] CBR for uncemented sand from mean sand relationship (Netterberg 2015a) : $CBR = 3\,000\ DN^{-1,46}$ for DN > 10) or UCS for cemented materials = $2\,900\ DN^{-1.08}$ (Kleyn 1984) in program

[11] Field water content (*n* = 3 on D (4.4 – 7.3%), 2 on E (8.2 and 8.9%) in Oct./Nov. 2013 by Freetrans, 3 on F (6.4 – 7.0%) in June 2017 by Geoplan

[12] Proctor (?) base OWC from as-builts on Table 1. MAASHO OWC 9.1% on disturbed sample (*n* = 1) in June 2017 by Geoplan

[13] Balance is for **mean** (50 % -ile) including surfacing: AB = averagely balanced, P = poorly, D = deep, I = inverted, structure.

[14] **Including surfacing** (i.e. zero taken at top of surfacing as in TMH6). Structural capacity to **additional** 20 mm rut depth : $MISA = Cm \times (DSN_{800})^{3.5} \times 10^{-9}$ in program from Kleyn (1984) assuming Cm = 30, i.e. ≈ OWC

[15] **Excluding surfacing** (i.e. zero taken at top of base as in Kleyn model). MISA calculated as in [14] above

[16] De Beer cemented model **including surfacing**. Structural capacity to **additional** 20 mm rut depth, including seal as in model

[17] BN_{100} **including** surfacing, recalculated manually: $BN_{100} = DSN_{100} / DSN_{800} \times 100$

[18] Mean load equivalency exponent $n = 0{,}044\ (BN_{100})^{1.24}$ (Kleyn and Savage 1982)

[19] BN_{100} **excluding** surfacing, calculated manually: $BN_{100} = (DSN_{125} – DSN_{25}) / (DSN_{825} – DSN_{25}) \times 100$

[20] Assuming linear increase in rut depth to 20 mm from existing rut depth and 1.0M E80 in 2013

[21] Assuming linear increase in rut depth to 20 mm from existing rut depth and DCP mean prediction to additional 20 mm. Mean or 80 %-ile as indicated

[22] **Relevant results for Category C road embolded**, where available

[23] Estimated residual structural capacity beyond range of DCP method bracketed

[24] Laboratory tests on Sections D and E by Freetrans, on Section F by Geoplan

Table 12 Summary of outer wheelpath December 2016 deflection and rut depth and June 2017 DCP test results on Sections G, HA and HB, and estimates of residual capacity

SECTION			**Section G (sulphite lye)**						**LHS: Section HA (8% bitumen emulsion)**						**RHS: Section HB (4% bitumen emulsion)**					
Layer	Test	Units	80% -ile	50% -ile	20% -ile	State (80% -ile)	Condi-tion (80% -ile)	Cap. ME80 (80% Ile)	80% -ile	50% -ile	20% -ile	State (80% -ile)	Condi-tion (80% -ile)	Cap. ME80 (80% -ile)	80% -ile	50% -ile	80% -ile	State (80% -ile)	Condi-tion (80% -ile)	Cap. ME80 (80% -ile)
						[1]	[2]	[1]				[1]	[2]	[1]				[1]	[2]	[1]
	Deflection (L or R)																			
	Y max	µm	**790**	753	716	V. flex.	Severe	**0.25**	788	732	676	V. flex	Severe	0.25	738	731	723	Flex.	Warn.	0.3
Pavement	RoC	m	100	89	**78**	-	Warn.	12	108	87	**65**	-	Warn.	4	115	101	**87**	-	Warn.	15
	BLI	µm	452	418	384	Flex.	Severe	0.3	**542**	411	280	V. flex	Severe	**0.2**	364	355	346	Flex.	Warn.	0.5
	MLI	µm	243	219	194	V. flex.	Severe	0.3	270	213	156	V. flex	Severe	0.25	**299**	276	253	V. flex.	Severe	**0.2**
	LLI	µm	54	47	41	Stiff	Warn.	2.5	50	45	41	Stiff	Warn.	3	52	35	17	Stiff	Warn.	3
[3]	Str. cap. gran. [8}	ME80	**0.25**	0.3	-	-	**-**	**0.25**	**0.2**	0.3	-	-	-	**0.2**	**0.2**	0.25	-	-	-	**0.2**
	Str. cap. stab. [8]	ME80	-	-	-	-	-	-	**0.15**	0.25	-	-	-	**0.15**	**0.15**	0.15	-	-	-	**0.15**
							[1]	[20]				[1]	[20]						[1]	[20]
Pavement	**Rut Depth** (L or R)	mm	**14**	10	-	-	Warn.	**-**	**8**	6	-	Sound	**2.5**	-	**9**	6	-	-	Sound	**-**
[4, 5]	Struct. capacity	ME80	0.4	1.0	-	-	-	**0.4**	1.5	2.3	-	-	-	**1.5**	1.2	2.3	-	-	-	**1.2**
	DCP (L or R)																			
	DN	mm/b	5.3	4.7	4.1				4.3	3.5	2.7				4.7	4.0	3.3			
	CBR [9]	%		58						83						70				
	CBR/UCS [10]	% / kPa		>100						(731)						(630)				
150 mm	FWC [11]	%		5.0						4.0						3.9				
Base [6]	OWC [12]	-		7.6						7.6						8.8				
	FWC/OWC	-		0.66						0.53						0.44				
	DN	mm/b	5.2	4.8	4.4				4.7	4.0	3.3				5.3	4.4	3.5			
Redefined	CBR [9]	%		56						71						63				
upper	CBR/UCS [10]	% / kPa		>100						(638)						(572)				
layer [7]	Thickness	mm		399						751						759				
Pavement						**Mean**	[1]	[21]				**Mean**	[1]	[21]				**Mean**	[1]	[21]
Balance No. (A), Category [13]		%.mm		2770		ABD	**Mean**	**Mean**		5454		PBD	**Mean**	**Mean**		3902		PBD	**Mean**	**Mean**
Balance No. (B) [13]		mm		0						11						3				
DSN_{800} (Incl. seal) [14]		Blows		249			Sound			299			Sound			238			Sound	
Struct. cap. (granular, incl.) [14]		MISA		7.3				3.6		(14)				10		6.2				4.3
DSN_{800} (Excl. seal) [15]		Blows		223			Sound			246			Sound	-		223			Sound	
Struct. cap. (granular, excl.) [15]		MISA		5.0				2.5		7.0				4.9		5.0				3.5
Struct. cap. (cemented, incl.) [16]		MISA		3.6				1.8		(14)				10		4.8				3.4
BN_{100} (Incl. seal) [17]		%		20						35						23				
Load equiv. expt (incl.) [18]		n		1.8						3.2						2.1				
BN_{100} (Excl. seal) [19]		%		11						20						15				
Load equiv. expt (excl.) [18]		n		0.9						1.8						1.3				

NOTES TO TABLE 12

[1] Behaviour state, estimated structural capacity and performance criteria for rut depth and DSN 800 for granular base according to TRH 12 : 1997
[2] Structural condition deflection rating according to Horak (2008) for granular base
[3] Deflections on 05 Dec 2016 by SRT in left outer wheelpath (LO) (*n* = 6 or 7) at 40 kN and 565 kPa. RoC at 200 mm. 80%-ile Parameter predicting lowest capacity embolded
[4] Rut depths in left outer wheelpath on 08 Dec. 2016, *n* usually about 14
[5] DCPs in June. 2017 by Geoplan; Section G (*n* = 4 in LO), HA (*n* = 3 in LO) and HB (*n* = 3 in RO). Processed using WinDCP 5.1Version 10002 by J Briedenhann and author assuming medium traffic, Category C road, moderate, i.e. optimum (OWC) moisture condition and granular base unless otherwise stated
[6] Surfacing (25 mm) removed after inspection of penetration curve during processing (i.e. zero taken at top of base)
[7] Uniform layer below surfacing as redefined by computer. Field water content of Section G subbase 5.0%, SSG 5.6%. MAASHO OWC Section G subbase 6.2%, SSG 5.9%. HA and HB not tested
[8] Minimum 80- and 50%- ile structural capacities shown assuming granular (gran.) for all sections, and stabilized (stab.), i.e. bitumen treated base for Sections H and J and cement treated base where base UCS ≥ 750 kPa. The critical parameter which predicts the lowest capacity is **embolded.** In all cases the RoC capacity estimate shown is for granular base using the 20 %-ile value.
[9] Mean CBR of layer from mean Kleyn (1984) relationship: $CBR = 410\ DN^{-1,27}$ for DN > 2 in program
[10] CBR for uncemented sand from mean sand relationship (Netterberg 2015a) : $CBR = 3\ 000\ DN^{-1,46}$ for DN > 10). UCS for cemented materials = $2\ 900\ DN^{-1.08}$ (Kleyn 1984) in program
[11] Mean field water content of base: Section G (*n* = 3; 4 7 – 5.2%) ; HA and HB (*n* = 1)
[12] Section G Proctor OWC from as-builts in Table 1; MAASHO OWC 6.6% (*n* = 1) in June 2017 by Geoplan. Sections H and HB MAASHO OWC from as-builts in Table 1.
[13] Balance is for **mean** (50 % -ile) including surfacing: AB = averagely balanced, P = poorly, D = deep, I = inverted, structure.
[14] **Including surfacing** (i.e. zero taken at top of surfacing as in TMH6). Structural capacity to **additional** 20 mm rut depth : $MISA = Cm \times (DSN_{800})^{3.5} \times 10^{-9}$ in program from Kleyn (1984) assuming Cm = 30, i.e. ≈ OWC
[15] **Excluding surfacing** (i.e. zero taken at top of base as in Kleyn model). MISA calculated as in [14] above
[16] De Beer cemented model **including surfacing**. Structural capacity to **additional** 20 mm rut depth, including seal as in model
[17] BN_{100} **including** surfacing recalculated manually: $BN_{100} = DSN_{100} / DSN_{800} \times 100$
[18] Mean load equivalency exponent $n = 0{,}044\ (BN_{100})^{1.24}$ (Kleyn and Savage 1982)
[19] BN_{100} **excluding** surfacing calculated manually: $BN_{100} = (DSN_{125} - DSN_{25}) / (DSN_{825} - DSN_{25}) \times 100$
[20] Assuming linear increase in rut depth to 20 mm from existing rut depth and 1.0M E80 in 2013
[21] Assuming linear increase in rut depth to 20 mm from existing rut depth and DCP predication to additional 20 mm. Mean or 80 %-ile as indicated
[22] **Relevant results for Category C road embolded**, where available
[23] Estimated residual structural capacity beyond range of DCP method bracketed
[24] Laboratory tests in June 2017 by Geoplan

Table 13 Summary of outer wheelpath December 2016 deflection and rut depth and June 2017 DCP test results on Sections JA, JB and K, and estimates of residual capacity

SECTION			**LHS: Section JA (4 % tar)**						**RHS : Section JB (8 % tar)**						**Section K (crusher-run)**					
Layer	Test	Units	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)	80% -ile	50% -ile	20% -ile	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)	80% -ile	50% -ile	20% -ile Defl. Min. DCP	State (80% -ile)	Con-dition (80% -ile)	Cap. ME80 (80% -ile)
						[1]	[2]	[1]				[1]	[2]	[1]				[1]	[2]	[1]
	Deflection (L or R)																			
Pavement	Y max	µm	**816**	761	706	V. flex	Severe	**0.2**	800	668	537	V. flex	Severe	0.3	**629**	559	489	Flex.	Warn.	**0.5**
	RoC	m	96	81	65	-	Warn.	5	144	103	62	-	Warn.	4	151	126	102	-	Sound	30
	BLI	µm	**536**	429	323	V. flex	Severe	**0.2**	**565**	369	174	V. flex	Severe	**0.15**	330	283	231	Flex.	Warn.	0.7
	MLI	µm	**305**	236	167	V. flex	Severe	**0.2**	239	182	126	V. flex	Severe	0.3	190	165	140	Flex.	Warn.	0.6
	LLI	µm	53	40	28	Stiff	Warn.	3.0	57	56	55	Stiff	Warn.	3.0	54	49	44	Stiff	Warn.	3.0
	Str. cap. gran. [8]	ME80	**0.2**	0.3	-	-	-	**0.2**	**0.15**	0.5	-	-	-	**0.15**	**0.5**	0.7	-	-	-	**0.5**
[1]	Str. cap. stab. [8]	ME80	**0.15**	0.2	-	-	-	**0.15**	**0.1**	0.3	-	-	-	**0.1**	**-**	**-**	-	-	-	**-**
							[1]	[20]					[1]	[20]					[1]	[20]
Pavement	**Rut Depth** (L or R)	mm	**9**	7	-	-	Sound	**-**	**11**	7	-	-	Warn	**-**	**6**	4	-	-	Sound	**-**
[4, 5]	Struct. capacity	ME80	1.2	1.9	-	-	-	**1.2**	0.8	1.9	-	-	-	**0.8**	2.3	4.0	-	-	-	**2.3**
	DCP (L or R)																			
	DN	mm/b	4.5	3.8	3.1				3.9	2.9	1.9				1.3	1.0	0.7			
	CBR [9]	%		77						104						290				
150 mm	CBR/UCS [10]	% / kPa		(682)						(897)						-				
Base [6]	FWC [11]	%		3.9						4.8						3.4				
	OWC [12]	-		7.5						5.3						5.5				
	FWC/OWC			0.52						0.91						0.8				
	DN	mm/b	4.9	4.2	3.5				4.8	3.8	2.8				4.4	3.6	2.8			
Redefined	CBR [9]	%		66						75						80				
upper	CBR/UCS [10]	% / kPa		600						668						708				
layer [7]	Thickness	mm		535						743						655				
Pavement						**Mean**	[1]	[21]				**Mean**	[1]	[21]				**Mean**	[1]	[21]
Balance No. (A), Category	[13]	%.mm		4746		PBD	**Mean**	**Mean**		5400		PBD	**Mean**	**Mean**		4507		PBD	**Mean**	**Mean**
Balance No. (B)	[13]	mm		0						13						25				
DSN $_{800}$ (Incl. seal)	[14]	Blows		313			Sound			341			Sound			475			Sound	
Struct. cap. (granular, incl.)	[14]	MISA		(16)				10		(22)				(14)		(70)				(56)
DSN $_{800}$ (Excl. seal)	[15]	Blows		267			Sound			291			Sound			434			Sound	
Struct. cap. (granular, excl.)	[15]	MISA		9.3				6.0		12				7.8		(50)				(37)
Struct. cap. (cemented, incl.)	[16]	MISA		8.7				5.7		(18)				12		(15)				12
BN_{100} (Incl. seal)	[17]	%		26						35						34				
Load equiv. expt (incl.)	[18]			2.5						3.7						3.5				
BN_{100} (Excl. seal)	[19]	%		12						24						32				
Load equiv. expt (excl.)	[18]	N		1.0						2.1						3.2				

NOTES TO TABLE 13

[1] Behaviour state, estimated structural capacity and performance criteria for rut depth and DSN 800 for granular base according to TRH 12 : 1997
[2] Structural condition deflection rating according to Horak (2008) for granular base
[3] Deflections on 05 Dec 2016 by SRT in left outer wheelpath (LO) (*n* = 6 or 7) at 40 kN and 565 kPa. RoC at 200 mm. 80%-ile Parameter predicting lowest capacity embolded
[4] Rut depths in left outer wheelpath on 08 Dec. 2016, *n* usually about 14
[5] DCPs in June. 2017 (*n* = 4 in LO) by Geoplan. Processed using WinDCP 5.1 Version 10002 by J Briedenhann and author assuming medium traffic, Category C road, moderate, i.e. optimum (OWC) moisture condition and granular base unless otherwise stated
[6] Surfacing (25 mm) removed after inspection of penetration curve during processing (i.e. zero taken at top of base)
[7] Uniform layer below surfacing as redefined by computer. Field water content of Section K subbase 4.4 %, MAASHO OWC 7.4%; SSG 4.2% (*n* = 1); Sections JA and JB not tested
[8] Minimum 80- and 50%- ile structural capacities shown assuming granular (gran.) for all sections, and stabilized (stab.), i.e. bitumen treated base for Sections H and J and cement treated or granular base where equivalent base UCS ≥ 750 kPa. The critical parameter which predicts the lowest capacity is **embolded.** In all cases the RoC capacity estimate shown is for granular base using the 20 %-ile value
[9] Mean CBR of layer from mean Kleyn (1984) relationship: $CBR = 410\ DN^{-1,27}$ for DN > 2 in program
[10] CBR for uncemented sand from mean sand relationship (Netterberg 2015a) : $CBR = 3\ 000\ DN^{-1,46}$ for DN > 10). UCS for cemented materials = $2\ 900\ DN^{-1.08}$ (Kleyn 1984) in program. UCS for Sections JA an JB base bracketed
[11] Base mean field water content for Sections JA and JB (*n* = 1), Section K (*n* = 3; 2.8 – 4.5%)
[12] Section K base OWC (*n* = 1), Sections JA and JB MAASHO Base OWC from as builts in Table 1; Section K in June 2017 by Geoplan
[13] Balance is for **mean** (50 % -ile) including surfacing: AB = averagely balanced, P = poorly, D = deep, I = inverted, structure.
[14] **Including surfacing** (i.e. zero taken at top of surfacing as in TMH6). Structural capacity to **additional** 20 mm rut depth : $MISA = Cm \times (DSN_{800})^{3.5} \times 10^{-9}$ in program from Kleyn (1984) assuming Cm = 30, i.e. ≈ OWC
[15] **Excluding surfacing** (i.e. zero taken at top of base as in Kleyn model). MISA calculated as in [14] above
[16] De Beer cemented model **including surfacing**. Structural capacity to **additional** 20 mm rut depth, including seal as in model
[17] BN_{100} **including** surfacing recalculated manually: $BN_{100} = DSN_{100} / DSN_{800} \times 100$
[18] Mean load equivalency exponent $n = 0{,}044\ (BN_{100})^{1.24}$ (Kleyn and Savage 1982)
[19] BN_{100} **excluding** surfacing calculated manually: $BN_{100} = (DSN_{125} – DSN_{25}) / (DSN_{825} – DSN_{25}) \times 100$
[20] Assuming linear increase in rut depth to 20 mm from existing rut depth and 1.0M E80 in 2013
[21] Assuming linear increase in rut depth to 20 mm from existing rut depth and DCP predication to additional 20 mm. Mean or 80 %-ile as indicated
[22] **Relevant results for Category C road embolded**, where available
[23] Estimated residual structural capacity beyond range of DCP method bracketed
[24] Laboratory tests in June 2017 by Geoplan

The field water contents were not added to **Tables 6 – 9** as they were either lost by the site laboratory or otherwise unusable. This was the main reason for repeating the DCPs and water contents in June 2017.

The zero point of the DCP test was taken when the top of the cone shoulder was level with the top of the seal, as required in TMH 6 : 1984.

Unfortunately, the testing for Kleyn's (e.g. 1984 etc.) work was carried out after a piece of the seal had been removed, i.e. the zero was taken at the top of the base. This means that a correction has to be applied to his DCP model (including in WinDCP) where the seal is unusually thick, thin, soft or hard.

In such cases this makes a significant difference to the results for the base course and the capacity predictions. Unfortunately, this requirement is little known.

For this reason estimates were made of both the structural capacity and, in the case of the June 2017 work, the load equivalency factors both with and without the current seal.

The BN_{100} balance numbers had to be recalculated manually for the average analyses as it was found that the WinDCP 5.1 calculations were incorrect due to an error in the program. This previously unknown error was conveyed to and has been confirmed by Dr M de Beer of the CSIR.

BN_{100} represents the percentage of the pavement strength (usually to a depth of 800 mm) in the upper 100 mm. However, the balance number B is used for this purpose in the DCP classification and BN_{100} is only used to calculate the load equivalency exponent n.

The De Beer model for lightly cemented pavements was developed with the zero taken at the top of the seal, as specified in TMH 6: 1984. This model does not require input with respect to the moisture regime. Except for the prediction of structural capacity on the first page the output is identical to that of the Kleyn granular base model and therefore has not been included in **Annex G,** but only added to the tables.

One DCP test was carried out on each shoulder at the 36m point on Section G and the upper 150 mm sampled for indicators and water content.

In order to assess the roadbed in as near its natural state as possible the author's double DCP test for collapsing sand was carried out close to the left-hand fence opposite Sections A and K and the results summarised in **Table 14.**

Interpretation of the DSN_{800} results for the condition of a Category C road pavement at the optimum water content according to TRH12 : 1997 is as follows: < 90 : severe/terminal; 90-190 : warning; >190 blows to 800 mm : sound.

Unless stated otherwise, all DCP - derived CBRs and UCSs were calculated according to the Kleyn (1984) equations:

$$CBR = 410\ DN^{-1.27} \qquad \%$$

$$UCS = 2\,900\ DN^{-1.08} \qquad kPa$$

Where DN is the arithmetic average penetration rate for the layer in mm/blow. (Note that WinDCP 5.1 uses weighted averages which can produce somewhat different results.)

Personal discussion with Mr EG Kleyn (2017) indicates that the above relationships should be used with caution on neat, cements; bitumen- and tar- treated Kalahari sands as such materials were not included in the testing used to develop these models. Indeed, it has since been shown that this model greatly underestimates the CBR of neat Kalahari sand – at least when confined in a CBR mould (InfraAfrica et al 2014, Netterberg 2015a, Netterberg and Elsmere 2015, Paige-Green et al 2015).

DCP-derived moduli for each layer are also shown on the DCP analyses but have not been summarised in the tables and will not be discussed in detail.

A set of DCP tests on neat sand from Section A compacted into CBR moulds was carried out both with and without a prior CBR in order to determine whether of not the prior CBR test affected the results. The results are shown in **Annex I** and discussed in **Section 17.3.**

8. Profiling, Sampling and In-situ Densities

Profiling and testing with phenolphthalein and acid to a depth of about 500 mm (i.e. into the subgrade) was carried out in all trial holes.

Although confirmation of the profiles is still awaited from Geoplan, those seen by the author in 2013, 2016 and 2017 and Mr M van der Westhuizen (pers. comm.) in 2017 showed all layers to be very close to those reported by Gregg (1963), i.e. a triple seal (currently very stiff with one reseal) about 25 - 30 mm thick on a 150 mm- thick, primed (on the basis of smell apparently with tar) base, on a 150 mm weakly cemented sand subbase on a 150 mm untreated sand selected subgrade, on a sand fill.

All layers, including those treated with PBFC cement (i.e. the base on Section F and the subbase on all sections) failed to turn red with phenolphthalein, indicating a field pH of less than about 8,4 and that the cement had become carbonated or otherwise inactivated.

The previously cement-treated layers all effervesced strongly with dilute HCl, indicating the presence of carbonate.

The untreated layers (similar to the sand used as raw material for the cement- treated layers) all failed to effervesce with HCl, indicating the absence of significant carbonate and confirming the carbonation of the cement.

The sulphite lye-treated base on Section G failed to turn red with phenolphthalein or to effervesce with acid.

The crusher-run base on Section K effervesced weakly with HCl, indicating that the sand binder used (to make what would now be called a G3) had contained a little carbonate.

Samples for water content and indicator tests were taken of the base at all points and also the subbase and selected subgrade at the middle point and the results of the June 2017 survey) shown in **Annex H.**

The water content of all three upper layers was assessed visually as at about OWC (optimum).

Larger samples were taken of the base from Sections G and F for CBR testing.

Separate small samples were also taken for chemical analysis by PPC and Sappi and mineralogical investigation.

The profiling and chemical testing confirmed that the sampling (and DCP) sites were located on the correct sections and that the nature and thickness of all layers were in accordance with the as-built record in **Table 1**.

A large bulk sample of neat sand base from Section A was taken for a full stabilization design and additional testing of the neat sand, including possible Sanral and Texas triaxial and other tests, and as a reference sample. Unfortunately most of that intended for the additional testing was inadvertently discarded by Geoplan.

Small blocks of cement-treated base, some exhibiting significant dry strength of about 10 MPa (as estimated with a Schmidt hammer) exposed along the left edge of Sections B – E were sampled for special testing, which has still to be completed.

In-situ nuclear density depth measurements of the base and subbase were taken at three points in the left outer wheelpath of Sections F, G and K using a Troxler 3430 instrument, in the direct transmission mode. Only the base course results have been received to date and are shown in **Annex H**. Results for the Section A neat sand base are shown in Netterberg (2015a) and Netterberg and Elsmere (2015) and indicated a relative compaction of about 100% MAASHO in the outer wheelpaths.

As the nuclear gauge could only determine water content in the backscatter mode (i.e. the upper approximately 200 mm) the laboratory water contents have been used to calculate all the dry densities, as in any event always required by COLTO : 1998.

In addition, the density from 0 – 300 mm was also determined at the two double DCP sites and the results summarised in **Table 14**.

9. Suction and Temperature

Thermo-hygrometer measurements in December 2016 in the upper base of the left outer wheelpath of Sections G and K using a Majortech MT667 instrument showed relative humidities of 89 – 90% at 35 – 42 °C, indicating a total suction (moisture tension) of about 8 – 9 MPa, equivalent to a pF of about 5 according to the standard relationships provided by the Road Research Laboratory (1952).

Such high suctions indicate a substantial potential undisturbed strength in the unsaturated state and are probably the key to the surprisingly good performance of the neat sand base.

10. Roadbed

The roadbed under all the sections consists of about 1,0 – 1.5m of Kalahari sand as previously described, although from the DCP testing it appears to thin out under Sections F – K.

The results of a double DCP and in-situ density test next to the fence in order to approximate the natural condition opposite Sections A and K confirm the suspected potentially collapsible nature of this soil (**Table 14**). This is particularly well-shown by the dramatic loss of strength to a CBR of only 2 when wetted, which also confirmed its high permeability.

This test is not new and has been used by the author and others since about 1970; and by Kleyn et al (1982) since at least about 1980. However, as it appears to be little known and used, and as a formal method never appears to have been previously published it has been written up as **Annex J.**

As the existing road was constructed over the old gravel road it does not appear to have been affected by roadbed collapse settlement. However, deep compaction should be considered for any new alignment and widening on such soil, especially if the drainage is suspect.

Table 14 Results of double DCP and density tests on roadbed

Position [1]		**A35L**		**K50L**	
Test/ Parameter	**Units**	**DCP**		**DCP**	
		Dry	**Wet [4]**	**Dry**	**Wet [4]**
DSN $_{300}$	Blows	25	5	17	6
Mean DN	mm/blow	12	60	18	50
Mean CBR	%	18	2	10	3
DSN $_{600}$	Blows	38	9	53	24
Mean DN	mm/blow	16	67	11	25
Mean CBR	%	12	2	19	7
DSN $_{800}$	Blows	45	13	71	41
Mean DN	mm/blow	18	62	11	20
Mean CBR	%	10	2	19	9
0-300 mm [2]					
Dry density	kg/m^3	1 544	-	1 598	-
Water content	%	7.8	15.8	2.5	22.0
Compaction [3]	%	85	-	88	-

NOTES

[1] Close to LHS fence approximately opposite SV 35 and 50 on Sections A and K respectively

[2] Troxler 3430 nuclear gauge with laboratory water contents

[3] Assuming a MDD of 1 809 kg/m^3

[4] Approximately 20 litres water used at each site

11. Maintenance

Only routine maintenance in the form of mostly edge patching had been carried out since the last visit in October 2013.

On Section A and onwards towards Hoopstad the sand shoulders had been graded over the edges and compacted as a temporary solution to the edge breaking problem. This prevented a later assessment of the edge breaking as in 2016.

The whole road had been handed over to the Contractor for rehabilitation, starting from the Bultfontein end.

12. Traffic

The maximum legally permissible axle loads on South African roads were raised from 1996. For example, the single dual wheel load was raised from 8 200 to 9 000 kg (88 kN/axle).

This is equivalent to a 45% increase in additional legal E80 for such an axle assuming a relative damage exponent of 4.0, and increased the typical E80 / vehicle for a legally loaded 7-axle truck from 3.82 to 6.01 (Jordaan 2013).

No traffic counts later than the June 2014 one of 30 – 35 six- and seven- axle trucks per day in each direction are available. However, on site it was clear that this count was still valid and may well have increased.

Although the road was only a low-volume road it was therefore not a lightly trafficked one.

Using the 2014 count, it is estimated that the sections have carried an additional approximately 1,0M E80 in both directions from the 2,0M in 2013, i.e. a cumulative total of about 1,5M in each direction to date assuming a 50 : 50 split.

Whilst no split counts are available, the poorer condition of the LHS (Bultfontein-bound) lane suggested a likely 60 :40 approximate split.

On this basis the Bultfontein-bound lane may have carried about 1.8M and the Hoopstad lane about 1.2M E80.

As is usual with all such estimates made without the benefit of measured axle weights and split counts, their accuracy should probably only be regarded as about ± 30%.

Whilst there are no weighbridges on this road, discussion with a local farmer indicated that overloading of trucks was minimal after they realised what damage it was doing to "their" road.

Although the cold tyre pressures used were said to be about 800 kPa, hot tyre pressures of over 1 000 kPa have been measured elsewhere.

13. Alignment, Cross-sections and Drainage

Cross-sections at intervals of 20 m were taken over the whole road by Propercon for rehabilitation design purposes and those relevant to the test sections as well as GPS coordinates and altitudes at the start and end of each section and at other salient points were supplied.

These coordinates and elevations have been added to the road log in **Annex A** and some to the layout plan in **Figure 1**.

The alignment of both sets of sections was straight, approximately NNE – SSE, and both were almost level.

Sections A to E rose from an altitude on the centreline of 1.284.19m at SV 0 m on Section A to 1284.79 m at SV 64 m on Section E, a rise of 600 mm in 428 m, i.e. about 0,14%.

Sections F to K similarly rose from an altitude of 1288.03 to 1289.36 m, a rise of 1.33 m in 348 m, i.e. about 0.38%.

The road reserve was 30 m in width and was entirely sand, covered with wild grass.

The sand shoulders were a nominal 2 m in width and were also covered with grass. However, from the cross-sections the effective width varied between about 1.0 and 1.90 m, averaging about 1.5 m.

The side drains were mostly wide and shallow with the inverts varying between about 1.5 and 7 m from the edge of the seal and about 0.3 - 0.4 m below the centreline on Sections A – E and about 0 - 0,2 m on Sections F to K and thus generally far less than the 600 mm considered desirable at that time.

From the fence the natural ground level appeared to fall about 0.5 m from right to left, but this varied due to the build-up of windblown sand, especially along the left fence, and in places was higher than on the right.

The drainage had clearly been compromised since construction due to the sand movement and was poorer on Sections F - K than on A - E, and in a few places such as on Section G the shoulder breakpoint was slightly higher than the edge of the seal – and even the centreline (see photos in **Annex C**).

According to local information the sand was free-draining and water did not stand along the sides of the road. After a short rain shower in December 2016 it was observed to stand for only about one hour along the edges of the seal on Sections F – K and in the outer wheelpath on Sections F - HB (see photos in **Annex C**), after which it was dry.

In short, although both the shape of the cross-section and the drainage had apparently been compromised, this negative factor was probably at least partly cancelled by the free-draining nature of the sandy pavement layers, shoulders, fill and roadbed.

There were only two culverts on the test sections : the one on Section A was open, but that on Section F was blocked.

14. Climate and Weather

The mean annual rainfall at the site is about 520 mm and for TRH 4 : 1996 pavement design purposes the experiment lies in a dry, borderline moderate macroclimate region. For a detailed climatic discussion see Netterberg (2015).

Table 15 shows the rainfall record for the area compiled from information kindly supplied by the owner of the farm Vesuvius (Mr D Naude) on which the sections lie.

The December 2016 work was thus carried out during the early part and the June 2017 work at the end of the rainy season.

The total annual rainfall for 2016 of 539 mm was close to the long-term average of 520 mm, but the October-November rainfall was above average.

The January-February 2017 rainfall was far greater than normal, but that of March and especially April below normal and only 9 mm was recorded during May.

Although the road may therefore have been expected to have dried out to slightly below average conditions during these three months before the June 2017 work, the above - OWC water contents showed that it did not (see **Discussion in Section 15**). The December 2016 visuals, rut depths, DCPs and deflections were carried out after two months of above average rainfall.

The 2016 work was delayed somewhat by the inclement weather on site – the heat (usually about 35 °C in the shade), and high winds for most of the day, followed by either a sandstorm or rain in the afternoon.

Table 15 Mean rainfall near the site for the period 1997 to 2013 and actual rainfall from 2012 to 2017 [1]

Statistic	Units	Jan	Feb	Mar [2]	Apr [3]	May	June	July	Aug	Sept	Oct	Nov	Dec [4]	Total
2017 [5]	mm	161	184	16	30	9	0	2						
2016	mm	90	23	25	127	44	19	14	0	0	44	87	66	539
2015	mm	46	74	78	18	36	21	4	0	28	14	0	23	342
2014	mm	50	144	62	4	16	0	0	5	0	22	163	120	586
2013	mm	25	25	98	54	0	0	3	2	0	35	38	75	355
2012	mm	41	49	43	14	0	14	7	3	15	21	59	66	332
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 291m (courtesy Mr BD Naudé, 2014, 2017, pers. comm., with statistics by author)
[2] 52 mm On or before 11 March 2014
[3] 4 mm On 01 April 2014
[4] 5 mm On 23 November, 25 mm On 08/09 December 2016
[5] For comparison, the following were recorded at the farm house on the farm Windehondenpan 217 on the opposite side of the road: Jan: 169 mm, Feb: 153 mm, Mar: nil, Apr: 19 mm, May: nil, June: nil, July: nil (courtesy Mr. A Ferreira, 2017 pers. comm.)

15. Mentoring

Mentoring in rut depth measurement, DCP and nuclear gauge work, profiling, sampling and bag labelling, testing for collapsing sand, and testing with phenolphthalein (for the presence of active cement) and acid (for the presence of carbonated cement), and suction was provided by Dr Netterberg.

16. Discussion of Performance Indicators and Performance

Both lanes of all sections were assessed visually – separately where different.

The 2013 DCP survey of Sections A – E was carried out in both lanes.

In the case of the 2016 – 2017 work DCP and deflection testing were carried out only in the outer wheelpath of the worse (the left – Bultfontein-bound) lane, as this showed slightly greater traffic-associated damage, and in the outer wheelpath of the two Sections HB and JB in the right lane as they were only in this lane. A few tests were also carried out in the midlane in order to simulate the effect of a sealed shoulder.

The experiment has been evaluated essentially as a TRH 4 : 1996 Category C lightly trafficked rural road with an approximate design reliability of 80 % intended for up to 3M E80 / lane over a 10 to 20 year design life. At the end of this period not more than 20 % of the length of the road should be in a terminal (i.e. "failed") condition requiring rehabilitation.

The rut depths have nevertheless been processed to include 90 and 50 percentiles more appropriate to Category B and D designs if required. However, the capacity estimates are only available as 50% -iles (averages) for all the sections.

Both the visual assessment and the rut depth survey showed that rutting was mostly in the sound (<10 mm) or at most the early warning (10 – 15 mm) range and was therefore not a problem. The condition of the sections will therefore be discussed only in terms of cracking, edge breaking, patching, deflections and DCP results, using the TRH 12 distress limits shown.

The condition of all the sections mainly as at December 2016 after about 55 years and probably about 1.8M E80 in the left (Bultfontein) and 1.2M E80 in the right (Hoopstad) lane is further summarised in **Table 16.** The later data shown in Tables **10- 13** have mostly been used in preference to the earlier data in **Tables 6 - 9**.

All of the sections except K (crusher-run) with 0% had **failed** in terms of **block plus transverse cracking** (up to 50% of length permitted).

However, such cracking is only structurally serious when it develops secondary, traffic-associated cracking such as crocodile cracking, which was most noticeable in the left inner wheelpath.

All except K with 0%, D (10% OPC) with 10%, JB (**RHS** 8 % tar) with 10% and HB (**RHS** 4 % emulsion) with 20 % had also **failed** with respect to the 25% limit for the far more serious **crocodile cracking**. In addition, Sections B (3% OPC), C (5% OPC) and HA (8% emulsion) were also showing signs of pumping in such cracking.

Apart from crocodile cracking the only other serious distress was **edge breaking** and the associated **cracking** and necessary **edge patching** sometimes extending into the outer wheelpath, which essentially extended to the very edge of the seal.

In this respect only Sections A (neat), E (5% PBFC) and G (2% sulphite lye) exceeded the **50% limit**, with F (5% PBFC) at a marginal 48%.

The reason for the duplication of the 5% PBFC section was not stated, but it may have been to act as a control because the two sets of sections were about 2 km apart. In any event, the condition of the second section (F) was significantly better than E and in some respects better than Section C with 5% OPC. Reasons for this may have included the greater strength of both the seal (DN of 0,8 and E-modulus of 1 400 MPa vs 3.1 and 330 MPa) and the base (DN of 2.3 and a UCS of 1 200 KPa vs 6.0 and 400 kPa, respectively), as well as a much higher DSN_{800}.

In terms of **deflection,** all the sections except K and HB (**RHS**) were in a **severe** condition with these two in a **warning** condition.

The state of **flexibility** and **condition** of the individual layers as indicated by the various deflection bowl parameters can be summarised as follows:

Ymax (Pavement) :
Very flexible state and **severe** condition on all sections except **flexible** and **warning** on HB (RHS) and K, with K (crusher-run), the lowest at 629 µm and A (neat sand) the highest at 900 mm.

Table 16 Summary of relative pavement TMH 9 condition, base course strength and estimates of residual structural capacity

Section		TMH 9 Visual Rating [1]	Cracking {1}		Edge Patching [1,4] E_p	Deflection **(80%-ile)** [2, 5, 6, 7]		Rutting **(80%-ile)** [2, 7]		DCP (**Mean**) [2, 7]					Overall Relative Rating
			Block + Trans-verse	Crocodile						Base (150mm)			Pavement [5, 8]		
No.	Base/ Stabilizer	[3]	Extent [3]	Extent [3, 4, 9]	% length + √area	Con-dition	Cap-acity	Con-dition	Cap-acity [10]	DN	CBR [11]	UCS [11, 12]	Con-Dition (DSN_{800})	Cap. acity [13]	[14]
			% of length		-	-	ME80	-	ME80	mm/b	%	kPa	-	MISA	
															Relative
A	Sand	Poor	80	>50	100	Severe	0.15	Warning	1.0	6.8	36	(349)	Warning	2.8	12
B	3% OPC	Fair	100	100+P	30	Severe	0.15	Sound	1.5	5.2	51	(475)	Sound	6.2	7
C	5% OPC	Fair	100	60+P	30	Severe	0.15	Warning	1.0	5.4	48	(455)	Warning	1.6	9
D	10% OPC	Fair	100	10	4	Severe	0.25	Sound	2.3	2.7	116	984	Sound	5.8*	2
E	5% PBFC	Poor	100	>50	64	Severe	0.2	Sound	1.5	6.0	42	(404)	Sound	3.8	10
F	5% PBFC	Fair	100	100	48	Severe	0.2	Sound	1.2	2.3	147	1210	Sound	11*	8
G	2% Sulphite lye	Fair	100	40	100	Severe	0.25	Warning	0.4	4.7	58	(545)	Sound	5.0	11
HA	8% emulsion	Poor	100	70+P	6	Severe	0.15	Sound	1.5	3.5	83	(731)	Sound	7.0*	5
HB	**RHS**:4% emulsion	Fair	90	20	22	Warning	(0.2)	Sound	1.2	4.0	70	(630)	Sound	5.0*	(3)
JA	4% tar	Poor	100	70	26	Severe	0.2	Sound	1.2	3.8	77	(682)	Sound	9.3*	6
JB	**RHS**: 8% tar	Fair	100	10	3	Severe	0.15	Warning	0.8	2.9	104	(897)	Sound	12*	(4)
K	Crusher-run	V. good	0	0	3	Warning	0.5	Sound	2.3	1.0	290	(2300)	Sound	(50*)	1
Criteria [15]		-	TRH12	TRH12	TRH12	Horak	-	TRH12	-	This	This	This	TRH12	-	All
Sound		-	<30	<15	<30	BLI <200	-	<10 mm	-	<3.6?	>80?	>730?	>190	-	-
Warning		-	30 - <50	15 - <25	30- <50	200-<400	-	10-<20	-	3.6-<70	>35-80	>350-730	>90 - 190	-	-
Severe		-	≥ 50	≥25	≥50	≥400	-	≥20	-	≥7.0?	≥35?	≤350?	<≤90	-	-

NOTES

[1] From Tables 3 – 5 [2] From Tables 10 – 13 [3] Both lanes except H and J [4] Mostly LHS (the worse) except HB and JB [5] Granular base assumed because of high flexibility . BLI behaviour state : Flexible : Sections D, F, G, HB, K; Very flexible : Sections A, B, C, E, HA, JA, JB [6] HB based on maximum of only two points [7] Outer wheelpath, LHS on all except RHS on HB and JB [8] Category C road, optimum moisture condition, granular base [9] With pumping (P) on Sections B, C and HA [10] To **80%-ile** 20 mm **total** rut depth [11] Kleyn DN-CBR-UCS relationship; CBRs all > 100 using Netterberg Hoopstad sand relationship [12] UCS on Sections H and J bracketed because of uncertainty regarding validity of Kleyn DN-UCS relationship on bitumen- and tar- bound sands. Others bracketed are "equivalent" UCS [13] To **mean** 20mm **additional** rut depth with seal removed in analysis. * = Capacity prediction probably unreliable because of poor pavement balance. All other sections had average balance. [14] RHS sections bracketed because of probable lower traffic [15] Category C road

BLI (Surfacing and Base) :
Very flexible and **severe** on all sections except **flexible** on D, F, G (marginal), HB (RHS) and K, and **warning** on HB (RHS) and K, with K at 330 µm the lowest and HB (8% tar, RHS) at 565 µm the highest.

MLI (Subbase) :
Very **flexible** and **severe** on all section except **flexible** and **warning** on K with K the lowest at 190 µm and JA (4% tar) the highest at 305 µm.

LLI (SSG, Fill & Roadbed :
Stiff and warning on all sections except **sound** on B and C, with B (3% OPC) the lowest at 38 µm and D (10% OPC) the highest at 63µm.

The deflection-predicted 80%-ile residual structural capacity of K was about 0.5 M E80 with all the others at about 0.2 M, which seems realistic. (This means that 20% of each section had higher deflections and therefore lower capacities than these.) The critical parameter was usually the maximum surface deflection (Ymax) and/or the BLI, which has been embolded in **Tables 10 - 13**.

In terms of **rutting,** all were in a **warning or sound** condition with a predicted 80%-ile residual capacity in excess of 1M E80 and K about 3M. This of course only applies to rut depth, and failure in terms of other criteria had already occurred on most sections.

As no base failures or significant rutting or potholing were present the mean **strength of all the bases** as measured with the DCP at that time (mostly June 2017) must be regarded as adequate. **On the sand sections** these ranged from a DN of 6.8 on the neat sand Section A, equivalent to a DCP CBR of only 36 according to Kleyn's relationship – but over 100 according to the DCP – CBR relationship for this sand developed by Netterberg (2015a), with Section G (sulphite lye) being somewhat better with a DN of 4.7, to 2.5 for the 10 % OPC sand section D (equivalent to a Kleyn CBR of about 128 and UCS of 1 070 kPa).

With mean DCP CBRs of 42-51 the Sections B (3% OPC), C (5% OPC) and E (5% PBFC) base courses would all have been in an **equivalent granular state** and were not greatly stronger than the 36 of the neat sand Section A and slightly less than the 58 of the sulphite lye Section G.

Inspection of the mean base strengths summarised in **Table 16** and shown in the DCP layer strength diagrams in **Annex G3** shows that the base of all the sections except K failed to meet the maximum DN requirement of 2.0 (equivalent to a DCP CBR of 170) for medium traffic (0.2 – 0.8 MISA) in WinDCP 5.1. Only, D, F, and K met the DN requirement of 2.8 (CBR 110) for light traffic (0.1 – 0.3 MISA and only D, F, HA, HB, JA, JB and K the DN of 4 for 0.03 – 1.0 MISA of the then Transvaal Provincial Administration (TPA) (1994). Sections A, B, C, E, G would only have satisfied their DN requirement for their very lightest design for up to 0.01 MISA of 7 (CBR 35).

Yet none of these bases had apparently ever failed except along the edges.

Most of the subbases except K (and marginally B, F, HA, HB, JA and JB) also did not meet the medium traffic requirements, and only the selected and lower layers all (greatly) exceeded the requirements.

A maximum mean base course DN of about 7.0 (equivalent to a Kleyn CBR of only 35 but a Hoopstad sand CBR of > 100) seems indicated for a neat sand base both from the present and the previous (Netterberg 2015a, Netterberg and Elsmere 2015) study. These should of course not necessarily be a design criterion, but show what can be tolerated over the long term. However, by chance they coincide with those required by the TPA (1994) for base course – but only for up to 0.01 MISA.

The strength of a base course under a thin seal should not of course be allowed to fall below the maximum tyre pressure of the vehicles travelling on it. In this respect significant protection may have been afforded by the relatively thick (25 - 30 mm) triple seal plus one reseal, which usually took about 20 blows to penetrate the 25 mm selected for the analysis and usually had a DCP-derived E-modulus of about 500 MPa on Section A – E, and 30 - 50 blows on Sections F - K, with a modulus of mostly more than 1 000 MPa.

Even in crocodile-cracked areas the DCP was always sited on uncracked surfacing and not exactly on a crack.

All of the sections except A (neat) and C (5% OPC) were in a **sound** condition in terms of the **DSN_{800}** minimum of 190 blows for a M2 (moderate, OWC moisture regime), which were in a **warning** condition (i.e. 90 - 190 blows).

However, these DSN_{800}s and the mean **DSN_{800} capacity predictions** are all unrealistically high because of the excessive contribution of the strong subgrade and roadbed to this count. This was especially the case from Section F onwards where the DSN from 450 to 800 mm averaged about 120 blows (i.e. a DN of 2.9 and an equivalent in-situ Kleyn CBR of 105). It is therefore **recommended** that they all be re-analysed on a DSN_{450} mm basis in order to normalise them and remove this effect and also by using pavement component analysis (Jordaan 2013). However, both of these are beyond the scope of this report.

It must also be borne in mind that DCP capacity predictions using DSN_{800} are only accurate for well-balanced (WB) pavements, less so for averagely balanced (AB) pavements and unreliable for those of poor balance (PB). The latter are starred in **Table 16.**

One reason for the large differences in capacity prediction is due to the different terminal criteria applied. The deflection criteria were not stated in TRH 12 and apparently varied among the sources used. That for rutting is to an **80%-ile total** rut depth of 20 mm from that measured on the sections and that for the DCP DSN method to a mean **additional** 20 mm for that measured. Corrections for the measured rut were made in **Table 10 - 13** and are thus more realistic but the former are used here in order to avoid combining another variable. The **purpose** of **Table 16** is actually to summarise the **measured condition** of the sections rather than to predict their residual capacity – this is only used as one such measure.

A suggested relative **overall rating** from best to worst is as follows, with only marginal and/or subjective differences between some.

1	K	:	crusher-run
2	D	:	10% OPC
3	HB	:	4% emulsion (but RHS)
4	JB	:	8% tar (but RHS)
5	HA	:	8% emulsion
6	JA	:	4% tar
7	B	:	3% OPC
8	F	:	5% PBFC
9	C	:	5% OPC
10	E	:	5% PBFC
11	G	:	2% sulphite lye
12	A	:	Neat sand

However, it is **concluded** that in the case of all these sections the absence of significant rutting (and shear failures) means that the **base was sound** and that the crocodile cracking was due to the fatigue cracking of the old, now relatively brittle, surfacing on a relatively flexible pavement (TMH 9 : 1992, p. 35).

In **Table 17** the effect of the **position of the zero point of the DCP** on the **residual structural capacity prediction and load equivalency exponent** is summarised from Tables 10 – 13.

Table 17 Effect of zero point of DCP on estimates of mean residual structural capacity and load equivalency exponent

Section	Units	A	B	C	D	E	F	G	HA	HB	JA	JB	K
Base		Neat	3% OPC	5% OPC	10% OPC	5% PBFC	5% PBFC	2% Sul. lye	8% Emul.	4% Emul.	4% Tar	8% Tar	Crusher -run
Bal. Cat.	-	ABD	ABI	ABD	PBD	ABI	PBD	ABD	PBD	PBD	PBD	PBD	PBD
Incl.	MISA	2.4	5.4	1.3	5.4	3.2	1.0	3.6	10	4.3	10	(14)	(56)
Excl.	MISA	1.8	4.7	1.0	4.6	2.9	7.2	2.5	4.9	3.5	6.0	7.8	(37)
Base DN	mm/bl	6.8	5.2	5.4	2.7	6.0	2.3	4.7	3.5	4.0	3.8	2.9	1.0
Incl.	n	1.4	1.2	1.7	2.7	1.1	3.2	1.8	3.2	2.1	2.5	3.7	3.5
Excl.	n	0.6	0.7	1.1	3.0	0.7	2.1	0.9	1.8	1.3	1.0	2.1	3.2

The data show that, as expected, the capacities excluding the seal are invariably lower – often substantially so – than those with the seal. This is simply due to the subtraction from the DSN_{800} of the 20 – 30 blows usually necessary to penetrate the seal (plus the number of blows from 800 mm to an extrapolated 825 mm).

As the seal was removed in the work carried out to develop the Kleyn model this is the correct procedure to use.

The pavement load sensitivity as indicated by the load equivalency exponent (LEE or n) is also affected by whether the zero point of the DCP is taken at the top of the surfacing or at the top of the base.

This effect is also substantial and with one exception (Section D with 10% OPC) the exponent is invariably lower – often by as much as half – when the seal is excluded, as in the Kleyn model.

In this case the validity of exponents of less than unity is questionable and those determined including the seal appear more reasonable. They are mostly relatively low, tend to increase with the strength of the base course and are in general agreement with those shown in TRH 4 : 1996 Table 8, including the post-cracked phase of cemented pavements.

The low exponents – especially those without the seal – of the sections in their present condition may provide another plausible reason for their good performance especially that of the neat sand base of Section A : it is relatively insensitive to the axle load.

The reason for the exponent for Section D being higher without the seal appears to be that the surfacing and upper pavement were of a uniform high strength with a DN of 2.4, as shown by the computer-redefined pavement.

Whilst this model has thus yielded plausible LEEs it must be used with caution as it was only a suggestion (Mr EG Kleyn 2014, pers. comm.) and its use should not be encouraged (Jordaan 1989) as it remains unproven (Dr GJ Jordaan 2013, pers. comm.). Indeed, work by De Beer (1990) suggested that deep balanced pavements with a BN_{100} of less than about 40 should have an LEE of about 1.5 and those shallow pavements with a higher BN an LEE of anything between about 1.5 and 4.5.

In short, all these **bases** were still there after 55 years and about 1.5M E80 / lane and no **base** appears to have ever failed except along the edges, as shown by the edge breaking and edge patching. It is therefore **concluded** that any of these designs can be used successfully provided that the seal is sufficiently wide and flexible to accommodate the expected traffic – and that it remains so. The final decision on which to use can probably be made on economic grounds.

A more detailed comparison of the left versus the right lane on each section may enable traffic limitations to be more closely defined.

In the interim it is suggested that a **neat or sulphite lye**-treated Kalahari sand base be limited to 0.3M E80 over 20 years, a cement-, bitumen- or tar- treated Kalahari sand base to 1.0M, and that bitumen-bound bases be designed according to existing criteria. Although this limitation is less than the 3M for a C3/C4 pavement permitted for a Category C road by TRH4 : 1996, a G6 or now even a G5 raw material would be required.

The very good performance and residual capacity of the **crusher-run section** even over what is now a totally carbonated, but still weakly cemented subbase with a DCP UCS of 860 kPa shows again the outstanding long-term value and return of such a design if the initial cost can be afforded.

Analyses by Sappi of three base course samples taken from Section G during the 2016 work showed the presence of only about 0.03% **residual sulphite lye** – i.e. about 1.5% of the original 2%. As a similar amount was found in the untreated subbase and selected subgrade, and as this product is relatively soluble, it is presumed to have leached downwards and perhaps sideways over the years. As this section (G) was only marginally better than the neat sand section (A) sulphite lye stabilization of such a sand base is probably not worthwhile unless close to the source of the sulphite lye and far from a source of cement.

However, the higher MDD, lower OWC and far higher as-built, dry-cured strength of 3.5 MPa – similar to the 3.6 MPa with 6.5% PBFC of section E – of the sulphite lye section than the 1.25 MPa of the neat sand **(Table 1)**, indicates that it both acted as a compaction aid and would have imparted significant cementation whilst relatively dry – presumably until it was leached out. For example, sulphite lye has been used for base course stabilization in the Sahara (Remillon and Narbonne 1960, **in** Fossberg 1966).

It is unfortunate that only the very short – and very long-term condition – nearly three times the normal design life of 20 years – is known.

Caution must be used in the application of the findings of this experiment and Fossberg (1966) as the sulphite lye currently available (Sappi/tugela Mill 2016) differs somewhat from that used.

It is therefore **recommended** that the compaction characteristics and CBR with the current product at different water contents after drying back and also soaking in comparison with neat material should be studied.

The paste electrical conductivity (EC) of the sulphite-lye- treated base course samples was all about 0.10 S/m, which was significantly higher than the 0.04 S/m of the subbase and the 0.03 S/m of the selected subgrade and a sample of the neat sand from Section A and higher than the equivalent EC of 0.02 previously found in all the layers of Section A (Netterberg 2015a, Table 17). The mean pH of the base course samples was 7.0 and the single samples of subbase 7.5, selected 7.8 and the Section

A sand 7.4. All of these were low in comparison with the mostly 8.0 – 8.5 previously found in all layers of Section A and in further samples of the Section A neat sand base shown later in **Table 17**.

These differences probably reflect the residual effect of the sulphite lye which had a pH of 3.2 and, probably, a high EC.

Both the December 2016 and June 2017 work were carried out after a period of about three months of below average rainfall.

The field water contents (FWC) for the December work were unfortunately either lost by the site laboratory or were otherwise unusable. However, the average base course FWC/OWC ratio in June 2017 for Section A was 2.0 (**Table 10**), Section F 0.74 (**Table 11**), Section G 0.76 (**Table 12**) and Section K 0.8 (**Table 13**) – all above MAASHO OWC. Whilst the Section A results are questionable (the base was assessed visually at only about OWC) the above average water contents indicate that at least the 2017 work was not carried out under unusually dry conditions and the DCP strengths and predictions of structural capacity are therefore conservative.

However, if such wet conditions are to be the norm (which is unlikely in this area) the structural capacity predictions should probably be halved to allow for the Kleyn DCP moisture condition factor of 14 instead of the 30 for optimum conditions used. Conversely, under dry conditions the Kleyn factor of 64 indicates that the capacity can be doubled (Kleyn and Van Zyl 1988, WinDCP5.1).

Visual observations confirmed by the limited sampling and DCPs confirmed that the **shoulders** on both sides of all sections were only sand similar to that used for the neat sand base on Section A.

The upper 150 mm of the left-hand shoulder on Section G in June 2017 had a DN of 5,7 mm/blow, at a water content of 5.1%, equivalent to a in-situ CBR of 45, and a DSN_{450} of 70 blows.

The right-hand shoulder had a DN of 8,5 mm/blow in the upper 150mm at a water content of 5.0%, equivalent to an in-situ CBR of 27, and a DSN_{450} of 129 blows.

No shear failures were seen anywhere on the shoulders during the visits made during 2013, 2016 and 2017 and only the upper part was being eroded by the large trucks now running on it.

It is therefore **concluded** that, although a sealed shoulder would be preferred, such a grassed sand shoulder is viable provided that the seal is wide enough to accommodate the expected traffic.

Examination of the deflections taken in the midlane on Sections A - E in December 2016 (**Annex F**) showed that the average deflection parameters were usually lower – and the BLI invariably so – than those in the outer wheelpath, indicating that such designs would benefit from a sealed shoulder.

However, the opposite was the case with Sections F – HB and only on Sections JA and JB was the BLI significantly lower. The reason for this is not clear, but may be associated with the poorer drainage and thinner sand cover associated with the second set of sections, which were about 2 km apart.

The few DCPs taken in the midlane on Sections A - F in Oct./Nov. 2013 and Dec. 2016 all yielded a higher DN in the base and a lower DSN_{800} suggesting that they would not benefit from a sealed shoulder. However, only one such test was carried out on each section in contrast to the three deflections.

Moduli as well as strengths are required for modern pavement design. Whilst moduli can be estimated from the deflections and the DCP tests and are shown on the DCP analysis sheets, in view of its huge potential as an inexpensive base course it is **recommended** that both shear and repeated load triaxial testing for the direct determination of its residual modulus according to the Sanral protocol be carried out on the neat sand used for Section A in order to provide a better understanding of its behaviour. (Such sand has not previously been subject to such testing.)

17. Material Test Results

17.1 Differences between South African and British and American laboratory test methods

Whilst internationally accepted tests such as for PI and CBR are used in South Africa and were used for this project, certain details of the methods differ significantly from those of other countries which use the British (BS), or American (AASHTO or ASTM) test methods.

The methods used in South Africa and some other southern African countries are those of TMH 1: 1986, now being superseded by the similar SANS (South African National Standard) series published by the South African Bureau of Standards (SABS).

The most significant differences between the South African and the BS and AASHTO methods and their implications have been discussed by Pinard and Netterberg (2017) and will only be briefly outlined here. (The AASHTO and ASTM methods are similar, although not always exactly the same.

Soil Preparation: The usual South African TMH 1 A1(a) (1986) grading and preparation method for the testing of soil constants is a wet method, but **requires** drying of the soil fines passing 425 µm (P425) at 105 – 110 ^{o}C whereas the BS 1377-2 (1990) and AASHTO T 87-86 (1996) require air- or oven- drying at <50 or 60 ^{o}C. If all other factors were equal the wet sieve grading used by the TMH and BS methods should both yield similar but finer results than the dry sieving used by AASHTO, but a TMH 1 PI might be significantly less than a BS or AASHTO PI on some soils.

Soil constants: The TMH 1, AASHTO and ASTM methods all use a similar Casagrande cup with a harder base than that used in the BS method. This means that, all other factors being equal, the LL – and therefore also the PI – using the BS cup (or the equivalent BS cone penetrometer) are 4 units higher than those determined using the TMH 1, AASHTO or ASTM device (Sampson and Netterberg 1984).

Compensation for oversize: The TMH 1 method **requires** compensation for oversize greater than 20 mm for the determination of both compaction characteristics and CBR, whereas AASHTO **allows** it, but the BS does not, and only the fraction passing 20 mm fraction is used.

In general, CBRs on the passing 20 mm fraction are lower than those on the same material which has been compensated. These differences are only relevant for this project in the case of the samples of crusher-run of Section K and possibly some of the cemented sand bases.

Compactive effort: The heavy compactive effort used in South Africa for both the compaction characteristics and the CBR is the old MAASHO of 2 415 kJ/m^3 as against the 2 670 kJ/m^3 of the BS 1377 and 2 700 kJ/m^3 of the AASHTO T180 methods.

This difference of about 11 or 12% can be expected to yield higher MDDs and CBRs than the South African practice, although the magnitude of these differences appears to be in dispute and is probably therefore material-dependent.

California bearing ratio: In South African practice only the CBR at a penetration of 2.54 mm is used, whereas in British and American practice (and that of most other countries) the higher of the CBRs at 2.54 or 5.08 mm is used. For most uncemented South African materials the 2.54 mm CBR is on average about 80% of that at 5.08 mm. In the case of this project this difference is only likely to be significant in the case of the Section K crusher-run. In the case of the neat sand the previous investigation (Netterberg 2015) found the CBR at 5.08 mm to be invariably **lower** than that at 2.54 mm. The CBR is always quoted at a particular percentage MAASHO compaction unless stated otherwise and the term 'MAASHO CBR' means at 100% compaction. Unless stated otherwise the CBR is always determined after 4 days of soaking.

Relative compaction: Compaction is always expressed as a percentage of MAASHO MDD unless stated otherwise.

Stabilizer contents are always expressed on an additive percentage mass basis.

17.2 Road samples

The results of the field and laboratory testing carried out by Geoplan according to the methods indicated there are shown in **Annex H** and will not be summarised in detail here.

The water contents of 9.1 – 12.8% on the CTB section at km 19 + 797 and Section A were all over OWC and much higher than the 2.8 – 6.9% reported for Sections F – K, which were all less than OWC.

All samples were SP or NP on the standard P425 fraction, but PIs of 6 – 11 were found on the P075 fraction – even on the cement-treated base and subbase – indicating that they survived the cement treatment and subsequent carbonation. The single-point method was used to determine the LLs and an estimated LL water content used for the LS when the LL could not be determined.

The GMs of 1.09 and 1.22 on the Section F base (5% PBFC) and the 0.87 – 0.89 for the three subbases to Sections F, G and K show that the base had partially and the subbases fully reverted to their equivalent granular state similar to the underlying untreated sand selected layer with a GM of (0.84 – 0.87).

The electrical conductivity (EC) and pH results should be considered with caution as they were not carried out according to the methods prescribed by COLTO : 1998 and are not discussed here.

The 98% (soaked) CBRs of about 40 of the cement-treated base and the subbases provide an indication of their residual disturbed strength after carbonation. However, their swells of 0.2 – 0,3% were surprisingly high.

With GMs of 0.8 – 0.9 and all slightly plastic the shoulder and veld samples were all very similar and confirmed that the shoulders were all just sand.

The MAASHO MDD OF 1 809 kg/m^3, OWC of 5.4% and 100% CBR of about 35 on the Section A neat sand were all significantly lower than those previously obtained by Geostrada and CSIR on samples from the same section.

The MAASHO MDD of 1 890 kg/m^3 for Section G (2% sulphite lye) was exactly the same as that reported at Proctor compaction by Gregg (1963) in **Table 1**. An average compaction of 98.1 % (97.0 – 99.3, *n* = 3) in the left outer wheelpath is therefore indicated.

No as-built MDD was reported for Section K (crusher-run), but using the MAASHO MDD of 2 294 kg/m^3 found indicated an average compaction of 95.1% (91.5 – 97.2, *n* = 3) in the left outer wheelpath, which seems low.

The relative compaction of the upper 150 mm of the left hand sand shoulder on Section G was found to be 97% using the neat sand MDD of 1 809 kg/m^3.

Using the same MDD the relative compaction of the upper 300 mm of the roadbed in the veld next to Sections A and K was found to be 85 and 88%, respectively (**Section 10**).

17.3 DCP tests in CBR moulds

The results of comparative DCP testing by CSIR of soaked CBR specimens of neat sand from Section A compacted at MAASHO OWC at MAASHO, NRB and Proctor efforts both without and after a prior CBR test are shown in **Annex I**.

Using the simple average DN from top to a maximum depth of 95 – 100 mm near the bottom of the specimens showed remarkably little difference, confirming the similar findings of Mr EG Kleyn (pers. comm.) :

DN (mm/blow)		CBR
AFTER	**WITHOUT**	**%**
10.9	14.0	41
19.6	19.4	26
33.3	33.7	12

DCP tests after MAASHO CBRs at OWC and about half OWC yielded a DN of 9.8 with a CBR of 50 and 4.7 with 116, respectively.

No significant heave occurred during any of the DCP testing.

Although all these results are consistent their relationship does not coincide with any of the Kleyn (1984) general, Netterberg (2015a) Hoopstad sand or the Paige-Green et al (2015) general sand equations, but lies somewhere between the first two. It does nevertheless predict about twice the CBR for the same DN than the Kleyn relationship.

Because of the huge DN strength gradient in the moulds it is **recommended** that both these results as well as those previously found also be analysed using the more sophisticated approach of Dr P Paige-Green.

Concern is often expressed with respect to the confining effect of the 150 mm-diameter CBR mould in relation to the in-situ CBR.

According to Croney (1977) the confining effect has little effect on the CBR of fine sands but increases with maximum particle size.

The work of Kleyn (1984) on Transvaal soils (mostly coarser than fine sands) showed that the confining effect of the mould dissipated at a diameter of 200mm and that its effect was cancelled out by the density gradient in the mould. The simple arithmetic average DN over the full thickness of the specimen was therefore used to develop the DN-CBR relationship. This was found to be valid for both soaked and unsoaked specimens.

In Kleyn's (1984) work the DCP was penetrated from the opposite side to that of the CBR on the same specimen. In the case of the present and previous work on this project the DCP was penetrated on the same side as the CBR, in the centre of the CBR indentation. However, this should make no difference to the average DN through the whole specimen. It is **recommended** that a test procedure should be agreed upon, using that of the author as a basis.

Whilst a correlation between DN and CBR is of interest in providing an interim crossover "feel", its chief value is to indicate that the CBR of such sands is far higher than that predicted by the Kleyn model and to thereby provide a plausible reason for the unexpectedly good performance of such sands.

In practice the use of DN only is **recommended**. However, caution is required because the strength of such a sand is apparently far higher than other materials at the same DN.

18. Stabilization Design and Durability

In South African practice as represented by TRH 13 : 1986 it is usual to determine the initial consumption of lime (the ICL) and now also cement (ICC) after curing for one hour in order to obtain an inexpensive, rapid indication of the minimum stabilizer content required. This amount is then used as the minimum for the more expensive and lengthy strength and durability testing and also represents the absolute minimum that should be added in the pavement layer in order to ensure that the stabilization – or at least the reduction in plasticity – is permanent.

18.1 Initial Consumption of Cement

The ICL/ICC test simply involves measuring the pH of the soil-stabilizer mix with different percentages of stabilizer. The minimum percentage of stabilizer at which the pH becomes constant or – in the case of lime a pH of 12.40 at 25 °C – is taken as the ICL/ICC.

The standard test requires only one hour of curing, after which the fast reactions with lime, i.e. those with organic matter, soluble silica, soluble alumina, phosphates and sulphates (all practically instantaneous) – and calcium adsorption on the clay minerals are mostly complete. This usually results in a reduction in plasticity and some improvement in strength. With the addition of more lime slower pozzolanic reactions with clay minerals and a substantial increase in strength take place.

Whilst the ICL test has been an accepted test in South Africa for many years, its application to cement stabilization is more recent and although now included in a national standard (SANS 3001 – GR57) is really still in the experimental stage. As very little of the cement is hydrated after one hour it is advisable to extend the curing time to 24h (or 48h as in cement testing) and at least 7 days. The interpretation of the results is also more difficult as cement may have a pH of 13 or more.

In the SANS method the pH is only recorded to the nearest 0.1 pH unit and the ICL/ICC is taken as the point where the plot departs from the straight line joining those points of constant pH. Common problems are that the pH readings do not become constant but creep upwards and/or a pH of 12.4 is not reached. A more rigorous interpretation is to use the minimum of at least two points which are within 0.10 units, which are also at least 12.3 at 25 °C (or in this case 12.6 at the test temperature of about 21 °C). However, in the author's opinion this will still only yield a third-class result (the author's first-class test requires three points all with 0.05 pH units).

The results of the ICC test using PPC CEM II 32.5N B-L "Roadsure" soil stabilization cement are shown in full in **Annex I** and summarised in graphical form in **Figure 3.**

According to these two methods the ICC was as follows.

Curing period	SANS	Two-point 0.10 pH
1 hr	2.5%	3.0%
24 hr	3.5%	4.0%
7 d	3.5%	(5.0%)

The ICC was therefore about 3.5 – 4.0% or, conservatively, 4.0%. In the absence of the requested pH on the pure cement or at least on a 10% mix the 7-day test must be regarded as invalid because the maximum pH reached was only 12,4 and was still increasing.

Although this test was carried out according to the SANS method – including oven-drying of the raw sand (air-drying was requested) – buffers of 7 and only 10 and not the required 12 were used. This and not having kept the specimens sealed for 7 days may account for the low pH recorded. Although not required by the method, the specimens were disturbed at regular intervals to prevent them forming solid lumps.

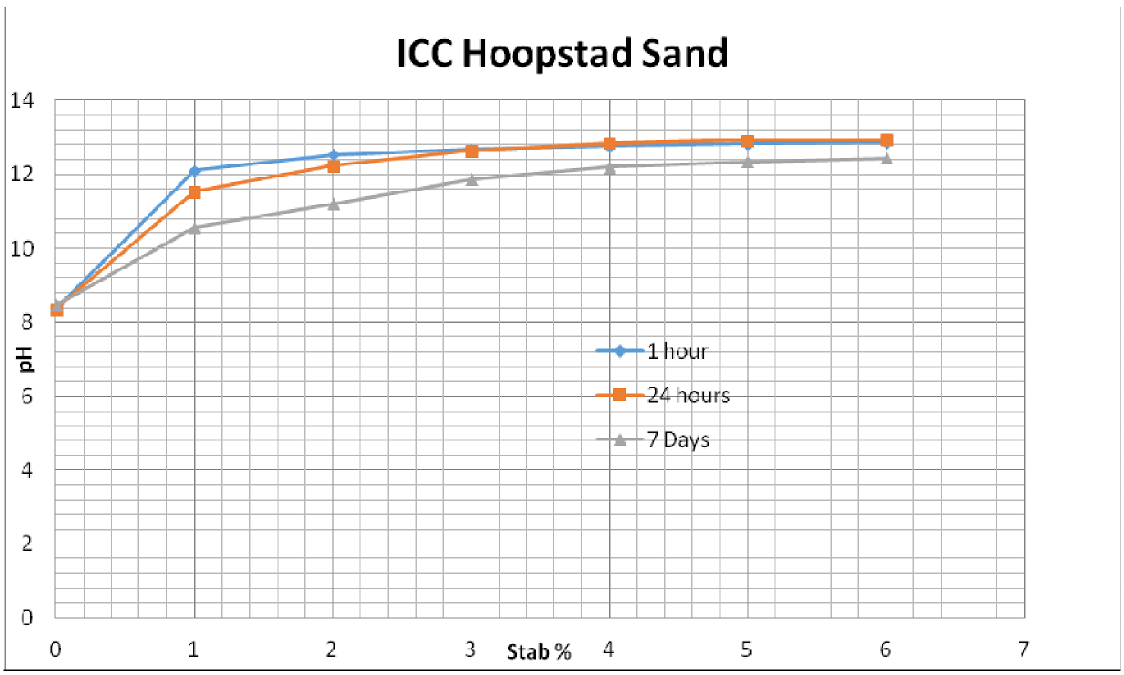

Figure 3. Initial Consumption of Cement by the pH Method

18.2 Engineering Testing

As there has been a strong move towards only using the MAASHO (i.e. 100%) specifications – and only using the ITS – most consideration is given here to these results. Although the South African pavement engineering manual (SAPEM) (Sanral 2013) provided specifications for both, it was recommended that precedence should be given to achieving the specified ITS over exceeding the maximum UCS.

Whilst it is – or should be – normal practice to consider more than one stabilizer, owing to cost and time considerations only one, a modern cement specially manufactured for soil stabilization in a factory in the Kalahari sand area has been used here. An OPC (i.e. a modern CEM I 32,5) would no longer normally be used for soil stabilization and a modern PBFC (i.e. a modern CEM III A) was not available from PPC in this area, although they did offer to make up a laboratory blend.

Modern cements legally sold in South Africa comply with SANS 50197 – 1 for common cements or SANS 50413 – 1 for masonry cements. SANS 50197 is essentially a local version of EN 197-1.

The results of the **UCS and ITS testing** by Geoplan are shown in **Annex H.**

Using normal 7-day curing with 2, 4 and 6% PPC Roadsure CEM II 32.5N B-L road stabilization cement from their Slurry factory showed that only 2% was sufficient to yield a MAASHO UCS of 960 kPa and an ITS of 267 kPa. This satisfies the respective COLTO : 1998 and the SAPEM laboratory design requirements of 0.75 – 1.5 MPa and a minimum of 200 kPa for a C4 material.

With a result of 1.43 MPa the use of 4% just failed to meet the MAASHO UCS requirement of 1.5 – 3 MPa, for a C3, but with 409 kPa comfortably exceeded the minimum ITS requirement of 250 kPa.

With 6% cement and a UCS of 1.90 MPa and ITS of 586 kPa both criteria were comfortably met.

The use of 2% and 5% Roadsure cement would therefore meet the laboratory requirements for a C4 and C3, respectively – and 3 or even 2% for a C3 on an ITS basis alone.

Although the G7 sand raw material did not meet the minimum COLTO and SAPEM quality of a G6 requirement for both a C3 and a C4 (Sanral usually now requires a G5 and a G6 respectively), and this would therefore not normally be permitted, this experiment has shown that this G7 sand can nevertheless be used successfully.

UCS testing at 28 days yielded higher results and accelerated testing even higher, which would at 1.62 MPa allow even 2% cement to qualify as a C3.

The as-built UCS test results can only be roughly compared with the current South African practice as the test methods, density and specimen shapes differ. However, assuming a factor of 1,25 for specimen shape alone, all the cured UCS results of 1.4 – 2.4 MPa shown in **Table 1** would be equivalent to about 1.8 – 2.9 MPa and would satisfy the COLTO requirements of a 97% MAASHO UCS for a C3 material of 1 -2 MPa.

In terms of specimen shape but not density the cured ITS results are probably comparable with current South African practice. If an allowance is made for density all the cured as-built results of 165 – 380 kPa would probably satisfy the current COLTO requirement of a minimum 100% MAASHO ITS for a C4 of 200 kPa and the 5% cement contents for a C3 of 250 kPa.

In short, the as-built results suggest that 3% OPC would have satisfied the COLTO requirements for a C4 and 5% of either OPC or PBFC those for a C3. (No durability testing was carried out.)

The results of **durability testing** by CSIR are shown in **Annex I** carried out according to SANS 3001-GR 55 (but with the normal 7-day curing instead of the prescribed accelerated method) at MAASHO compaction.

Duplicate wet-dry brushing tests with 2, 4 and 6% of the Roadsure cement yielded average losses of 60, 27 and 17%, respectively.

Four and 6% cement respectively would thus be required to meet the maximum loss of 30% for a C4 and 20% for a C3 usually required by Sanral.

With accelerated curing the losses would probably have been less and the requirement for a C4 might have been met with an estimated 3% and a C3 with 4%.

Phenolphthalein and HCl tests on the brushings and the outside of the specimens at the end of the test showed them to be totally carbonated. Later testing of the deliberately broken specimens showed all to be totally carbonated except for the 140 mm diameter x 120 mm high cylindrical 6% specimens, which showed a light red colour in their central 50 x 50 mm cylindrical portions, i.e. they were carbonated to a depth of about 40 mm on all surfaces.

Accelerated carbonation tests on small 117 x 102 mm (i.e. non-standard size) cylindrical specimens at MAASHO compaction prepared using normal curing showed losses of UCS after carbonation comparable with those yielded by the wet-dry brushing test :

Cement	**UCS Uncarbonated**	**UCS Carbonated**		**Carbonated Loss**	**Wet-dry brushing Loss**
%	kPa	kPa	**%**	%	%
2	764	352	46	54	60
4	1 078	685	64	36	27
6	1 430	1 240	87	13	17

According to the phenolphthalein test on broken pieces the carbonated specimens had been fully carbonated. Only the use of 6% cement would therefore satisfy a residual UCS (RUCS) requirement of 80% recommended by Sampson and Paige-Green (1990) or to meet the full C4 requirement of 0.75 – 1.5 MPa.

However, 4% would probably be sufficient if the specimen size is taken into account. (The smaller specimens were used because of material and time constraints and should be regarded as only indicative.)

Extrapolation of the results suggests that no loss of strength would have resulted with 8% cement.

In **summary**, whilst there is some disagreement, allowing for the addition of an extra 1% on the road to allow for mixing the results of this testing support the use of 3% cement for a C4 subbase and 5% for a C3 base on the road itself – as used on Section C and apparently on the rest of the road and on other roads in the area.

These amounts were sufficient to provide adequate performance in spite of carbonation and, as shown (e.g. **Table 16**), even Section B with only 3% in the base was sufficient and in some respects better than Section C.

Whilst economic, CEM II B-L cement contains 21 – 35% limestone filler which is virtually inert. It is therefore likely that the use of other cements with less filler and/or with reactive additives would give better test results and enable lower percentages of cement to be used on such a NP or SP sand.

The density testing after carbonation was unsatisfactory, will have to be repeated and is not discussed here.

18.3 Chemical Composition and Mineralogy

The purpose of this work was to check some of the original cements contents, calculate the degree of carbonation, and to identify the nature of the cement in fully carbonated but strong specimens.

The results of chemical analyses by PPC of selected samples are shown in **Annex K** and recalculations of some of these by the author in **Table 17**.

These recalculations showed the:

- pH to be 8.9 – 10.5 (*n* = 11), with 5 ≥ pH10, even though all failed to turn red with phenolphthalein in the field;
- cement contents to be 1.8 – 7.4% (*n* = 11);
- cement contents of Section B (3% OPC) to be 1.8 – 3,0% (*n* = 2);
- cement contents of Section C (5% OPC) to be 4.1 – 6.2% (*n* = 5);
- cement contents of Section D (10% OPC) to be 6.6 – 7.4% (*n*=2);
- cement contents of Section E (5% PBFC) to be 3.0 – 5.0% (*n* = 2);
- carbonate content of Section A (neat sand) to be 0,0 – 0,5% *(n* = 5);
- carbonation of Sections B – E to be 82 – 100% (*n* = 11);
- residual cement contents to be 0,0 – 1,1% (*n* = 11);
- degree of hydration of the cement to be mostly 50 – 100% (*n* = 11);
- highest original cement contents (6.0 – 7.4%) to be mostly associated with the highest UCS (1 500 – 2 400 kPa), *n* = 4;

and that there was:

- little correlation between pH and UCS;
- no correlation between the UCS and the carbonate content, degree of carbonation, residual cement content, degree of hydration, or effective residual cement content.

From these observations it is **concluded** that the laboratory pH can be about 9 – 10 even when the material fails to colour with phenolphthalein in the field; substantial strengths of over 1 500 – 2 500 kPa can occur even when the cement is over 80 % carbonated; but that the reason for these strengths is not due to the carbonate or the total or effective residual cement, and remains unknown.

The high strengths retained with 6% or more cement support the 6% indicated by the RUCS testing.

Preliminary mineralogical and petrographic works on three cemented samples by Dr S Verryn shown in Annex L were inconclusive with regard to identification of the cementing matrix.

In theory the cement after carbonation should be amorphous silica and alumina.

However, attempts by the author to identify the nature of the cementing matrix using selective dissolution tests have also proven inconclusive to date.

It is **recommended** that this work be continued under the PPC-sponsorship.

19. Base Course Material Specifications

The specifications for **untreated sand base** provided in the May 2015 Netterberg report and Netterberg and Elsmere (2015) are summarised here without repeating their discussion.

However, this later work indicates that the conservative traffic limit of 0.1M E80 can now be raised to 0.3M.

Table 18 Chemical analyses of base course samples taken in 2013 and 2014: Key results, derived data and in-situ strengths

PPC Lab. No.	Field Ref. Section, SV, Lane, Wheelpath	Additive		CaO	CO_2	LOI	pH	H_2O+	Cement content	$CaCO_3$ from CaO	$CaCO_3$ from CO_2	Cement Carbonation	Cement Residual	Cement Hydration	In-situ sDCP trength DN	In-situ sDCP trength UCS	In-situ sDCP trength CBR Kleyn	In-situ sDCP trength CBR Sand	Effective Cement content
[1]	[2] [3]			[4]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]	[12]	[13]	[14]	[15]	[16]	[17]	[18]
		Type	%	%	5	%		%	%	%	%	%	%	%	mm/blow	kPa	%	%	
03120	A 10 LO	-	0	0.00	0.00	1.45	8.36	1.45	0.0	0.00	0.00	-	-	-	5.6	-	46	>100	-
03118	A 30 LO	-	0	0.00	0.20	1.37	8.25	1.17	0.0	0.00	0.45	-	-	-	4.7	-	58	>100	-
3121	A 30 LI	-	0	0.00	0.04	1.40	8.35	1.36	0.0	0.00	0.09	-	-	-	8.2	-	28	>100	-
3117	A 55 LO	-	0	0.00	0.12	1.45	8.48	1.33	0.0	0.00	0.27	-	-	-	7.0	-	34	>100	-
3119	A 55 RO	-	0	0.04	0.23	1.58	9.21	1.35	(0.1)	0.07	0.52	100	-	(87)	5.6	(450)	46	>100	-
3116	B 55 LO	OPC	3	1.13	0.93	2.45	9.61	1.52	1.8	2.02	2.11	100	0.0	46	6.4	390	39	>100	0.0
3112	B 55 LI	OPC	3	1.90	1.36	3.15	10.2	1.79	3.0	3.40	3.09	91	0.3	67	5.8	430	44	>100	0.2
0032	C 20 LOU	OPC	5	3.49	2.26	5.08	9.9	2.82	5.5	6.25	5.13	82	1.0	(118)	5.4	470	48	>100	1.0
3123	C 20 LOU	OPC	5	3.52	2.38	4.82	10.0	2.44	5.5	6.30	5.40	86	0.8	88	5.4	470	48	>100	0.7
3115	C 20 LO	OPC	5	2.60	1.73	3.78	10.3	2.05	4.1	4.66	3.93	84	0.7	76	7.5	330	32	>100	0.5
3122	C 20 LIU	OPC	5	3.97	2.61	5.28	9.92	2.67	6.2	7.11	5.92	83	1.1	94	1.7	1600	200	>100	1.0
3113	C 20 LI	OPC	5	3.85	2.64	4.95	10.0	2.31	6.0	6.89	5.99	87	0.8	71	0.9	2400	(320)	>100	0.6
3114	D 30 LO	OPC	10	4.75	3.06	5.47	10.5	2.41	7.4	8.50	6.95	82	1.0	64	1.8	1500	190	>100	0.6
3111	D 30 LI	OPC	10	4.20	2.86	5.07	8.86	2.21	6.6	7.52	6.49	86	0.9	58	0.8	2500	(340)	>100	0.5
00031	E 30 LO	PBFC	5	1.44	1.12	2.34	9.7	1.22	3.0	2.58	2.54	98	0.1	0	6.0	410	42	>100	0.0
00030	E 30 LI	PBFC	5	2.39	1.58	3.21	9.9	1.63	5.0	4.28	3.59	84	0.8	26	1.3	1900	250	>100	0.2

NOTES

[1] Chemical analyses by Pretoria Portland Cement Co. Derived data and strengths calculated by author

[2] km 19,771; 19,796 and 19,821 sampled Jan. 2013 and stored air-dry; E 30 March 2014 and remainder Oct. 2013 and kept sealed until tested. All full thickness (approx. 150 mm) except for U samples (upper 50 mm only)

[3] Key example: Section A 10 LO = Section A (neat sand) at section stake value ("chainage") 10 m in left outer wheelpath R = right lane, I = inner wheelpath, M = midlane

[4] CaO by XRF, CO_2 by volumetric evolution with nitric acid

[5] At 1 000 °C [6] Saturated paste [7] H_2O + = LOI – CO_2 [8] Cement content = 100 (CaO **/** 64) for OPC and 100 (CaO **/** 48) for PBFC [9] $CaCO_3$ = 1.79 CaO [10] $CaCO_3$ = 2.27 CO_2

[11] Carbonation = 100 ($CaCO_3$ from CO_2) **/** ($CaCO_3$ from CaO [12] Residual cement = calculated cement content (100 **–** % carbonation) **/** 100 [13] Hydration = 100 (H_2O + **–** 1.33) **/** (0.23 cement content)

[14] Upper 50 mm for U samples and 150 mm for rest) [15] UCS = 2900 DN $^{-1.08}$ (Kleyn 1984) [16] CBR = 410 DN $^{-1.27}$ (Kleyn 1984) [17] CBR = 3000 DN $^{-1.46}$ for DN > 10 for this sand (Netterberg & Elsmere 2015)

[18] Effective residual cement = residual cement x % hydration / 100

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. (When PI075 = NP or 0, then IF075 = P075.)
- 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 : 0.30% or, less reliably, minimum Fe_2O_3 content by XRF analysis : 1.2 % Fe_2O_3

Probably desirable:

- Sand equivalent : 13 – 40
- Dust ratio : 0.15 – 0.30; preferably 0.20 – 0.30
- pH (saturated paste or 1 : 2.5 soil : water ratio) : 7.5 – 8.5
- Particle angularity : Minimum uncompacted voids (ASTM C 1252) on the plus 075 µm fraction : 45 % **or** mostly angular particles visible under stereo microscope
- Dominant clay mineral : kaolinite

Raw sand to be used for a cement; bitumen-, tar- or sulphite lye-treated base should comply with the untreated sand base specification.

A 2% sulphite lye-treated sand base can also be used for up to 0.3M E80. However, on economic and performance grounds it cannot be recommended in this area on this material.

The above specification is supported by the eight Geoplan GM, P075, PI, PI on the P075 and sand equivalent results on the neat sand base, selected subgrade and veld sand samples and the one CSIR result which, with two minor exceptions of a PIs on the P075 of 7 and 8 on two shoulder samples, all fall within the limits derived from the pervious work. As the Geoplan liquid limit tests were all carried out by the less reliable one-point method it is considered that the upper limit of 6 should stand.

The high permeability of the sand is probably an important factor contributing towards its good performance in that it will be free-draining and permit the rapid dissipation of pore pressures under dynamic loads. It is therefore **recommended** that these aspects be included in any further study of this sand.

Regarding specifications for cement-treated sand bases, 3% OPC was the minimum used and has proven to be successful on the experiment, and 10% gave performance next best to that of a crusher-run base (although with the usual block cracking).

In essence the TRH 13 : 1986 and COLTO UCS and ITS requirements are supported although maybe only the ITS is needed. However, the fact that these sections were still there and carrying traffic indicates that carbonation can be tolerated and that severe durability requirements are not warranted provided that the base is sealed before it can undergo surface weakening.

The results with the Roadsure cement are unusual in that acceptance on an ITS basis alone would result in lower cement contents being acceptable.

The experiment has shown that 4% of a 60% anionic SS emulsion or 4% of a 30 / 35 EVT coke oven tar are also adequate stabilizers for this sand.

Specifications for bitumen emulsion-treated sand bases are already covered by TH14: 1985 (Asphalt Academy 2009) and Netterberg (2015b) guidelines and will therefore not be considered further here.

As a G7 sand such as this would not normally be allowed, the raw sand to be used should comply with the untreated sand base specification.

20. Other Conditions

Terrain

- Relatively flat
- Sandy
- Permanent or perched water table (e.g. due to rock bars): at least 1,0 m below top of roadbed

Construction

- Surface drainage : surface camber or adequate (≥ 3% ?) crossfall
- Seal : at least a double seal of width adequate for anticipated traffic
- Prime : required
- Compaction :
  - Base to refusal or at least 100% MAASHO, whichever is the greater
  - Shoulders, subbase (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 potentially collapsing
- Uncladded sand side slopes : 1 : 6 or flatter
- Side drains : at least 5 m from edge of seal and inverts at least 0,5 m below finished road at centreline

Maintenance

- Seal and shoulders : good

Shoulders : to comply with neat sand and base specification and preferably sealed.

The experimental terrain was flat. Significant gradients might result in greater erosion of the shoulders and the development of water channels next to the seal.

Other factors which have probably contributed to the good performance include the good surface drainage due to the camber, the triple seal (plus one reseal and one or two rejuvenation sprays), the substantial penetration of the prime, and the sand roadbed.

Experience in Botswana has shown that new, uncladded Kalahari sand sideslopes steeper than about 1 : 6 are likely to suffer severe erosion due to rainfall.

It was clear from the sandstorms experienced during all four site visits and the cross-section of the road reserve that sand movement was still active in this area even under the current mean annual rainfall of about 520 mm. This appeared to have negatively affected the cross-section and drainage.

Whilst the grassed **sand shoulders** have performed surprisingly well – at least until the advent of the 6- and 7- axle trucks – they should preferably be sealed. An A-2-4(0) classification with some plasticity – at least on the P075 – is essential if left unsealed, and the growth of grass should be encouraged.

Problems due to burrowing animals such as dune moles, suricates (meerkats) and termites can be experienced in such areas. However, only a few meerkat holes on the shoulders and up to about 500 mm from the edge of the seal were noted on the experiment. At least one DCP result had to be rejected on this account.

21. Relative costs

Estimates of the current cost of pavement layers and seal construction in this area have been compiled by Propercon and are shown in **Annex M**. These are at current market-related prices and exclude other project-related costs.

The costs in South Africa Rands per km of an 8.2 m-wide, 150 mm-thick base course construction only, including 600 mm-wide sealed shoulders, but excluding prime and seal are as follows:

Item	R
• Neat sand base compacted to 100%	153 750-
• CTB sand base with 3% OPC	315 150-
• CTB sand base with 5% OPC	418 950-
• ETB sand base with 3% SS60 emulsion	559 650-
• ETB sand base with 4% SS60 emulsion	694 950-
• G3 crushed stone base compacted to 98%	385 728-

The above costs for ETB bases include 1% cement at R54 200/km which may not be necessary with the A-2-4(0) sand specified and was apparently not used on the experiment.

The approximate costs of 3% stabilization (the probable minimum desirable) or a crushed stone base relative to neat sand base are therefore :

- 3% cement : 2 X
- 3% emulsion : 3.5 X
- Crushed stone : 2.5 X

The cost savings of a neat Kalahari sand base for a low-volume road – especially in remote areas where the cost of cement, emulsion and crushed stone would be even higher – are obvious.

However, it still has to be shown that a neat Kalahari sand base is viable in normal, full-scale construction.

From the purely engineering point of view, an emulsion-treated Kalahari sand base would probably be the preferred choice on considerations of flexibility and freedom from carbonation, cracking and, according to TRH 4 : 1996, the necessity for a cemented subbase for traffic in excess of 0.3M E80 on a Category C road.

Only 4 and 8% emulsion were used on the experiment. On this basis an emulsion-treated base (ETB) of say a TRH 14 : 1985 BT3 or (Asphalt Academy 2009) BSM standard containing 4% emulsion (2,4% net bitumen) suitable for up to 1M E80 would cost about 4.5 times that of a neat sand base.

The Orapa emulsion-treated Kalahari sand experiment in Botswana conservatively indicated that 2% emulsion (without cement) was adequate for up to at least about 0.5M E80 and that the same amount in only the upper 75 mm was adequate for up to 0.3M E80 (Netterberg 2015b).

As the gate price of lignosulfonate is about R4 500 / ton as against about R500 / ton for cement it is not an economically viable stabilizer in this area.

Recent developments in nanotechnology additions to the emulsion also suggest that the amounts of emulsion could be greatly reduced, possibly to a level which would make it competitive with the larger amounts of cement necessary in both the base and subbase.

22. Conclusions

Practically all of the work proposed in the Mobilization Report was eventually carried out, although the sampling and most of the laboratory engineering testing had to be done three times, the DCP surveys twice, and a few of the less important test results are still outstanding.

Although it was carried out after a period of three months of below average rainfall, the June 2017 field water contents in the base were found to be **above** MAASHO OWC – presumably due to the previous exceptionally wet season.

The surface drainage was also only poor to fair and the results of the work are therefore conservative for normal conditions.

In spite of this the few suction measurements indicated moisture suctions of 8 – 9 MPa in the bases tested and are probably an important factor in their good performance.

All 12 sections were still there after 55 years, about 1,5M E80 per lane, and were still carrying at least 30 6–7 axle trucks per lane per day.

All the cement-treated layers were totally carbonated according to the phenolphthalein test.

Although all except the crusher-run section were only in a poor to fair condition because of extensive cracking and edge breaking, all were structurally sound with no shear failures, very few potholes, and no excessive rutting.

However, considering the maximum length of 20% on which severe distress is permitted for a Category C rural road at the end of its structural design life of 10 – 20 years, all **except those indicated below** were in a terminal condition with respect to the following distress modes:

- Block plus transverse cracking : crusher-run (Section K);
- Crocodile : 10% OPC (D), 4% emulsion (HB) (marginal), 8% tar (JB), crusher run (K);
- Edge patching : 10% OPC (D), 8% emulsion (HA), 4% emulsion (HB), (marginal); 8% tar (JB), crusher-run (K);
- Edge breaking : crusher-run (K)

All of them had far exceeded their presumed design life of 20 years and analysis period of 30 years.

The crocodile cracking was largely caused by the old, brittle thick seal rather than a structurally unsound pavement. However, the carbonation-induced weakening of the cement-treated layers would have contributed to this by increasing the flexibility of the pavement.

The edge breaking and necessary patching was caused by the narrow 6.0-wide seal being now too narrow for the very large, heavy vehicles currently using it.

All sections except the crusher-run (K) and 4% emulsion (HB) were in a severe condition with respect to deflections in the left outer wheelpath, with a residual structural capacity of about 0.5M E80 indicated for the crusher-run and about 0.2M for the rest, including the neat sand section.

A program error was found in WinDCP 5.1 for the calculation of the average BN_{100} which was brought to the attention of the CSIR.

The visual observations in pits, deflections and DCPs indicated that all the sections were in a flexible or very flexible state and that all the cemented layers had reverted to an equivalent granular state. Although some were small blocky the DCP surveys indicated all the cement-treated bases to be essentially uncemented in the left outer wheelpath with a UCS of about 500 kPa, except for the 10% OPC section (D) and one 5% PBFC section (F), each with about 1 000 kPa. All of the bases except the crusher-run failed to meet the Win DCP 5.1 DN strength requirement for 0.2 – 0.8 MISA.

In spite of this, and although (except for Section K and, neglecting the block cracking, D, HB, and JB), all were in a terminal condition with respect to the **surfacing**, all sections were still carrying at least 30 six- and seven- axle trucks daily in each lane with only edge maintenance. Unfortunately, their condition after 20 years (in about 1983) is unknown, but must have been much better, as the heavy traffic (>50 E80/lane/day) only started in about 1993 and in 2013 was over 250 E80/day.

All of the designs evaluated can therefore be used for lightly trafficked Category C or D rural roads with a structural design life of 20 years.

For neat Kalahari sand of this type the previously recommended limit of 0.1M E80 can be increased to 0.3M. The specifications for this base have been reviewed but have been left unchanged.

The DCP method greatly overpredicted the residual structural capacity because of the exceptionally strong selected and lower layers.

Both the capacity and the load equivalency exponent were found to be affected by the position of the zero point of the DCP.

The low exponents found indicate that most of the sections – especially the neat sand – are relatively insensitive to the axle loading and thereby provide a plausible reason for their good performance.

A provisional upper limit of 1.0M E80 is recommended for cement-, bitumen- and tar-treated Kalahari sand of this type. (The use of tar is no longer recommended on health and safety grounds.)

Three percent cement or 4% bitumen or tar should be adequate for base course provided it is well-mixed and a cement-treated base protected from surface carbonation and physical damage during construction. This should be designed using current design methods to conform to existing requirements. However, severe durability requirements appear unwarranted. As all these bases had a 3% PBFC-treated subbase the nature of the subbase must also be considered. Omission of this may result in severe pavement imbalance and excessive traffic-associated cracking. Whether of not a neat Kalahari sand subbase compacted to 100% would be adequate requires further consideration. This is one reason for resilient modulus determination as part of the triaxial testing **recommended.**

In all cases the surfacing must be sufficiently wide for the traffic expected and adequately maintained**.**

Complete field carbonation of all cement stabilized layers was indicated by phenolphthalein and acid tests and 84 – 100% carbonation confirmed by chemical analysis. A degree of hydration of 0-100% (but mostly 50 – 100%) and effective residual (i.e. hydrated but uncarbonated) cement contents of only 0 – 1.0% (but mostly less than 0.7%) were also indicated. The high strengths of some of the sand-cement cannot be explained by these low effective cement nor the 4 – 6% carbonate contents found.

In spite of them being totally carbonated and generally weakened, all the cement-tested sections had performed satisfactorily in the carbonated and post-cracked phases and were still structurally sound. A DCP UCS of up to about 1 000 kPa was found in a few places as well as a Schmidt hammer-estimated UCS of up to about 10 MPa on blocks of base exposed along the edges. Several chemical and a few mineralogical analyses and selective dissolution tests have so far failed to determine the nature of the cementing medium in such cases and further work is continuing.

The naturally grassed shoulder performed moderately well with no shear failures but was subject to erosion by the heavy vehicles.

ICC tests should be carried out after 48 hours and 7 days or more in addition to the usual one hour.

The approximate costs of 3% stabilization (the probable minimum desirable) or a G3 crushed stone base in this area relative to a neat sand base are cement 2x, emulsion 3.5x and crushed stone 2.5 x.

As the performance of the sulphite lye (lignosulfonate) section was only slightly better than that of the neat sand its use is therefore uneconomic in this area.

The reasons for the exceptionally good performance of all of these sections, most of which must have had bases of grossly substandard strength for quite some time, appear to be as follows:

- neat material and all layers including shoulders and roadbed of free-draining sand;
- no perched or shallow water table;
- only just sufficient fines and plasticity, and probably free iron oxides, in the A-2-4 (0) sand to provide cohesion, but not to impede drainage and dissipation of pore pressures;
- high suctions of about 8 – 10 MPa in the base;
- far higher DCP CBRs than expected according to the Kleyn relationship – the neat sand base had an apparent in-situ CBR of over 100;
- exceptionally strong sand layers below a depth of 450 mm to at least 800 mm;
- low apparent load equivalency exponent;
- strong and thick (25 – 30 mm) triple seal and one reseal, with one or two rejuvenations;
- cambered seal;
- absence of weak interlayer between seal and base;
- good prime penetration.

23. Recommendations

A more detailed comparison should be made between the performance of the left versus the right lane.

The comparative DCP capacity predictions should be re-analysed on a DSN_{450} basis in order to remove the strong but variable selected and lower layers, and also using pavement component analysis.

The results of the comparative DCP tests on CBR specimens both with and without a prior CBR test should receive further analysis and comparison with existing DN-CBR relationships.

A standard test procedure for such work should also be agreed upon.

The work on stabilization durability should be continued under what is left of the existing PPC sponsorship. However, some additional funding may be necessary.

The optimum specifications for cement stabilized Kalahari sand bases – and the raw material used – should receive further consideration especially as such a G7 sand would not normally be allowed (a G6 or even a G5 is currently required), severe durability requirements appear unwarranted, and acceptance on an ITS basis alone would appear to permit lower cement contents to be used.

Sand-cement bases clearly have the potential to carry more and heavier traffic than neat sand bases. However, this project was confined to short experimental sections probably constructed under good supervision and sealed without delay.

This may have led to better performance than might be expected in the case of long lengths of normal construction, on some of which problems have been experienced. A literature and experience survey of such roads known to the author to have been built in the Free State, Botswana and Zambia should therefore be carried out. The object of this would be to identify those factors which have led to premature distress and how to avoid it. For example, the condition of all the cement-treated experimental sections appeared to be better than the adjacent long lengths of road which apparently used a 5% cement-sand base on a 3% cement-sand subbase.

At least a Zimbabwe-type Texas Triaxial test, but preferably also shear and repeated load (for resilient modulus) triaxial tests with pore pressure measurements according to the Sanral protocol should be carried out on the remaining bulk sand sample from Section A in order to provide data for more modern pavement design. The latter should also provide an answer to the important question as to whether or not a neat sand subbase compacted to 100% can be safely substituted for the 3 % PBFC- treated subbase used in the experiment. This is particularly important in the case of cemented bases.

The BS 1377 cone penetrometer LL and the cone penetration index (CPI) (Sampson and Netterberg 1985) together with the AASHTO T93 field moisture equivalent (FME) should be tried as a possibly better measure of both the plasticity, cohesion and angularity characteristics of neat sands than the Casagrande method.

Investigation by Sappi of the effect of their current lignosulfonate product in comparison with neat material should be carried out as this differs from that used in the experiment.

The water-suction and permeability characteristics of the neat sand should be determined.

Similar triaxial, CPI, FME, suction and permeability testing should also be carried out on the neat red Kalahari sand used in the Orapa experiment, which still exists.

The offer of a free study of the efficacy of nano-additives on the reduction of the emulsion requirement on both the Hoopstad and Orapa Kalahari sands should be taken up. However, this would probably require nominal funding for the author for the collection of samples.

As this part of the road has been rehabilitated it is no longer possible to obtain more neat sand from Section A. However, similar sand is obtainable from the road reserve should the remaining material be insufficient, but would require characterisation by indicator and CBR testing.

As recommended in the Netterberg (2015a) report:

- The causes of the poor local BS vibrating hammer work should be investigated and overcome if this method is to be used in future work on sands.
- The use of vane shear and Clegg hammer tests as additional or alternative simple laboratory and field tests on slightly cohesive sands such as used on the Hoopstad and Orapa experiments should be investigated.
- The use of the sand equivalent test as a simple, rapid and inexpensive test for Kalahari sands should be investigated.
- Longer test/demonstration sections of neat Kalahari sand base should be constructed in order to identify any construction problems and confirm its suitability for full-scale construction.

Surplus sample material must be stored safely until all concerned consider that no further testing or archive /reference samples are necessary.

Payment for storage and for transport will probably be required.

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