

# THE JWANENG KALAHARI SAND-ASPHALT LTPP EXPERIMENT: PERFORMANCE OVER 14 YEARS AND DERIVATION OF PAVEMENT DESIGNS

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## ABSTRACT

The Jwaneng long-term Kalahari base course experiment on the Kanye-Jwaneng road in Botswana was opened to traffic early in 1980. It consisted of 12 sections of Kalahari fine sand- asphalt using various tars and bitumens and 11 sections of calcrete with two control sections of gravel base and subbase, all under a double surface treatment and most over an untreated Kalahari fine sand as subbase and lower layers. The purpose of the experiment was to evaluate various alternative base material and thickness designs for roads in the Kalahari where good quality gravels are scarce. However, only the sand-asphalts and their performance are described here as the calcretes have been reported elsewhere. After about 0,4 ME80 in 13 years when the experiment was terminated an analysis of the data available enabled conservative designs to be derived for pavements with Kalahari sand-asphalt bases on untreated sand lower layers to carry traffic in five categories ranging from 0,1 to an extrapolated 1,0 ME80. The experiment also showed that it is not necessary to import a gravel subbase as sand compacted to 97-100% MAASHO performed well and, if confined, could even be considered as untreated base course for a lightly trafficked road.

## 1. INTRODUCTION

Approximately 80% of Botswana is covered by fine Kalahari sand (Roads Department (RD), 2010) in which areas conventional rock and gravel road construction materials are scarce to non-existent, the only other material usually being isolated occurrences of mostly poor quality calcrete (Lionjanga et al., 1987).

A set of long-term pavement performance (LTPP) experimental sections comprising both Kalahari sand-asphalt and calcrete base courses were therefore constructed as part of the new Kanye - Jwaneng road in 1979 in order to evaluate such materials for their potential use in Botswana.

The calcrete sections have already been described (Lionjanga et al, 1987; Greening and Rolt, 1997; 2014) and a comparison of their performance relative to the sand-asphalts provided by Netterberg and Pinard (This Conf.). It is therefore the purpose of this paper only to provide an overview of the sand-asphalts and their control sections of gravel base course and their performance up to 1993, after which the whole experiment was rehabilitated.

## **2. THE SITE**

The sand-asphalt experiment was located in flat, sand-covered terrain between km 113,9 and 115,2 on the Kanye-Jwaneng road in Botswana, about 7 km from Jwaneng and 130 km west of Gaborone. The alignment was straight and level, at an altitude of about 1 000 m. The area has a semiarid warm climate with a moisture deficiency in all seasons (Schulze, 1958), a Thornthwaite moisture index of about minus 29 cm (Emery, 1992), a Weinert (1980) N-value of about 6, and lies within the 1982 Botswana road design manual (BRDM) and the TRH4:1996 (COLTO, 1996) Dry Macroclimatic Region for pavement design purposes. The annual rainfall at Jwaneng varies between about 200 and 800 mm, with a mean of about 420 mm, falling mostly between October and April. The above-average rainfall of 530–800 mm during the first three years after construction was followed by a five-year drought of only 210–340 mm, followed by two years with about 580 mm, one of 340 mm, and 515 mm in 1991, followed by two years with only 250 and 290 mm (Botswana Meteorological Services, pers. comm.).

The mean monthly maximum and minimum air temperatures during the hottest months of November to February are about 31 (but with maxima up to 36) and 17°C, respectively. The mean monthly maximum and minimum temperatures during the coldest months of June and July are about 21 and 2°C, respectively.

## **3. SOILS AND ROADBED**

The roadbed, embankment and subgrade consisted of a reddish-brown, fine-grained Kalahari sand extending to a depth of at least 5 m possessing a collapsible grain fabric and with about 98% passing 0,425 mm and about 15% passing 0,075 mm, classifying as an AASHTO M-145 (1995) A-2-4 (0) soil rather than as an A-3 single-sized sand.

The permanent water table was believed to lie at a depth of about 110 m, but a temporary, perched water table during the rainy season at a depth of about 1,0 m due to a less permeable, clayey sand B horizon appeared likely.

## **4. EMBANKMENT AND SUBGRADE**

The road was raised on a low embankment such that the centreline at surfacing level was about 750 mm above the natural ground level. The embankment was raised in a 600 mm side fill borrow operation using motor scrapers. Two 1,85 m-wide shoulders of a plastic, calcified, clayey sand were constructed on the platform to leave a 7,0 m-wide trough for the sand-asphalt base courses.

## **5. PAVEMENT DESIGN AND LAYOUT**

The road is a TRH4:1996 Category C and BRDM:1982 Category II Type 3A with the following features on the experiment:

- Cross-section: 2,5% fall on seal, 3,5% on shoulders.
- Surfacing: 6,7 m-wide with yellow edge lines, 9,5/19 mm double seal (80/100 pen.) plus fog spray (cationic emulsion) except on seal experiments.
- Prime: 7,0 m-wide MC-30 at 0,75 l/m<sup>2</sup> on Sections 1–6; at 0,98 l/m<sup>2</sup> on Sections 7-12.
- Shoulders: 1,85 m-wide, as for gravel base on Sections A, B, C and D, and preconstructed calcified clayey sand on others
- Subbase: Gravel on sections A, B, C and D, calcified clayey sand on Sections 1 and 2 and reddish Kalahari sand on others. The sand subbase was primed on Sections 3-7.

- Subgrade and fill: Side-borrowed Kalahari sand compacted to 93% MAASHO; fill to 90%.
- Roadbed: Kalahari sand compacted to 90% MAASHO to 1,25 m below top of subgrade

The whole experiment consisted of 12 sections of sand-asphalt and 11 of calcrete bases plus two seal experiments Sections B (sand seal) and C (single seal) and a control section of normal construction at each end (A and D). Each section was 100 m long. Sections A, B, C and D had surfaced shoulders. The thickness of all base sections was intended to be 150 mm except for Sections 8 (100 mm), 9 (75 mm), 10 (75 mm) and 11 (100 mm). Only the sand-asphalt and the control Sections A and D (Table 1) and, for comparison, the adjacent calcrete Section 13 will be discussed here. Further details are provided by Netterberg and Pinard (This Conf.).

**Table 1: Layout of Jwaneng sand-asphalt experiment**

Section No.	A	1	2	3	4	5	6	7	8	9	10	11	12	D
<b>Base [1]</b>	<b>Gravel</b>	<b>In-situ Wet Process</b>						<b>Foamed Asphalt</b>						<b>Gravel</b>
Binder	-	RC-250	RTH 35/40	RTH 30/35 + 1,5 % PVC	RTL 35/40	RC-250	60/70 pen.			RTH 55/60				
Design [2] (%)	-	4,5	5,0	5,0	5,0	5,0	4,5	3,5	3,5	3,5	4,5	4,5	4,5	-
Actual (%)	-	3,4	5,2	5,2	5,0	5,0	3,6	3,9	3,0	4,1	4,9	4,1	4,6	-
Thickness														
Design (mm)	150	150	150	150	150	150	150	150	100	75	75	100	150	150
Actual (mm)	150	140	140	140	140	140	140	150	90	70	40	70	120	150
Compaction [1, 3] (%)	≥ 98	90	99	96	96	95	87	91	93	90	89	90	87	≥ 98
Strength [4]														
Mean (% kPa)	61	510	380	340	300	330	380	320	310	390	170	210	190	120
Min. (kPa)	-	450	320	300	250	250	320	280	240	220	70	150	140	-
<b>Subbase (150 mm)</b>	Gravel	Cal. clayey sand	Red Kalahari sand to ≥ 93 % MAASHO										Gravel	
Comp. (%)	≥ 95	100	102	106	98	96	104	90	97	97	98	95	93	≥ 95
	SV 67 + 100 km 113,900											SV 68 + 400 km 115,200		

Notes :

[1] Actual percentages binder, thicknesses and compactions are means. [2] Optimum. [3] Specified : 98% MAASHO for gravel base controls, 95% for gravel subbase controls, 95% for sand -asphalt base, 95% for calcified clayey sand subbase, 93% for sand "subgrade" (subbase). [4] CBR at 98% for Section A and D gravels; mean and minimum vane shear strengths at 7 days at 40°C for sand-asphalts (n = 6)

The in-situ strength on all the sections reached the required 200 kPa within 24 hours and then **generally** increased rapidly for up to about 3 weeks (the setting-up process) then increased more slowly or leveled off. Mean and minimum 7-day strengths of at least 300 and 220 kPa were achieved on all sections except for Sections 10, 11 and 12. However, the specified minimum compaction of 95% MAASHO was only met on Sections 2-5.

## 6. MATERIALS

Table 2 shows a summary of the properties of the gravel base and subbase materials used for the control Sections A and D, the raw sand used for the sand-asphalt Sections 1–12, the untreated calcified clayey sand subbase used for Sections 1 and 2, and the untreated sand subbase used for Sections 3–12. The base course used for the control Section A was a natural, ferruginised, weathered granite gravel and that on Sections B – D a crushed, alluvial quartzite gravel similar to those used over the rest of the road. The subbase to Sections A and B – D was a ferruginised quartzite gravel. Only Sections 1 and 2 of the sand-asphalt sections had a separate subbase of plastic calcified sand. Similar material was initially used for the shoulders of the sand-asphalt sections only.

The Kalahari sand used for the sand-asphalt came from an approved borrow pit and was a mostly nonplastic, reddish-brown, silty, fine sand (SM) according to the Unified classification (ASTM, 2017) with a grading modulus (GM) of about 0,87, classifying as an A-2-4 (0) material according to AASHTO M-145-91 (1995), and as a G7 in the BRDM:1982

and TRH14:1985 (National Institute for Transport and Road Research (NITRR, 1985). No filler had to be used as the sand complied with the BRDM and TRH14 requirements of a maximum PI of 6 and a minimum P075 of 5% for a BT3 sand.

**Table 2: Summary of engineering properties of untreated materials**

Layer	(Ref.) or [Note]	Section no.	GM	Max. Size mm	P425 %	P075 or P063 %	PI %	LS %	Mean soaked CBR @ % BS Heavy			Sand Equiv. %
									100 %	98 %	95 %	
<b>Base</b>												
Gravel	(1)	A	≥ 2,0	38	38	12	5	3,0	-	60	-	-
Gravel	(1)	D	≥ 2,0	50	23	10	NP	-	-	120	-	-
Sand	(2)	1-12	0,85-0,89	2,00	97-98	13-16	NP-3	0-0,7	-	-	-	11-14
<b>Subbase</b>												
Gravel	(1)	A, D	≥ 1,5	75	47	12	6	2,0		120	70	-
Cal. sand	(1) [1]	1,2	0,87-1,15	20	75-86	20-34	13-17	5,7-7,6	38 [3]	-	-	-
Sand	[1]	3-12	0,95	1,8	95	10	NP [2]	-	90 [4]	-	-	-

References:

(1) Lionjanga et al (1987); Greening and Rolt (1997, 2014), using BS 1377:1975 test methods (P063 and BS Heavy (BSH) results reported) (2) Using South African TMH1:1979 test methods (P075 and MAASHO results reported),  $n = 12$  for grading, soil constants and sand equivalents

Notes:

[1] Subbase to Sections 1 & 2 calcified clayey sand with 16% total  $\text{CaCO}_3$  & 15 % in P425; mean  $\text{LS} \times \text{P} 425 = 486$  ; mean CBR 23 soaked, 90 at OMC on road samples

[2] P063: 7–15%, PI on P063: approx 16, LS on P063: 7,1–9,3%; PI on P063 x P063:160–180

[3] 20–65 Obtained. BSH MDD 1 836  $\text{kg/m}^3$ , OMC 12,2%

[4] BSH MDD 1 900  $\text{kg/m}^3$ , OMC 7,5%; soaked CBR 90, at OMC 110 ; Vibrating Hammer (BSVH) MDD 1960  $\text{kg/m}^3$ , OMC 7,3%; 4d soaked CBR 120, at OMC 120 (typical results)

The vane shear method (Marais, 1966; TMH6:1984) was used for the design and construction control of both the cold wet-mix and the foamed sections. A design value of at least 200 kPa at 100% MAASHO converted to 40°C using the Marais graph was required in both cases, as for a BT3 material as in what became the BRDM:1982 and TRH14:1985. In the case of the foam process a Hveem Rt value exceeding 78 was also required. However, only the vane method was used during construction.

## 7. CONSTRUCTION

Access ramps were constructed at intervals of 100 m to facilitate the turning of the stabilization train.

### 7.1 Wet-mix process (Sections 1–6)

The sand was brought up to above its optimum fluid content of 7% for maximum density to about 10% with water. The hot binder was then applied at about 100°C and mixed in with a Rex Pulvimixer and a motor grader. The mix was then aerated with the grader until vane shear tests showed that it had reached the optimum fluid content for 95-100% of the maximum vane shear strength and 80-90% of the maximum dry density (MDD) on compaction. This usually took 3-6 hours. This optimum varied between 4 and 6% depending on the binder. Initial compaction was carried out in three equal lifts.

Except in the case of Sections 1 and 6 (RC-250) the average binder contents were within 0,2% of those specified. In the former cases the measured binder contents were only 3,4% and 3,6% respectively in comparison with the optimum of 4,5% specified. Good and uniform coating of the sand with the binder was reported. The mean compacted thicknesses of 140 mm on the centreline compared reasonably with the design value of 150 mm. However, on Section 2 the sand-asphalt was only 100 mm thick in the outer wheelpath of the Kanye-bound lane, below which was a 50 mm-thick layer of clean sand.

The minimum specified density of 95% MAASHO was met in the case of all sections (95-99% achieved) except in the case of Section 1. This section had to be ripped three times because of poor levels and only 90% was achieved.

## 7.2 Foamed process (Sections 7–12)

After bringing the sand to its optimum of 7% moisture content it was then lightly compacted with a smooth steel-wheel roller. The hot binder at about 160°C for the bitumen and 140 °C for the tar, and water at 1 200 kPa were then applied and mixed in one pass, using the Pulvimixer alone. Compaction was carried out in a single lift.

The average binder contents achieved tended to vary more from those specified than in the case of the wet process. The actual thicknesses of Sections 10-12 were also about 30 mm thinner than specified. None of the sections met the minimum density requirement of 95% MAASHO and only 87-93% was achieved. Although the estimated expansion ratios of 8-10 times are just acceptable according to current guidelines (Asphalt Academy, 2009) foaming appeared inadequate and plums of bitumen in clean sand were common. However, there was a risk of damage to the tar if a higher temperature was used.

## 7.3 History

The sand-asphalt bases were constructed in April – May 1979 and primed within two weeks of construction, but only surfaced in October 1979 due to a shortage of surfacing chippings. The control Section A was constructed and surfaced in October 1979 and Section D in March 1980. No traffic was permitted on the sand-asphalt bases before surfacing, but cattle badly damaged the prime on Sections 7–12. After three months under traffic the sections were closed and finally reopened to traffic on the 28 March 1980.

## **8. MAINTENANCE**

June 1981: Shoulders regravelled with a ferruginised, weathered granite gravel over Sections 1–12 and side slopes flattened from 1 in 4 to 1 in 6 due to severe rainfall runoff erosion of the unsatisfactory calcified sand shoulders and the sand side slopes.

Distress was apparent on most of the sand-asphalt sections soon after opening to traffic and most required some patching. Sections A and D received a fog spray in July 1984, all sections were resealed with 10 mm chips and crusher dust in 1984, and Sections A and D received a slurry seal in 1990.

## **9. TRAFFIC**

Details of the traffic history of the experiment have been provided by Lionjanga et al (1987) and Greening and Rolt (2014). Regular automatic (up to 1985) and manual counts as well as axle load surveys were carried out.

The average annual daily traffic (AADT) in both directions rose from about 200 vpd in the first two years with 40% over 5 tons unladen mass to about 250 thereafter until 1986, after which it rose to about 400–550, the heavy vehicles remaining constant at about 20%.

In 1980 the average E80 per commercial vehicle in the Jwaneng lane was 4,3. After 12 years it had decreased to 2,0 with an average of 2,2 from year 7 to 12 due to the completion of the mine and town infrastructure. This had also decreased in the Kanye lane from 1,1 to 0,4 in year 12 of which was also the average between year 7 and 12.

The cumulative number of millions of equivalent standard 80 kN axles (ME80 or MESA) using a load equivalency exponent of 4,5 is shown in Figure 1 (courtesy Mr PAK Greening of TRL). The loading in the Jwaneng lane was about five times that in the Kanye lane. At the time of final monitoring after 13 years in 1993 the Jwaneng lane had carried about 0,4 and the Kanye lane only about 0,1 ME80. (As this was a research project the BRDM:1982 Dry region traffic reduction factor of 0,7 has not been applied to the results.)

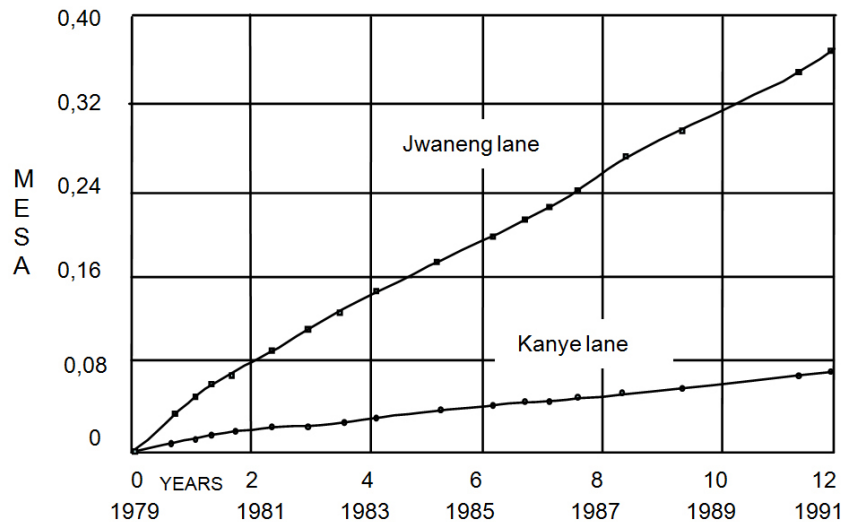


Figure 1: Cumulative axle loads (Courtesy PAK Greening)

## 10. MONITORING

Visual inspections by a panel consisting of representatives of the BRD, TRL and NITRR (usually Dr F Netterberg) were held once or twice a year nearly every year since construction in 1979 until March 1993, after which the whole experiment was rehabilitated. The structural pavement condition rating used (Table 3) was essentially a forerunner of TRH6:1985, TRH12:1984, Roads Department (2000) and TMH9:1992 (CSRA, 1992), supplemented by stringline and straight edge rut depth measurements.

Table 3: Jwaneng panel overall pavement condition ratings compared with TMH9:1992 [1]

Degree	Description (Panel) [2]	Description (TMH9: 1992)
Very good	No distress: Nearly perfect, no cracks or deformation. Ruts $\leq 10$ mm, normally $\leq 6$ mm	Very few or no defects. Degree of defects $< 3 <$ warning
Good	Minor distress: Only "sound" TRH12 ratings, no significant defects, i.e. $\leq$ Degree 2 (TRH6). Ruts $\leq 10$ mm	Few defects. Degree of structural defects mostly $<$ warning
Fair	Significant distress : Generally not severe, but ruts can be 10 – 20 mm	Few defects and seldom severe. Extent only local if degree is severe (excluding surfacing defects)
Poor	Considerable distress : Severe deformation, patching or rutting, or warning PSI. Ruts can be $> 20$ mm but no shears. (Ruts max. 30 mm added 1992-03-03)	General occurrence of particularly structural defects with degrees warning to severe
Very poor	Failed: Severe deformation, patching, rutting and/or PSI. One or more serious types of defects $\geq$ Degree 4. However, rutting alone not required	Many defects. Degree of majority of structural defects severe and extent predominantly general to extensive

Notes:

[1] Ratings are averages for the central 50 m of the section. [2] In general agreement with threefold TRL system (Kennedy and Lister, 1978). See TRH6:1985: & TRH12: 1984 for details, and use appropriate percentiles for ruts and TRH12 for extent

More comprehensive monitoring including rut depth and riding quality were carried out approximately every three years and deflections by the BRD /TRL team using the TRL transient deflection method with a 6 350 kg axle load and 590 kPa tyre pressure (Smith and Jones, 1980) at least once a year. Roughness was initially measured by means of a TRL rolling straight edge, later supplemented by a vehicle-mounted linear displacement integrator (Netterberg and Pinard, This Conf.). A 3 m-long straight edge as used for

construction control was initially used to measure the as-built transverse profile. This was later replaced by a 2 m straight edge and a 20 mm-wide wedge for rut depth measurements. Only one falling weight deflectometer (FWD) survey was carried out, in September 1990, and the control sections were unfortunately not tested.

Ad-hoc investigations of failures or distress were also carried out as necessary by various persons and their reports have been freely drawn upon in this paper.

## 11. RESULTS

A survey soon after construction (see also Netterberg and Pinard, This Conf.) showed that most of the sections were considerably more uneven both longitudinally and transversely towards their construction joints. Only data representative of the middle 50 m of each 100 m section were therefore considered in an attempt to eliminate the effects of the much rougher transition zones between the sections.

**Table 4: Summary of visual condition in 1986, 1989 and 1993 and inspection panel maintenance recommendations in 1993 after 13 years**

Section [1] No.	Jwaneng Lane					Kanye Lane					Rut Depth OWP 0,08M In 12 y [2] (80 %) mm	
	Rut Depth OWP 0,36 M in 12 y [2] (80 %) mm	General Pavement Condition after ME80 in years [3, 4]				Minimum Maintenance Recommended by Panel in March 1993 After 13 years	General Pavement Condition after ME80 in years [3, 4]					
		Panel			TMH 9		Panel			TMH 9		
		0,22	0,30	0,40	0,40		0,04	0,06	0,09	0,09		
	6	9	13	13		6	9	13	13			
A	6	VG	G	G	VG	None	None	VG	G	G	VG	8
1	18	F	P	VP	VP	Patch, reseal	Reseal	VP	F	VP	VP	14
2	6	G	G	F	F	Reseal	Reseal	G	G	F	F	6
3	14	F	G	F	F	Reseal	Reseal	F	G	F	F	5
4	10	VG	G	F	G	Fog	Fog	VG	G	F	G	5
5	10	VG	F	F	G	Fog	Fog	VG	G	F	G	8
6	10	P	P	P	F	Patch, reseal	Patch, fill rut, reseal	F	G	P	P	21
7	25	G	F	P	P	Fill rut, reseal	Fill rut, reseal	VG	G	P	P	24
8	33	F	P	VP	VP	Fill rut, reseal	Fill rut, reseal	VG	F	P	P	26
9	48	F	P	VP	VP	Fill rut, reseal	Fill rut, reseal	F	F	VP	P	38
10	53	P	P	VP	VP	Fill rut, reseal	Fill rut, reseal	F	F	P	P	27
11	40	P	P	VP	VP	Fill rut, reseal	Fill rut, reseal	F	F	P	P	23
12	31	F	F	VP	P	Fill rut, reseal	Fill rut, reseal	G	G	P	P	27
13	11	VG	VG	F	G	Slurry	Slurry	VG	G	F	F	12
D	8	VG	VG	G	VG	None	None	VG	VG	VG	VG	8

Notes:

[1] Sections 1 – 12 sand-asphalt, A & D gravel & 13 calcrete controls. [2] Over central 50 m in Oct. 1991;  $n = 21$ , t-distribution assumed for 80 %-ile. [3] Best average over central 50 m. Mostly structural condition assessed from visuals and rut depths in June 1986, Feb. 1989, Oct. 1993. [4] Panel and TMH 9:1992 methods: VG = Very good, G = good, F = fair, P = poor, VP = very poor

The riding quality in June 1986, as indicated by the Quarter Car Index (QI) and South African Present Serviceability Index (PSI) values of mostly 19–34 and 2,8–3,6 respectively were all sound for a TRH12:1997 Category C (PSI) > 2,0), or Botswana Category II road with only Section 11 with a PSI of 1,7 in a warning condition (Netterberg and Pinard, This Conf.). With PSIs of 3,3 and 3,7 respectively the control sections A and D were typical of good, newly constructed rural roads with surface treatments in South Africa.

## 12. DISTRESS

At the first inspection held in October 1979 just after surfacing all the sections were in good condition and apparently to the correct shape, but the construction transition zones between the sections were corrugated. Severe shoulder erosion was already evident, whilst the exposed edges of the bases of Sections 7–12 and the shoulders were soft.

Distress in the form of rippling and shoving of the surfacing, depressions and/or rutting and even shear failures up to 15 mm in depth were noticed on most of the sections except A and D (controls) and 2, 3 and 4 in February 1980 after only about 20 000 E80 in the Jwaneng lane. The worst distress was mostly confined to the first and last 20–30 m of each section and it was considered that some of this had been built-in. By April 1981 the distress had become worse over most of the sections (except 2, 3 and 4) and patching was evident. The distress on Section 1 (the first to be constructed) was largely in the more lightly trafficked Kanye lane and the Jwaneng lane was at first little affected. Distress continued to increase and by May 1982 after about 0,10 ME80 the Jwaneng lane of Section 10 and the Kanye lane of Section 1 had failed, after only 20 000 E80.

In 1984 after about 0,17 ME80 it was recommended that Section 1 should be reconstructed and that Sections 9, 10 and 11 should probably be reconstructed. It was agreed that only Sections A, 2, 3 and 4 and D had given satisfactory performance. However, only maintenance was carried out, all sections continued to be monitored, and in 1986 after about 0,22 ME80 the Jwaneng lane of Section 1 was rated as fair, with a rut depth of 11 mm. However, both lanes had been extensively patched and all sections resealed. All sections exhibited Degree 2 or 3 longitudinal and/or transverse cracking tending to a block pattern and on the poor sections after 10 years multiple longitudinal cracking in the wheelpaths had progressed to a TMH9:1992 Degree 3 to 4 crocodile pattern.

## **13. DISCUSSION**

### 13.1 Performance

In 1993 after about 0,4 ME80 in 13 years only Sections 4 (5,0% PVC tar) and 5 (5,0% RTL 35/40) were in a condition almost comparable with the control sections A and D and the nearest calcrete section (13). Sections 2 and 3 had given fair performance and Section 6 had performed fairly well in the Jwaneng lane but poorly in the more lightly trafficked Kanye lane. Sections 7–12 (all foamed) were in a terminal condition or had already failed, a rut depth of 20 mm being regarded as one terminal condition.

Early investigation of the distress on the cold wet-mix Sections 1, 5 and 6 found that it generally consisted of disintegration of the surface and top 20 to 30 mm of the base on a compaction plane and that the sand-asphalt was dry and brittle and lost cohesion after disturbance. In contrast, depressions and shear failures associated with binder contents 1,4–2,2% lower than the adjacent undistressed areas were more typical of Sections 7–12 (foamed asphalt) and blobs of binder were common, apparently due to poor foaming caused by an inadequate water pump. Fattening-up near the joints of most sections and erosion of the calcified sand shoulders was also common.

Table 5 shows a summary of the performance in comparison with the controls and the adjacent calcrete Section 13 arranged in order of their estimated total structural capacities, taking all the available information into account. These are based mostly on the performance of the more heavily trafficked Jwaneng lane on the assumption that most of the distress in the Kanye lane had been built-in.



**Table 5: Summary of performance and estimated residual and total structural capacities [1]**

Section No. [2]	Process or Material	As-built Base			Structural Condition and Estimated Residual Capacity									Estim. Total Cap. [1] ME80
		Binder		Mean Thickness. Comp. mm %	Visual after ME80 [4]			FWD after 0,33 ME80 (Sept. 1990) [5]						
		Mean %	Type		Cond.	Rut	Cap.	Max. Deflection			Base Layer Index			
				0,40 0,22 [3]				0,36 0,22 mm	0,40 ME80	D <sub>o</sub> µm	Cond. [6]	Cap. ME80	BLI µm	
A	Gravel Control	0	None	150 98	<b>VG</b> <b>VG</b>	6 6	1,6 2,6	-	-	-	-	-	-	2,0 3,0
4	Cold Wet	5,0	RTH 30/35 PVC	140 96	<b>G</b> CB3 <b>VG</b>	10 7	0,4 0,6	520	Warn.	0,7	300	Warn.	0,6	0,8 1,0
5	Cold Wet	5,0	RTL 30/40	140 95	<b>GP</b> CB3 <b>VG</b>	10 2	0,2 0,4	540	Warn.	0,7	310	Warn.	0,6	0,6 0,8
3	Cold Wet	5,2	RTH 35/40	140 96	<b>F</b> CB4 <b>F</b> CB3	14 4	0,1 0,2	660	Warn.	0,35	370	Warn.	0,35	0,5 0,7
2	Cold Wet	5,2	RTH 35/40	140 99	<b>F</b> CL3 <b>G</b>	6 3	0,1 0,2	920	Sev.	0,12	510	Sev.	0,15	0,5 0,7
7	Foam	3,9	60/70 pen.	150 91	<b>P</b> CL3 <b>G</b>	25 11	0 0,1	690	Warn.	0,30	420	Warn.	0,25	0,2 0,3
8	Foam	3,0	60/70 pen.	90 93	<b>VP</b> B4 CL3 <b>F</b> B4	33 14	0 0,1	590	Warn.	0,5	340	Warn.	0,5	0,2 0,3
12	Foam	4,6	RTH 55/60	120 87	<b>VP</b> CL3P2 <b>F</b> P1	31 20	0 0,1	720	Warn.	0,25	420	Sev.	0,25	0,2 0,3
6	Cold Wet	3,6	RC- 250	140 87	<b>FP</b> 3CL3 <b>P</b> P3	10 9	0 0	690	Warn.	0,30	430	Sev.	0,25	0,1 0,2
9	Foam	4,1	60/70 pen.	70 90	<b>VP</b> CL4B4 <b>F</b> B4	48 20	0 0,1	640	Warn.	0,40	380	Warn.	0,35	0,1 0,2
11	Foam	4,1	RTH 55/60	70 90	<b>VP</b> CL3P3 <b>P</b> P2	40 30	0 0	590	Warn.	0,5	340	Warn.	0,4	0,1 0,2
1	Cold Wet	3,4	RC- 250	140 90	<b>VP</b> P5 <b>FP</b> 5CL3	18 11	0 0	1020	Sev.	0,09	740	Sev.	0,06	0,05 0,1
10	Foam	4,9	RTH 55/60	40 89	<b>VP</b> CL3P3 <b>P</b> P2	53 30	0 0	620	Warn.	0,40	370	Warn.	0,35	0,05 0,1
13	Powder Calcrete	0	None	150	<b>G</b> <b>VG</b>	11 6	0,4 0,6	890	Sev.	0,20	670	Sev.	0,12	0,8 1,0
D	Gravel Control	0	None	150 98	<b>VG</b> <b>VG</b>	8 6	1,6 2,4	-	-	-	-	-	-	2,0 3,0

Notes:

[1] Based mostly on performance in Jwaneng lane after 0,40 ME80. [2] Gravel subbase on Sections A and D, calcified clayey sand on 1 and 2 and Kalahari sand on others. [3] March 1993 : TMH9:1992 method after 0,4 ME80 in Jwaneng lane; May 1986: panel method after 0,2 ME80 in Jwaneng lane. VG = Very good, G = Good, F = Fair, P = Poor, VP = Very poor. Distress mode codes: B = Bleeding in wheelpath, CB = cracking, block, CL = Cracking, longitudinal, multiple, in outer wheelpath, tending to crocodile; TM9 Degree 3 or 4; P1 = Patching in wheelpath < 5 %, P2 5 – 10 %, P3 > 10 – 20 %, P4 >20 – 50 %, P5 > 50 % of area. [4] 80 %-ile in Oct. 1991 in outer wheelpath (OWP) of Jwaneng lane, n = 21, n = 5 in May 1986. [5] Normalised to 40 kN, mean of 3 in OWP of Jwaneng lane at about 35°C; condition criteria according to Horak (2008); residual capacity estimates according to TRH 12:1997. [6] D<sub>o</sub> <500 sound, 500 – 750 Warning, > 750 Severe. [7] BLI < 200 sound, 200 – 400 Warning, > 400 Severe

Two capacity estimates are provided. The lower one is for a terminal 80%-ile rut depth of 20 (TRH 4, TRH12) or 25 mm (Roads Dept, 2000) or a maximum total area of patching and severe distress of 20% for a TRH4 Category C or a Botswana Category II road and the higher one to a maximum 80%-ile rut depth of 30 mm or 50% of patching and severe distress probably adequate for a TRH4 Category D road in a dry environment.

The best sections were Sections 4 (PVC tar) and 5 (RTL 30/40), both with an estimated total capacity of about 0,8 ME80. With the exception of Sections 1 and 6 the performance of the wet-processed sections was far better than the foamed sections which, however, were less well-compacted and mostly thinner. The best sections 2, 3, 4 and 5 with an estimated total capacity of at least 0,5 ME80 were all 140 mm thick, had a binder content

of at least 5,0%, a compaction of at least 95% and a mean and minimum in-situ strength at 7 days of at least 300 and 250 kPa, respectively. Those with a capacity of 0,2 ME80 or less all had binder contents of less than 5,0%, were all compacted to 93% or less, were mostly thinner and/or mostly had mean and minimum strengths of less than 300 and 250 kPa, respectively. Those with a capacity of 0,1 ME80 or less were all compacted to 90% or less.

Capacity estimates from FWD data can be embarrassingly inaccurate (Horak, 2008). In this case only those for Sections 4 and 2 agreed reasonably with those estimated from performance data, whilst those which had already failed in terms of rutting and/or other criteria were mostly optimistic. A surprising result was the plastic powder calcrete Section 13 which had performed very well, but which received worse  $D_0$  and BLI ratings and only slightly better middle (MLI) and lower layer (LLI) index ratings (not shown), to the far poorer sand-asphalt Section 12, even at the end of the dry season.

No adjustments for temperature, seasonal effects or past traffic have been made to the FWD results in Table 5. However, previous work on Sections 5 and 12 showed that Benkelman deflections were little affected by temperature, at least up to 40°C at a depth of 40 mm. Daily temperature ranges recorded at a depth of 40 mm were about 5–35°C in winter and 20–55°C in summer.

### 13.2 Causes

As much of the early distress was in the form of localised shear failures, rutting, depressions and rippling or shoving, in 1986 after about 0,22 ME80 in the Jwaneng lane, a selection of unpatched “good” and “bad” sites was further investigated by means of DCP testing and coring for binder content, thickness and density determinations and visual examination. At least five sites on each section, with two in the outer wheelpaths on each lane and one on the centreline were investigated, a total of 64 sites and cores.

Table 6 shows a summary of the visual and coring work in both lanes in comparison with the design and the as-built results and the individual results for the outer wheelpath of the Jwaneng lane only. The DCP results were unfortunately never processed. Although many of the binder contents appeared low due to incomplete extraction, the generally lower than the as-built design contents and thicknesses were confirmed, together with an extremely high variability and poor binder distribution, but usually good compaction, both in the wheelpaths and on the centreline (not shown). Although mean residual binder contents of at least 3,0% were found after construction, individual results down to 0,8% with many less than 2,0% (even in good areas) were found. The individual results indicate that adequate thickness and compaction rather than binder content and distribution were the most important factors and that an attained minimum residual binder content of 1,5–2,0%, a thickness of 75 mm and compaction of 95% would ensure a performance which is still good at about 0,2 ME80 with a capacity to a rut depth of 20 mm of at least 0,3 ME80. Compaction to 100% should permit a thickness of only 50 mm to be used. In all cases the distress appeared to be confined to the base course and the subbase was unaffected.

The cold wet process, had been successfully used on an experiment in Namibia (Marais and Freeme, 1977), but the construction team had no past experience of the then new foamed asphalt process and such distress should now be preventable by the application of guidelines such as those of Morton et al (2004) and the Asphalt Academy (2009).

## 14. SAND SUBBASE AND SHOULDERS

Both the Benkelman beam deflections (not shown) and practically all the FWD parameters (Table 5) including the MLI and LLI indices (not shown) for Sections 3 and 6 with a Kalahari sand subbase were lower than those for the respectively nominally identical Sections 1 and 2 with a calcified clayey sand subbase. At 27  $\mu\text{m}$  only the LLI of Section 1 was lower than those of the two sand sections (49 and 26  $\mu\text{m}$ ). Indeed, practically all of the beam and FWD deflections of Sections 3–12 on the sand subbase were lower – including the average MLI of 156 – than those of Sections 1 and 2 on the calcified sand subbase with an average MLI of 254 – even though the latter was compacted to 101 % against the mean of 97% MAASHO for the Kalahari sand subbase.

The deflections are supported by the far higher CBR of the Kalahari sand of 90 in comparison with the 38 of the calcified sand (Table 2), DCP testing in March 1985 which showed the average in- situ DCP CBR of the subbase to Sections 1 and 2 to be only 10, in comparison with the 46 of Sections 3 and 4, and the absence of any shears or rutting extending into the subbase found on any sand-asphalt or calcrete section.

It is unnecessary to import a subbase as the use of well-compacted A-2-4(0) Kalahari sand was adequate, for this category of road and traffic at least and, if confined, could also be considered for base course for a low volume road. (Such an experimental base has since proved successful for over 20 years (Roads Dept, 2010; Paige-Green et al, 2015; Netterberg et al, 2016). However, at 1 in 4 the sand sideslopes became badly eroded by rainfall and were flattened to 1 in 6. Although not evaluated as such, for the same reason this sand is unlikely to be suitable as shoulder material unless sealed or effectively grassed.

**Table 6: Comparison of base course properties as-built and by coring in June 1986 [1]**

Section No.	Both lanes (0,22 ME80 in Jwaneng and 0,04 ME80 in Kanye lane) [1]								Jwaneng lane on cores in outer wheelpath					
	Structural Condition [2] Lane Jwaneng Kanye	Binder Type	Binder Content		Thickness		Compaction		Surface		Binder		Thick-ness mm	Com-paction [5] MAASHO %
			Design [3] Total As-Built Mean	Cored [4] Residual Min-Max Mean	Design	Cored	Design	Cored [5]	Con-dition [2]	Rut Depth	Content [4]	Distri-bution.		
			%	%	As-Built Mean mm	Min-Max Mean %	As-Built Mean MAASHO %	Min-Max Mean MAASHO %	-	mm	%	-		
1	Fair V. Poor	RC-250	4,5 3,4	1,8-2,5 2,2	150 140	113-134 123	95 90	92-96 94	Good Rut	10 12	2,0 2,0	Good Good	125 113	95 93
2	Good Good	RTH 35/40	5,0 5,2	2,4-3,6 3,1*	150 140	122-143 132	95 99	98-102 100	Good Good	0 0	2,4* 3,6*	Poor -	122 -	102 101
3	Fair Fair	RTH 35/40	5,0 5,2	2,2-4,5 3,4*	150 140	96-126 113	95 96	101-102 101	Good Good	3 4	4,5* 3,6*	- Poor	96 113	101 101
4	V. good V. good	RTH 35/40PV	5,0 5,0	4,0-5,3 4,5*	150 140	103-125 114	95 96	102-106 104	Good Good	3 4	4,0* 5,2	Fair Good	115 115	104 105
5	V. good V. good	RTL 35/40	5,0 5,0	1,4-2,7 2,6*	150 140	98-190 161	95 95	95-97 96	Good Good	0 0	1,4* 2,7*	Poor Poor	180 185	96 97
6	Poor Fair	RC-250	4,5 3,6	1,0-8,5 3,3*	150 140	145-200 178	95 87	93-95 94	Good Shear	3 11	- 1,6	- -	181 145	- 94
7	Good V. Good	60/70 pen.	3,5 3,9	3,2-5,1 4,4*	150 150	96-127 113	95 91	91-97 94	Good Good	0 8	- 5,1	Poor Poor	96 105	- 97
8	Fair V. Good	60/70 pen.	3,5 3,0	1,7-4,2 3,0	100 90	70-110 84	95 93	97 97	Good Rut,B4	0 13	1,7 -	Poor Good	98 75	- -
9	Fair Fair	60/70 pen.	3,5 4,1	2,2-5,3 3,6	75 70	35-60 46	95 90	93-98 96	Good Rut	8 18	2,2 -	V. Poor V. Poor	60 40	97 97
10	Poor Fair	RTH 55/60	4,5 4,9	0,8-3,3 2,1*	75 40	40-52 43	95 89	93-97 96	Rut Rut	14 18	2,1* -	Poor Poor	52 40	93 99
11	Poor Fair	RTH 55/60	4,5 4,1	1,2-1,7 1,5	100 70	25-55 35	95 90	99-104 101	Good Good	2 8	4,0 1,2	Poor Poor	25 55	99 100
12	Fair Good	RTH 55/60	4,5 4,6	0,8-6,4 5,8	150 120	75-90 79	95 87	97-100 99	Good Rut Shear	7 14 14	2,6 3,5 3,8	Poor Poor Poor	75 85 70	100 98 100

Notes: [1] Dry cored;  $n = 4-7$  for binder contents, 5–7 for thicknesses, 4-5 for densities per section, including centreline. [2] Panel method. [3] Binder content: Design = optimum; as-built by TMH1:1979 Method CC (7b), corrected for solubility of fresh binder in toluene (bitumens 100%, RTL 35/40 97%, RTHPV 90%, RTH 35/40 88%, RTH 55/60 85%, [4] On cores after removing surfacing (15mm) and primed base; by TMH1:1979 CC7(b), solubility - corrected as for fresh binder; result assumed residual for cutbacks (i.e. total corrected for solubility less volatiles); \* = probably low. [5] Waxed method for density after drying at 50°C (2-5% volatiles found)

## 15. DERIVED KALAHARI SAND-ASPHALT BASE COURSE DESIGNS

Table 7 shows recommended, conservative base course designs using the vane shear method for a BRDM Category II or TRH4 Category C road. They are neither a slight on particular binders nor on the process used, but simply what could be derived from the results available, which were complicated by the construction defects. Better performance would be expected from all of them without such defects and either process with any suitable binder could probably be used except possibly for 0,8/1,0 ME80. However, these designs appear compatible with those for a BSM3 material on a >15 foundation CBR for up to 0,3 and 1,0 ME80 in the Asphalt Academy (2009) guide. In all cases an optimum quantity of binder is intended, with a minimum mean of that actually used on the particular section, and 2,0% residual for the 0,1/0,2 and 0,2/0,3 classes, all provided that the strength requirements are met. Although a respective compaction of only 90% and 93% for the 0,1/0,2 and 0,2/0,3 ME80 classes appears feasible, 95% is recommended for all.

**Table 7: Derived Kalahari sand-asphalt designs**

Cumulative Max. ME80 / lane Distress [1]		Thick-ness mm	In-Situ 7-day Vane Shear Strength @ 40°C		Represented by		
			Min. kPa	Mean kPa	Section No.	Binder [2] Type %	
20 %	50 %						
0,1	0,2	50	100	200	10, 11 9	RTH 55/60 60/70 pen.	4,9; 4,1 4,1
0,2	0,3	75	150	200	8, 9 11	60/70 pen. RTH 55/60	3,5 4,1
		100	250	300	7, 8	60/70 pen.	3,9; 3,0
		150	150	200	12	RTH 55/60	4,6
		150	250	300	6, 1	RC-250	3,6; 3,4
0,5	0,7	150	250	300	3, 2	RTH 35/40	5,2; 5,2
0,6	0,8	150	250	300	5	RTL 35/40	5,0
0,8	1,0	150	250	300	4	RTH 30/35 PVC	5,0

Notes :

[1] Max. 80 %-ile rut depth 20 or 30 mm or 20 or 50 % total area of patching and severe distress in 20 years, respectively

[2] Residual binder (total binder less volatiles for cutbacks), min 2,0 % actually attained for 0,1/0,2 and 0,2/0,3 classes

Sand to be treated and for untreated subbase and subgrade: AASHTO M145 class: A-2-4(0) with P075 10-20 %, PI NP-3.

Compaction: Min. 95 % MAASHO for sand-asphalt for all traffic classes

Compaction for subbase and lower layers for A-2-4(0) Kalahari sand (100% MAASHO for A-3 sand for fill and upper 0,5 m of roadbed):

Traffic category (ME80) :	≤ 0,5	≥ 0,5
Minimum subbase compaction	93	95
Minimum subgrade & fill compaction	90	93
Minimum roadbed compaction	90% in top 0,5m	90% in top 0,5 m, 85% from 0,5-1,0m

Shoulders : Gravel or sealed sand-asphalt

## 16. CONCLUSIONS

- 1 The experiment has shown that a Kalahari sand-asphalt base course on a well-compacted Kalahari sand subbase and lower layers with a structural capacity of about 1,0 ME80 is viable and pavement designs with thicknesses ranging between 50 and 150 mm for capacities between 0,1 and 1,0 ME80 have been derived.
- 2 The best sections were those with 5,0% RTH 30/35 with 1,5% PVC or 5,0% RTL 30/40, both 140 mm-thick compacted to 95 % MAASHO, with mean in-situ 7-day vane shear strengths of at least 300 kPa, and a capacity of 0,8–1,0 ME80.
- 3 The use of a gravel subbase is unnecessary provided the Kalahari sand is well compacted, preferably to at least 97% MAASHO. The ease with which this A-2-4 (0) sand could be compacted indicates that 100% should be specified, at which a laboratory CBR of over 80 and in-situ DCP CBRs of 45 or more were obtained, and which has even proven adequate as base course for a low volume road.

4. The form taken by the failures on the cold wet-mix sections was generally that of disintegration and slippage of the upper lift of base course and that on the foamed asphalt sections that of rutting and shearing.
5. The primary causes of these failures were inadequate thickness and/or compaction. Contributory factors were low mean and localized low and/or ineffective binder contents on most sections, compaction planes on the cold wet-mix sections and possibly erosion of shoulder support in general, and a weak subbase on Section 1.
6. Most of the causes of the distress were traced back to construction defects and it is clear that the use of the correct equipment and good construction is crucial to the successful performance of sand-asphalt and that both it and untreated sand must also be protected from damage by the hooves of cattle before covering or sealing.
7. The Sections which performed well had a good thickness, high binder content and were well-mixed and well-compacted on a well-compacted sand or calcified sand subbase, i.e. they complied with the design specifications, whereas most of the other sections did not. The general validity of the original designs is thus confirmed, although they could usefully be modified from this experience.

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