CURING RATES OF EMULSION-TREATED KALAHARI SAND BASES

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ABSTRACT

The curing rates of ten full-scale experimental road base sections of nonplastic, grey-white and red Kalahari sands treated with 2,5, 5,0 and 6,5% of SS 60 emulsion without added cement or inert filler were monitored by means of in-situ vane shear and Clegg Hammer tests at intervals of 1, 2, 4, 8 and 20 weeks. Rainfall at about ten days after compaction on the unprimed bases seriously weakened the grey sand ETB sections to less than that of the untreated red sand control section but had less effect on the red sand sections. After about 8 weeks only the red sand sections had achieved their design strength of 200 kPa at 40ºC. The sections were primed after 14 weeks and, with shear strengths of 230 – 475 kPa in comparison with the 440 kPa of the untreated nonplastic red sand, all had easily exceeded their design strength when surfaced after 20 weeks at FFCs of about half their OFCs. However, Clegg Hammer CBRs uncorrected to 40ºC were mostly only 48-74, with only the 2,5 and 6,5% red sand sections − as well as the untreated, red sand− exceeding an in-situ CBR of 80. Clegg Hammer impact values correlated well with vane shear strength and may be a more convenient alternative to the vane shear test. The red sand ETBs were less temperature-sensitive, less affected by rain, and cured more rapidly to higher early strengths than the grey sand ETBs, probably due to a positive charge imparted by the sesquioxides accelerating the breaking of the emulsion, but did not all develop higher strengths when surfaced after nearly five months. With anionic emulsion the red sands are to be preferred to the grey, for which a cationic emulsion should be more suitable. Specifications for an untreated red sand base course in terms of vane shear strength or Clegg Hammer tests are derived.

1. INTRODUCTION

Materials treated with stable grade anionic emulsion without added cement may take a few months to a few years to achieve their full strength (Marais & Tait, 1989). Although they suggested that this should only take six months in a dry climate, a period of dry curing is usually still necessary before they can be sealed and trafficked. This requirement can be a disadvantage to using such treatment.

The curing rates of the two most common types of Kalahari (Setswana: *Kgalagadi*) sand in Botswana identified most easily by colour as grey to white, and red treated with 2,5 – 6,5% of SS60 emulsion (i.e. 1,5 – 4,0% net bitumen) used in ten full-scale, experimental, long-term pavement performance (LTPP) sections on the Serowe-Orapa road in Botswana were therefore monitored during the curing period of April – August, 1989.

Although both G7 sands were nonplastic and had similar remoulded engineering properties as determined by the standard road indicator and California bearing ratio (CBR) tests, the red sand contained about 2 – 3% of free Fe_2O_3 and Al_2O_3 (collectively called sesquioxides) and exhibited somewhat better CBR penetration characteristics and unsoaked CBRs than the grey sands.

As full details of the experiment and the sands have been provided elsewhere (Netterberg *et al.*, 2021a, b) it is the purpose of this paper only to report on the curing of the emulsion-treated bases (ETBs) using an untreated red sand section as a control.

2. CLIMATE AND WEATHER

The area lies within the Botswana Road Design Manual (BRDM) (1982) and the Committee of Land Transport Officials (COLTO) TRH4:1996 dry macroclimatic region for pavement design purposes. The mean annual rainfall of about 400 mm falls almost entirely between November and March with usually less than 25 mm in April, less than 10 mm in September and practically nothing from May to August.

As the relative humidity at midday in winter seldom rises much above 30% evaporation is high – the mean annual pan evaporation of 3 000 mm is about five times the rainfall and only varies between a low of 180 mm for the month of June and a high of 340 mm in October.

The mean annual shade air temperature is about 20ºC, with a mean daily maximum and minimum of about 28 and 13ºC, respectively. The highest and lowest temperatures of about 30 – 32ºC (occasionally up to 40) and 3 – 10ºC (occasionally down to –5) occur during September to February and May to August, respectively.

The only significant rain reported during the curing period was “showers” on April 21, 1989. During this period 58 mm was recorded during April at Orapa and 29 mm in October with nothing in-between.

Extrapolation of the maps in Sabita (2020a) suggests that the 7-day average maximum asphalt temperature at a depth of 20 mm and the 1-day minimum temperature at the surface in this area should be about 59 – 60 and 1 − 3ºC respectively, and a design summer temperature of 40ºC for ETBs has been suggested (Marais & Tait, 1989).

According to an extrapolation of the map in TRH3:2007 (Sanral, 2007), the temperature for the correction of ball penetration tests should be between 50 and 55ºC.

3. METHODS AND RESULTS

Tests for in-situ vane shear strength (S) according to Method ST7 in TMH6:1984 National Institute for Transport and Road Research (NITRR), 1984), (developed by Marais (1966) specifically for sand-bitumen mixes), temperature, field fluid content (FMC), and in-situ CBR using a 4,5 kg Clegg Hammer Impact Soil Tester (CIST) with a 50 mm-diameter foot (Clegg, 1983, 1999) at intervals of approximately 7, 14, 28, 60 and 140 days (i.e. almost 5 months) after construction are shown in Table 1.

Table 1: Curing rates of emulsion-treated bases [1]

BINDER (net bit.)	Upper / Lower	Units %	2	3	4	5	6	7	8	9	10	11	12
			← GREY – WHITE SAND			→	← RED SAND						→
	Upper		3,0	1,5	1,5	3,0	3,0	1,5	4,0	3,0	1,5	1,5	0,0
	Lower		1,5	1,5	0,0	0,0	0,0	0,0	2,0	1,5	1,5	1,5	0,0
CBR (lab.) [2]		%	52	48	48	39	39	53	29	29	42	42	103
Compaction[2]		%	103	103	102	101	102	101	104	103	93	99	99
FFC/FMC [3]		%	4,4	4,8	4,2	4,3	5,9	6,0	6,2	5,2	3,6	4,7	4,3
S (in-situ) [4] @	**7 d.**	kPa	175	211	138	148	186	-	-	-	-	-	379
	Temp.	ºC	20	31	26	26	29	-	-	-	-	-	22
[5]@	**40ºC**	kPa	**51**	**120**	**58**	**62**	**151**	-	-	-	-	-	-
[4]	**14 d.**	kPa	153	192	146	183	216	286	339	290	408	409	89 [11]
	Temp.	ºC	20	20	21	21	19	23	34	26	25	24	19
[5]@	**40ºC**	kPa	**44**	**56**	**44**	**55**	**145**	**206**	**302**	**223**	**306**	**303**	-
[4]	**28 d**.	kPa	254	260	146	277	261	183	321	273	120	284	158 [12]
	Temp.	ºC	20	18	**18**	19	18	18	18	21	18	16	18
[5]@	**40ºC**	kPa	**56**	**65**	**37**	**75**	**172**	**121**	**212**	**191**	**79**	**182**	-
[4]	**60 d**.	kPa	342	533	389	287	348	300	444	327	225	320	440
	Temp.	ºC	16	13	15	12	13	18	13	13	15	15	9
[5]@	**40ºC**	kPa	**75**	**96**	**82**	**49**	**209**	**198**	**266**	**196**	**142**	**202**	-
[4]	**140 d.**	kPa	364	600 +	482	424	288	530	594	436	282	565	440
[6]	Temp.	ºC	35	28	37	39	29	19	29	20	30	31	30
[5] [6]@	**40ºC**	kPa	**266**	**282**	**400**	**399**	**233**	**355**	**481**	**301**	**231**	**480**	-
CBR (in-situ) [7]	**140 d.**	%	55	72	67	67	59	67	107	74	48	83	86
[8]	**140d.**	%	47	78+	63	55	37	69	77	57	37	73	57
FFC / FMC [9]	**140 d.**	%	3,5	1,6	2,1	2,3	4,0	3,1	3,6	3,4	5,6	2,8	3,4
CBR (lab.) [10]	**30 d.**	%	-	-	25	40	64	40	53	40	49	22	-
	60 d.	%	-	-	87	143	130	80	87	88	40	20	-
UCS (lab) [11]	**30 d.**	MPa	1,00	0,70	1,00	1,10	1,80	1,40	1,85	1,60	1,25	1,50	-
	60 d.	MPa	0,90	0,95	0,82	1,00	2,00	1,20	1,75	1,60	-	1,30	-

Notes:

[1] All sections 150m in length; constructed 03-12 April 1989, primed 22 July after 101-110 days, surfaced 30-31 August after 140-149 days.
[2] At 98% MAASHO, standard soaked CBR procedure, n = 1 for CBR; mean of n = 8 for compaction (% MAASHO MDD), according to TMH1:1986; 103 for Section 12 probably at 100%
[3] Field fluid (FFC) or moisture (FMC) content at 105 - 110 ºC from field density tests after compaction; mean of n = 9.
[4] Vane shear strength (S) (TMH6:1984) at road temp. after approx. no. of days curing indicated; final one at 140 days just before first seal spray on 30 Aug. Vane and thermometer in **centre of treatment**, mean of 10 positions in lanes (shoulder ETBs excluded) x 5 measurements, i.e. n = 50.
[5] Corrected to design 40°C using separate laboratory-derived relationships for grey and red sand (Eqs 1 and 2).
[6] Road surface temp. for ball penetration tests (TMH6:1984) used.
[7] Clegg Hammer-derived CBR using the fourth blow (CIV) and Eq.3, **not** corrected to 40 °C; mean of 8 positions in lanes (shoulders excluded) x 5 measurements, i.e. n = 40.
[8] Calculated using Eq. 4 for traditional (not DCP) surface in-situ CBR using uncorrected S as in Note 4.
[9] FFC/FMC; mean of n = 3.
[10] At 100% MAASHO, unsealed shelf curing in site lab at about 15°C for period shown and soaking for 4 d. for CBRs and 4–6 h. for UCS, otherwise according to TMH1:1986; **not** corrected to 40°C.
[11] On 22 April after rain on 21 April.
[12] On 15 May, 24 days after rain.

Attempts to derive reliable, in-situ temperature-correction factors for both shear strength and CBR were mostly unsuccessful due to an insufficiently wide temperature range and variable results. Only the relationships for the grey sand Section 3 with 1,5% net bitumen in the whole base (1,5%) and the red sand Sections 9 with 3 and 1,5% net bitumen in the upper and lower half of the base respectively (3/1,5%) and 11 (1,5%) appeared reasonable and – with considerable extrapolation – were fairly close to the laboratory-derived relationships, although the few valid results for the red Section 8 (4/2%) suggested a relationship closer to that for the grey sand ETBs.

The shear strengths at the temperature of test (t) during curing were therefore all corrected to the design temperature of 40°C using the following laboratory-derived factors (F):

$$\text{Grey sand ETBs: } \log F = (0{,}0275\,t - 1{,}097) \text{ or } F = 0{,}0812e^{0{,}0626t} \qquad (1a, 1b)$$

$$\text{Red sand ETBs : } \log F = (0{,}00797\,t - 0{,}319) \text{ or } F = 0{,}4746e^{0{,}0184t} \qquad (2a, 2b)$$

and are embolded in Table 1 for ease of comparison.

From these equations partly supported by the field work it is clear that the grey sand ETBs were far more temperature-sensitive than the red. For example, the factor to convert from 20 to 40ºC was 0,28 in the case of the grey but only 0,69 for the red.

Laboratory testing for soaked CBR and soaked unconfined comparison strength (UCS) after open shelf curing for 7, 14, 30 and 60 days was also carried out, essentially following guidelines later included in Sabita (1999) Manual 21. However, only the results at 30 and 60 days are shown in Table 1.

In converting the fourth hammer blow (CIV) readings to CBR the later Clegg (1999) relationship was used:

$$CBR = (0{,}24\,CIV + 1)^2 \qquad (r = 0{,}957) \qquad (3)$$

In using this relationship the user should be aware that Australian practice is to report the CBR at whichever is the higher at the 2,54 or 5,08 mm penetration depth whereas in South Africa only the CBR at 2,54 mm is used. As the 5,08 mm CBR is on average about 27% higher for typical South African materials (Pinard & Netterberg, 2012), Equation 3 is probably significantly overestimating the CBR in South African terms. (In the case of untreated Kalahari sands and lightly cemented materials the 5,08 mm CBR is usually lower than that at 2,54 mm.) A few unpublished results in the author's possession suggest that the CIV does indeed overestimate the South African CBR by about 25% on a sand-bitumen base course.

Also shown in Table 1 are traditional (i.e. not DCP) in situ surface CBRs at about day 140 calculated from the uncorrected vane shear strength results at the road temperature using a relationship derived by the author from a graph in Marais (1965) on the Andoni sand-bitumen bases (Equation 4):

$$CBR = 0{,}13S \qquad (4)$$

Equation 4 should be used with caution as it was derived from CBRs converted to a surface temperature of 40ºC and vane shear strengths and temperatures at 75 mm corrected to 40ºC and was not used by Marais and Freeme (1977).

The results nevertheless appear quite reasonable and it is also significant that, on average, the Australian CIV-derived in-situ CBRs are 25% higher than the shear strength-derived from South African in-situ CBRs.

4. PREVIOUS WORK

The laboratory design strength of a minimum of 200 kPa at 40ºC (BRDM, 1982; TRH14:1985) was derived from the performance in terms of early rut depths of 18 sections of mostly grey-white Kalahari sand base courses with and without 15% added inert powder calcrete filler on the Andoni LTPP experiment in Namibia which included four sections treated with 4,5 and 6,5% cationic bitumen emulsion (Marais & Freeme, 1977), and appears to be the only case study of Kalahari sand ETBs. The experiment was monitored for eight years, two of which were under a light Kalahari sand seal and six under 30 mm of asphalt and 0,1 MESA in **both** lanes, with a maximum rut depth of about 7 mm after eight years.

Laboratory strengths − apparently means − of less than 130 kPa at 40ºC were regarded by these authors as “unsatisfactory” and those between 130 and 200 kPa as “critical” with respect to the rut depth developed. The six sections regarded by them as having given good performance with respect to both deformation and crack resistance had laboratory strengths of 220 − 300 kPa, and 242 − 380 and 314 − 555 kPa after one and two years, respectively. Of these, the one “good” ETB had strengths of 130 kPa after compaction, and 280 and 314 kPa after one and two years, respectively. Although no laboratory strength was reported, it was probably about 200 kPa.

The author’s interpretation of their results is that only the three sections which subsequently cracked badly had laboratory strengths exceeding 250 kPa (three out of seven such sections) **and** developed field strengths – apparently means – exceeding 280 (out of four such sections) **and** 400 kPa (also out of four such sections) after one and two years, respectively. None of the ETBs cracked significantly, all of which had laboratory strengths of 80 – 200 kPa and 170 – 280 and 235 – 345 kPa, after one and two years, respectively. Only the strongest one with 4,5% emulsion with filler (345 kPa) cracked slightly whereas the equivalent one without filler (130, 170 and 235 kPa, respectively) did not.

5. DISCUSSION

After 14 days in the last week of April all the red sand sections except Section 6 (3/0%) had achieved their design strength of 200 kPa at 40ºC at a mean base course temperature of 24ºC and could have been primed and sealed if the field fluid content had decreased sufficiently. At that time the white sand sections had only achieved 44 - 56 kPa – less than the 89 kPa of the untreated control Section 12 after rain on the 21 April. (Proof-rolling or penetration tests for strength are now recommended for ETBs rather than drying back to about half OFC (Sabita, 2020b).

However, the red sand Section 6 had been tested the day after the rain and the other red sand sections just before or up to 4 days before the rain and the white sand sections 3 – 4 days after the rain. When tested at about 28 days in the first week of May all the red sand sections except Section 6 (which gained strength) had lost about one-third of their shear strength, with Section 10 with an old emulsion having lost more than two-thirds. At 212 kPa only Section 8 (4/2%) had maintained a strength above 200 kPa, with Sections 9 (3/1,5%) and 11 (1,5%) not far behind, but there was little improvement in the white sand sections.

The importance of protection from rain during curing by priming as soon as feasible was therefore established – even in the case of the red sand ETBs. (However, surface enrichment of ETBs during construction rather than priming is now recommended (Sabita, 1999; 2020b) and a cutback primer should not be used (Sabita, 2020b).

With 196 – 266 kPa all the red sand sections had achieved 200 kPa at about 60 days except Section 10 (1,5%) with the old emulsion. (This concern was the reason for repeating it as Section 11 with a new emulsion.) At that time the white sand sections had only achieved 49 – 96 kPa and took (an interpolated) approximately four months to achieve 200 kPa.

After 140 days with 231 – 481 kPa developed at an average base course temperature of about 21ºC when sealed at FMCs of mostly 1,6 – 4,0% (all except Section 10 at less than 60% of OFC) all the sections had more than achieved their design strength. Whilst the strength of most of them was less than the 440 kPa of the untreated control Section 12, the rain showed that it could not be maintained when wet.

However, even at these strengths ball penetration tests taken just before surfacing showed the necessity of taking potential embedment into account in the design of the seal (Netterberg *et al.*, 2021a).

The in-situ CBR (not corrected to 40ºC) after 140 days determined by Clegg Hammer varied between 55 and 72 on the grey sand ETBs and 48 – 107 on the red, or 59 – 107 if the old emulsion Section 10 is disregarded, in comparison with the 86 of the untreated control Section 12.

The traditional in-situ CBR calculated from the uncorrected shear strength after 140 days varied between 55 and 78+ for the grey sand ETBs and 37 and 77 on the red, on average about 20% lower than the CIV-derived CBRs. These were all much higher than the 4 – 23 of the four ETBs on the Andoni experiment before opening to traffic (Marais, 1965), although those were at about 25 days and corrected to 40ºC.

Comparison of the laboratory shelf-cured CBRs and UCSs at 30 and 60 days in Table 1 shows that the CBR increased substantially in curing for a further 30 days whereas the UCS did not. The Sabita (1999) recommendation of a 28-day curing period was therefore adequate for a UCS but too short for a CBR.

Considering the CBR requirements alone, all the 60-day CBRs of Sections 4 – 9 (although at 100%) met the Sabita (1993) requirement of a minimum 98% CBR of 80 for an E2 ETB suitable for 0,8 – 3 MESA and Sections 5 and 6 the minimum of 100 at 100% for an E3 material for 3 – 12 MESA (later (Sabita, 1999) downgraded to E2 and up to only 0,3 MESA for a Category C road on a COLTO:1996 G6 quality subbase). (No results were available for Sections 2 and 3.)

In terms of UCS alone all sections except Section 3 met the minimum 1993 Sabita requirement of 1,0 MPa for a G3 material code, but only the red sand sections met the requirement of 1,2 MPa for a G2 code and Sections 6, 8 and 9 the 1,5 MPa for a G1 material. All met the later (1999) Sabita requirement of 0,70 MPa for an E2 material, but only the red sand sections met the 1,20 MPa requirement for an E1 material.

These results cannot be evaluated according to the latest ETB guide (Sabita 2020b) as its material requirements now require indirect tensile strength and triaxial testing and not CBR and UCS.

As the raw sand used for the experiment was practically all only of G7 quality and at best only G6 for Sections 10 and 11 (Netterberg *et al.*, 2021a), none of the sections would be acceptable if the worst normally acceptable raw material requirement of a COLTO G5 (Sabita, 2020b) or even the Sabita (1993) substandard G7 grading and sand equivalent requirements are applied. However, both sands were acceptable for a TRH14:1985 BT3 material and also classified as “good” for use in sand/soil asphalt mixes according to the 1965 Dunning and Turner guidelines in Theyse and Horak (1987), still regarded as valid by Horak and Mukandila (2008). As only Section 4 failed, the experiment therefore provides a significant advance in ETB pavement material design for low volume roads in the vast area of southern Africa covered by Kalahari sands.

In short, all the sections developed satisfactory laboratory and in-situ strengths, but the red sand ETBs cured more rapidly, were less temperature-sensitive and more resistant to weakening by rain than the grey, but did not all develop higher strengths than the equivalent grey sand ETBs when surfaced after nearly five months.

The reason for this difference in behaviour apparently lies in the mineralogy rather than in their engineering properties and is probably because of the positive charge normally possessed by sesquioxides at a pH of less than about 7 (Wooltorton, 1954; Uehara, 1982; Bloom, 1999) in soils low in clay and organic matter such as the red sand, resulting in more rapid breaking and curing and better adhesion of the negatively charged anionic emulsion, whereas the opposite would be expected with a plain quartz sand. In either case evaporation of the water is necessary for complete curing.

A comparison between the vane shear strength and Clegg impact values during curing both corrected to 40ºC on the assumption that Equations 1 and 2 are valid for both methods is shown in Table 2.

Although there are no impact value data at 60 days and this method is probably less accurate than the vane method, inspection of the available data shows similar trends for both methods. The effect of the rain on the 21 April and subsequent recovery are shown by both methods. Both surface wetting and surface drying should have a greater effect on the CIV than the vane shear strength.

It is concluded that the Clegg Hammer appears to provide a viable, non-destructible, more convenient and faster method of testing Kalahari sand ETBs during curing than the vane shear strength method. However, the actual CIV reading should probably be used rather than the predicted CBR, and it may not be suitable soon after priming or after sealing.

Table 2: Comparison between vane shear strength and Clegg impact value corrected to 40°C during curing [1]

SECTION	← GREY−WHITE SAND							→	← RED SAND												→	
No.	**2**		**3**		**4**		**5**		**6**		**7**		**8**		**9**		**10**		**11**		**12**	
BINDER	3,0		1,5		1,5		3,0		3,0		1,5		4,0		3,0		1,5		1,5		0,0	
(net bit.)	1,5		1,5		0,0		0,0		0,0		0,0		2,0		1,5		1,5		1,5		0,0	
																					[6]	[6]
DAY	S	CIV	S	CIV	S	CIV	S	CIV	S	CIV	S	CIV	S	CIV	S	CIV	S	CIV	S	CIV	**S**	CIV
No.	kPa		kPa		kPa		kPa		kPa		kPa		kPa		kPa		kPa		kPa		kPa	
7 [2]	51	5,2	120	9,1	58	5,6	62	6,4	151	16,7	-	(18,6)	-	(18,5)	-	(18,5)	-	-	-	-	379	21,8
14 [3]	44	5,2	56	5,0	44	4,9	55	6,2	145	14,4	206	17,6	302	19,7	223	17,0	306	17,9	303	20,7	89	5,5
28 [4]	56	7,3	65	5,8	37	4,5	75	6,8	172	18,1	121	16,2	212	19,1	191	20,2	79	9,1	182	17,3	158	14,9
140 [5]	266	20,4	282	15,1	400	24,9	399	29,0	233	23,5	355	20,7	481	29,4	301	22,4	231	21,4	480	24,7	440	35,1

Notes:

[1] Each result is mean of n = 50 (5 measurements at 10 positions) for shear strength and n = 40 (5 measurements at 8 positions) for CIV at 7; 14 and 28 days both excluding shoulders; and n = 10 (5 measurements at 2 positions on centreline) for both at 140 days; results in brackets are estimates.

[2] Rain on 21 April 1989; Sections 2 – 5 tested on 17 - 19 April; Sections 6 – 11 on 10 – 15 April (CIV only); Section 12 on 22 April.

[3] Sections 2 – 5 tested on 24 – 25 April 1989; Section 6 on 22 April; Sections 8 and 9 on 21 April; Sections 7, 10 and 11 on 17 – 18 April; Sections 8 and 9 on 21 April; Section 12 on 30 April.

[4] Sections 2 – 5 tested on 08 − 09 May 1989; Sections 6 – 11 on 01 - 06 May; Section 12 on 15 May.

[5] Primed on 22 July 1989; 140-day results “just before” first seal spray on 30 Aug.1989.

[6] Not corrected to 40 °C as no bitumen added.

In the case of the Andoni experiment in Namibia (Marais & Freeme, 1977) the sections were apparently open-cured for one month before priming, sealing and opening to traffic. Although no vane shear strengths were reported during the curing period, from Marais (1965) at about 30 days they would have been about 200 kPa for the 4,5% ETB with filler and only 70 kPa for the equivalent one without. The in-situ strengths corrected to 40ºC of 25 – 168 kPa at compaction increased to 170 – 280 kPa after one year and 242 – 345 kPa after two years in comparison with the laboratory strengths of 80 – 200 kPa at 40ºC.

If grey or white sand is used a cationic emulsion should therefore be used although either an anionic or cationic emulsion could probably be used for a red sand.

Figure 1 shows a highly significant combined relationship for the grey and red sand ETBs between the actual section ETB mean vane strengths and the mean CIVs **not** corrected for temperature (Equation 5):

$$S = 55{,}279e^{0{,}0649CIV} \qquad (R^2 = 0{,}84;\; N = 36;\; P < 0{,}001) \qquad (5)$$

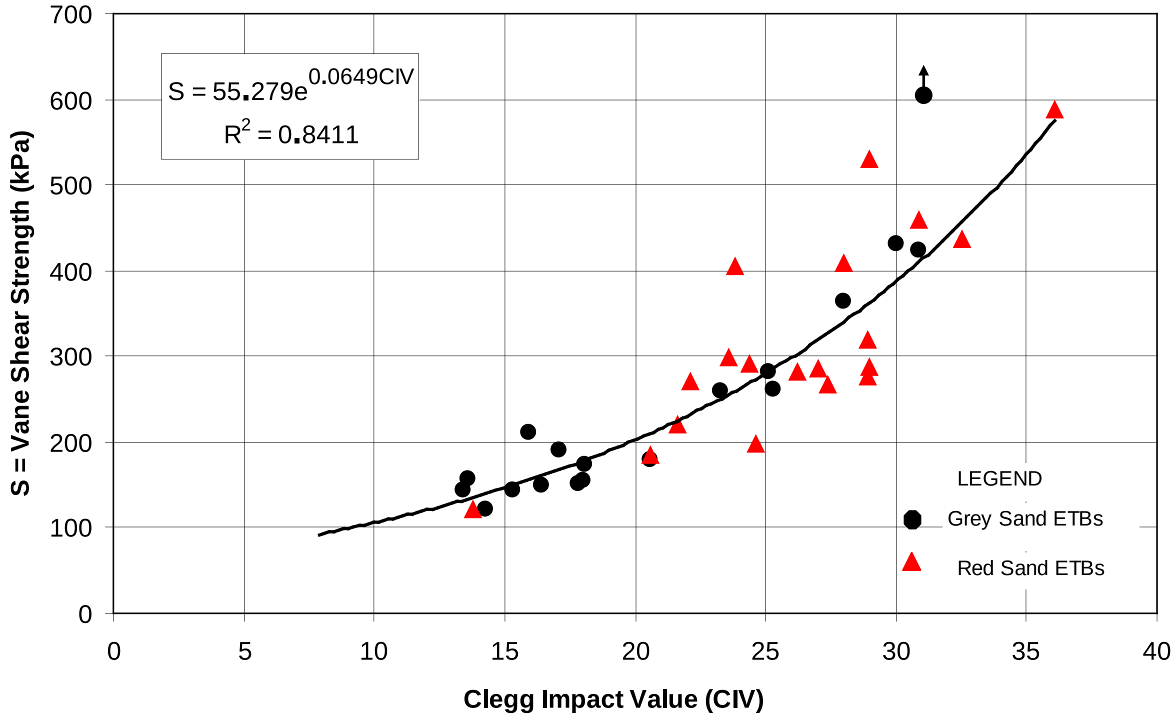

Figure 1: Relationship between vane shear strength and Clegg impact value on grey and red sand ETBs

Each of the 36 points on this graph represents pairs of the mean of forty measurements of vane shear strength and CIV on each section at that time, i.e. 17 on grey sand and 19 on red sand ETBs, before and after priming (the latter accounting for most of the worst outliers). (Because of overlap not all points could be shown.) Although the red sand ETBs showed a wider scatter, the relationships for both are practically identical (graphs not shown) and Equation 4 could be used for both grey and red sand ETBs:

$$S = 55{,}067e^{0{,}0654CIV} \qquad (R^2 = 0{,}90;\; N = 17;\; P < 0{,}001) \text{ for grey sand ETBs} \qquad (6)$$

$$S = 53{,}842e^{0{,}0657CIV} \quad (R^2 = 0{,}71;\ N = 19;\ P < 0{,}001) \text{ for red sand ETBs} \tag{7}$$

A relationship between the now-recommended (Sabita, 1999, 2020b) rapid compaction control device (RCCD) which also measures shear strength and the Clegg Hammer CIV was found by Guiamba *et al.* (2010) on sand ETBs in Mocambique:

$$CIV = 551{,}072\ RCCD^{-1{,}004} \quad (R^2 = 0{,}71) \tag{8}$$

From Equation 5 and Figure 1 it appears that the Marais and Freeme (1977) critical shear strengths of 130 and 200 kPa on these ETBs were equivalent to CIVs of approximately 13 and 20, respectively.

As the vane measures shear strength and the hammer a surface elastic modulus this is probably as good a relationship as one can expect. Moreover, if a Boussinesq stress distribution under a statically loaded circular footing is also valid for impulse loading, the upper 50 mm of the base would be expected to have accounted for about 70% of the 50 mm-diameter hammer response, whereas in the in the case of the full-depth sections the top of the 50 mm-long vane would have been at about this depth.

The temperature distribution in the base course was probably an additional factor. During a typical day at the same time of the year on the Andoni experiment in a similar environment the temperature at the surface and at 75 mm were only equal at about 09 h and 18 h (Marais, 1965). During most of the day the temperature at 75 mm was 5 – 10ºC less than that at the surface. Other than for ball penetration tests, on the Orapa experiment the temperature was only measured near the middle of the vane, i.e. at about 40 and 75 mm in the case of the half- and full- depth sections, respectively.

Adding the mean results on the untreated red sand Section 12 (with CIVs of 2 – 16) to the analysis in Figure 1 made a negligible difference to the relationship and extended it to below 100 kPa, suggesting that it could also be used on untreated red sand.

However, this could be confirmed only up to a CIV of about 20 by Figure 2 and Equation 9, which represent 38 pairs of individual pairs of the mean of five measurements each of vane shear strength and CIV at 12 specific points on Section 12 at 10 and 33 days (i.e. $N = 26$) and 24 pairs on the untreated red sand shoulders of Sections 6 – 11 at 7 and 28 days, all before priming. (No reliable results were available for the grey sand sections.)

$$S = 59{,}88e^{0{,}0585CIV} \ (R^2 = 0{,}71;\ N = 38;\ P < 0{,}001) \tag{9}$$

This suggests that the Clegg Hammer would also be a good and very convenient means of design, process and acceptance control on at least untreated red Kalahari sand pavement layers after suitable specification limits for it have been derived. Minimum CIV and vane shear strength limits derived from the actual results on Section 12 at 4 and 7 days after compaction are shown in Table 3. Either method could be used to supplement the conventional CBR approach used by Netterberg *et al.* (2021b).

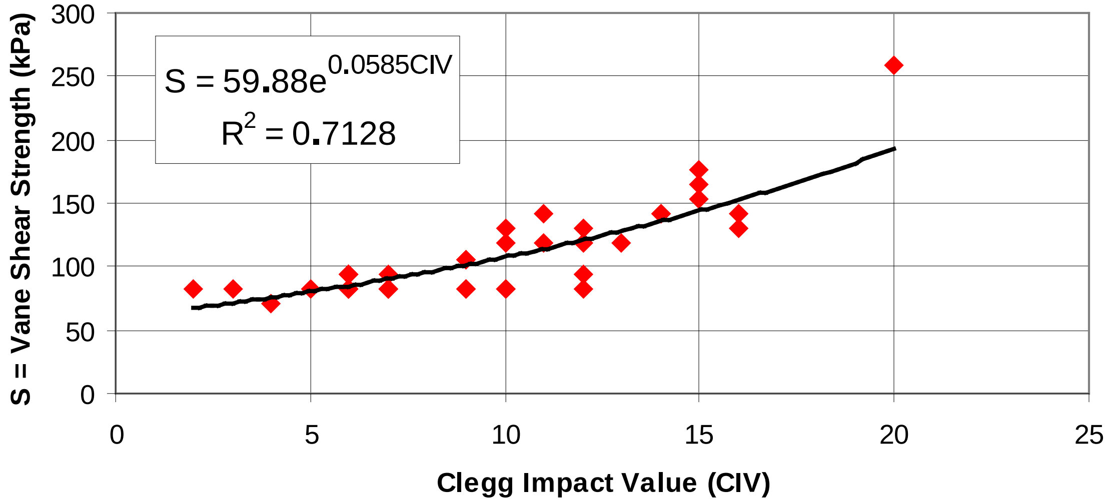

Figure 2: Relationship between vane shear strength and Clegg impact value on untreated red Kalahari sand base and untreated shoulders of red sand ETBs

The vane shear strength limit of 200 kPa is the same as that derived by Marais and Freeme (1977) from the Andoni experiment, specified in the BRDM (1982) and TRH14:1985 and supported by the Orapa experiment (Netterberg *et al.,* 2021a). From Equation 4, 200 and 380 kPa should be equivalent to in-situ CBRs of 26 and 49, respectively.

Table 3: CIV and vane shear strengths of Section 12 untreated red sand and suggested specification

DAY	TEST	UNITS	LANES AND SHOULDERS [1] (As found)				SPECIFICATION [2] (Minimum)		
			Mean	SD	Min.	*n*	Mean	Min.	*n*
4	CIV	-	15,5	2,6	11	12	15,0	10	12
	S	kPa	207	48	142	14	200	140	10
7	CIV	-	20,3	3,3	16	12	20,0	16	12
	S	kPa	385	112	189	14	380	180	10

Notes:
[1] Relative density by sand replacement: (mean 102,0 % MAASHO; SD 0,94; min. 100,9; *n* = 7; MDD 1917 kg/m^3)
[2] Mean 100% MAASHO; min. 99; *n* = 8

As a high level of compaction is probably essential for this kind of base course it should be compacted to as high a density as possible without shearing and the number of tests increased from the usual six to eight.

Rain at 10 days reduced the mean CIV and S in the lanes to 5,6 and 89 kPa respectively, and even after a further two weeks in April with no rain it had not fully recovered: at 33 days the mean CIV and S were 14,1 (min. 12) and 137 kPa (min. 118) respectively. With a mean S of 437 kPa (min. 248) (no CIV available) at 65 days it had more than recovered and it maintained this strength (without further rain) until primed at 101 days and sealed at 134 days at a mean CIV of 35,1 and S of 440 kPa.

The results in Tables 1, 2 and 3 show the value of dry curing (drying back) even this nonplastic sand after compaction, its extreme sensitivity to water but its ability to recover, and also that the strength was not due to self-cementation but only a surprisingly high apparent cohesion due to the extremely small clay and/or sesquioxide content. However, construction should obviously be limited to dry weather and it should be protected from rain and traffic by priming after a period of dry curing and sealing as soon as practicable. In spite of these limitations this sand provides by far the cheapest form of base course and Section 12 had not failed in 20 years.

Local limits in terms of a Clegg CBR (Equation 5) are problematic due to the difference between South African and Australian test methods and CIV alone should rather be used. However, it would be useful to have a local correlation with CBR.

6. CONCLUSIONS

Although of similar engineering properties as determined by the standard road indicator and CBR tests, the red sand ETBs proved better than the grey with respect to a lower temperature sensitivity, faster curing rate and better early strength water resistance, although not necessarily higher strengths when surfaced after nearly five months. After softening by rain the red sand ETBs achieved their design strength of 200 kPa at 40ºC two months after construction, whereas the grey sands took about four months.

The reason for this difference in behaviour was probably due to the positively charged sesquioxides in the red sands promoting more rapid curing of the negatively charged anionic emulsion.

The red sands are therefore to be preferred when using anionic emulsion. Although both sands achieved adequate strengths after 140 days when surfaced, grey-white sands are probably best treated with cationic emulsion.

A highly significant relationship was found between vane shear strength used as the primary method to monitor the curing and the Clegg Hammer impact value and the latter may be a more convenient and rapid substitute for the vane test.

A similar highly significant relationship was also found for the untreated red sand, suggesting that the Clegg Hammer would be a good and very convenient means of non-destructive control testing of untreated red Kalahari sand pavement layers.

Although it was softened by rain the nonplastic sand control section benefited greatly by dry curing and had developed strengths comparable to the stronger of the ETBs when sealed and a specification for such a base in terms of either vane shear strength or impact value is suggested.

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However, the opinions expressed are those of the author and not necessarily any of the above.

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