The Development of Specifications for the Use of Low-Grade Calcretes in Lightly Trafficked Roads in Botswana

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A full-scale road experiment that incorporated four representative types of calcrete was constructed in Botswana in 1979 to determine more appropriate criteria for sealed calcrete bases in lightly trafficked roads. The suitability of calcrete for stabilization with Portland cement or hydrated lime and its mechanical stabilization with local Kalahari sand were examined. A test section was also constructed in which a calcrete was compacted at a moisture content substantially below the normally required level as a means of saving water in arid areas. Calcretes were also used in the experiment as unsurfaced shoulder materials. Standard laboratory tests were made on the calcrete materials before, during, and after the construction of the experiment. The condition of the experiment was monitored at regular intervals by means of measurements of surface deformation, rutting, cracking, longitudinal roughness, and surface deflection. Panel inspections were also conducted. In addition, traffic counts and axle-load surveys were performed, climatic data were collected, and the density, moisture content, and strength of the pavement layers and subgrade were measured on a number of occasions. Satisfactory performance was obtained from all untreated calcrete base sections, although it is apparent that more strict selection of unsurfaced shoulder material is required. The behavior of cement- and lime-stabilized calcretes and a mechanically stabilized calcrete was unsatisfactory. In the case of the stabilized calcretes, this was attributed to the lack of a stabilization reaction (only modification occurred) and the instability of the bases under traffic, during which time they behaved similarly to single-sized sands. The results of the experiment have led to specifications being recommended that permit a wider range of calcretes to be used as road bases for design traffic levels up to 160,000 equivalent standard axle loads. This, combined with existing specifications for higher traffic categories, now enables limits for the use of calcretes to be clearly defined. Limitations on the use of calcretes as unsurfaced shoulder materials are also given.

Calcrete deposits exist as virtually the only sources of hard aggregate or gravel materials suitable for use in pavement layer construction throughout the central and western regions of Botswana. Their occurrence is generally confined to those regions overlain by Kalahari sands, which occupy approximately three-quarters of the country (Figure 1), although they are also found further east overlying older rocks and along drainage lines.

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The engineering properties of calcretes are extremely varied, and they range in texture and composition from slightly calcified uniform sands and soft, powdery soils to hard gravels and rock layers. In comparison with most guidelines used in the tropics to select road pavement materials, calcretes would normally be considered low-grade with only a small number of deposits satisfying the commonly applied criteria in regard to plasticity, grading, and strength. However, research performed in southern Africa has shown that wider specification limits can be applied to calcretes without a consequent loss of performance (*1*). This has also been recognized by some other authorities (*2*).

In Botswana the greater proportion of calcretes still fails to meet the less stringent requirements proposed by Netterberg (*1*). Because of the development and road construction planned in the Kalahari region, a need existed to further consider the appropriateness of available specifications. Factors that can contribute to the creation of a lower specification include the strong free-draining sand subgrades (with CBR values often greater than 25 percent), low traffic levels (less than 100 vehicles per day), and the low rainfall and semi-arid climate of the region.

A cooperative research project involving the Botswana Ministry of Works and Communications and the Transport and Road Research Laboratory of the United Kingdom has been in progress since 1978 to determine appropriate selection criteria for sealed calcrete bases in lightly trafficked roads. As part of this project, a full-scale road experiment that comprised nine different experimental sections and two control sections, each 100 m in length, was constructed. The site of the experiment was 9 km from Jwaneng on the new road that was being built from Kanye to Jwaneng. A new diamond mine and the town of Jwaneng were being developed, so it was expected that the experiment would carry more traffic during its early life than would normally be expected on the trans-Kalahari routes.

In the experiment, four different calcretes representing the range that occurs in the region were assessed as road base materials under a double bituminous surface dressing. One of the calcretes of lower quality was also used to evaluate its suitability for stabilization with Portland cement or hydrated lime; another was mixed with the local Kalahari sand in equal proportions to determine whether savings could be made in the haulage of material. In addition, another of the calcretes was compacted at a moisture content considerably below the normally required level as a means of saving water in dry areas. A further section was constructed and left unsurfaced for a number of months prior to surfacing and opening to traffic to determine whether any gain in strength on drying would be maintained in subsequent years.

Other important aspects of the experiment were to assess the performance of the different calcretes as unsealed shoulder

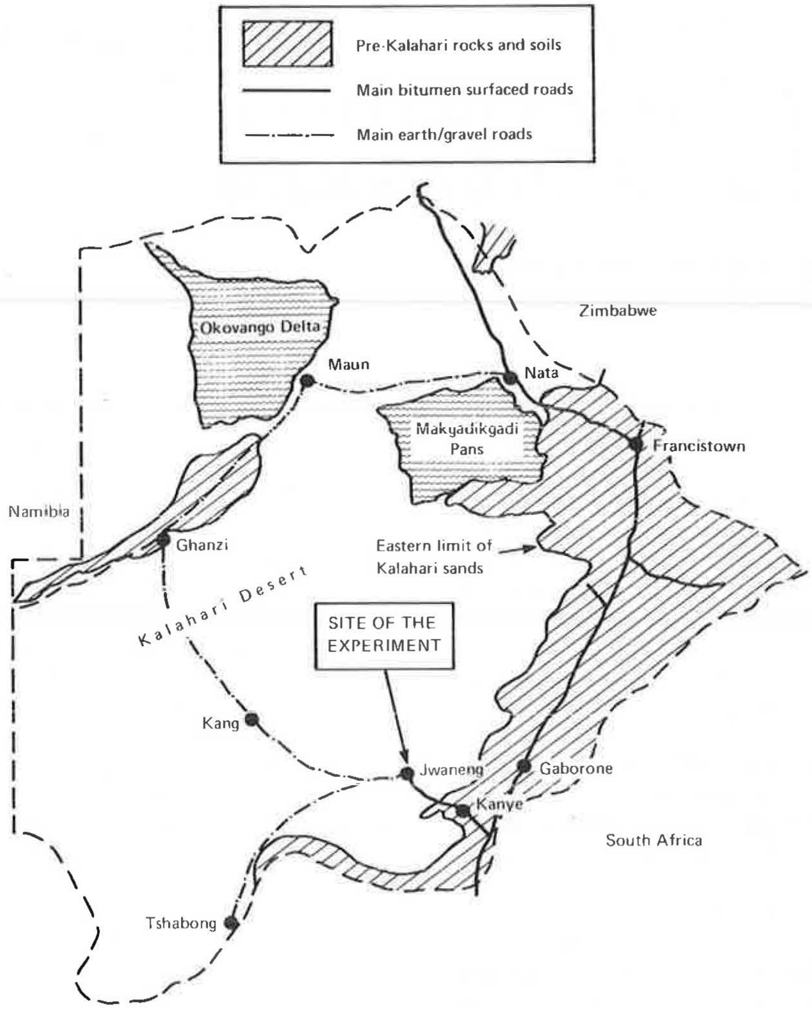


FIGURE 1 Map of Botswana in which the location of the experiment is shown.

materials and to observe the behavior of the side slopes of the sand embankment.

A description is provided of the classification and properties of the materials used in the experiment, its design and construction, and its performance over a period of 7 years. Recommendations are made from the results obtained for the use of calcretes up to the level of traffic carried so far.

THE DESCRIPTION AND CLASSIFICATION OF BOTSWANA CALCRETES

Calcrete is a member of the pedogenic soils group in which a host soil is modified by the precipitation of soluble minerals introduced by groundwater. In Botswana the host soil is commonly the medium- to fine-grained Kalahari sand, to which carbonates of calcium or magnesium are added to form rubbly, nodular, or powdery horizons within the sand. In certain areas the host soil may alternatively be natural gravels or weathered rocks, particularly in the Kalahari fringe areas.

The amount and type of carbonate present, and its degree of crystallinity, create a variety of calcareous deposits, all called calcrete, that vary from loose, calcified sand to massive, hard "rock." The proportion of sand to carbonate and the degree of induration determine its hardness and to some extent its grading, two of its most important engineering properties. A certain amount of mineral fine material, for example, water-soluble salts and clay minerals, may also be present in calcretes.

In Botswana a simple morphological classification of common calcrete types based on the South African groupings has been adopted and these are distinct both physically and in their engineering properties (*1*). They are calcified (or calcareous) sand, powder calcrete, hardpan calcrete (including "boulder calcrete"), and nodular calcrete. In other areas of the world, similar accumulations of material rich in calcium carbonate exist and are known locally by a variety of names including caliche (United States), kunkar (East Africa, India, and Australia), kurkar (Israel), and jigilin (Nigeria). A description of the types recognized in Botswana follows.

Calcified (or calcareous) sand consists mostly of host soil grains and may exhibit the characteristic uniform particle size distribution of an unaltered Kalahari sand. Weak cementation between sand grains may have occurred, although the in situ deposit is generally loose. The amount of calcium carbonate is normally low, often less than 10 percent by weight, and any gravel-sized particles are easily broken.

Powder calcrete contains a higher proportion of calcium carbonate (greater than 30 percent) and the host soil grains are proportionately less visible. The in situ material may have acquired a laminar, blocky, or massive structure; it can normally be excavated with a pick axe, although it can be very difficult to remove. The material may retain its original proportions of the various fractions with restricted handling, but because of its essentially soft nature, being composed largely of a fragmented mass, it is easily broken down in handling.

Hardpan calcrete could be considered a development of a powder calcrete into a calcrete rock. The calcrete is generally much harder, and excavation requires the use of a mechanical ripper. Subsequent breakdown of particles during handling and compaction is considerably less than powder calcrete, and crushing, screening, or grid rolling is frequently necessary to remove oversized pieces or reduce them to a specified maximum size.

Completely indurated hardpan separates into discrete boulders upon weathering to form "boulder" calcrete. Unweathered, massive hardpan and large boulders can be very difficult to remove and blasting may be necessary. However, the material could be useful in regions in which no conventional rocks exist, because it can be hard enough to be used as a high-quality road base or surfacing aggregate.

Nodular calcrete is a natural calcrete gravel composed of a high proportion (greater than 50 percent by weight) of rounded gravel-sized nodules of calcium or magnesium carbonate and quartz, in a matrix of powder calcrete or calcified sand. The material is typified by a generally well-graded particle size distribution and a hard aggregate fraction, although the hardness of the nodules varies greatly.

Nodular calcretes are often considered the most useful calcrete for pavement layer construction, by virtue of their relatively good grading, mechanical interlock, and ease of excavation. Nodular calcretes may become partly cemented; in this stage of development they are referred to as honeycombed calcretes. Further cementation would lead to an indurated hardpan deposit.

Any one or a number of the types described may occur in a calcrete profile, which creates deposits of variable quality, the properties of which may change both in a lateral and vertical direction.

PROPERTIES OF THE MATERIALS USED IN THE EXPERIMENT

Calcrete Base Materials

Four different calcretes were chosen to represent the types that occur in Botswana and to obtain a range of materials that possessed engineering test properties that both satisfied and extended outside the lower quality limits of existing specifications (e.g., TRRL Road Note 31) and those developed for calcrete road bases in southern Africa (see Table 1) (*1, 3*). The calcretes chosen included a hardpan calcrete, a nodular calcrete, a powder calcrete of low plasticity, and a plastic calcified sand. The calcretes were assigned the sample numbers BG1, BG4, BG6, and BG7, respectively.

The results of grading tests that were performed on samples taken from the completed road are given in Table 2 and their mean particle size distributions are shown in Figure 2. Also shown in Table 2 are the test results for the mechanically

TABLE 1 EXISTING GUIDELINE SPECIFICATIONS FOR THE SELECTION OF LIGHTLY TRAFFICKED CALCRETE BASES (*3-5.*)

Test	TRRL Road Note 31 Design Traffic	NITRR Bulletin 10 Expected Traffic Category (vdp) <20% >3 tonnes			
	$<2.5 \times 10^6$ ESA	<500	500-1000	1000-2000	2000-4000
Maximum size of material (mm) or range of sizes	37.5	19-37.5	37.5-53	37.5-53	37.5-53
Minimum grading modulus	ns	1.5	1.5	1.5	1.5
Percentage passing the 425-μm sieve	12-25	15-55	15-55	15-55	
Percentage passing the 63-μm sieve	5-15	ns	ns	ns	ns
Maximum liquid limit (%)	25	40	35	30	25
Maximum plasticity index	6[a]	15	12	10	8
Maximum linear shrinkage (LS) (%)	4	6	4	3	3
Maximum LS times percentage passing 425-μm sieve	ns	320	170	170	170
Minimum CBR after 4 days of soaking[b]	80	60 at 2.5 mm 80 at 5.0 mm	80 at 2.5 mm 80 at 5.0 mm	80 at 2.5 mm 100 at 5.0 mm	80 at 2.5 mm 100 at 5.0 mm
Minimum dry 10% FACT value (kN)	ns	ns[d]	80	110	110
Minimum soaked 10% FACT value (kN)	50[c]	ns	50	50	50
Minimum relative field compaction (%)	100 BS[b]	98% modified AASHTO			
Total soluble salts (%)	ns	<0.2	<0.2	<0.2	<0.2

Note: ns = not specified.
[a]Can be raised to 12 in arid and semi-arid areas.
[b]At 100% BS, 2.5-kg rammer method. In practice, a higher field compaction of 97% BS, 4.5-kg rammer method is often specified and the CBR is based on this.
[c]A minimum 10% Fines Aggregate Crushing Test value is suggested by TRRL; a maximum modified aggregate impact value of 40% is suggested as an alternate.
[d]An aggregate pliers value of 50% is usually specified.

TABLE 2 LABORATORY TEST RESULTS FOR THE CALCRETE BASES

Soil classification tests

Sample designation	Liquid limit (LL)		Plastic limit (PL)		Plasticity index (PI)		Linear shrinkage (LS)		Maximum particle size (mm)	% passing 425 μm sieve		% passing 63 μm sieve	
	Range	$\bar{x}$	Range	$\bar{x}$	Range	$\bar{x}$	Range	$\bar{x}$		Range	$\bar{x}$	Range	$\bar{x}$
BG 1	22-28	25	17-20	18	5-8	7	3.6-3.8	3.7	37-75	33-47	39	6-20	12
BG 4	34-57	44	17-30	24	13-31	20	6.3-13.8	8.5	50-75	32-45	37	10-23	14
BG 6	36-44	39	28-35	30	7-9	9	2.9-4.5	3.6	26-53	58-65	62	29-36	33
BG 7	32-41	36	19-24	21	13-17	15	5.7-7.6	6.2	20-37	75-86	81	20-34	28
BG 4 plus sand	25-33	28	15-17	16	8-17	12	5-7.7	5.8	37-75	56-69	63	14-22	16

Compaction and strength tests

Sample designation	BS 4.5 kg rammer compaction test		4-day soaked CBR test result			
			BS 4.5 kg rammer test		BS 2.5 kg rammer test	
	Maximum dry density (kg/m^3)	Optimum moisture content (%)	Range	$\bar{x}$	Range	$\bar{x}$
BG 1	1972	9.5	110-172	148	42-67	54
BG 4	1964	10.5	81-180	123	44-141	65
BG 6	1490	25.0	21-92	51	10-39	23
BG 7	1836	12.2	19-64	38	8-26	14
BG 4 plus sand	2058	9.4	19-103	48	20-73	38

Aggregate tests

Sample designation	Ten per cent fines crushing value (KN)				Modified aggregate impact value			
	10-14 mm size		>20 mm size		10-14 mm size		>20 mm size	
	Dry	Soaked	Dry	Soaked	Dry	Soaked	Dry	Soaked
BG 1	18-20	9.5-11	37-45	29-31	96-130	120-150	54-64	51-57
BG 4	37-42	43	68-122	-	42-84	40-87	17-27	15-26

stabilized sand/calcrete that consisted of the nodular calcrete (BG4) mixed with an equal proportion of the uniformly graded local Kalahari sand. British Standard (BS) procedures and preparation methods were used in the conduct of the soils classification tests (*6*).

The results of the classification and strength tests indicated that the calcretes BG1 and BG4 were potentially better road pavement materials than BG6 and BG7, although they did not satisfy all specification requirements, particularly in regard to plasticity and grading. The testing did, however, show that high soaked CBR values could easily be obtained with the better-graded calcretes.

The nodular and hardpan calcretes also possessed a sufficient proportion of hard aggregate particles, which enabled their aggregate strength to be evaluated using both the BS Ten Percent Fines Aggregate Crushing Test and the Modified Aggregate Impact Test (see results in Table 2) (*4, 7*).

The more poorly graded calcretes displayed low and variable CBR values and possessed few gravel-sized particles; those that were present were easily broken. The low particle strength of powder calcrete BG6 was reflected in its change in grading, as shown in Figure 3, which occurred as a result of field compaction.

The mixing of BG4 and the local sand produced a gap-graded material, the particle size distribution of which is shown in Figure 4. The results of testing samples taken following field mixing also illustrated the variability that can be expected, particularly in relation to CBR values.

The low quality of BG7 led to its being selected for stabilization by the addition of Portland cement and hydrated lime. An initial series of tests performed in the laboratory on samples taken from the borrow pit had shown that high strengths could be obtained by the addition of a low amount of stabilizer. The results of the testing of samples taken after field mixing and curing in the laboratory confirmed this (see Table 3). The addition of cement and lime also led to a decrease in the plasticity index of the calcretes (see Table 4) and the aggregation of the silt and clay particles (Figure 5). This caused

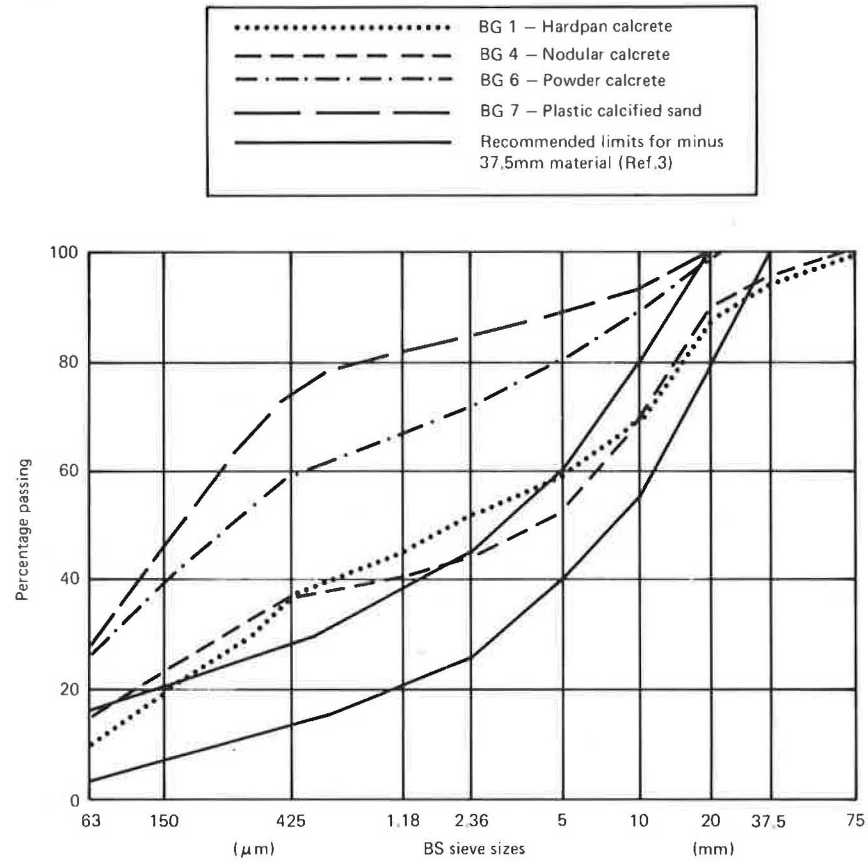


FIGURE 2 Mean particle size distribution of samples of four untreated calcretes taken after field compaction.

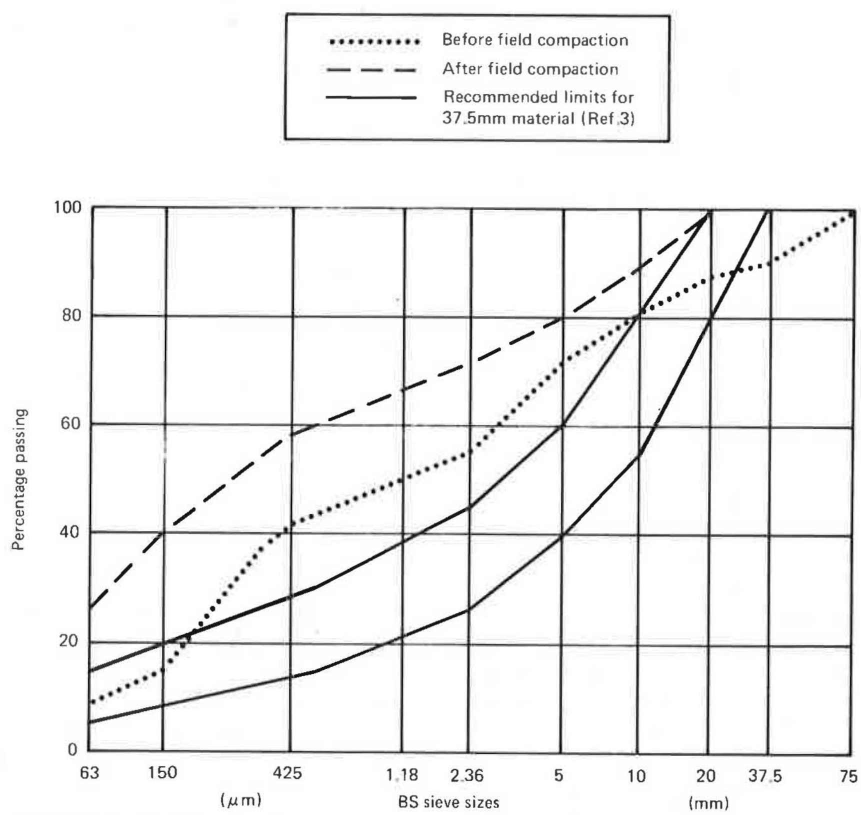


FIGURE 3 Particle size distribution of powder calcrete BG6 before and after field compaction.

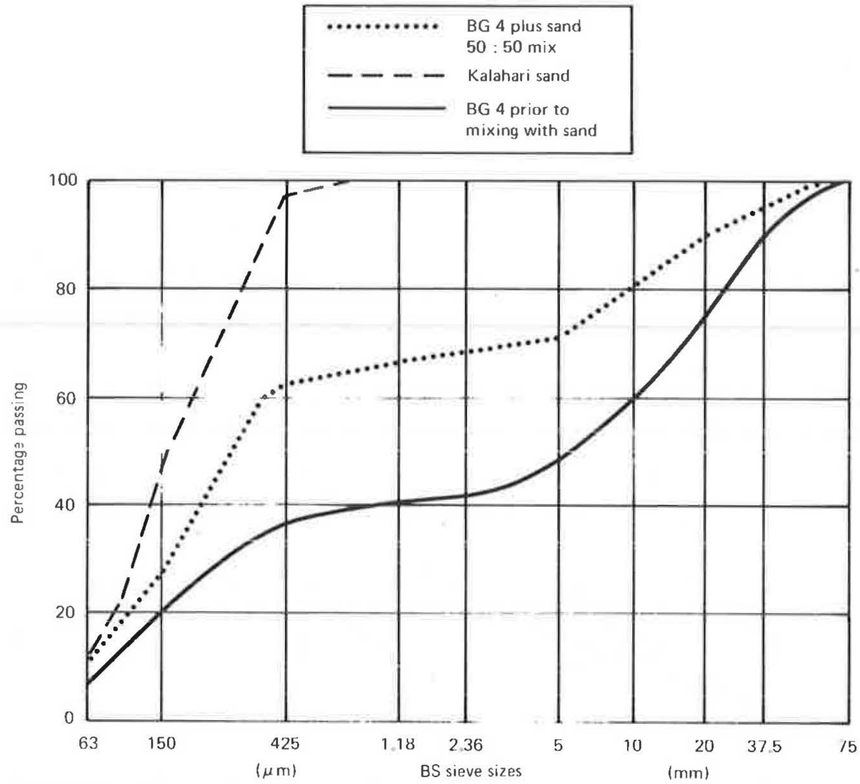


FIGURE 4 Particle size distribution of the calcrete/sand base and the component materials.

TABLE 3 LABORATORY TESTS ON STABILIZED SAMPLES OF BG7 TAKEN AFTER FIELD MIXING

Section	Material description	Curing period (days)	Moisture content (%)	BS 4.5 kg rammer test	
				Dry density (Kg/m^3)	CBR (%)
4	Calcrete BG 7 and 3% cement	7	9.3 and 9.8	1803 and 1814	117 and 122
		28	8.1 and 8.2	1742 and 1849	135 and 194
5	Calcrete BG 7 and 3% hydrated lime	7	8.5 and 10.5	1751 and 1736	65 and 126
		28	7.6 and 9.8	1686 and 1741	124 and 197

the amount of material passing the 63-μm sieve to decrease from between 25 percent and 37 percent in the untreated material to between 11 percent and 24 percent in the treated material.

Chemical and mineralogical tests were performed on the calcretes to determine their composition and to identify the mineral types present and their amount. The test results shown in Table 5 confirmed the presence of a large amount of quartz in the samples, which reflected the contribution of the host material, and varying amounts of calcium and magnesium carbonate. The low amount of calcium carbonate in BG7 confirmed its classification as a calcified sand.

Kalahari Sand

The particle size distribution of the Kalahari sand shown in Figure 4 is that of a uniform, medium-to-fine sand with a coefficient of uniformity between 3 and 4. The sand was nonplastic in the standard plasticity tests but was plastic when the fraction finer than 0.063 mm was tested (see Table 6). This plasticity and the small amount of iron oxide that was present were responsible for the slight cohesion exhibited by the sand.

The results of CBR tests (Table 6) showed that the sand satisfied the requirements for subbase for lightly trafficked

TABLE 4 PLASTICITY CHARACTERISTICS OF THE STABILIZED MATERIALS

Section	Material Description	Before Field Mixing				After Field Mixing and Compaction				After 3 Years in the Road				After 4 Years in the Road			
		LL	PL	PI	LS	LL	PL	PI	LS	LL	PL	PI	LS	LL	PL	PI	LS
4	Calcrete BG 7 and 3 percent cement	41	24	17	7.6	28	24	4	2.1	23	18	5	2.5	27	21	6	3.6
5	Calcrete BG 7 and 3 percent hydrated lime	41	24	17	7.6	29	24	5	2.1	26	16	10	4.1	27	19	8	2.1

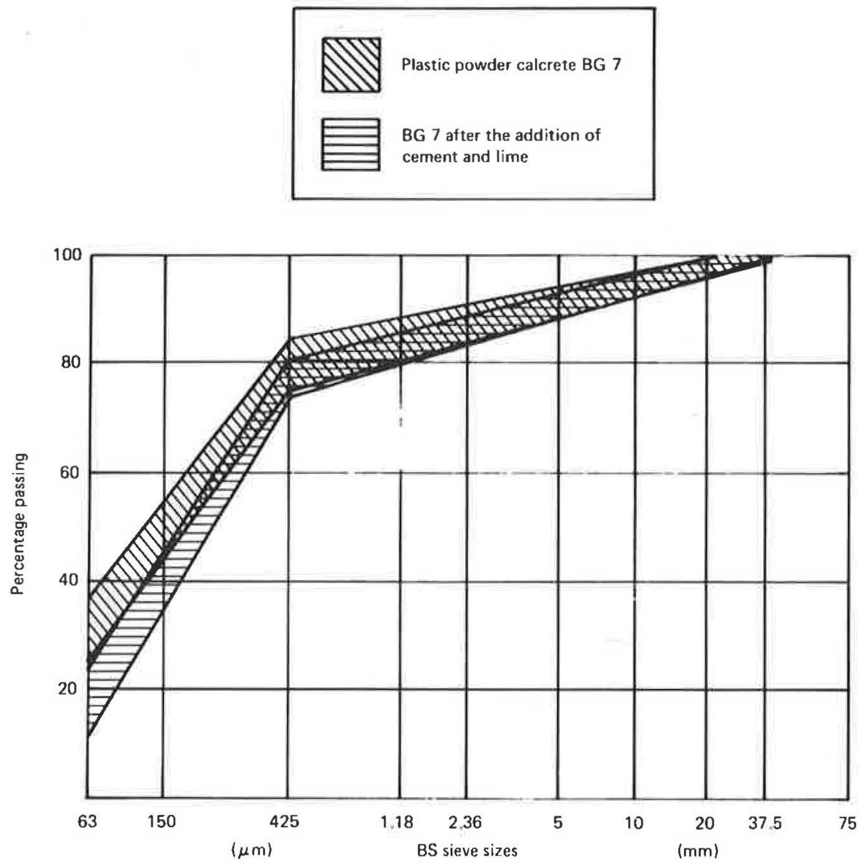


FIGURE 5 Grading envelope of the plastic calcified sand before and after it was mixed with cement and lime.

roads (*3*). The sand therefore was used to form the low embankment of the road, the subgrade, and the subbase beneath the calcrete bases in the experimental sections. In the experiment, the upper layer of the sand was referred to as the finished subgrade.

Control Materials

Conventional gravel materials that were derived from quartzite and granite and the properties of which complied with the specifications used in the main construction contract were used as base and subbase materials in adjacent sections of the road. These materials were incorporated into the experiment to act as control materials. Their test properties given in Table 7 confirm their suitability in relation to the specifications given in Table 1.

DESIGN AND CONSTRUCTION OF THE EXPERIMENT

The layout of the experiment is shown in Figure 6. A typical cross-section of the original design is illustrated in Figure 7. Construction of the experiment took place in April 1979 and formed part of the main construction contract for the Kanye to Jwaneng road.

TABLE 5 RESULTS OF CHEMICAL AND MINERALOGICAL TESTS ON THE CALCRETES

Sample description	Calcium carbonate content (%)		X-ray diffraction results			
	Total sample	Fraction passing the 425 μm sieve	Main minerals	Other minerals	Trace minerals	Dominant clays
BG 1	40	29	Cc	Q	F,D	K
BG 4	32	22	Cc,Q	D	F	P,Mm
BG 6	38	32	Cc	Q	F,D	-
BG 7	16	15	Q	Cc	F,D	Mm

Abbreviations: Calcite (Cc); Quartz (Q); Dolomite (D); Feldspar (F); Palygorskite (P); Montmorillonite (Mm) ; Kaolinite (K).

TABLE 6 RESULTS OF LABORATORY TESTS PERFORMED ON THE SAND SUBGRADE MATERIALS

Level of compaction	Maximum dry density (Kg/m^3)	Optimum moisture content (%)	CBR (%)	
			After 4 day soak	At OMC
BS vibrating hammer	1950	7-7.5	80-150	110-130
BS 4.5 kg rammer	1895-1915	7-8	80-100	75-140
BS 2.5 kg rammer	1880	7.5-9	35-65	60-70

Properties of the material passing the 63 μm sieve	
% passing 63 μm sieve	7-15
Linear shrinkage	7.1-9.3
Fineness index	160-180

Note Fineness index:- Defined as the product of the plasticity index determined on material less than 63 μm and the % passing the 63 μm sieve

TABLE 7 LABORATORY TEST RESULTS OF THE MATERIALS USED IN THE CONTROL SECTIONS

Section	Layer	Soil plasticity				Particle size			BS 4.5 kg rammer test		4 day soaked CBR (%)[a]	
		LL	PL	PI	LS	Max. size (mm)	%-425 μm sieve	%-63 μm sieve	Maximum density (Kg/m^3)	Optimum moisture content (%)	98%	95%
A	Base	16	11	5	2.9	37.5	38	12	2190	7.1	61	-
A and B	Sub-base	19	13	6	2.2	75	47	12	2170	6.4	115	70
B	Base	18	-	NP	-	50	23	10	2200	5.7	120	-

[a]Calculated at 98% and 95% relative compaction of the maximum density of the BS 4.5 kg rammer test which corresponds to required field compaction levels for bases and sub-bases respectively.

Section	A	1	2	3	4	5	6	7	8	9	B
Surfacing	Double surface dressing										
Base	WG	BG6	BG6	BG7	BG7 + cement	BG7+ lime	BG4 + sand	BG4	BG4	BG1	QG
Sub-base	FG	Medium – fine grained, uniformly graded 'Kalahari' sand									FG
Subgrade											

Note 1 Each section = 100m in length

2 Design thickness of base = 150mm

3 A and B = control sections

4 Description of pavement materials :–

BG1 - Hardpan calcrete
BG4 - Nodular calcrete
BG6 - Powder calcrete
BG7 - Plastic calcified sand
WG - Weathered granite gravel
FG - Ferruginised quartzite gravel
QG - Quartzite conglomeratic gravel

FIGURE 6 Layout of the full-scale experiment.

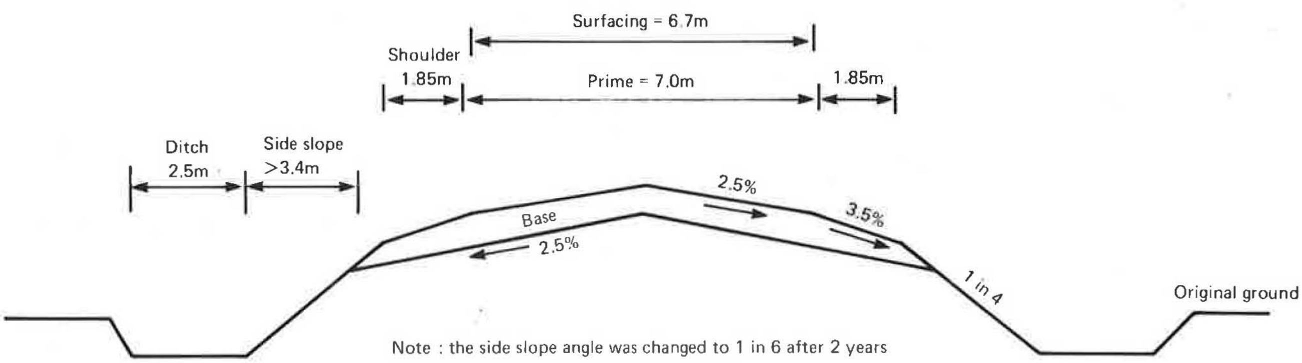


FIGURE 7 Typical cross-section and pavement details.

Preparation of the Road Bed and Construction of the Sand Embankment

The construction involved the clearing of vegetation followed by the compaction of the in situ Kalahari sand road bed at its natural moisture of 1 to 2 percent using a 15-ton towed vibrating roller. The sand embankment was raised by a motor scraper operation; compaction was performed at a moisture content of 3 to 4 percent using pneumatic-tired and self-propelled vibrating rollers.

High levels of compaction, in relation to the maximum density obtained in the BS 4.5-kg rammer test, were achieved in the sand (see Table 8) down to a depth of over 1 m below the finished subgrade level (*6*).

Construction of the Pavement Layers

The calcrete base materials were laid and compacted in a single 15-cm layer directly onto the prepared sand subgrade. Careful control of trafficking and the provision of access ramps at convenient intervals ensured that no damage was done to the sand subgrade. The untreated bases were mixed and processed by a motor grader. Compaction was performed with a grid roller, a 15- to 30-ton pneumatic-tired roller, and a 9-ton towed vibrating roller. Although the inclusion of the grid roller was only recognized as being a necessity for the nodular and hardpan calcretes, it was used on all sections.

The cement- and lime-stabilizing agents that were added to Sections 4 and 5, respectively, were initially spread by hand over a loose thickness of calcrete after which they were introduced into the base material by a multi-pass pulvimixer operation. The mixing process, including the addition of water, took over 3 hours to complete. Compaction followed immediately and a prime coat of MC30 was applied within 24 hours.

The sand/calcrete base was constructed by spreading the calcrete to a loose thickness before dumping and spreading an equal amount of sand. The mixing process was performed by pulvimixer and compaction was performed as described earlier.

The compacted densities, mixing moisture contents, and constructed thicknesses for the calcrete base materials are given

TABLE 8 RESULTS OF IN SITU TESTS IN THE SAND SUBGRADE AND EMBANKMENT (*4*)

	Depth Below the Bottom of the Base Layer (mm)		
	0-150	150-600	600-1200
Average relative compaction (%)[a]	97.8	96.8	94.1
Coefficient of variation (%)	1.8	1.8	3.5
Lowest relative compaction result (%)	94.4	93.7	86.9
Number of tests	36	10	15
Specified minimum relative compaction (%)	95	93-90	90

[a]Relative to the maximum density in the BS 4.5-kg rammer test.

in Table 9. The relative compaction levels for the stabilized sections are based on samples in which stabilizer was introduced before laboratory compaction.

Specifications normally require relative compaction levels for bases to be above 97 percent of the maximum density obtained in the BS 4.5-kg rammer test. In this respect, the densities that were obtained in Sections 1 and 2 using the powder calcrete (BG6) were extremely low. It was believed that the reason for this was related to the high amount of energy input in the laboratory compaction test, which allowed a higher density to be obtained as a result of a greater amount of particle breakdown than occurred in the field. However, no supporting evidence exists to substantiate this.

Another reason for the low relative densities of Sections 1 and 2 could be the result of differences between the optimum moisture contents obtained in laboratory compaction tests and those of the compaction plant used.

Water for Construction

Substantial quantities of water were required at all stages of construction. Estimated quantities used in the construction of the experiment are given in Table 10. The water requirements for embankment construction are based on the minimum embankment height of 0.6 m. On the project water was available at 5-km intervals along the alignment from 300 m^3 reservoirs constructed of sand with a lining of plastic sheeting. A pipeline connecting the reservoirs was fed by a borehole system at the southern end of the project.

The variation in water requirements illustrates the differences that can exist between in situ moisture content and compaction moisture content for the materials used.

Surfacing of the Experiment

An MC30 bituminous prime was applied to all experimental sections following the completion of the stabilized base sections. A double surface dressing, consisting of a 19-mm first stone layer followed by a 9.5-mm second layer using an 80/100 penetration grade binder was applied in September 1979. The surfacing stone was a "Witwatersrand quartzite." The shoulders of the calcrete sections were not sealed.

Construction of the Control Sections

The two control sections were constructed under normal contract conditions. The pavement comprised two layers, both 15 cm in design thickness, consisting of a quartzite gravel subbase with a base of naturally occurring weathered granite gravel in Section A and a crushed and screened conglomeratic quartzite gravel in Section B. The carriageway of the control sections was surfaced with a double surface dressing; their shoulders received a bituminous seal with graded crushed fines as the covering aggregate.

TABLE 9 IN SITU TEST RESULTS ON THE CALCRETE BASES

Section No.	Field density tests			Average mixing moisture content (%)	Thickness of base material (mm)	
	Dry density (Kg/m3)	Coefficient of variation (%)	% Relative compaction[a]		Average depth	Coefficient of variation (%)
1	1233	1.0	82.7	30	155	4.5
2	1266	3.0	84.9	23	142	12.9
3	1750	5.0	95.3	12	150	11.5
4	1741	8.0	97.2	9.5	166	4.6
5	1659	3.0	97.4	9.4	167	8.5
6	1958	2.4	95.0	7.3	166	10.2
7	1946	2.2	99.1	10.2	198	3.5
8	1958	3.5	97.9	10.2	198	11.4
9	1791	4.2	90.8	11.0	170	9.5

Note: Six tests performed per section.
[a]% of maximum dry density obtained in the BS 4.5 kg rammer test.

TABLE 10 QUANTITIES OF WATER REQUIRED FOR COMPACTION

Section	Material Designation	Natural Moisture Content (%)	Compaction Moisture Content (%)	Quantity of Water Added per 100 m (liters)[a]
1	BG 6	9.0	32.0	45 000
2	BG 6	9.0	22.0	26 000
3	BG 7	9.8	13.0	10 000
7 and 8	BG 4	9.4	10.2	2 350
9	BG 1	5.2	11.0	17 000
Sand embankment		1-2	3-4	110 000

[a]Additional 3 percent added for evaporation losses.

POST-CONSTRUCTION MEASUREMENTS AND RESULTS

Since construction was completed in April 1979 and the experiment was opened to traffic in November 1979, a program of measurements and observations to assess the performance of the experiment was undertaken at regular intervals. This included both direct and indirect testing of the road pavement and individual layers. Traffic and climatic data were also recorded. Further details of the types of measurements are included in Table 11. The results are discussed in the following sections.

Testing of the Calcrete Bases

The results of moisture content determinations are shown in Figures 8a to 8c and are expressed in terms of the equilibrium moisture content ratio (EMCR) as follows (*8*).

$$\text{EMCR} = \frac{\text{Percent of in situ moisture content}}{\text{Percent of optimum moisture content (BS 4.5-kg rammer test)}}$$

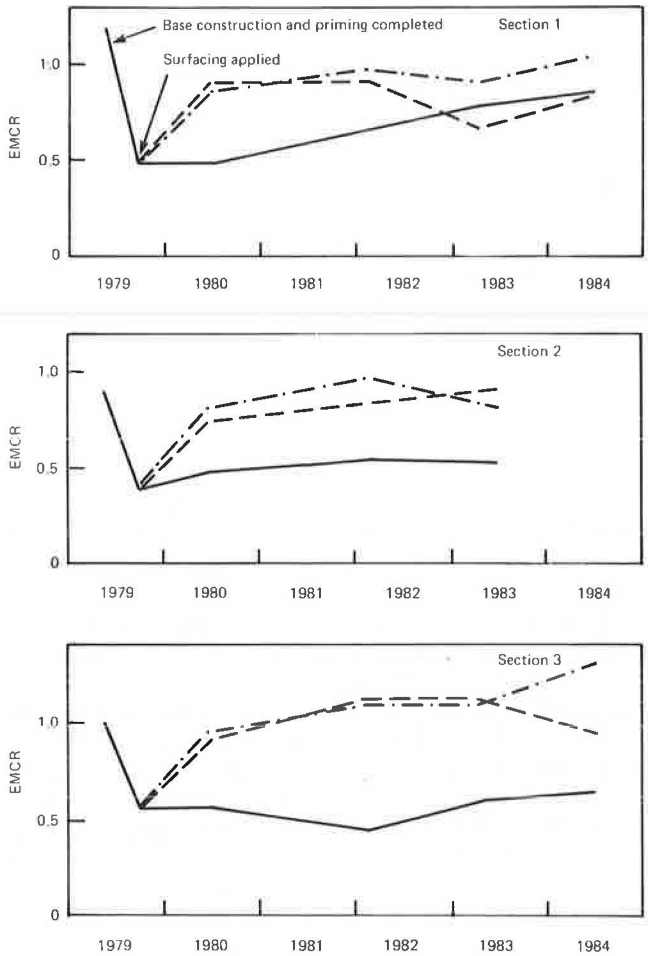
The results show that the EMCR in the outer (verge-side) wheelpath of all sections increased with higher values, often above unity, that were obtained in sections that contained the plastic and more fine-grained materials (Sections 1 to 6). In most cases the EMCR in the center of the road was substantially lower.

The results of in situ strength tests, reported as equivalent CBR values, are shown in Figures 9a to 9c. In situ CBR values that ranged between 20 and 50 percent were obtained in the verge-side wheelpath of Sections 1 to 6. Included in these sections are those sections that were stabilized with cement and lime (Sections 4 and 5). The low strengths of Sections 4 and 5 indicate that effective stabilization did not occur and only slight modification of the plasticity of the materials was achieved (see Table 4). The absence of any hardening of the bases by a cementitious reaction was observed in the months immediately following construction and before surfacing, during which time the bases were left primed. Changes in moisture content in the base during this period, as a result of climatic variations, were mirrored by strength changes that indicated the lack of cementing and the inefficiency of the MC30 prime coat as a curing and waterproofing membrane. The use of MC30 in this instance was a result of the lack of a suitable curing membrane. It was also expected that the surfacing would be applied soon after construction, although this was not the case.

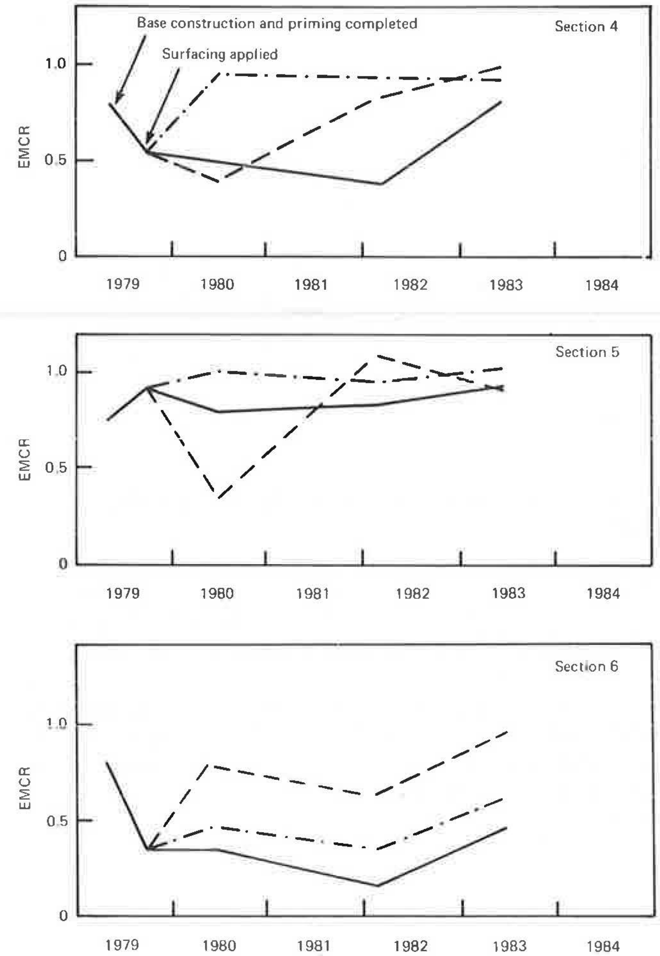
Subsequent field testing has also shown that the stabilizers had carbonated within 2 years following construction, although this in itself may not have affected the cementitious reaction. In situ CBR values in the nodular and hardpan calcrete bases

TABLE 11 POST-CONSTRUCTION MEASUREMENTS

	Activity	Frequency
Measurements on test sections	Visual examination to note general condition of the road, such as the formation of cracks, potholes, and fretting. Shoulders and side drains are also included.	Monthly
	Cross-section profiles to measure deformation and rut depths. Ruts are also measured using a 2-m straightedge.	Every 6 months
	Deflections and radius of curvature.	Every 6 months
	Surface roughness (riding quality) using a vehicle-mounted bump integrator.	Monthly
	Subsurface measurements on all pavement layers and below subgrade level to include density, moisture, and strength.	Every 12-24 months
Traffic	Traffic flows from an automatic traffic counter.	
	Classified traffic counts and axle load surveys at six monthly intervals.	
Climate	Daily measurements of rainfall and air temperature.	



8a



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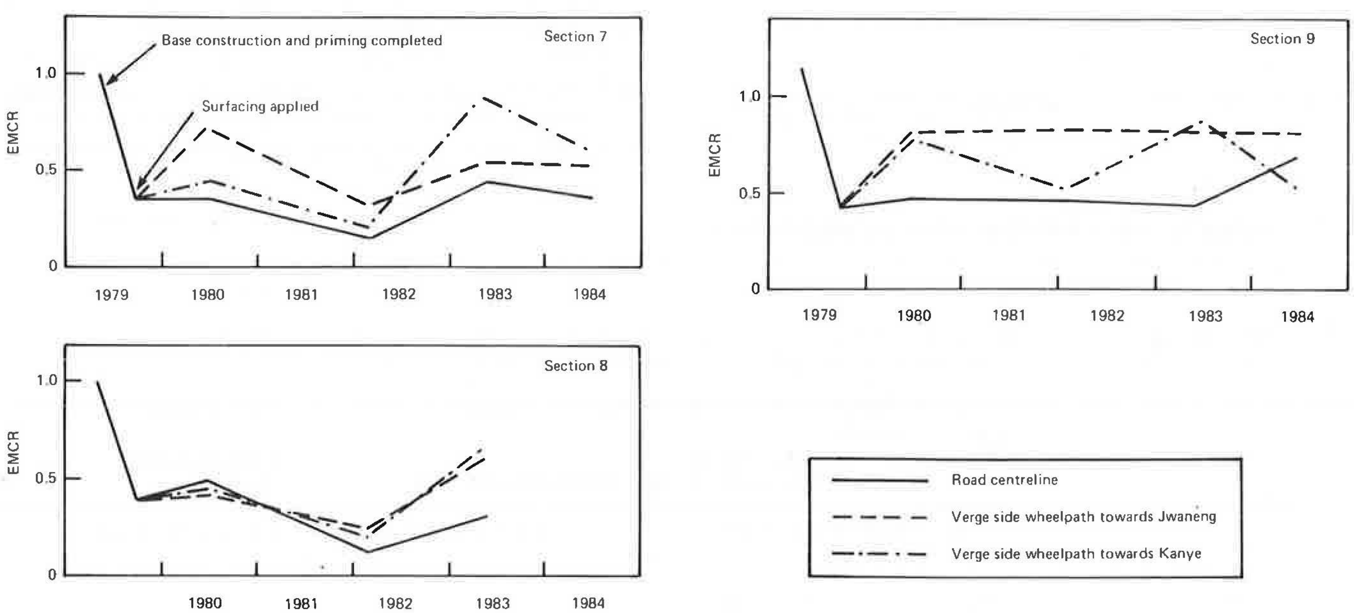
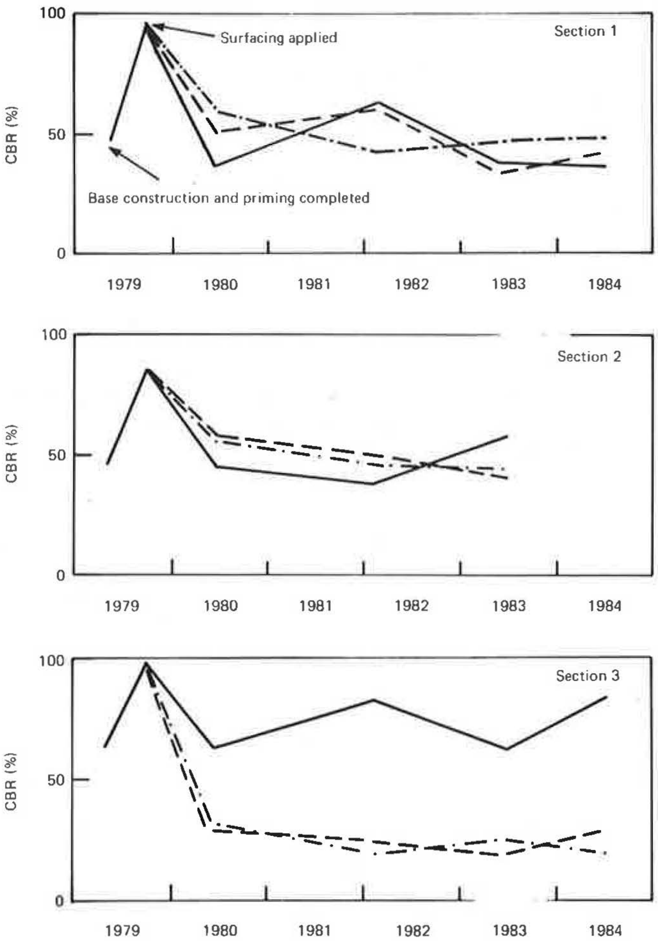
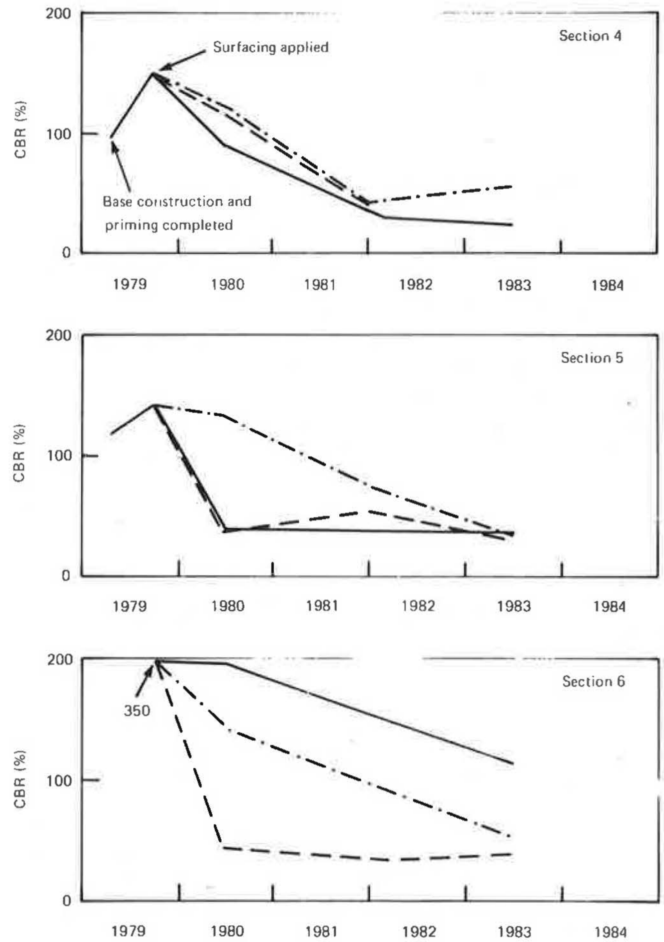


8c

FIGURES 8a-8c Equilibrium moisture content ratio of the calcrete bases.



9a



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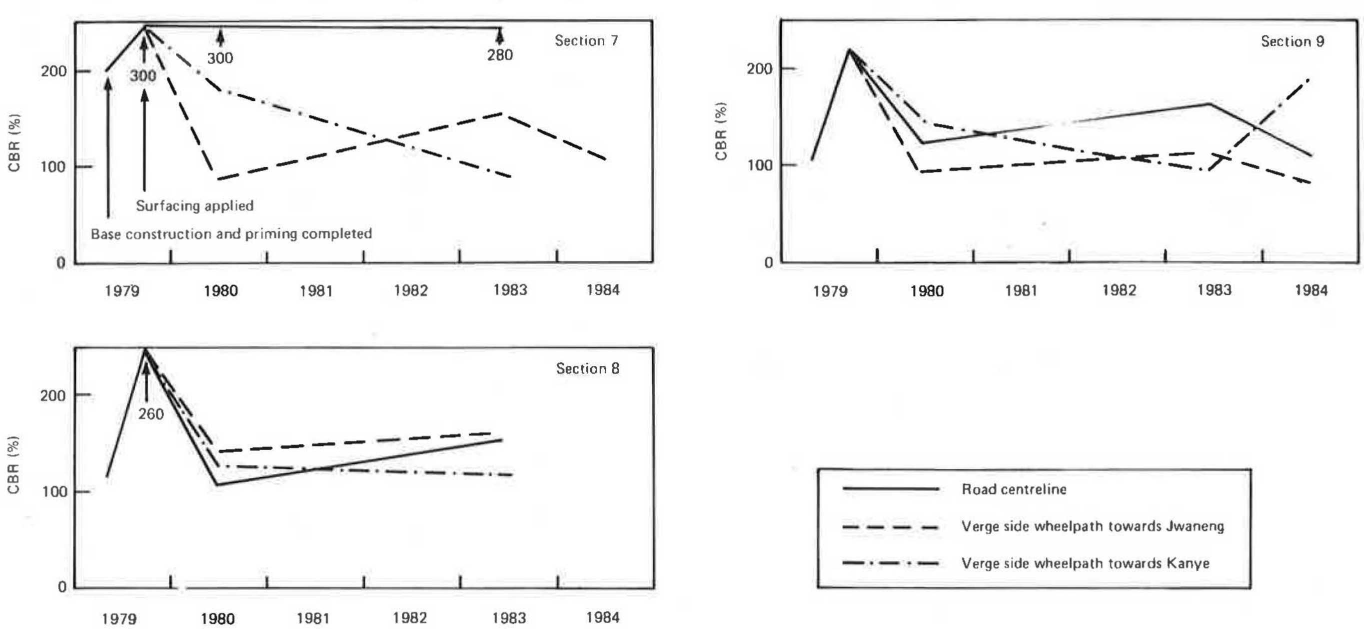


9c

FIGURES 9a-9c In situ CBR values of the calcrete bases.

(Sections 7 to 9) remained in excess of 80 percent, which validates their initial classification as better pavement materials. No difference emerged between the strength of Section 8, which was left unprimed for a number of months, and Section 7.

Testing of the Sand Subgrade and Embankment

The strength and moisture content of the sand subgrade and embankment were measured to a depth of 600 mm at positions along the centerline and in the outer (verge-side) wheelpaths. The results of tests on the top 150 mm of the sand subgrade are shown in Figures 10 and 11. The CBR values in particular show that a progressive decrease in strength occurred under the verge-side wheelpath of all sections. The rate of decrease was greatest under the poorer calcretes. This was reflected in a slight increase in the equilibrium moisture content ratio. CBR values under the outer wheelpath were as low as 15 to 20 percent. These strengths were considerably lower than those obtained at equivalent densities and moisture contents in the laboratory. This indicates that considerable overestimation occurred in the laboratory test, which was probably because the laboratory tests on sands were influenced by the confining effect of the mold.

Deflection Measurements

Transient deflection tests using the standard TRRL test method were performed at 28 test points in each section (*9*). The overall mean deflection histories for the nine calcrete sections are

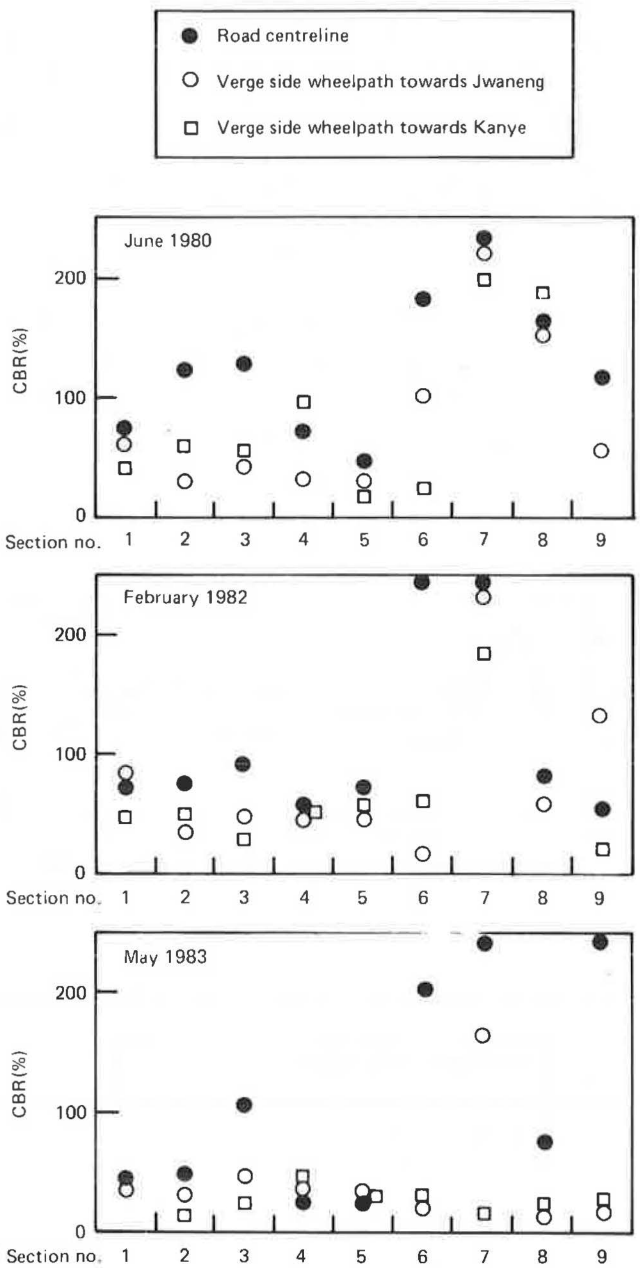


FIGURE 10 Subgrade CBR values for three monitoring dates.

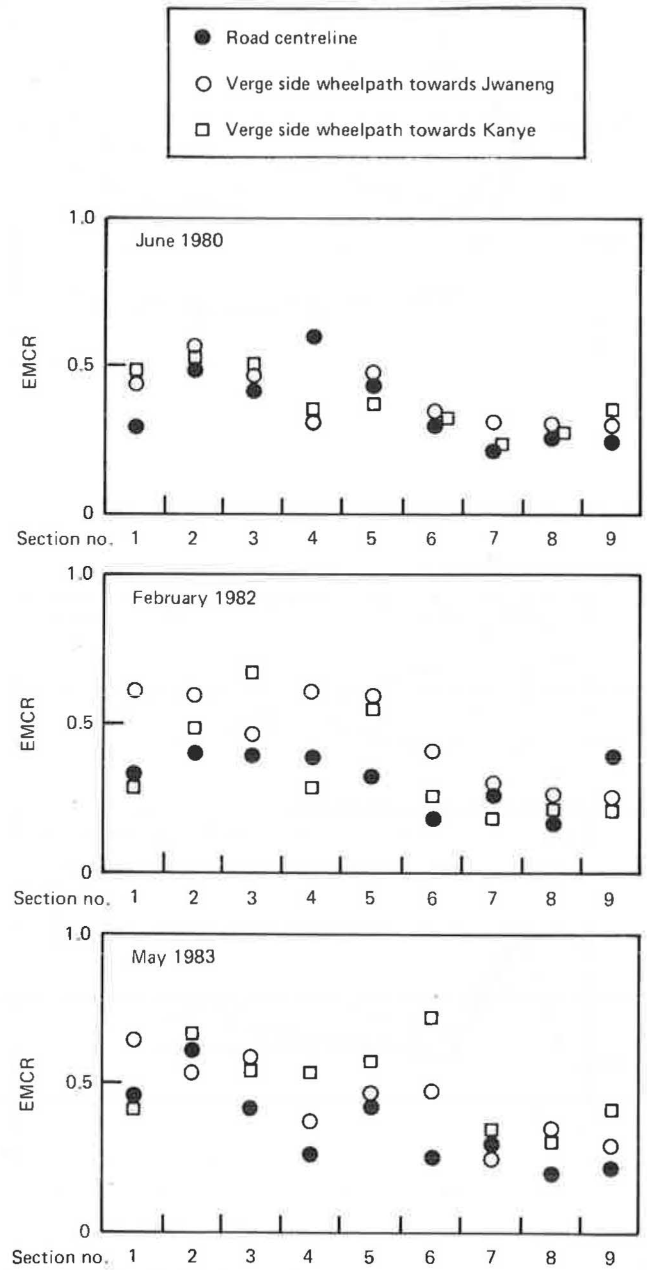


FIGURE 11 Equilibrium moisture content ratios in the subgrade.

shown in Figure 12. The highest deflections were obtained following the wet season of 1980/81; since then reasonably constant, but lower, values were measured. On the basis of deflections, the experimental pavements can be divided into two groups: Sections 1 to 5, with bases of the powder calcretes and calcified sands, and Sections 6 to 9, with bases of nodular and hardpan calcretes and the sand/calcrete mixture. A more detailed illustration of the deflection histories of two of the sections, Section 3 (the untreated plastic calcified sand) and Section 7 (the nodular calcrete), is shown in Figure 13. It can be seen that the verge-side wheelpath deflections of Section 3 are considerably higher than the centerline position. This effect is less noticeable in Section 7, because of the high strength of the nodular calcrete base material and the higher subgrade strength for the full width of the road.

Radius of Curvature

Radii of curvature were measured by conducting deflection tests with the loaded vehicle moving forward in increments to enable a deflection bowl to be obtained. The value of the test is that it can often provide more information on the load-spreading properties of the upper pavement layer than can be determined by the normal transient deflection test.

The tests were conducted in the off-side wheelpath of the more heavily trafficked Jwaneng-bound direction. The results shown in Figure 14 provide further evidence that no cementitious reaction occurred in the stabilized bases of Sections 4 and 5. These sections had lower radii of curvature than the unstabilized section that was constructed using the same original material.

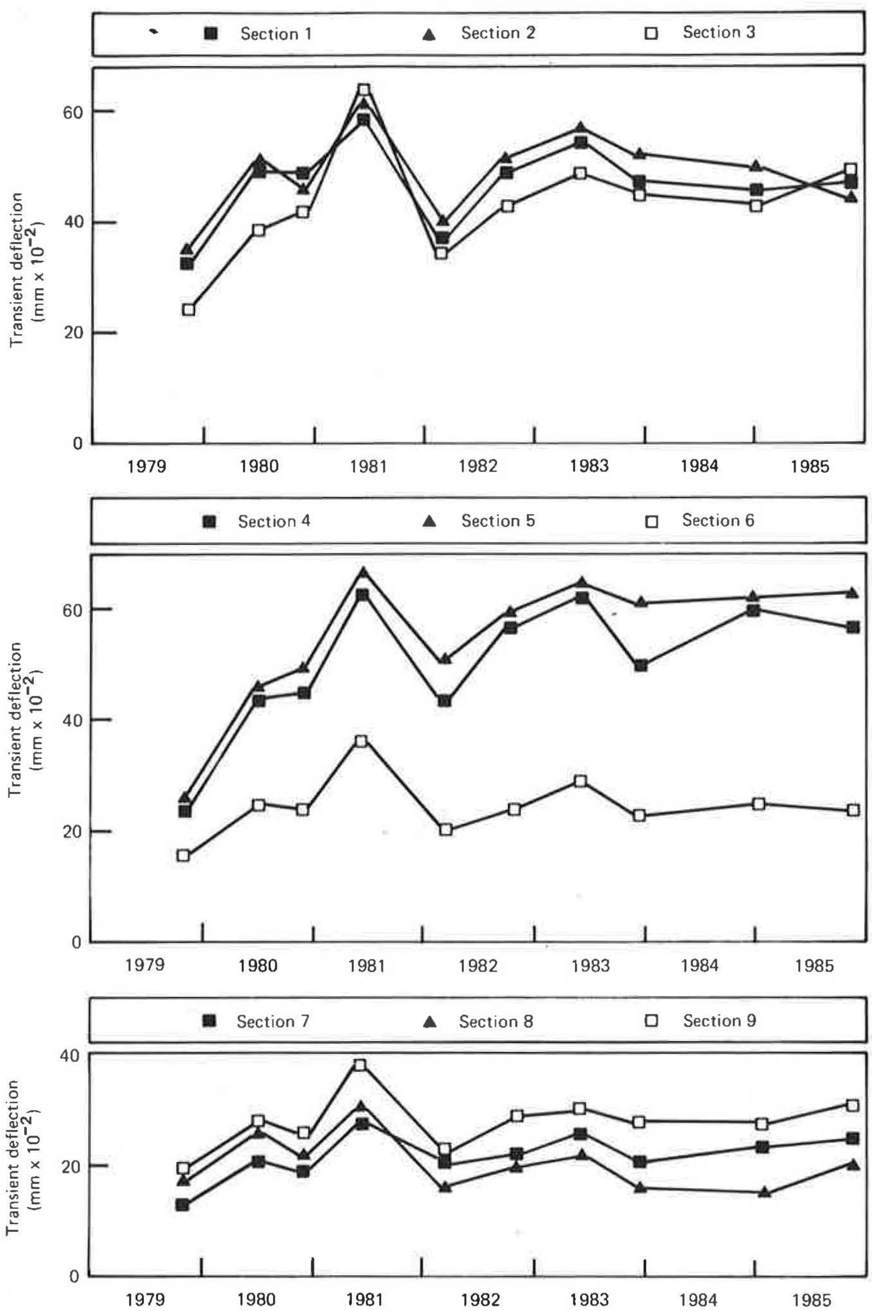


FIGURE 12 Mean transient deflection values versus time.

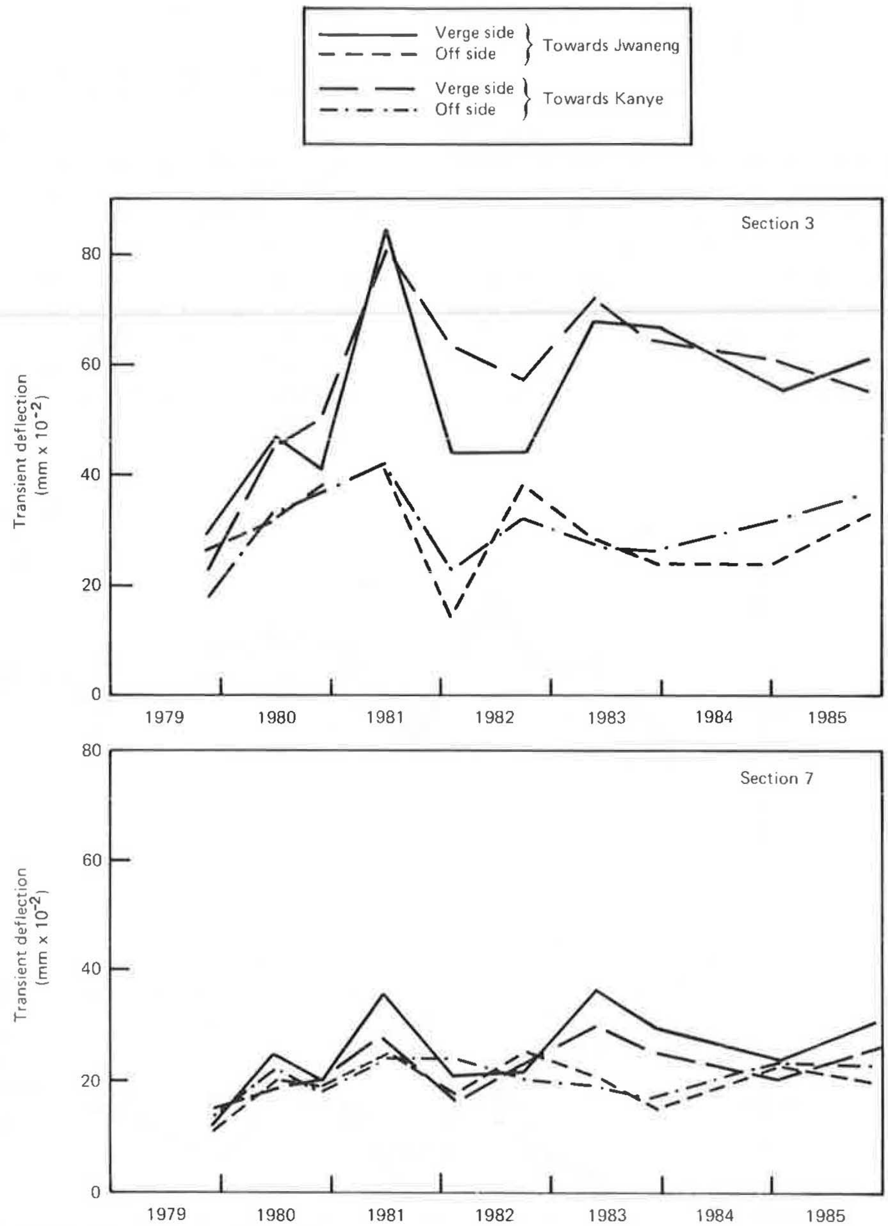


FIGURE 13 Deflection history for Sections 3 and 7.

Transverse Deformation

Rut depths were measured under a 2-m-long straight edge using a calibrated wedge. The results of measurements taken in the verge-side wheelpath of the Jwaneng-bound traffic direction are shown in Figures 15a and 15b. Deformation in all other wheelpaths was insignificant at less than 5 to 10 mm.

The sections that showed the most deformation were those sections that were originally stabilized, both mechanically and chemically. The rate of progression of deformation in these sections was greatest early in the life of the experiment, which coincided with both a high rate of loading and the high rainfall in 1980/81.

In May 1986 trenches were dug across the width of the road in those sections with excessive deformation and it was observed that the rutting was confined to the base layer.

Riding Quality

Measurements of riding quality were taken using a vehicle-mounted bump integrator. The vehicle response readings were converted to equivalent towed, fifth-wheel roughness values (expressed in mm/km) by use of the TRRL road roughness calibration beam (*10*).

The results obtained distinguished between the surface roughness of Sections 1 to 6, which had values in the range of 2500 to 3500 mm/km, and Sections 7 to 9, the values of which were between 3000 and 4500 mm/km. The relative differences in roughness confirmed the visual assessment that the more coarsely graded calcretes were rougher. The values, however, were higher than expected and may be the result of the short 100-m experimental lengths, which made it difficult for the plant to obtain smooth surface finishes, particularly in the zones of intersection.

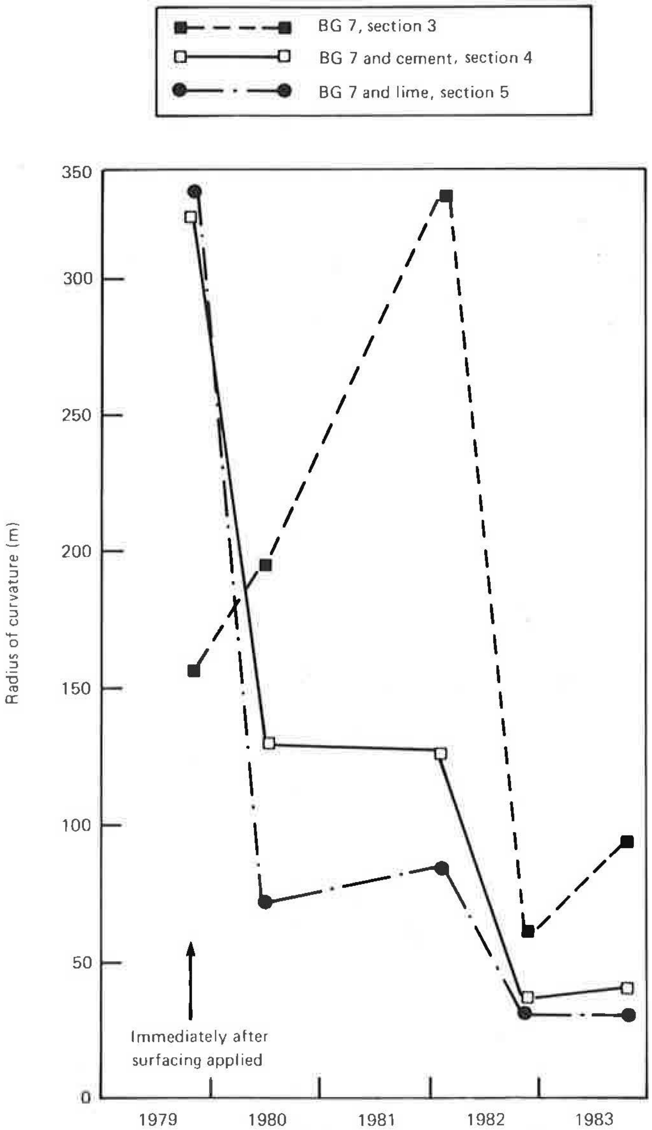


FIGURE 14 Radius of curvature history for Sections 3, 4, and 5.

Traffic

Manual, classified traffic counts conducted at six monthly intervals were supplemented by readings obtained from an automatic, pneumatic tube counter installed at the site of the experiment. Daily traffic (24-hr) has increased from 180 vehicles in November 1979 to 260 in June 1986. In this time, the proportion and number of heavy commercial vehicles have decreased as a result of the completion of the construction of the mining complex and town at Jwaneng.

Axle load surveys were conducted using the TRRL portable weighbridge system (*11*). Factors derived from the AASHO Road Test were used to express all axle loads in terms of an equivalent number of standard (80-kN) axle loads (ESA). The equivalence factors used were calculated from the following formula:

$$\text{Equivalence factor} = (\text{axle load in kgf}/8160 \text{ kgf})^{4.5}$$

In the period up to mid-1986 it is estimated that 160,000 ESA were carried in the more heavily trafficked Jwaneng-bound traffic direction. Details of the traffic and axle loading are given in Table 12.

Rainfall

Measurements taken at a nearby meteorological station at Jwaneng indicate a large variation in annual rainfall, ranging between 200 mm and 800 mm, during the life of the experiment (see details in Table 13).

The reported mean annual rainfall for the area was 400 mm, which normally falls in the summer months of November to March. The experiment therefore experienced the drought conditions of recent years and the excessively wet rainy seasons earlier in the decade that affected southern Africa.

PERFORMANCE OF THE EXPERIMENT

Calcretes As Road Bases

The results of the experiment have shown the following:

- The four untreated calcretes used in six of the sections (Sections 1 to 3 and 7 to 9) have performed satisfactorily and as well as the control sections up to the present cumulative traffic loading of 160,000 ESA during the 7 years since construction. This has been supported by the minimal amount of rutting that has occurred (values are less than 10 mm) and the absence of other forms of deformation or of cracking in any of the sections.
- The effect of initially drying the nodular calcrete base (Section 8) before surfacing was largely negated by the 6-month delay that occurred after construction of the base and the application of the surfacing. No subsequent difference in the performance between Section 8 and Section 7 has emerged.
- The powder calcrete section (Section 2), which was compacted dry of the laboratory-determined optimum, has performed well and similar to Section 1, which was compacted at a higher moisture content.
- The mechanical stabilization of the nodular calcrete with an equal amount of Kalahari sand has not been successful and rut depths greater than 20 mm have been measured in the more heavily loaded traffic direction.
- Both the cement- and lime-stabilized plastic calcified sand sections have not shown any gain in strength as a result of the introduction of the stabilizers. Substantial rutting has occurred in both sections, although it was worse (>20 mm) in the section mixed with lime. The net effect of the addition of the stabilizers was to bring about a decrease in the plasticity index of the materials and the aggregation of the silt and clay particles (see Table 3 and Figure 5).

All available evidence suggests that the primary mode of failure in the development of rutting in both the mechanically and chemically stabilized bases can be attributed to instability in the base layer. A number of possible explanations for this exist, including low bearing capacity, densification, and low internal friction in the base material. The first explanation does not provide the complete answer because lower in situ CBR values of between 20 and 30 percent were measured in the plastic, calcified sand base (Section 3) without any significant deformation occurring (see Figures 9a and 15a). Deflection measurements have also not proven suitable to differentiate

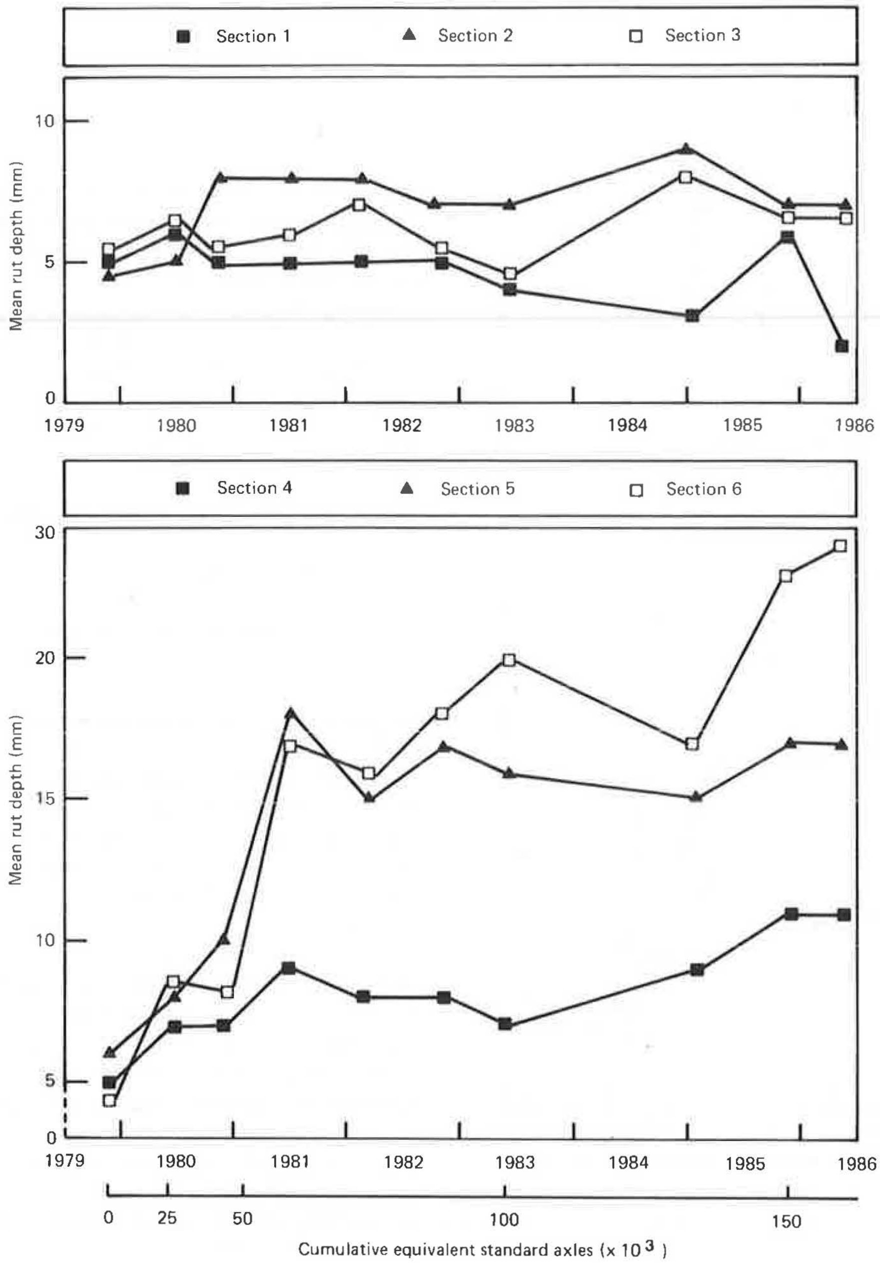
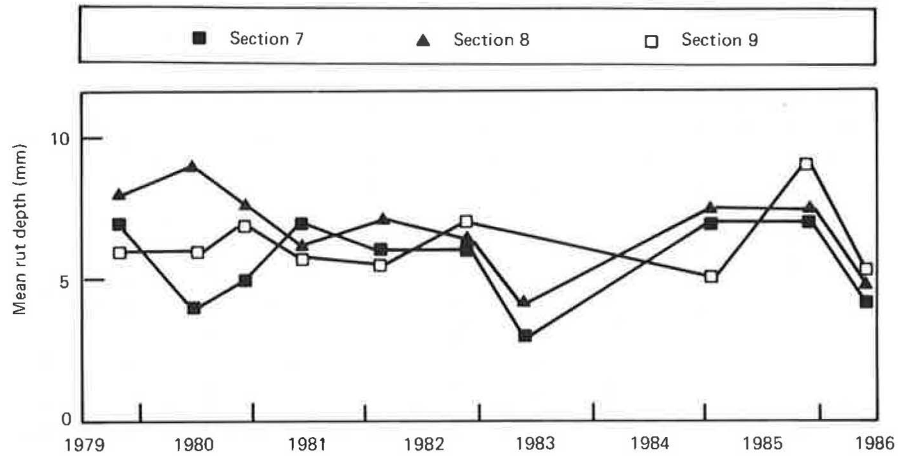


FIGURE 15a Rut depth histories for Sections 1 to 6 in the verge-side wheelpath towards Jwaneng.

between the condition of the bases. This is illustrated by comparing the deflection histories of the unfailed plastic, calcified sand base and the failed sand/calcrete base shown in Figures 16a and 16b, in which lower deflections in Section 6 were not supported by low deformation measurements. The symbols in the figures represent the classification of transverse deformation, as described in Figure 17. It is also unlikely that any substantial densification of the bases took place, because the initial constructed densities were high (95 to 98 percent relative compaction) in comparison with lower densities achieved in other sections (see Table 8).

The most likely explanation is that the low internal friction in the materials led to movement in the base, which led to rut formation. This probably resulted from the large single-sized sand component in the materials (see Figure 5), the excessive gap in the grading of the sand/calcrete mix (see Figure 4), the lower amount of silt and clay particles, and the lower plasticity (see Table 3). The net effect of this is that the bases were susceptible to traffic-induced vibration and transverse shearing forces that led to the displacement of the material. The greater resilience and cohesion of the untreated plastic calcified sand, and the more stable grading in the sand fraction with a corresponding higher silt and clay content of both this material and the powder calcrete, differentiate these materials from those of the failed sections. The properties of the soil fines that could best express stability in these materials are the ratio of the percentage passing the 425-μm sieve and that passing the 63-μm sieve, and the product of the linear shrinkage (or plasticity index) and the percentage passing the 63-μm sieve. The values for the materials used in Sections 1 to 6 are given in Table 14. The sieve sizes chosen represent standard sizes that are commonly used in the determination of the particle size distribution



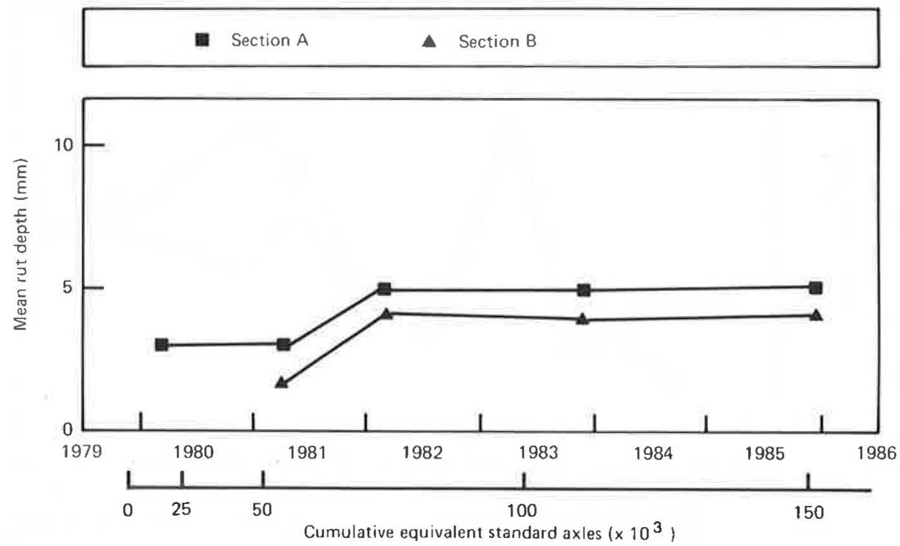


FIGURE 15b Rut depth histories for Sections 7 to 9 and A and B in the verge-side wheelpath towards Jwaneng.

TABLE 12 TRAFFIC AND AXLE LOAD DATA

Date	ADT	Cumulative vehicle passes in both directions	Vehicles >5 tonnes unladen weight				
			% of total traffic	Average ESA		Cumulative ESA	
				To Jwaneng	To Kanye	To Jwaneng	To Kanye
Nov. 1979	180	0	40	–	–	0	0
June 1980	205	43694	40	3.5	0.91	30586	7952
Feb. 1981	276	93000	22	2.3	0.23	52749	12308
Feb. 1982	250	194020	25	1.8	0.59	77082	17174
Oct. 1982	260	253327	20	2.0	0.36	89759	20019
May 1983	226	307707	22	1.6	0.23	100526	21703
Dec. 1983	260	358737	20	0.75[a]	0.12[a]	116081	23739
Jan. 1985	260	454722	16	2.34	0.53	133099	27021

Note: Estimated cumulative ESA to mid-1986 = 160,000.
[a]A typical survey.

TABLE 13 RAINFALL DATA (mm)

Year	Month												Annual rainfall (mm)
	Jan.	Feb.	March	April	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	
1979	ND	ND	ND	ND	ND	ND	ND	38	16	88	194	52	-
1980	110	111	52	11	1	-	-	-	61	24	116	48	534
1981	256	80	102	139	2	-	-	8	-	19	44	148	798
1982	61	30	102	107	-	-	4	-	-	100	106	20	530
1983	47	46	39	4	27	1	1	-	-	46	44	51	306
1984	73	18	23	1	1	2	5	10	5	32	38	9	217
1985	67	48	86	3	1	-	-	-	-	33	27	69	334
1986	41	59	13	29	-	ND	ND	ND	ND	ND	ND	ND	-

ND - No data

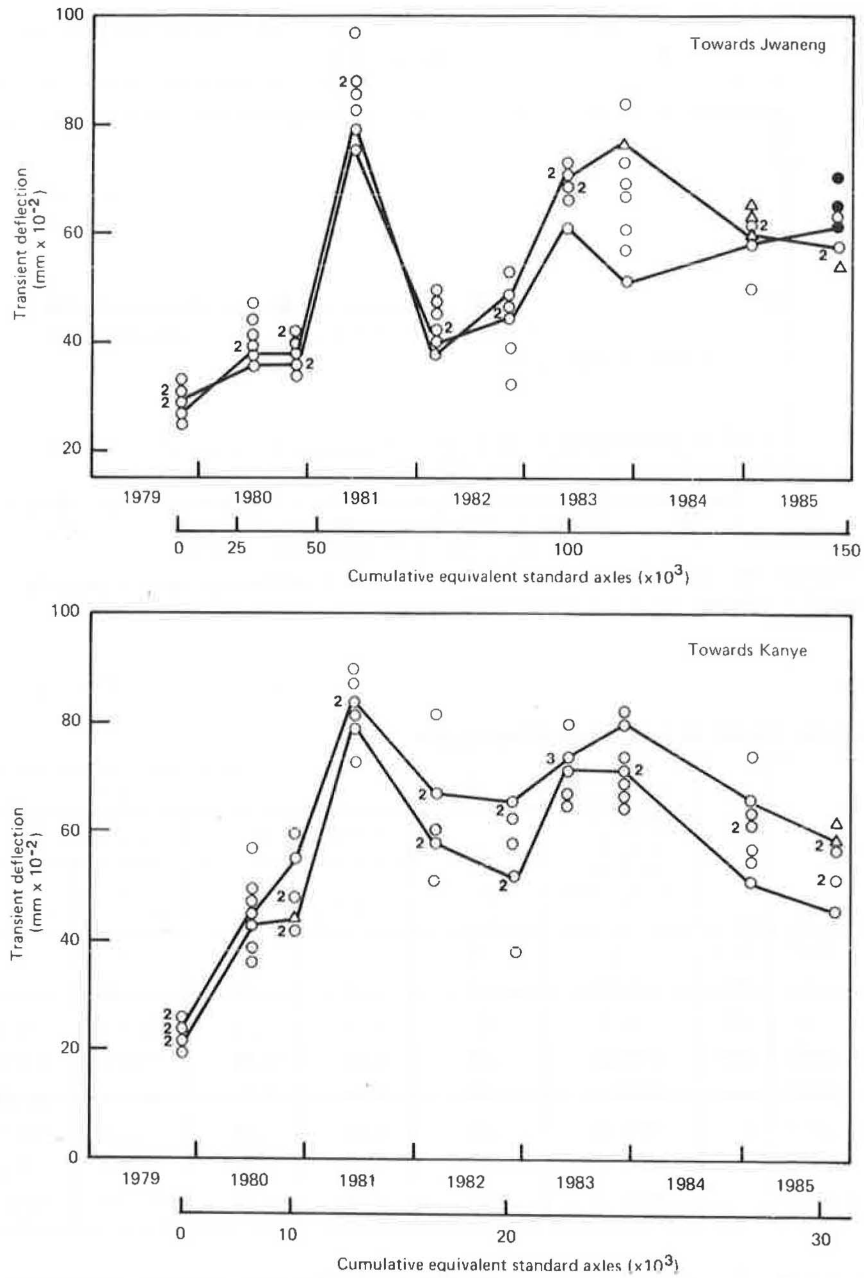


FIGURE 16a Deflection history and trend line for Section 3.

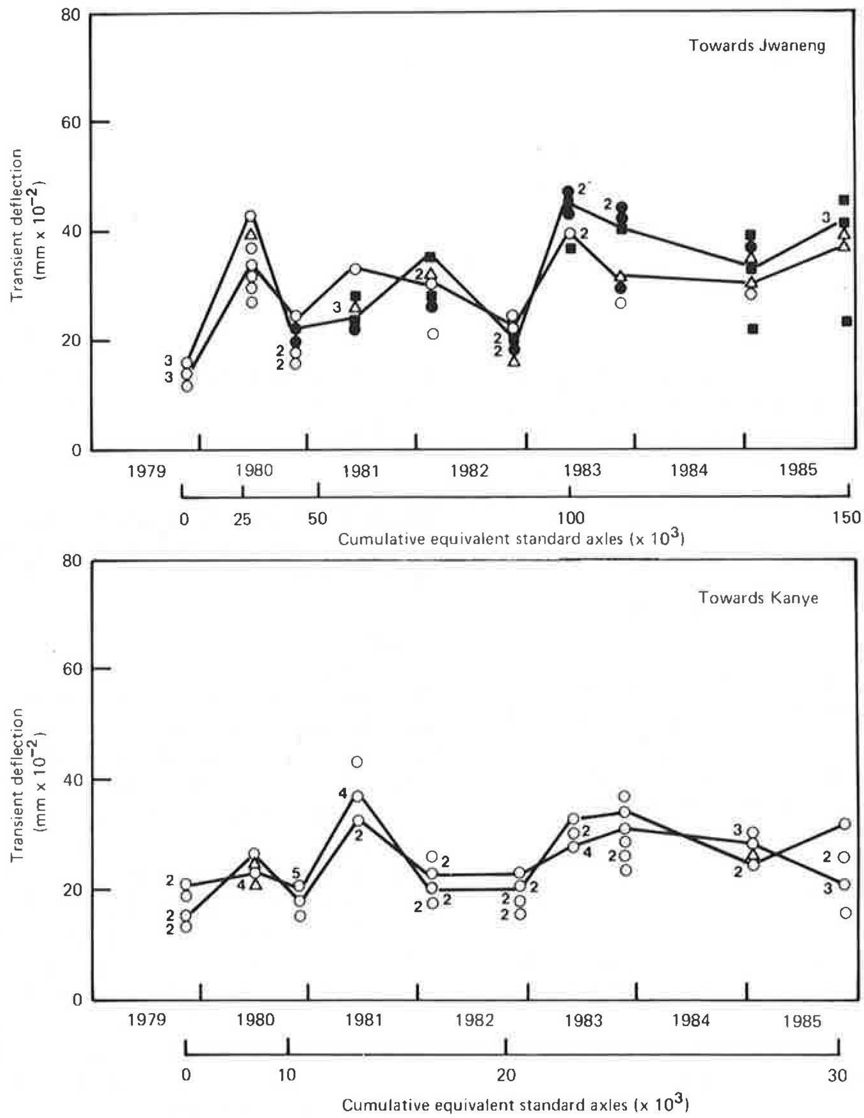


FIGURE 16b Deflection history and trend line for Section 7.

Symbol	Transverse deformation under a 2m long straightedge
○	Less than 10mm
△	10mm to 14mm
●	15mm to 19mm
▲	20mm to 25mm
■	Greater than 25mm

FIGURE 17 Symbols used to record transverse deformation (rutting) in deflection history charts.

of a soil sample by wet sieve analysis methods, thus avoiding the need for hydrometer analysis (*6*). The use of an alternative expression such as the coefficient of uniformity would, in the case of the calcrete materials, require hydrometer analysis.

In the case of the more coarsely graded materials, their stability is achieved through the mechanical interlock of the coarser particles present and the range of sizes in the grading, which follows the accepted Fuller-Talbot principles.

The reasons for the lack of strength gain following cement- and lime-stabilization are not clearly known. A lengthy mixing time and inefficient curing were undoubtedly contributory factors. The rate at which carbonation of the lime component of the chemically stabilized materials occurred may also have had an influence, although this was only first tested and detected after 2 years, by which time any hardening and flocculation should have taken place. Further research and laboratory investigations are necessary before any firm conclusions can be drawn.

Tests performed thus far on the sand/calcrete base in Section 6 have not explained the deformation that has occurred.

TABLE 14 PROPERTIES OF THE SOIL FINES OF THE CALCRETE BASES IN SECTIONS 1 TO 6

Materials Description	Section	Percentage Passing 425 μm	Percentage Passing 63 μm	Percentage Passing 425 μm / Percentage Passing 63 μm	LS (%)	PI	LS × Percentage Passing 425 μm	LS × Percentage Passing 63 μm
Powder calcrete	1 and 2	52-66	12-35	1.8-4.3	2-4.5	8-10	120-290	30-150
Plastic calcified sand	3	75-86	25-37	2.3-3.3	5.7-5.8	13-15	427-500	145-210
Plastic calcified sand and lime	4	74-81	12-18	4.3-6.6	2.1-4.1	8-11	163-324	38-53
Plastic calcified sand and cement	5	76-82	11-24	3.3-6.9	2.5-3.6	4-6	190-288	27-86
Nodular calcrete and sand	6	56-69	14-22	3.1-4.6	2.1-6.6	8-14	130-455	29-99

It is possible that inefficient mixing, either vertically or throughout the whole section, and a higher proportion of sand than calcrete compared to the intended 1:1 ratio could be responsible for the poor performance.

Calcretes As Unsurfaced Shoulder Materials

The results of the experiment have shown the following:

- The plastic calcified sand proved to be unsuitable as a shoulder material and required replacement within 2 years after completion of construction.
- The other three untreated calcretes, which included the hardpan, nodular, and powder calcretes, and the sand/calcrete mix have performed satisfactorily. However, in Section 2, which was compacted drier than the laboratory optimum, up to 50 mm of material was lost within 6 years.

The better performance of the powder calcrete indicates that a slightly better grading than that of the plastic, calcified sand and a higher calcium carbonate content are necessary requirements for poorly graded shoulder materials. No distress occurred on the shoulders of the control sections.

Performance of the Sand Side Slopes

Erosion, in the form of gullying, occurred in the side slopes of the sand embankment and led to the reduction of the slope angle from the original gradient of 1 in 4 to that of 1 in 6 or less. No subsequent erosion has occurred since this operation.

Condition of the Bituminous Surfacings

A substantial loss of the top stone layer occurred on a number of sections progressively over a period of 5 years. The loss was greatest on the coarsely graded nodular and hardpan calcrete sections (Sections 7 to 9). Loss also occurred on other sections, particularly Sections 3 and 4. Resealing of these sections was performed in 1984 using a single bitumen application with a 10-mm covering aggregate applied to the Jwaneng-bound lane of Sections 6 to 9 and a graded crusher waste (<10 mm) on the other lanes and sections. The resealing was performed to stop the loss of stone and prevent damage to the road bases.

The greater loss of stone on Sections 7 to 9 is attributed to the lack of sufficient bituminous binder. Both the hardness of the base, into which little stone embedment occurred, and the possible increased absorption of prime and binder into their more open-textured surfaces are the likely sources of the problem.

RECOMMENDED SPECIFICATIONS FOR LIGHTLY TRAFFICKED CALCRETE BASES AND SHOULDER MATERIALS

Road Bases

The results of the experiment have shown that a wider range of calcretes can be recommended for use in lightly trafficked roads than those specified previously (see Table 1). This applies to the cumulative traffic loading of 160,000 ESA that the experiment has carried so far and enables a revision to be made to the specifications that were introduced in Botswana at an earlier stage of the experiment when the traffic loading had reached 100,000 ESA (*12*). The revised specification, which is shown in Table 15, is based on a more critical examination of the calcified sands, powder calcretes, and gap-graded materials described in this paper. In particular, the importance of the distribution of sizes in the soil fines is highlighted. A minimum calcium carbonate content is also recommended, which may help to further classify calcretes, although the role of calcium carbonate in the performance of calcrete as a road-building material is not fully understood. There was no evidence from the experiment that it promoted self-stabilization, which has been suggested elsewhere (*1*).

Recommended CBR values are given for the selection of calcretes, which refer to laboratory tests performed on soaked samples. The minimum value given was that obtained for the calcified sample (Sample BG 7). The differences between laboratory test results and the values determined in situ in the full-scale experiment are a cause for concern, but a CBR criterion is included because it is the most commonly used soil strength test for road-making materials.

Of further concern is the problem of achieving high levels of compaction for some calcretes, as shown by the results obtained for the powder calcrete sample BG 6. Because this behavior is not fully understood, it is recommended that normal compaction requirements should continue to be specified and that field compaction trials should also be conducted to determine whether the problem exists.

Finally, by comparing the recommendations given in Tables 1 and 15, the calcrete requirements for a range of traffic categories can be obtained. Calcretes that satisfy the specifications of TRRL Road Note 31, provided that the criteria for aggregate hardness and soluble salts are also met, should be suitable for traffic levels up to 2.5×10^6 ESA. As described

TABLE 15 REVISED GUIDELINE SPECIFICATIONS FOR LIGHTLY TRAFFICKED CALCRETE BASES

Test	Percentage passing the 425 μm sieve[a]		
	10-50	50-65	65-85
Maximum particle size (mm)	75-10	75-10	75-10
% passing 63 μm sieve	5-25	15-35	20-35
Maximum ratio of % passing 425 μm and 63 μm	Not specified	3.5	3.5
Linear shrinkage (%)	<10	<6	<6
Plasticity index	<25	<15	<15
Maximum LS x % passing 425 μm	600	500	500
LS x % passing 63 μm	Not specified	30-150	120-210
Minimum 4-day soaked CBR at field density (%)	40	40	40
Minimum calcium carbonate content (%) of the material passing 425 μm	12	12	12

Note: Up to a design traffic level of 160,000 ESA.
[a]After compaction using wet sieve analysis methods.

earlier, limits are given for the permissible range of calcretes that can be used for low traffic categories (160,000 ESA). Between these extremes, in four other traffic bands, are the calcrete requirements recommended in NITRR Bulletin 10 (*1*). Assuming an axle load factor of 1.0 per commercial vehicle greater than 3 tons, which is given in TRH14, these bands are equivalent to $<0.25 \times 10^6$ ESA, $0.25 - 0.5 \times 10^6$ ESA, $0.5 - 1.0 \times 10^6$ ESA, and $1.0 - 2.0 \times 10^6$ ESA (*13*). It can therefore be seen from the two tables that there is a requirement of increasing quality of calcrete to meet increasing traffic loading categories. This is important when it is considered that the combined information drawn from three separate and independent sources of research fits into a smooth trend.

Shoulder Materials

As a result of the unsatisfactory performance of the plastic, calcified sand as an unsurfaced shoulder material, separate guidelines for material selection are required. These are contained in Table 16 and include restrictions on the maximum allowable amount of material passing the 425-μm sieve and a minimum calcium carbonate content. The guidelines in Table 15 should be adopted in cases in which shoulders are to receive a protective bituminous seal.

ACKNOWLEDGMENTS

The work described in this paper forms part of the research program of the Overseas Unit of the Transport and Road Research Laboratory. Much of the supervision and monitoring of the research was performed by Mr. M. Stewart and Mr. P. A. K. Greening of TRRL. The research was undertaken as part of the Joint Road Research Project of TRRL and the Botswana Ministry of Works and Communications. The authors are indebted to the staff of the Roads Department who participated in the work.

TABLE 16 GUIDELINE SPECIFICATIONS FOR UNSURFACED CALCRETE SHOULDER MATERIALS IN LIGHTLY TRAFFICKED ROADS

Test	Percentage passing the 425 μm sieve[a]	
	10-50	50-65
Maximum particle size (mm)	75-10	75-10
% passing 63 μm sieve	5-25	15-35
Maximum ratio of % passing 425 μm and 63 μm	Not specified	3.5
Linear shrinkage (%)	<10	<6
Plasticity index	<25	<15
Maximum LS x % passing 425 μm	600	500
LS x % passing 63 μm	Not specified	30-150
Minimum 4-day soaked CBR at field density (%)	50	50
Minimum calcium carbonate content (%) of the material passing 425 μm	25	25

[a]After compaction using wet sieve analysis methods.

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An Aggregate Thickness Design That Is Based on Field and Laboratory Data

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A summary is provided of the results of a research program sponsored by the Federal Highway Administration entitled "Design and Operation of Aggregate-Surfaced Roads." Major emphasis is devoted to correlating field and laboratory data and developing the Clegg Impact device as an alternative method for determining in-place density and strength evaluation. Twelve sites in Michigan, Iowa, Texas, Oregon, North Dakota, Montana, West Virginia, and South Carolina were selected for tests. The determination of sites included a variety of climatic and subgrade conditions to allow these factors to be included in the analysis. Field data collected at each field site included roadway dimensions, thickness of the aggregate surface, Clegg Impact Values, in-place density, and moisture content. Bag samples of subgrade and surface aggregate were collected for laboratory tests that included classification tests, gradation, durability, abrasion tests, Clegg Impact Value, and California Bearing Ratio (CBR). Results from laboratory and field tests were analyzed to develop relationships between the various test parameters. A regression analysis of laboratory results showed good agreement between Clegg Impact Value and CBR. The relationship that was developed compared with the results given by Clegg in his work. Statistical relationships that related surface and subgrade conditions in the field to Clegg Impact Value and field moisture content were also obtained and were related to the equation for aggregate thickness design. A discussion that relates the results to the design of low-volume, aggregate-surfaced roads is included.

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Millions of miles of roads in the United States are aggregate surfaced. In most cases they have relatively low traffic levels and depend on inexpensive design, construction, and maintenance. Maximum use of local materials and empirical procedures that are based on experience are required to minimize cost.

The results discussed in this paper were collected as part of a project to develop a design procedure for aggregate-surfaced