COLLABORATIVE RESEARCH PROGRAMME ON HIGHWAY ENGINEERING MATERIALS IN THE SADC REGION

Performance of Chemically Stabilised Roadbases: Results and Recommendations from Studies in Southern Africa

Transport Research Laboratory, Crowthorne, Berkshire, United Kingdom

Performance of Chemically Stabilised Roadbases: Results and Recommendations from Studies in Southern Africa

by C S Gourley and P A K Greening

PROJECT REPORT PR/OSC/168/99

Performance of Chemically Stabilised Roadbases: Results and Recommendations from Studies in Southern Africa

by C S Gourley and P A K Greening

Copyright Transport Research Laboratory November 1999

Subsector:	Geoscience
Theme:	G4
Project Title:	Natural gravels for road building
Project Reference:	R6020

APPROVALS

Project Manager		Quality reviewed	

PERFORMANCE OF CHEMICALLY STABILISED ROADBASES:

Results and Recommendations from Studies in Southern Africa

Contents

ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
BS	British Standard
CAH	Hydrated Calcium Aluminates
CBR	California Bearing Ratio
CEBTP	Centre Experimental de Recherches et d'Etudes du Bâtiment et des Travaux Publics
CSH	Hydrated Calcium Silicates
CSIR	Council for Scientific and Industrial Research
DCP	Dynamic Cone Penetrometer
FHWA	Federal Highways Authority
GOZ	Government of Zimbabwe
ICC	Initial Consumption of Cement
ICL	Initial Consumption of Lime
MDD	Maximum Dry Density
MOWS	Ministry of Works and Supply
NITRR	National Institute for Transport and Road Research
ORN	Overseas Road Note
RRL	Road Research Laboratory
RUCS	Residual Unconfined Compressive Strength
SADC	Southern African Development Community
SATCC	Southern African Transport and Communications Commission
SFRDP	Secondary and Feeder Road Development Programme
SNC	Modified Structural Number
TPA	Transvaal Provincial Authority
TRH	Technical Recommendations for Highways
TRL	Transport Research Laboratory
TRRL	Transport and Road Research Laboratory
UCS	Unconfined Compressive Strength

Executive Summary

Background and aims

Soil stabilisation, using cement and lime, has been widely used throughout the world for over 40 years. However, recently there has been loss of confidence in the chemical stabilisation process in parts of Southern Africa. Some studies have indicated that road failures could be attributed partly to the degradation of the stabilising agents and their cemented products through the process known as 'carbonation'. As a result, some countries in the region have discontinued the use of chemical stabilisation in road projects. Conversely, there are also many examples within Southern Africa where the use of chemical stabilisation has been very successful, even on roads that have received little maintenance. This conflicting evidence has resulted in considerable uncertainty about the continued use of stabilisation as an option for road projects.

Against this background, a project has been undertaken to investigate the performance of a variety of chemically stabilised roadbases to assess their performance over time. The project had the following objectives:

   a) Establish reasons for the disparate performance of chemically stabilised roadbases in the region

   b) Evaluate the performance of chemically stabilised gravel roadbases in relation to current pavement design criteria

   c) Make recommendations and provide guidelines for the chemical stabilisation of granular materials which are based on performance data, so that confidence is restored in this method of improving gravels for use in roadbases

Method of study

Test sections were selected on roads in Zambia, where chemical stabilisation had been used extensively, and where road performance could be measured. Information from these was supplemented with some test sections in Zimbabwe, and by drawing on work from a TRL project in Botswana carried out in the 1980s. Sections of the road network of widely different ages (approx. 5-25 years), and with different base material and stabiliser types, were investigated. The degree of carbonation was determined using a number of different methods. The general condition of the road was assessed using visual condition surveys, which also included rut depth measurements. The field investigations aimed to measure the residual strength of the roadbase and to sample materials for laboratory investigations to determine if the stabilised materials had carbonated and whether or not they had lost sufficient strength for the layer to cease to be an effective roadbase.

Findings

Reasons for disparate performance in the region

1)   The distress effects reported elsewhere in the region, including rutting, shearing, pumping and scabbing of the surfacing were not observed on the sites investigated. Pot-holing of the surface was observed, but this was primarily due to poor surface maintenance.

2)   Shrinkage, causing block cracking, was prevalent in Zambia. This was probably due to the use of a high percentage of stabiliser, typically between three and six per cent. In Zimbabwe, only a one to two per cent cement content is used, resulting in a lower incidence of observed cracking. Further investigation is required to determine the factors which result in this improved pavement performance with a lower cement content.

3)   Many of the bases in Zambia appear to have lost some strength, assuming that the materials met the original CBR specification of 180 per cent. Over 30 per cent of the bases in Zimbabwe and Zambia showed *in situ* strengths greater than this. However, nearly all the sections had *in situ* CBRs greater than 80 per cent and, from a structural point of view, carried much higher volumes of traffic than suggested by the original designs.

4)   The importance of recording and archiving field records in a convenient and reliable way, such as in a computerised database, was highlighted during the study. Records which would have proved invaluable to the investigation were destroyed due to poor management and storage.

Evaluation of performance relative to pavements design criteria

5)   Correlations between CBR and UCS and also between the strength coefficients of stabilised and unstabilised materials were developed for use in structural number computation. A new relationship was developed for calculating the structural coefficient in terms of UCS which should be more appropriate for stabilised granular materials.

6)   The point load test proved quick and reliable, and the relationship:
       $UCS = 5.631(Is(50))$
     was developed which enables the UCS of the material to be estimated from the point load index.

7)   Increases in plasticity due to carbonation were not generally observed. It was unclear whether the few high values measured were due to reversal of plasticity or to poor mixing at construction.

Recommendations on the use of chemical stabilisation

8)   The results of the study showed that although all cement and lime stabilised bases investigated were carbonated, the pavements performed reasonably well.

9)   Chemical stabilisation of laterite, quartz and other common roadbase materials in the region can be carried out successfully. Thus, chemical stabilisation still provides a

practical and cost-effective option of improving materials for the construction of durable road pavements in many circumstances.

10) A revised guideline is not required as there are adequate recommendations available. However, an upper strength limit for roadbase material of 2MPa should be considered to prevent the development of excessive shrinkage cracking and an imbalance in the stiffness of the upper layers of otherwise lightly designed pavements.

11) Chemical stabilisation of calcretes appears to present particular problems which are still not fully explained and need further investigation.

12) Where materials are encountered which could give potential durability problems, additional testing using mechanical wet/dry brushing and residual UCS (carbonated) are recommended. It is also strongly recommended that the Initial Consumption of Lime (ICL) or Initial Consumption of Cement (ICC) test is carried out as a matter of course when aiming to establish a design stabiliser content for pavement materials.

Recommendations are also made on construction practices that can reduce the problems of carbonation in the field.

1.   Introduction

1.1   Background

In their report on *Road Deterioration in Developing Countries*, the World Bank (1988) recommends several policy actions aimed at arresting deterioration of road networks in developing countries. One action is to identify methods of road construction that produce robust pavements which can survive with a minimum of maintenance. Soil stabilisation offers one practical means of securing this objective, and can be used both for the construction of new roads and for the rehabilitation of older roads.

Soil stabilisation has been widely used in the Africa Region and elsewhere in the world for over 40 years. Both lime and cement have been used to chemically stabilise roadbase materials. Where gravels which meet the roadbase specification are scarce, or cannot be located close to the road, chemical stabilisation of the local materials often provides a cheaper option than hauling higher quality materials over long distances, or using expensive crushed stone. Improving locally available gravels on roads where traffic levels are relatively low can be a particularly cost-effective alternative.

Studies in South Africa carried out by the Council for Scientific and Industrial Research (CSIR) and others in the early 1980s, firstly on the Grootfontein-Runda road in Namibia, indicated that road failures could be attributed partly to the degradation of the stabilising agents and their cemented products through the process known as 'carbonation'. As a result of reports circulated at that time to road authorities both in South Africa and elsewhere in the region, there was a general loss of confidence in the chemical stabilisation process and some countries in the region discontinued the use of chemical stabilisation in road projects. Later reports issued by CSIR outlining methods to mitigate the carbonation problem did not dispel the industry's fears. Conversely, there are also many examples within Southern Africa where the use of chemical stabilisation has been very successful, even on roads that have received little maintenance. This conflicting evidence has resulted in considerable uncertainty about the use of stabilisation as an option for road projects.

1.2   Objectives

Against this background, a project has been undertaken with the following objectives:

   a) Establish reasons for the disparate performance of chemically stabilised roadbases in the region

   b) Evaluate the performance of chemically stabilised gravel roadbases in relation to current pavement design criteria

   c) Make recommendations and provide guidelines for the chemical stabilisation of granular materials which are based on performance data, so that confidence is restored in this method of improving gravels for use in roadbases

1.3   Content of the report

Chapter 2 of the report contains a description of the basic principles of chemical stabilisation, and this is followed in Chapter 3 with a description of the problem of carbonation. Chapter 4 puts forward the methodology adopted by the study, and the results are summarised in Chapter 5. Detailed results are given in Appendix A. The findings of the research are discussed in Chapter 6, including those on the impact of carbonation on the performance of stabilised roadbases. A detailed discussion is also provided of the question of the impact of stabilisation on road strength. Finally the conclusions of the study are given in Chapter 7.

2. Chemical stabilisation

2.1 Purpose of stabilisation

The engineering properties of soils and gravels, such as plasticity, strength and permeability, can often be improved significantly by the addition of chemical stabilising agents. In this study, stabilisation of roadbase materials with Portland cement or lime are considered. These are the most commonly used chemical stabilising agents for roadbases.

Where a small amount of chemical stabiliser is added to material, the process is known as 'modification'. With some materials, relatively little hardening or increase in compressive strength occurs with the addition of small amounts of lime. There may, however, be benefits from an improvement in material properties. Moderate strength gains can occur from modification by the addition of small amounts of cement. Although relatively small amounts of stabiliser are used in 'modification', considerable improvements to the performance of the materials can result. Where larger amounts of stabiliser are used, and a larger increase in strength results, the term stabilised (or cemented) is normally used. A suitable boundary between modification and stabilisation has been suggested as a seven-day unconfined compressive strength of 0.8MPa.

2.2 Types of stabilisation

2.2.1 Cement

Cement comprises calcium silicates and aluminates, and calcium oxide. In the presence of water, these form hydrated compounds which harden over time to produce a strong cemented matrix in which the particles of soil or granular material are embedded. Initially, this chemical reaction is quite rapid, but the rate decreases with time.

Cement can be used to stabilise most soils. Exceptions are those with a high organic content, which retards the hydration process, and those with a clay content outside the normal specification, where it is difficult to mix the soil/cement mixture evenly. Addition of cement to base materials results in a reduction in plasticity and swell, and an increase in strength and bearing capacity. CBR values well in excess of the minimum requirement for unstabilised gravels (usually 80 per cent, soaked at the required field density) normally result.

2.2.2 Lime

Lime (calcium oxide, or quicklime) can be used to modify or lower the plasticity of the available materials, or to dry out materials which are wetter than the optimum moisture condition at construction. Some cementation and strengthening of the matrix also occurs. The chemical is normally used in its hydrated or slaked form (calcium hydroxide), and is most often applied where soils have a high plasticity and where mixing in cement is difficult. Sufficient excess lime is added to the material to ensure that the stabilised mixture retains a high pH of at least 12.4. This high pH environment is essential to ensure the long term stability of the hydration products.

During the hydration of cement, lime is also produced as a by-product of the reactions and this aids the production of a suitable pH environment.

2.3   Tests and guidelines for stabilised soils

2.3.1   Materials and stabilisers

The *TRL Overseas Road Note 31* (ORN 31) guidelines for materials to be stabilised are given in Table 2.1.

Table 2.1 Desirable properties of material before stabilisation

BS test sieve	Percentage by mass of total aggregate passing test sieve		
	Roadbase 1	Roadbase 2	Stabilised sub-base
53.0mm	100	100	-
37.5mm	85 – 100	80 - 100	-
20.0mm	60 – 90	55 - 90	-
5.0mm	30 – 65	25 - 65	-
2.0mm	20 – 50	15 - 50	-
425µm	10 – 30	10 - 30	-
75µm	5 – 15	5 - 15	-
	Maximum allowable value		
Liquid limit	25	30	-
Plasticity index	6	10	20
Linear shrinkage	3	5	-
Note: It is recommended that materials should have a coefficient of uniformity of 5 or more. *Source:* TRL 1993			

A guide to the appropriate use of cement and lime stabilisers is shown in Table 2.2.

Table 2.2 Guide to the type of stabilisation likely to be effective

	Soil properties					
	More than 25% passing the 75µm sieve			Less than 25% passing the 75µm sieve		
Type of stabilisation	PI ≤ 10	10<PI ≤ 20	PI>20	PI ≤ 6 PP ≤ 60	PI ≤ 10	PI > 10
Cement	Yes	Yes	*	Yes	Yes	Yes
Lime	*	Yes	Yes	No	*	Yes
Lime-Pozzolan	Yes	*	No	Yes	Yes	*
Notes: PP Plasticity product = percent passing 75µm x Ip * Agent will have marginal effectiveness						

2.3.2 Test methods

Cement or lime stabilised materials are usually approved on the basis of strength tests carried out on the materials after the stabiliser has had sufficient time to cure. The most commonly used methods are the Unconfined Compressive Strength (UCS) Test, for cement stabilised materials, and the California Bearing Ratio (CBR) Test for lime stabilised or modified materials.

Unconfined compressive strength
The UCS Test is carried out on cylindrical or cubical samples by mixing the material at the desired moisture and stabiliser content, and compacting the material into a mould to either a pre-determined density or a given compactive effort. The use of cylinders or cubes depends partly on the particle size of the material, with cubical samples preferred for coarser materials. As the increase of strength of stabilised materials occurs over relatively long periods of time, samples are cured for 7, 14 or 28 days prior to testing. A seven-day curing period, although arbitrary, is often chosen as a convenient reference for cement treated materials, whilst a longer twenty-eight day period is chosen for lime treated materials to take into account their slower strength gain.

California bearing ratio
The CBR test is carried out on samples compacted into cylindrical moulds. The reference value for the results is the standard value expected for a well-graded and non-cohesive crushed stone, which has a CBR of 100. Stabilised and modified soils can have CBR values well in excess of this value and at these strength levels the UCS test is preferred.

2.3.3 Specifications

The *ORN 31* (TRL 1993) and the *TRH 13* specifications (Department of Transport South Africa 1986) do not differentiate between lime and cement stabilised materials. *ORN 31* gives strength criteria for three stabilised materials which can be used in the structural layers of pavements, and these are shown in Table 2.3. The *TRH 13* recommendations for stabilised gravel materials are a seven-day strength of 0.75-3.0MPa at 100 per cent mod AASHTO, or 0.5-2.0MPa at 97 per cent mod AASHTO density depending on the traffic level.

Table 2.3 *ORN 31* strength criteria

Pavement layer	Strength requirements (MPa)
Cement or lime stabilised sub-base (CS)*	0.75 - 1.5
Roadbase 1 (CB1)	1.5 - 3.0
Roadbase 2 (CB2)	3.0 - 6.0

Note:
*   For CS materials, a CBR value of 70 after 7 days moist curing
         and 7 days soaking is also given
CB1   This material is actually a cemented sub-base in high traffic
         designs under an asphalt wearing course and crushed rock
         roadbase
CB2   This material is used as roadbase with design traffic up to
         10 million cumulative equivalent standard axles
Source: TRL 1993

These specifications are for tests on cylindrical samples and should have the correction factors in Table 2.4 applied for comparison to Overseas Road Note 31.

Table 2.4 Correction factors for cylindrical samples

Size (mm)	Correction factor
200 x 100	1.25
115 x 105	1.04
152 x 127	0.96

Experience suggests that excessive cracking is likely to occur at the cement contents required to give strengths over 3.0MPa, and that lower strengths are adequate especially for roadbases carrying relatively low levels of traffic.

2.3.4 Comparison of specifications

Examples of some current specifications for stabilised natural gravel roadbases and sub-bases are given in Table 2.5. Comparison of specifications is complicated by differences in test methods, sample size, curing time and moisture content. Further details can be found in Netterberg (1991) and Sherwood (1993).

The *ORN 31* strength recommendations for chemically stabilised base (at 97 per cent mod AASHTO compaction), of 1.5-6.0MPa, appear to be high when compared with the *TRH 13* recommendations of 0.5-2.0MPa and other specifications shown in Table 2.5. The *ORN 31* recommendations appear to be particularly conservative for relatively dry regions and may reflect a broader range of climate. It is also worth noting that the grading and Atterberg limits recommended in *ORN 31*, and shown in Tables 2.1 and 2.2, seem particularly severe, especially in conditions with a relatively dry climate and low levels of traffic. In these conditions, these materials could be suitable without modification.

Table 2.5 Examples of current specifications for stabilised natural gravels

Specification source and reference	UK (TRRL) (1977)		France (CEBTP) (1980)					UK (TRL) (1993)			Malawi (MOWS) (1978)	Zambia (GOZ) (1973)	S. Africa (NITRR) (1985)			
Traffic category (10^6 esa)	n.s.		T3-T5 (more than 10)		T1-T2 (less than 10)		T3-T5 (more than 10)	>6	<10	<10	n.s.	n.s.	n.s.	n.s	<12	<0.8
Material type	Soil stabilised with:		Sub-base lateritic gravel treated with:		Roadbase gravel improved with:		Roadbase gravel stabilised with cement	CB1 Base 2	CB2 Base 1	CS Sub-base	Roadbase	Roadbase	C1	C2	C3 Road-base	C4 Road-base
	cement	lime	cement	lime	cement	lime										
Maximum size (mm)	n.s	n.s	10-50	10-50	10-50	10-50	10-50	53	n.s.	n.s.	37.5	53	37.5	37.5	63	63
Uniformity coefficient	>5	>5	n.s	n.s	n.s	n.s	>10	>5	n.s.	n.s	n.s	n.s	n.s	n.s	n.s	n.s
% passing 425 µm	n.s	≥15	n.s	≥15	n.s	<15	n.s	10-30	n.s	n.s	n.s	n.s	n.s	n.s	n.s	n.s
% passing 80 µm	n.s	n.s	<35	<35	<35	<35	<35	5.15	n.s	n.s	n.s	<10	n.s	n.s	n.s	n.s
Plasticity index (%)	n.s	>10	<30	10-30	n.s	10-25	<25	≥6	≥10	<20	<12	n.s	<6	<6	n.s	n.s
Liquid limit	n.s	n.s	n.s	n.s	n.s	n.s		<25	n.s	n.s	<40	n.s	n.s	n.s	n.s	n.s
Linear shrinkage	n.s	n.s	n.s	n.s	n.s	n.s		3	n.s	n.s	n.s	n.s	n.s	n.s	n.s	n.s
Plasticity modulus	n.s	n.s	<2500	<2500	<2000	<2000	mix-in-place <1500 plant mix <700	n.s	n.s	n.s	n.s	n.s	n.s	n.s	n.s	n.s
Organic content (%)	n.s	n.s	<1.5	<1.5	n.s	n.s	n.s	n.s	n.s	n.s	n.s	n.s	n.s	n.s	n.s	n.s
CBR after 3 days curing in air and 4 days soaking (%)	100	n.s	>100	>60	<160	<160	n.s	n.s	n.s	≥70[1]	>100	≥150	n.s	n.s	n.s	n.s
CBR after 28 days curing in air (%)	n.s	≥100	n.s	n.s	n.s	n.s		n.s	n.s	n.s	n.s	n.s	n.s	n.s	n.s	n.s
Required compaction • Method • Dry density (% mdd) • Moisture content	BS 1924 Test 13[2]		BS 1377 Test 13 95% at omc					BS 1377 TEST 13 97% at omc			BS 1377 TEST 13 98% at omc	n.s	n.s	n.s	n.s	n.s

/continued

Specification source and reference	UK (TRRL) (1977)	France (CEBTP) (1980)		UK (TRL) (1993)			Malawi (MOWS) (1978)	Zambia (GOZ) (1973)	S. Africa (NITRR) (1985)			
Unconfined compressive strength: 7 days curing in air (N/mm^2)	n.s	n.s	1.8-3.0	n.s	n.s	n.s	n.s	3.5	n.s	n.s	n.s	n.s
Unconfined compressive strength: 3 days curing in air, 4 days soaking (N/mm^2)	1.7[1]	n.s	≥0.5	3.0-6.0[1]	1.5-3.0[1]	0.75-1.5[1]	n.s	1.8	4 - 8	2 - 4	1 - 2	0.5 - 1

Notes:

n.s.   not specified

(1)   Seven days moist curing, seven days soaking

(2)   Compaction to the density and moisture content expected in the field

(3)   If the required CBR specification is met at a lower stabiliser content than compressive strength specification, then the CBR value is adopted

Notes on tests:

a)   Minimum RN31 is 1.5MPa; minimum NITRR roadbase is 0.5MPa

b)   NITRR specification is on CBR size samples, whereas TRL samples are tested in cubes; correction factor is 0.96

3.   Carbonation

3.1   Evidence of carbonation

Although the phenomenon of carbonation of cemented materials has been recognised for some time, the suggestion that this can result in premature failure of road pavement structures has been only recent. This has caused sufficient concern to deter many consultants from recommending the use of chemical stabilisation of road building materials. In addition, some road administrations in Southern Africa have also refused to accept road designs which include a chemically stabilised roadbase. The inability to use locally available gravels which have been stabilised chemically can increase costs of construction considerably.

3.2   The carbonation process

3.2.1 Lime

The process of carbonation is most easily explained with reference to lime, which is produced by heating mineral deposits containing calcium carbonate. This process is represented in Equation 3-1.

Formation
$$CaCO_3 + heat \ \sqrt{} \ CaO + CO_2$$ Equation 3-1

Calcium is a relatively active element and calcium oxide (CaO), commonly known as quicklime, is quite dangerous. Therefore, it is often slaked with water to form its hydrated hydroxide, $Ca(OH)_2$, as shown in Equation 3-2. In this form it is commonly known as slaked lime and is much easier to handle.

Hydration
$$CaO + H_2O \ \sqrt{} \ Ca(OH)_2 + Heat$$ Equation 3-2

As with the many chemical reactions, the formation process is reversible, depending on the environmental conditions. Both the oxide and hydroxide forms can react with carbon dioxide to form calcium carbonate as shown in Equation 3-3 and Equation 3-4. It should be noted that calcium carbonate is not effective as a soil stabilising agent.

Carbonation
$$Ca(OH)_2 + CO_2 \ \sqrt{} \ CaCO_3 + H_2O$$ Equation 3-3
$$CaO + CO_2 \ \sqrt{} \ CaCO_3$$ Equation 3-4

The above equations demonstrate that the amount of available lime will be reduced unless it is stored in conditions which reduce exposure to the air. In fact, there is a view that premature deterioration in some roads containing stabilised materials may be due as much to the quality of the lime used as to other processes following construction. The reaction products are also susceptible to carbonation. It is therefore important to minimise exposure to CO_2 during the

manufacture and storage operations, to mix and compact materials quickly, and to cure the stabilised material properly.

3.2.2 Cement

The reactions with cement are more complex, but the cemented products, represented as hydrated calcium silicates (CSH) and hydrated calcium aluminates (CAH), are also susceptible to carbonation as shown in Equation 3-5.

Carbonation of hydration products

$$CSH + CAH + CO_2 \; \surd \; CaCO_3 + SiO_2 + Al_2O_3 + H_2O$$

Equation 3-5
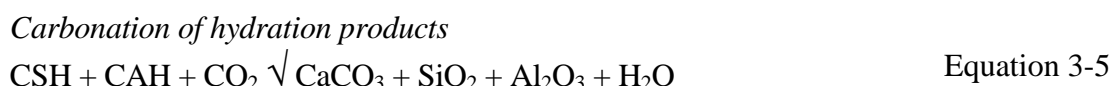
Both CSH and CAH require a minimum pH of about 12.4 to ensure that the reactions progress and that the hydration products remain stable. This high pH environment is possible because of the presence of $Ca(OH)_2$ in the soil water. Above pH 12.4, there is a dramatic increase in the solubility of the silica and alumina in the soil, and reactions proceed to form calcium silicates and aluminates. These are the cementitious materials which bind the soil together. Sherwood (1968) showed that materials treated with 10 per cent cement would not gain significant strengths (greater than 1MPa at 7 days), unless the reaction pH was greater than 12.4. With higher pH, greater long term strengths were possible.

3.3   Tests for carbonation

The presence and depth of carbonation can be detected by testing the pH of the stabilised layer with phenolphthalein indicator and phenol red, and checking for the presence of carbonates with dilute hydrochloric acid (Netterberg 1984). Phenolphthalein will remain colourless at a pH less than 8.4, and will turn red at a pH greater than 11. Phenol red turns red at a pH greater than 8.0, suggesting the presence of carbonate, lime or cement. If it remains yellow, then these are absent. Dilute hydrochloric acid will effervesce in the presence of $CaCO_3$. No effervescence will occur with $Ca(OH)_2$, CSH, and CAH. Measurement of the pH of soil pastes, using either laboratory methods or indicators in the field, enable a better representation of the pH to be gained. $Ca(OH)_2$ and Portland cement will have pH>12.4; CSH and CAH will have pH from 11.0 to 12.6; and $CaCO_3$ will have pH=8.3. In the field, freshly exposed faces of stabilised materials can be tested for carbonation using a combination of these procedures, as shown in Table 3.1.

Table 3.1 Determination of the presence of carbonation (Based on BS1924, Part 2, 1990)

Phenolphthalein	Dilute hydrochloric acid	Phenol red	Indication of carbonation	Carbonate in original material
Red	No Effervescence	-	None[1]	Absent
Clear	Effervescence	Red	Carbonated	Absent
Clear	No Effervescence	Red	No Stabiliser added[1]	Absent
Red	Effervescence	-	None	Present[3]
Clear	Effervescence	Red	Carbonated[2]	Present
Notes:				
(1) Partial carbonation cannot be ruled out				
(2) Cannot rule out that stabiliser was not added				
(3) If carbonates not present some partial carbonation must have occurred				

3.4   Effects of carbonation

Since carbon dioxide can also react with the hydration products of lime and cement, there is a risk that this process can also counteract the benefits of chemical stabilisation. Potentially, an increase in plasticity and a reduction in strength can result. These effects can also be exacerbated by the presence of organic matter. This decomposes and increases the concentration of CO_2 in the soil air to levels much higher than those normally found in the atmosphere. Reported rates of carbonation in the road range from 0.5mm-2mm/day in air to 2mm-50mm/year at the bottom and sides of stabilised layers in the road (South African Roads Board 1990).

The theoretical and observed effects of carbonation on the stabilisation process are summarised in Table 3.2.

Table 3.2 Summary of the effects of carbonation

Process	Observed effect
$Ca(OH)_2$ destroyed, pH decrease from 12.4 (lime) to 8.3 ($CaCO_3$)	$CaCO_3$ formed, plasticity increases
CSH, CAH destroyed	Decrease in UCS, CBR or tensile strength
Shrinkage of calcium silicates	Cracking and micro-cracking
Reduced relative compaction	Road surface deformation and rutting

3.5   Experience in Southern Africa

A report published by the South African Roads Board (1990) showed the extent of the problem of carbonation in the Southern African region. Distress manifested in roads constructed with stabilised bases were reported as:

- Surface disintegration of the primed base during construction
- Loss of the seal during service
- Partial or complete loss of cementation and strength
- Rutting, shearing, pumping and cracking
- Increasing plasticity index

These are inter-related effects since some lead on to others. The effects suggest that carbonation can contribute to premature distress in some circumstances.

The results of the South African study are summarised in Table 3.3. The results suggest that carbonation was a factor in distress in about half of the cases investigated. However, the study also concluded that the presence of carbonation or the loss of stabilisation did not necessarily lead to distress. Lack of distress was attributed to possible:

- Lack of traffic
- Use of abnormally high stabiliser contents
- Use of durable aggregate or durable cement

These statements have been interpreted as indicating that chemical stabilisation, using low cement content with materials containing sub-specification aggregate, is a high risk process which is only effective under the above conditions. Yet there are many other circumstances, such as on the Zimbabwe trunk road network (Hewitt et al, 1998), in which chemical stabilisation of these materials has been highly effective.

Table 3.3 Percentage of sites where stabilised materials deteriorated

Occurrence of distress	Carbonation	Return of Ip	Surface loosening	Loss of strength
No (18 sites)	78	11	-	39
Yes (69 sites)	64	9	100	45

The South African study was initiated after an investigation of extensive failures of a two year old road in Namibia. This road was constructed with a lime stabilised calcareous sand base and sub-base. Problems observed included acute cracking and movement of the seal, requiring extensive patching of the outer wheel-tracks. Beneath the intact seal at the top of the roadbase was a loose layer of calcareous sand above a relatively hard stabilised layer. Weakening was also observed in the top and bottom of the sub-base. Carbonation was attributed as the cause of the problem.

The South African study report suggests that, on average, soils lose about 40 per cent of their unconfined compressive strength through carbonation. It has also been reported (Pinard 1987, Bagonza et al 1987) that soil plasticity returns to its original unstabilised value due to insufficient addition of lime. However, as indicated by the South African report, it should not be possible for the plasticity to return within the service life of the road if sufficient lime has been added to destroy the clay and to maintain a sufficiently high pH. These will inhibit the ability of the reactions to be reversed. Therefore, where apparent reversion of plasticity does occur, it is usually due to insufficient stabiliser, insufficient mixing or pulverisation, or poor quality of the stabiliser such as high carbonate content.

The addition of lime to a soil produces an increase in the optimum moisture content and a reduction in the maximum dry density (MDD), as shown in Table 3.4 for a typical sandy clay in Malawi. The postulation that materials subjected to carbonation revert to their natural state implies that the field density of the carbonated material will be at lower relative percentage compaction of MDD of the natural material. In these circumstances, the material could become particularly moisture sensitive, weaker and prone to densification under traffic. Rutting would result.

Table 3.4 Effect of addition of lime on density, optimum moisture content and strength

Percentage of lime	Maximum dry density	Optimum moisture content (%)	California bearing ratio
0	1 956	9.3	13
2	1 944	11.3	56
4	1 920	11.4	76
6	1 890	11.6	92
Note: Results are for a Malawian sand clay			

3.6   Experience of stabilisation in Zambia and Zimbabwe

In the case of Zambia, much of the network has received little maintenance since construction. Despite this, many main roads constructed with stabilised gravels appear to be structurally sound, although the lack of resealing has meant that surfacings have deteriorated. By contrast, some roads have deteriorated significantly, including those constructed with crushed stone roadbase.

In Zimbabwe, cement modification of base materials has been the preferred option since the early 1970s. For many years, 1-2 percent of cement was added routinely to the base gravels. Maintenance of the state road network has been of a high standard. The findings of a recently completed study on the road network (reported by Hewitt et al at the *SFRDP Workshop* in Harare in June 1998) has shown that, in general, the network is in very good condition, despite its age. In Zimbabwe only one case of return of plasticity of the type reported in Table 3.3, and attributable to carbonation, has been reported. This is perhaps surprising in the light of the South African information, and is particularly significant since the proportion of cement added is low.

3.7   Other experience of stabilisation

There have been few recent comprehensive studies of stabilised roads but the results of one such review are given in Table 3.5. This study reported some cracking, deformation and stripping on roads stabilised mostly with cement, but some with lime, in a number of African countries. The proportion of the networks where deterioration was reported was still very low, as shown in Table 3.6. These results illustrate the durability and benefits of stabilisation.

Table 3.5 Soil stabilisation practice in 11 African countries

Country	km of roads with stabilised bases	km of bitumen surface roads	Cement stabilisation					Lime stabilisation					Remarks
			Specification(s)	Per cent stabiliser normally used	km of roadbase stabilised	Per cent failure	Type of failure	Specification(s)	Per cent stabiliser normally used	km of roadbase stabilised	Per cent failure	Type of failure	
Algeria	30 000	40 000	French specification	5-7	100			French specification	1-3	1 000			Bitumen stabilsed bases preferred for heavy traffic.
Angola	1 500	6 000	Durability test UCS & grading	3-8	1 500	nil							Soil stabilisation used when costs are in its favour.
Gambia	165	3 000	Density spec no strength spec	3	165	nil							Soil stabilisation used for new main roads.
Ghana	135	3 800	UCS 1.7 MN/m^2 at 7 days & CBR 200	4-5	135	nil							Soil stabilisation for major projects only, natural gravel normally used.
Kenya	1 440	3 100	UCS 1.8 MN/m^2 PI $\leq$ 6	4-6	1 290	20-40	Cracking, deformation & stripping	As for cement	3-7	150	77	Cracking, deformation & stripping	Soil stabilisation not favoured-only used if alternatives significantly more expensive.
Mozambique	1 154	2 780	UCS 1.7 MN/m^2 Durability test	6-9	896	0.5	Cracking, deformation & stripping	CBR>80 PI $\leq$ 6	2-4	110	nil		Soil stabilised bases limited to medium and lightly trafficked roads.
Nigeria	800	12 500	CBR 160-180 field CBR 80-100	4-7	720	15	Cracking, deformation & stripping	As for cement		nil	nil		Soil stabilisation only used when suitable natural gravels unobtainable.
Zimbabwe	1 700	3 500	Texas triaxial μ3.0 & PI $\leq$ 6 for natural material UCS 700 KN/m^2	2	1 500	nil		Texas triaxial $\leq$ 3.0 & 6<PI<12 for natural material	2-4	200	nil		Soil stabilisation preferred when costs comparable with alternatives.
RSA	1 600	4 300	UCS 3 to 5 MN/m^2 Grading & PI limits	3-5	1 600	5-20	Stripping				nil		Soil stabilisation preferred when costs comparable with alternatives.
Tanzania	800	2 500	UCS 1.7 MN/m^2	5	640	10-20	Cracking & deformation	Standard percentage no strength test	5	160	nil		Soil stabilisation preferred when costs comparable with alternatives.
Zambia	3 000	3 000	CBR 180 Grading & PI	3-4	approx. 1 000	nil		As for cement	4	approx.	nil		Soil stabilisation used for all main roads.

Source: Bulman 1972

Table 3.6 Deterioration of cement and lime stabilised roads in 11 African countries

	Cement	Lime
Kilometres of road stabilised	9 446	2 620
Percent showing deterioration (cracking, deformation and stripping)	8[1]	4[2]
Notes: (1) Recorded in Kenya, Mozambique, Nigeria and Tanzania (2) Recorded in Kenya *Source:* Bulman 1972		

4.   Study methodology

4.1   General approach

The aim of the study was to investigate the performance of a variety of chemically stabilised roadbases to assess their performance over time. In particular, the study aimed to clarify the situation with regard to the possible weakening of the stabilised materials through the effects of carbonation.

Test sections were selected on roads in Zambia, where chemical stabilisation had been used extensively, and where road performance could be measured. Information from these was supplemented with some test sections in Zimbabwe, and by drawing on work from the TRL project in Botswana carried out in the 1980s. Sections of the road network of different ages, and with different base material and stabiliser types, were investigated. The degree of carbonation was assessed and the *in situ* strength of the bases were determined using a dynamic cone penetrometer (DCP). Unconfined compressive strength measurements were conducted in laboratory tests on undisturbed block samples of the roadbase recovered from the test sections (see Plate 4-1 and Plate 4-2). The general condition of the road was assessed using visual condition surveys which also included rut depth measurements.

Plate 4-1 Hand saw used to cut stabilised roadbase

Plate 4-2 Blocks of roadbase lifted from road for laboratory testing

It was originally intended to compare present road condition and material properties with the as-built strength and properties by referencing construction records. Unfortunately, the Zambian Road Department's records were accidentally destroyed in the period between project design and inception, so a revised methodology was required. It was considered reasonable to make the assumption that the stabilised materials satisfied the minimum test requirements specified in Zambia at the time of construction. In the absence of as-built information, current strengths were compared with these specified values. In those cases where supervision of construction was carried out by consultants, as-built information was often available and used.

The field investigations aimed to measure the residual strength of the roadbase and to sample materials for laboratory investigations to determine if the stabilised materials had carbonated and whether or not they had lost sufficient strength for the layer to cease to be an effective roadbase.

4.2   Site selection

A visual survey was conducted on most of the paved road network in Zambia and, following an examination of the available records and discussions with Roads Department Staff, 23 sites were established on nine trunk road routes for further investigation. The sites were designated names and site numbers based on the first and last letters of the origin and destination points in the direction of increasing chainage (e.g. KASE1 was the first test section along the <u>K</u>asungul<u>a</u> to <u>S</u>heshek<u>e</u> road). The location of these sites is shown in Figure 4-1 for Zambia and in Figure 4-2 for Zimbabwe, and is summarised for both countries in Table 4.1. Detailed site characteristics are given in Appendix A: the Zambia sections are listed in Table A.1, and the construction data for the Zimbabwe sections in Table A.2. The types of roadbase material used in the test sections in Zambia and Zimbabwe are shown in Table 4.2.

Figure 4-1 Location of test sections in Zambia
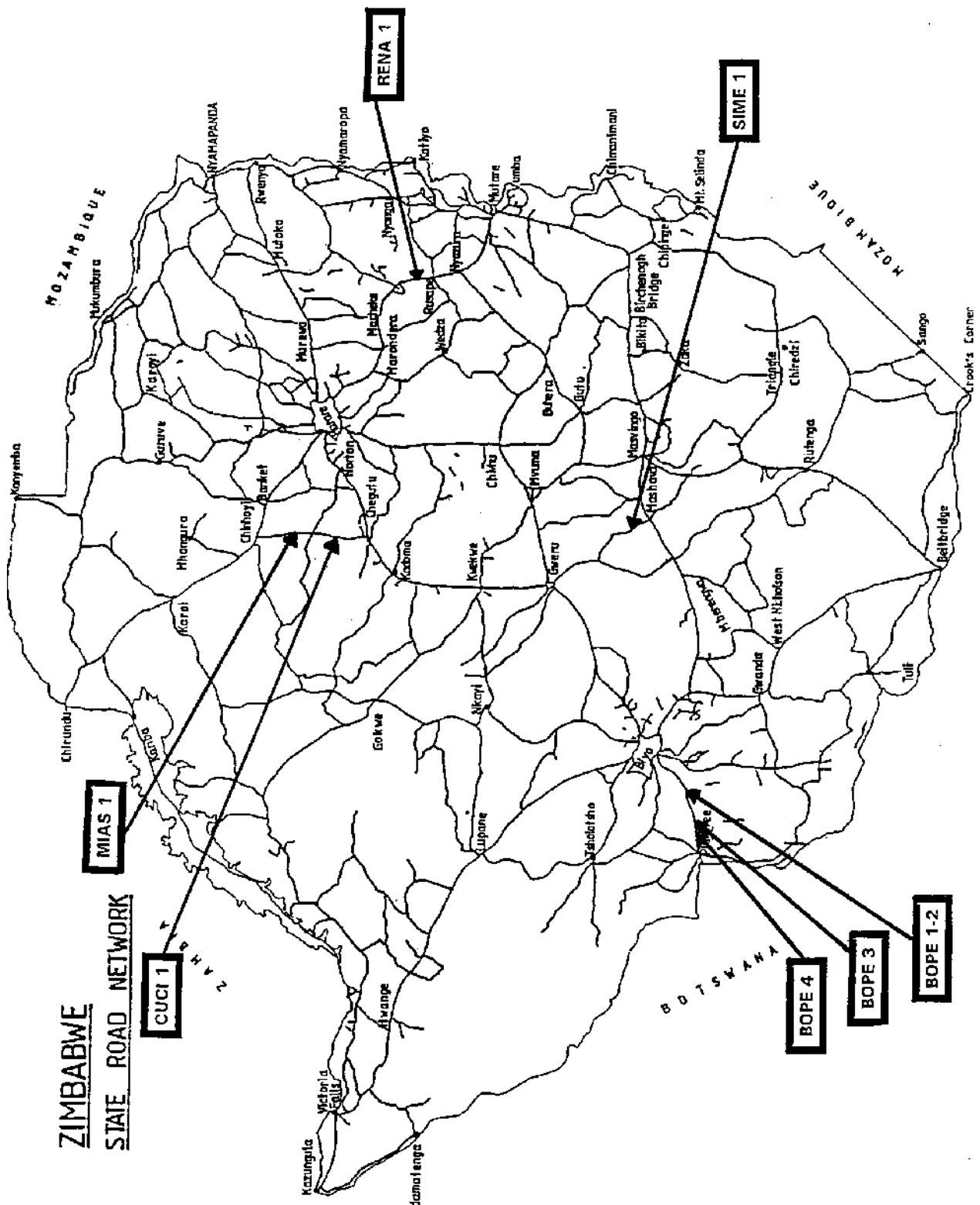
Figure 4-2 Location of test sections in Zimbabwe

Table 4.1 Location and details of test sections

Zambia			Zimbabwe		
Road name	Site name	Chainage	Road name	Site name	Chainage
Kazungula - Sesheke	KASE (3)	91+040	Bulawayo - Plumtree	BOPE (1)	21+360
Livingstone - Sesheke	KASE (1)	74+960	Bulawayo - Plumtree	BOPE (2)	21+895
Livingstone - Sesheke	KASE (2)	80+400	Bulawayo - Plumtree	BOPE (3)	61+125
Livingstone - Zimba	LEZA (X1)	51+400	Bulawayo - Plumtree	BOPE (4)	66+910
Lusaka - Mongu	LAMU (T)	104+290	Chegutu - Chinhoyi	CUCI (1)	70+010
Lusaka - Mongu	LAMU (4)	126+990	Murombedzi - Access	MIAS (1)	1+300
Lusaka - Mongu	LAMU (R)	24+510	Rusape - Nyanga	RENA (1)	6+010
Lusaka - Mongu	LAMU (S)	58+300	Shurugwi -Mandamabwe	SIME (1)	38+500
Katete - Mozambique	KEME (X)	18+250			
Katete - Mozambique	KEME (Y)	18+650			
Chipata - Lundazi	CALI (Z1)	34+350			
Chipata - Lundazi	CALI (Z2)	34+650			
Chipata - Lundazi	CALI (Z4)	40+850			
Chipata - Lundazi	CALI (Z3)	35+700			
Mpika - Nakonde	MANE (L)	158+000			
Mpika - Nakonde	MANE (M)	104+600			
Mpika - Nakonde	MANE (X3)	119+000			
Mpika - Serenje	SEMA (X4)	91+450			
Serenje - Mukuku	SEMU (X5)	90+000			
Mansa - Nchelenge	MANE (N)	25+000			
Mansa - Nchelenge	MANE (O)	36+000			
Ndola - Kitwe	NAKE (P)	15+500			
Ndola - Kitwe	NAKE (Q)	17+600			

Table 4.2 Number of sections of each generic class of roadbase material

	Zambia	Zimbabwe
Laterite	7	4
Laterite+quartz	9	-
Quartz	1	4
Calcrete	3	-
Decomposed granite	1	-

4.3   Field measurements

The following procedures were carried out on site:

- Visual condition surveys
- Density measurements
- Sampling
- Carbonation testing
- Rut depth measurements
- Strength tests
- Point load testing

Visual condition surveys
Each of the sites were marked out on a 10 metre grid in each lane, and the visual condition assessed using a standard survey condition technique. The type, position, length and width of cracking and other surface defects were recorded.

Density measurements
Densities of the base, sub-base and subgrade were measured using a CPN Stratagauge.

Sampling
Sample pits, approximately $0.8m^2$, were dug in the outer and inner wheel-tracks. The thickness of the surfacing, base, and subgrade was also recorded.

Carbonation testing
The condition of the surfacing and its bonding to the underlying roadbase was recorded. Block samples, measuring approximately 300mm x 300mm, were cut out of the roadbase using a power-saw. The top and bottom of the samples were marked, and the samples were waxed and wrapped in plastic film. These samples were transported to the laboratory for UCS testing. The cut faces of the sample holes and off-cuts from smaller blocks were brushed, cleaned and tested for carbonation using phenolphthalein, phenol red, and universal indicators. The non-calcareous materials were also tested for carbonate using dilute hydrochloric acid. The reactions observed were recorded. Additional off-cuts were retrieved for point load testing and further samples of roadbase, sub-base and subgrade materials were taken for laboratory classification and testing for cement content.

Rut depth
Rut depths were measured along the section at one metre intervals in the outer and inner wheel-tracks prior to sampling.

Strength tests
In situ strength tests on the pavement layers were carried out using a DCP. In those cases where the strength of the roadbase was too high to allow penetration of the DCP, the test on the sub-base was continued from the exposed top of the sub-base layer following excavation to collect the sample of roadbase material.

Point load tests
This test is used to measure the point load strength index on cut blocks, cores and irregular offcuts. Blocks or irregular lumps of size in the 15mm to 85mm range were recovered for testing *in situ* and in the laboratory. Where specimens of this size and shape were not available, they were prepared by chiselling or trimming. The target number of tests was 20 for any particular sample. The test was originally developed as an index strength test for rock materials. It is quick and simple and uses the easily portable equipment (see Plate 4-3 and Plate 4-4). It can be carried out in the laboratory or field. The results can be correlated with other test results, such as uniaxial tensile strength and compressive strength, and the point load test can then be used as a proxy for these tests.

Plate 4-3 Point load testing of stabilised roadbase

4.4   Laboratory testing

The grading, plasticity and linear shrinkage were measured on the roadbase, sub-base and subgrade materials. The pH and cement contents of the stabilised base materials were also determined.

The block samples of roadbase materials collected from the test sections were trimmed, where possible, to provide 150mm cubes. On the samples taken from roadbases thinner than 150mm, the blocks were trimmed to produce 100mm cubes. Where necessary, a thin fine sand mortar was applied to the top and bottom of the cubes to provide the flat and level surfaces required for the test. UCS tests were then carried out on the samples either dry (*in situ* moisture content) or after soaking for a period of 7 days.

Further point load testing was carried out in the laboratory on the off-cuts from the block samples at their field moisture condition and again after soaking for 7 days.

Plate 4-4 Point load testing of stabilised roadbase

5.   Results

5.1   Field measurements

The results of the visual survey data are given in Appendix A, in Table A.3 for Zambia, and in Table A.4 for Zimbabwe. The results are summarised in Table 5.1 and Table 5.2.

Table 5.1 Summary of observed degree of cracking in both lanes

Percent cracking	Zambia	Zimbabwe
0 - 15	24	13
16 - 30	12	3
31 - 45	1	0
46 - 60	3	0
51 - 75	6	0
> 75	0	0

Table 5.2 Summary of observed crack width in both lanes

Crack width (mm)	Zambia	Zimbabwe
0	20	8
<1	2	2
1 - 3	5	5
> 3	19	0
Spalled	0	1

The results of the carbonation tests are given in Table A.5 and Table A.6 of Appendix A. All 30 of the road sections (60 sites) in Zambia and Zimbabwe where stabiliser had been added showed evidence of carbonation. No stabiliser had been added to two sites in Zimbabwe and one in Zambia. The bond between the surface and roadbase was also examined but evidence of disbonding was observed only on calcareous materials on the Livingstone-Shesheke road in Zambia.

Detailed rut depth results are given in Table A.7 and Table A.8 of Appendix A, and are summarised in Table 5.3.

Table 5.3 Results of rut depth survey in both lanes

80th percentile rut depth (mm)	Zambia		Zimbabwe	
	Outer wheel-track	Inner wheel-track	Outer wheel-track	Inner wheel-track
0 - 5	9	17	0	4
5 - 10	26	24	11	11
10 - 15	5	3	5	1
15 - 20	0	0	0	0
20 - 25	0	0	0	0
> 25	1	0	0	0
No record	5	2	0	0

The *in situ* CBRs are given in Table A.9 and Table A.10 of Appendix A.

The results for the samples tested with the point load test are shown in Table A.11 and Table A.12 of Appendix A. The effect of prolonged soaking was recorded daily and some samples disintegrated before completion of the soaking period.

5.2   Laboratory tests

Classification test results for Zambia are given in Appendix A: in
Table A.13 for roadbase materials, and in
Table A.14 for sub-base materials.

The unconfined compressive strength test results are given in Table A.11 and in Table A.12 of Appendix A, and are summarised in Table 5.4.

Table 5.4 Summary of average unconfined compressive strength test results

UCS range (MPa)	Zambia		Zimbabwe	
	Unsoaked	Soaked	Unsoaked	Soaked
0 - 0.5	6	3	6	3
0.5 - 1.0	3	9	1	0
1.0 - 1.5	2	0	0	0
1.5 - 2.0	2	0	0	0
2.0 - 2.5	2	1	0	0
2.5 - 3.0	0	1	0	0
> 3.0	5	2	0	0
No result	1	5	1	5

6.   Discussion of results

6.1   Carbonation

None of the stabilised bases had a pH greater than 12. Of the materials tested in Zambia, only three indicated a value of pH greater than 9, whilst no bases on the Zimbabwe sections attained this value. In all but one case, in which no stabiliser was added, the samples were carbonated. Confirmation of carbonation on the calcrete sections on Kasungula-Sesheke could not be identified positively because of the presence of calcium carbonate in the host material. However, the field values of pH recorded were of the same order as magnitude as the carbonated samples. In all cases, the carbonation extended to the full depth of the layer, but in only one case was any evidence found of the presence of soft or loose materials at the base-surfacing interface. Some flecks of red coloration were occasionally observed indicating the presence of unhydrated or uncarbonated cement.

6.2   Laboratory tests results

A comparison of the specifications based on grading and coefficient of uniformity showed that over half of the Zambian stabilised bases were outside the recommended grading envelopes for CB1 and CB2 given in *ORN 31*, and only two sites (KEMEX and MANEX3) had coefficients of uniformity less than 5, as shown in Table 6.1.

Table 6.1 Compliance of the Zambia stabilised roadbase materials with ORN 31 guidelines

	Within recommendations	Outside recommendations
Grading envelope	9	12
Coefficient of uniformity	21	1
Note: Recommendations are for CB1 roadbase as specified in *ORN 31*		

Only one site showed an apparent plasticity reversal from a value of 13, for the natural material, to a current value of 17, on the basis of the limited as-built information available. One other section also showed high plasticity in the roadbase although, since no as-built data was available, it is not possible to state conclusively that the plasticity had increased. Other sections, for which data were available, remained non-plastic after carbonation. The remainder of the roadbases sampled in Zambia were non-plastic or slightly plastic, as shown in Table 6.2. Although the precise original as-built plasticity index values are not known for most of the Zambian sites, the fact that chemical stabilisation was required is likely to indicate that the original materials had a moderate to high plasticity for a granular roadbase material.

Table 6.2 Plasticity of stabilised roadbases for Zambia

Range of plasticity indices	Number
NP to SP	18
SP to 5	1
6 - 10	0
11 - 15	1
>15	1
Note:	
NP Non-plastic	
SP Slightly plastic	
Figures for Zimbabwe are not available	

The results suggest that there has not been any significant reversal of plasticity in most of the materials used in the stabilised based in Zambia. Plasticity reversal has been recorded in other situations, particularly in weathered materials containing specific clays. Pinard (1987) has reported reversal in the case of a weathered basalt in Botswana, although insufficient or poor quality lime were more likely to be the cause of this.

One of the major concerns expressed in the South African work was that the improved properties of the materials measured after stabilisation or modification, in terms of reduced plasticity, may not be permanent. The South African work suggests that it is modified rather than stabilised materials that are susceptible to this reversal in plasticity properties and carbonation.   This is because there is insufficient lime present to satisfy the *initial consumption of lime test* (ICL) requirements and to enable permanent cementation and hardening products to form. This may be the case for some materials, such as the gravels with highly plastic or expansive minerals, and possibly also the calcretes, particularly where these are stabilised with lime. However, there is ample evidence in Zimbabwe to show that modification with low cement contents works well on most materials. It is also important that differentiation is made between those materials which respond well to lime and cement stabilisation and those where there is the risk of durability problems. At present, it is known that basaltic, doleritic, some calcareous and sedimentary gravels could pose durability problems, although the reasons for this are still unclear.

6.3   Overall Performance

The field performance of the test sections is summarised in Table 6.3.

Table 6.3 Summary of performance of test sections

Deterioration	Degree	Zambia Percent of sites	Zimbabwe Percent of sites
Carbonation		100	100
Wide cracking	>3mm	41[1]	0
Cracking	0-10%	52	50
	11-25%	7	50
	26-50%	22	0
	>50%	19	0
Rutting	0-4.9mm	22	0
(80th percentile)	5-9.9mm	63	68
in outer wheel-track	10-14.9mm	12	32
	>15mm	3	0
Pot-holes		24[2]	0
SNC[2]	>3.6 (T6)	52	88
SNC[2]	>3.3 (T4)	85	100
Base *in-situ* CBR>80[2]		93	100
Base *in-situ* CBR>200[2]		49	37
In-situ UCS > 3.5[2]		25	4
In-situ UCS > 2[2]		35	15
In-situ UCS > 1[2]		55	18
In-situ UCS < 1[2]		45	82
Plasticity index	>SP	14	-
Notes: (1) High stabiliser contents in excess of 4 per cent were used on these sites (2) Measured in outside wheel-track SP Slightly plastic			

The sites in Zambia exhibited a much higher degree of cracking than did those in Zimbabwe, with block type cracks wider than 3mm being most prevalent. This probably reflects the lower proportion of stabilising agent used in Zimbabwe. No significant rutting was observed, even where wide cracks would increase the likelihood of wetting of the underlying pavement layers. Rutting levels in the outer wheel-track were generally low, with 80th percentile rutting over 15mm occurring at only three per cent of the Zambian sites. No rutting of this level was observed on any sites in Zimbabwe.

Some pot-holing was prevalent on sections, although in most cases this could be attributed to the age of the seals and lack of maintenance operations. The seals in Zambia were mostly over 15 years old. The bitumen in surface dressings of this age becomes very brittle and cracked. However, this study suggests that pavements remain structurally sound, and it is likely that the strength of the stabilised roadbases contributed significantly to this. Although road sections tested were carbonated to the full depth of the base, only those roads with calcareous bases were not in a serviceable condition.

Many of the roads investigated have undoubtedly taken very heavy traffic loads, although detailed information on traffic volumes and loading is scarce. However, the stabilised roadbases have, for the most part, performed exceptionally well, and the evidence from this research indicates that chemical stabilisation remains a cost-effective option for the construction of durable road pavements.

6.4   Strength

6.4.1 General structural condition

Many of the roads in Zambia show evidence of cracking. Furthermore, many have probably been subjected to severe over-loading, as there have been periods without adequate axle load control. Despite this, and poor maintenance, most of the roads remain structurally sound. On all but the most heavily trafficked roads, crack sealing, patching and a re-sealing would probably be sufficient to ensure that these roads continue to function adequately for many years. If decisions are taken in Zambia to rehabilitate road structures, particularly of secondary and feeder roads, this could be an inappropriate and unnecessarily action, since maintenance measures alone might be sufficient and could be carried out at a much lower cost.

Some reduction in the strength of the stabilised materials has occurred since construction but, in general, the bases remain well-cemented and relatively strong with CBRs often well over 80 per cent. Therefore, although carbonation may have resulted in some loss of strength, the evidence suggests that chemical stabilisation, with a few exceptions, is an effective and durable method of improving material strength.

6.4.2 Appropriate strength parameters

The method of analysis for comparing the strength of the *in-situ* materials and estimating the traffic carrying capacity of the roads is based on the concept of 'structural number' and 'strength coefficients' of the pavement layers. Relationships are available based on both the CBR and UCS of the materials. The standard equation for calculating the strength coefficient of natural unstabilised roadbase materials is:

$$a_i = \{29.14(CBR) - 0.1977(CBR)^2 + 0.00045(CBR)^3\} \times 10^{-4} \qquad \text{Equation 6-1}$$

where

$$a_i \quad = \quad \text{pavement layer strength coefficient}$$
$$CBR \quad = \quad \textit{in situ } \text{California bearing ratio (per cent)}$$

Hodges et al (1975) derived the relationship given in Equation 6-2 between the structural coefficient and UCS for cemented bases:

$$a_i(UCS) = 0.075 + 0.039(UCS) - 0.00088(UCS)^2 \qquad \text{Equation 6-2}$$

In this relationship, the UCS values are at 14-day strength, although it is not clear whether this is 14-day moist cured, 14-day soaked, or 7-day moist cured and 7-day soaked.

If two pavements have the same structural number then the structural number concept means they should be capable of carrying the same traffic before structural rehabilitation is required, other things being equal. This is a simple idea that needs modification for some types of pavements, but it has been shown to work quite well for straightforward forms of road

construction.   Thus the strength coefficients themselves should be determined by the *performance of the roads* and should subsequently be related to standard laboratory measures of strength such as CBR and UCS.   Since relatively weakly-stabilised materials can be characterised by either their CBR value or their UCS value, it is possible to use the results of this project to verify the compatibility of Equations 6-1 and 6-2. The performance of the roads is compared with the anticipated performance in the next section.

6.4.3 Analysis of structural performance

The structural numbers for each of the pavements are shown in Appendix A (Table A.9 for Zambia, and in Table A.10 for Zimbabwe). Where penetration into the roadbase was possible, the *in situ* CBR values were calculated from the *in situ* DCP measurements on site. UCS values were obtained from laboratory tests on samples taken at the same position as the DCP measurements. These samples were sealed on site to retain the same moisture condition and density. The strength coefficients of the roadbases used to determine the structural numbers were calculated using equations 6-1 and 6-2.

In Zimbabwe, where cement was added routinely to Base 1 material, the addition of two per cent cement gave average strengths of around 1MPa in the Texas Triaxial Test (moisture conditioned) or approximately 180 per cent CBR.

The subgrade soils were generally strong, typically with *in situ* CBRs greater than 15 per cent. The modified structural numbers for these designs were compared with the designs for S5 and S6 soils in *ORN 31*, as shown in Table A.15. The maximum design traffic class is given for the roads based on the layer thicknesses required from *ORN 31*. Comparison of the *in situ* structural numbers with the design structural numbers in *ORN 31* show that these roads are capable of carrying higher levels of traffic than the design values would suggest.

From the strength tests, a relationship between *in situ* UCS and *in situ* CBR (by DCP) was obtained (Figure 6-1). The equation is:-

$$CBR = 300 \text{ x } UCS \hspace{4cm} \text{Equation 6-3}$$

Where UCS is expressed in MPa.

The relationship derived from this project is very similar to that developed by the Federal Highways Administration (1979) (for granular materials similar to those used in the roadbases in this project), as illustrated in Figure 6-2.
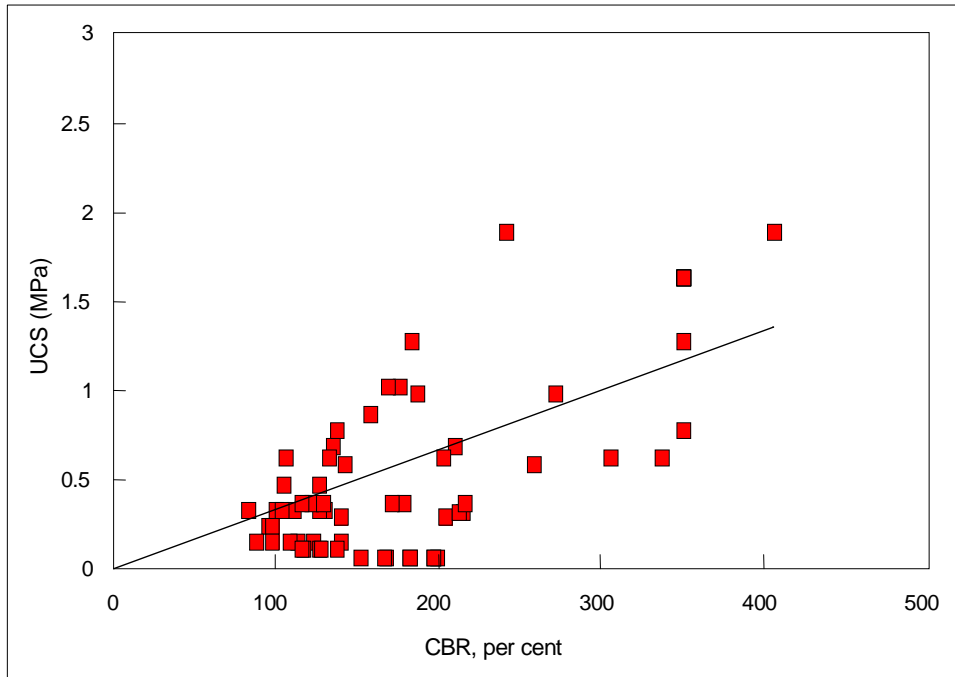
Figure 6-1 Unconfined compressive strength versus California bearing ratio
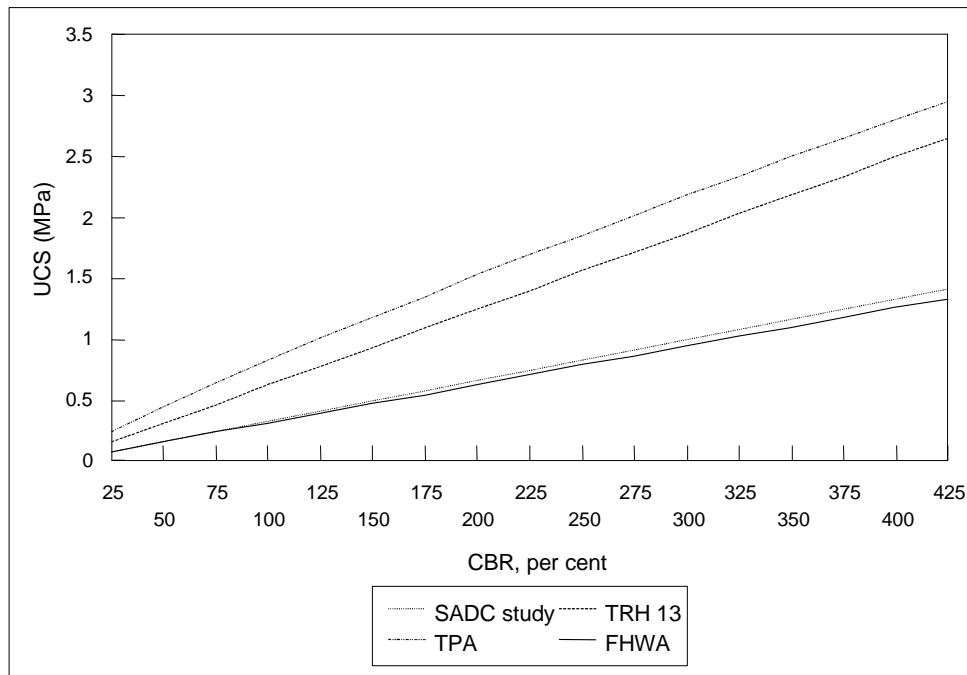
Figure 6-2 Comparison of relationship derived for UCS and CBR (Equation 6-4) with other published data (FHWA and SADC are for granular soils)

Substitution of this equation into Equations 6-1 or 6-2 will show that equations 6-1 and 6-2 are not compatible with each other. The same material should give the same strength coefficient

irrespective of whether CBR or UCS is used. As explained in section 6.4.2, the strength coefficients are determined from the field performance.

At low values of CBR and UCS the physical behaviour of both stabilised and unstabilised material is expected to be the same. At all values, the structural number concept means that pavements with the same SN should carry the same traffic. Using these principles, new relationships were developed relating the strength coefficients to CBR and to UCS. These are illustrated in Figures 6.3 and 6.4.

The equations are:-

For granular materials

$$a_2 = \{0.0054(CBR)^3 - 4.5(CBR)^2 + 1350(CBR) + 43300\}.\,10^{-6} \qquad \text{Equation 6-4}$$

For stabilised materials, with UCS expressed in MPa.,

$$a_2 = 0.167 \text{ x } UCS^{0.33} \qquad \text{Equation 6-5}$$
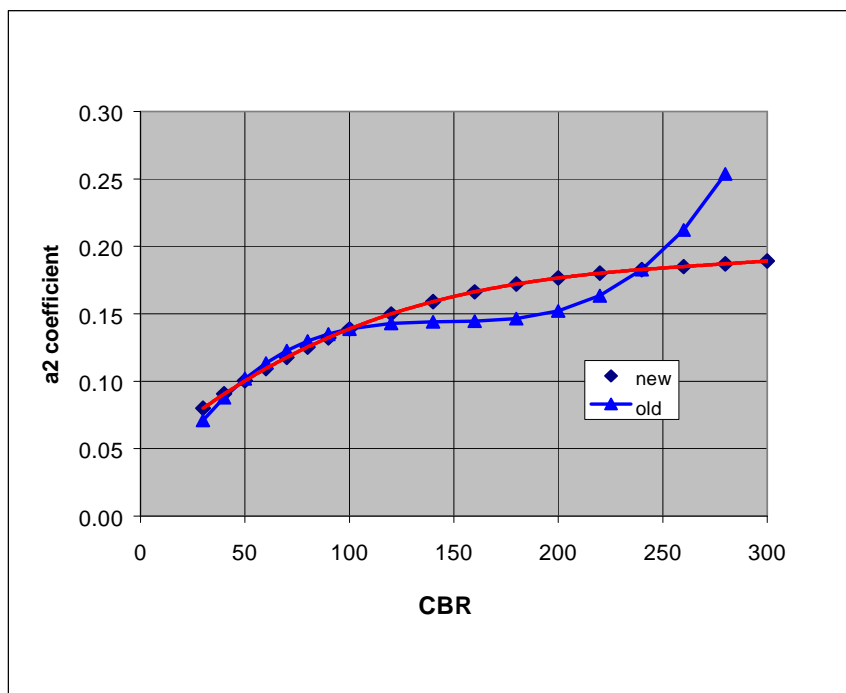
Figure 6-3 Revised coefficients for granular material

In HDM-III, the strength coefficient for unstabilised roadbase is calculated from *in situ* CBR whereas, for stabilised roadbases, the soaked UCS values are used. Equations 6-4 and 6-5 give an alternative method of calculating the strength coefficient for *in situ* conditions using either CBR or UCS.
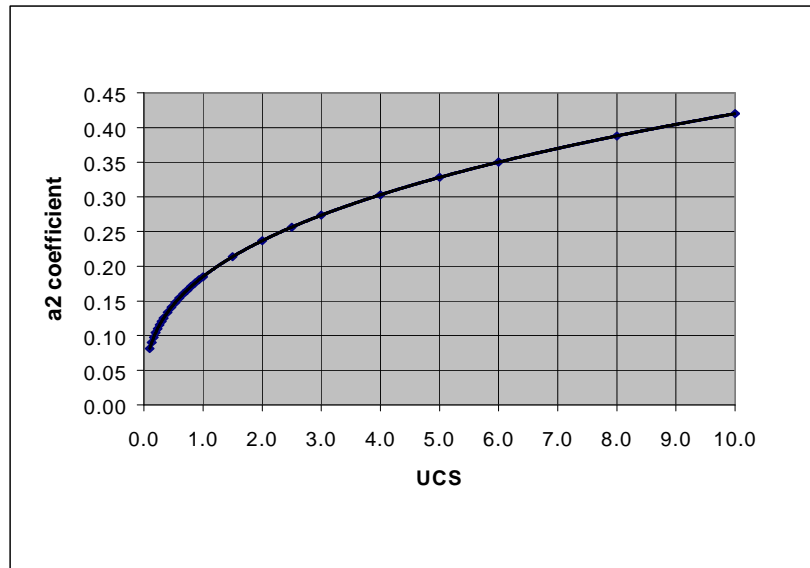
Figure 6-4 Revised coefficients for cemented materials

Using these equations, the equivalent values of CBR and UCS for each a_i are shown Table 6.4.

Table 6.4 CBR and UCS values that give equivalent values of strength coefficient

CBR	a_2	UCS
30	0.080	0.095
40	0.090	0.134
50	0.100	0.179
60	0.109	0.228
70	0.118	0.280
80	0.125	0.334
100	0.139	0.445
120	0.150	0.553
140	0.159	0.653
160	0.166	0.740
180	0.172	0.815
200	0.177	0.876
220	0.180	0.926
240	0.183	0.966
260	0.185	1.000
280	0.187	1.031
300	0.189	1.064

The relationship between CBR and UCS is not linear, the ratio CBR/UCS varying from about 215 to 315, slightly less than indicated in equation 6-3. This is because (a) the UCS relationship must be extrapolated for use to much higher values of UCS than found in this study and (b) the coefficient cannot approach an asymptote in the same way as the coefficient for CBR.

Using these new relationships, the structural number of the road sections were compared with the traffic that the roads had carried and the expected performance based on an average

SN/Traffic relationship obtained from *ORN 31*. The results are shown in Figure 6.5 where it can be seen that the structural numbers calculated using either CBR or UCS are now broadly compatible with each other. The Figure also illustrates that some of the roads have carried more traffic than anticipated, confirming the good performance discussed above.
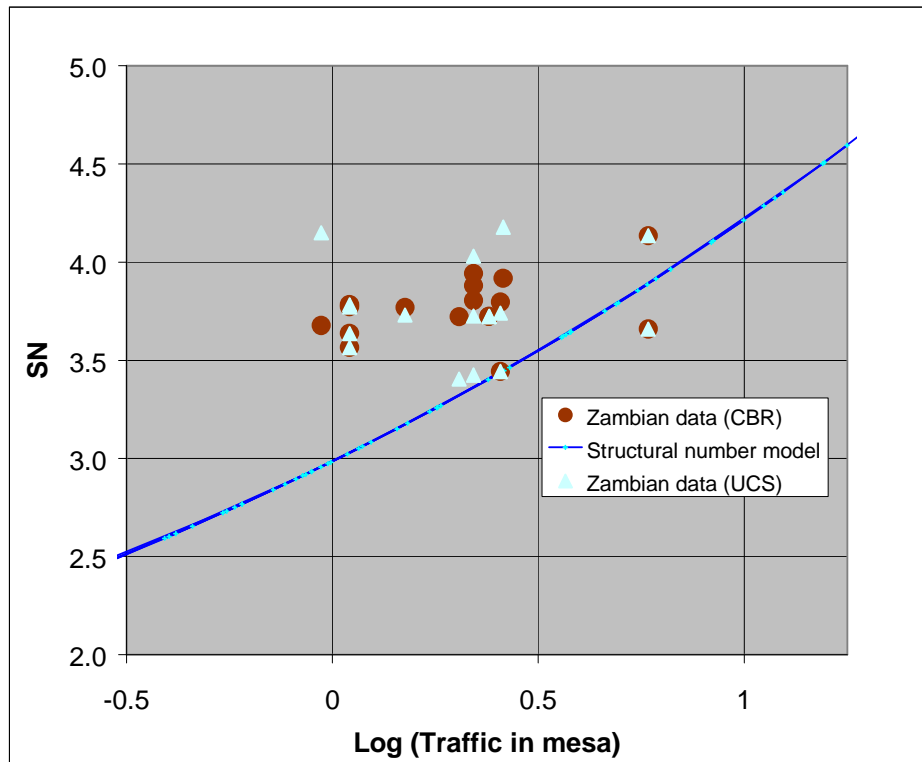
Figure 6-5 Structural number versus traffic for stabilised bases

6.4.4 Point load test

The point load strength index (Is(50)) was related to the UCS determined from testing the blocks. The following relationship was derived:

$$UCS = 5.631(Is(50)) \quad\quad\quad (R^2=0.85) \quad\quad\quad\quad \text{Equation 6-6}$$

Using this relationship, the point load test gives a quick and reasonably accurate method of estimating the *in situ* UCS of cemented roadbase materials which are often difficult to penetrate with a DCP.

6.5   Comparison of findings with other work

Many of the references to carbonation in the literature do not specifically mention the type of roadbase in which carbonation has occurred. The results of this study show that carbonation affects a wide range of materials. Chalk, which is high in calcium carbonate, can be successfully stabilised with cement(Pocock1970), and the presence of calcium carbonate in the soil is generally seen as beneficial (Sherwood 1993). However, most of the reported cases of carbonation in Southern Africa, which are thought to have led to subsequent pavement distress or failure, have been associated with the use of calcareous materials. The use of calcrete in the roadbase is a feature of the studies reported by, for example, Bagonza and other (1987),

Netterberg et al (1987), and the South African Roads Board (1990). It should also be noted that the only road showing poor performance in this study (the Kasungula-Sesheke road) also had a calcrete roadbase.

Bagonza et al (1987) have reported the performance of a road trial in Botswana where lime and cement were used to stabilise a calcareous sand. The characteristics of the original material are shown in Table 6.5. Although high strengths were obtained in laboratory tests, as shown in Table 6.6, this was not achieved in the road. The 'terminal' rut depth of 20mm occurred on the lime stabilised section after seven years, with the road having carried 240 000 equivalent standard axles, and on the cement stabilised section after 12 years having carried 380,000 equivalent standard axles.

Table 6.5 Properties of untreated calcareous sand

Test method	Test result
Liquid limit	39
Plasticity index	20
Linear shrinkage	7
Passing 2.00mm sieve	84
Passing 425:m sieve	77
Passing 75:m sieve	18
Carbonate content (%)	16
pH	8.0
Source: Bagonza et al 1987	

Table 6.6 Laboratory test results of stabilised samples

Material	Moisture content (%)	Compaction (BS 4.5kg rammer)	
		Dry density (kg/m^3)	CBR (%)
Calcrete	9.3	1803	120
and 3% cement	9.8	1814	120
Calcrete	8.5	1751	65
and 3% hydrated lime	10.5	1686	120
Notes: Tests carried out after field mixing and seven-day cure			

The poor performance was attributed entirely to accelerated carbonation of the base due to poor curing. However, there may well be other factors, not yet substantiated, such as the speed of the reaction process with some calcretes which could be equally important. Other factors, including low moisture content, low density, poor construction practice and ineffective priming, may also have contributed to the subsequent poor performance. The difficulties of compacting cement stabilised materials after long time delays are well known (Netterberg 1971). It has been suggested that the reaction times may be particularly rapid with calcretes due to the presence of non-plastic pozzolans in the form of reactive silica. This could have a significant effect on both the production of the cemented products and on compaction. The effect on density of delays between mixing and compaction is shown in Figure 6-6. Corresponding cured strengths are shown in Figure 6-7.
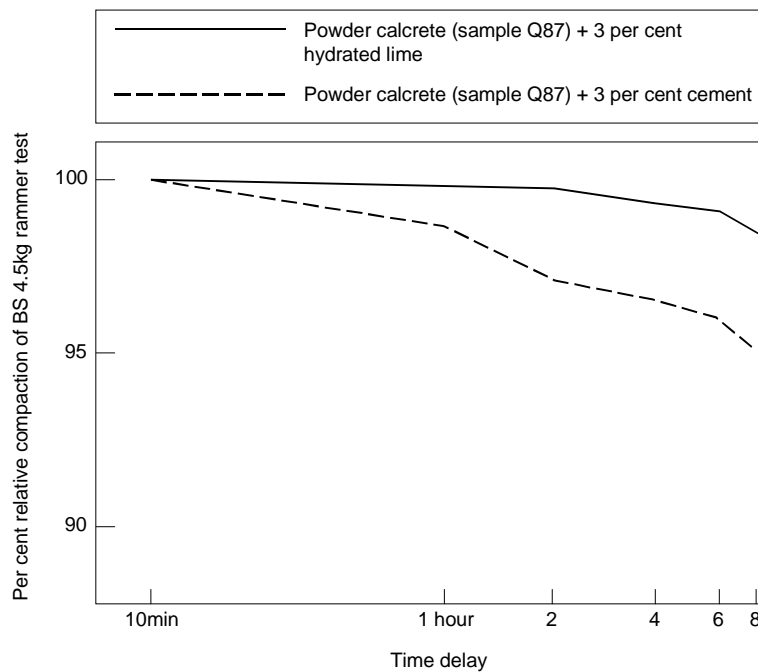
Figure 6-6 Relative compaction after different time delays following stabilisation (from Lionjanga et al, 1987b)

Lime and calcium carbonate do not react with each other so any adverse reactions due to the presence of carbonate should at least be neutral (Sherwood 1993). However, there may be some question as to the source of pozzalanic material available in calcretes with which the lime can react. Evidence does exist (Lionjanga et al 1987) which suggests that the initial flocculation reactions between lime and poorly-graded calcretes, which reduce plasticity, are unstable and fail to show the permanent strength gains expected.

Most of the road failures in the region attributed to carbonation have occurred with calcretes and this suggests these materials are particularly susceptible to the process. If the chemical reactions take place so rapidly in these materials that it presents construction problems, then this would not be detected by the standard laboratory tests. This indicates that further research on the chemical stabilisation of calcretes is urgently required as the weaker calcretes would appear to be ideal carbonate materials for stabilisation. Yet roads constructed with chemically stabilised calcretes continue to fail prematurely.

Strength loss due to density and effect of time delay on cementitious products

– – – – – – 7 day cure

———— 28 day cure
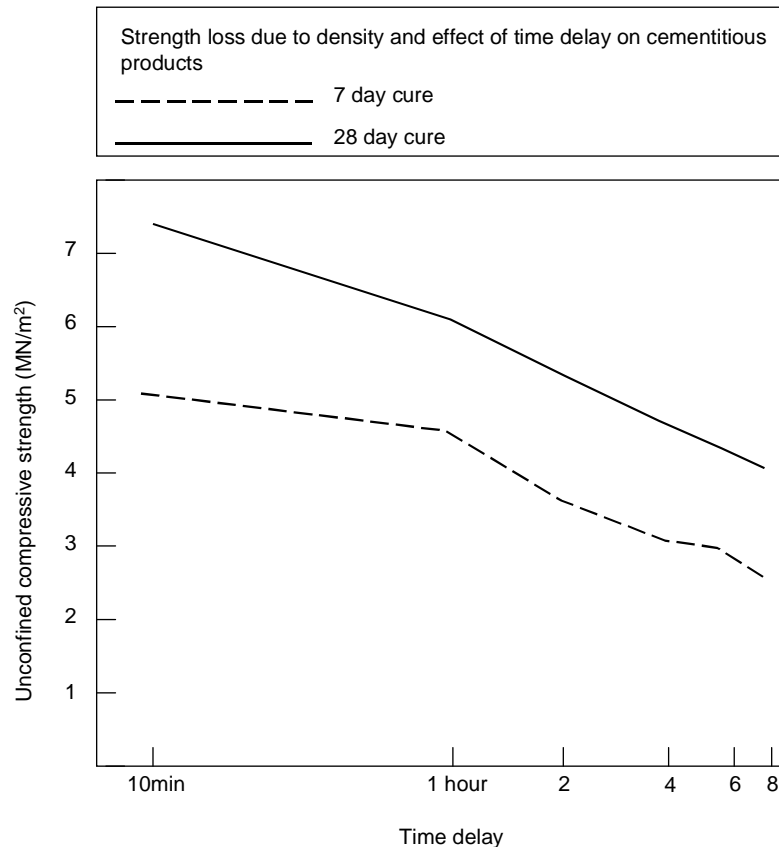
Figure 6-7 Effect of time delays on the strength of cement stabilised calcrete (from Lionjanga et al, 1987b)

6.6   Recommendations for suitable durability tests and limits

The evidence from this research suggests that the existing test methods are sufficient for the design of chemically stabilised bases. However, some chemically stabilised roadbases, such as those constructed using calcretes and basic igneous materials, have not performed well. Typical modes of distress in these cases are:

- Disintegration at the surface leaving a loose interface with the overlying layer and leading to a loss of bond between the base and seal
- Reduction in strength of the layer in service possibly due to carbonation with loss of structural capacity with time

For these materials, the additional test methods developed by CSIR for roadbase aggregates (Sampson 1990) should be considered. Three criteria were identified by Sampson to ensure adequate durability and long term performance of the stabilised layer. These criteria are to ensure:

- Addition of adequate stabiliser to maintain the required pH at a level where the cementation reaction will proceed normally, so that bonding and strength gain within the layer occurs and the hardening products form

- Suitable durability testing is carried out which identifies the potential of the material to degrade in the presence of adverse conditions, such as, cyclical wetting and drying and/or carbonation
- Minimum residual UCS (RUCS) is sufficient to provide adequate structural capacity under the most severe conditions likely to be encountered in service

From an analysis of the available test methods, their degree of sophistication and the equipment required, a practical suite of tests was developed to fulfil the above requirements. The tests and limits are given in Box 6-1.

Box 6-1 Durability tests and limits for lime and cement stabilised materials

Gravel initial consumption of lime (ICL) or cement (ICC) test

       Stabiliser content = ICL or ICC + 1%

Mechanical wet and dry brushing

       Stabilised material under concrete pavement   < 5%
       Stabilised roadbase   <8%
       Stabilised sub-base   <13%

Residual UCS (carbonated)

| TRH 4/14 | 100% modified | | UCS (MPa) | |
classification	Maximum	Minimum	Maximum	Minimum
C1	12	6	8	4
C2	6	3	4	2
C3	3	1.5	2	1
C4	1.5	0.75	1	0.5

Source: Sampson 1990

The following notes relate to the tests.

Gravel ICL/ICC

It is recommended that, during any assessment of the suitability of materials for stabilised base or sub-base, the first test to be carried out should be the ICL or ICC. The inclusion of the gravel ICL/ICC test into specification limits should alleviate most of the durability problems presently experienced, for example with basic igneous materials.

Should the wet/dry brushing test and/or the residual UCS test be required, they should be carried out on samples with a stabiliser content one per cent higher than the ICL/ICC value. In some cases, the suggested stabiliser content from the gravel ICL/ICC value may exceed that which is economically viable. Using stabiliser contents in excess of about three per cent is often uneconomic. However, there is concern in South Africa that reducing the stabiliser content is more likely to increase the risk of durability problems resulting in costly premature failures in service. This is likely to be true for the more difficult materials like the calcretes

and basic igneous gravels. However, there is much evidence in the region which shows that many materials can be successfully modified with small (1-2 per cent) quantities of stabiliser.

A significant amount of work on the ICL test and its interpretation has been carried out by the KwaZulu Natal Roads Department. They now recommend that the one-hour ICL test should be supplemented by a delayed, 28 day ICL test on the same material, sealed after the original one-hour test has been carried out. Calculation of the ICL value is then made from the breaking point of the graphs of the two test results as follows:

$$ICL\ (\%) = a + (b - a) / 3$$

where:

   a = one-hour ICL breakpoint
   b = 28-day ICL breakpoint

While this method may give a more accurate ICL value, it is time consuming to obtain a result. For the purposes of evaluating stabiliser contents for other durability tests, the gravel ICL value is considered suitable.

Wet/dry brushing

The existing wet/dry brushing test given in method A19 of TMH 1 (NITRR, 1986) is very operator dependent (Sampson 1986). This has led to the development of the mechanical wet/dry brushing test described by Sampson (1988). A limited number of performance-related results were available for method A19 of TMH 1 from which tentative specification limits were developed. A correlation is shown by Sampson (1990) for converting the limits based on method A19 of TMH 1 to the limits shown above for the more reproducible mechanical brush test.

Residual UCS

The original residual UCS test described by De Wet and Taute (1985), included a UCS after 12 cycles of wetting and drying and a UCS after vacuum carbonation. However, the correlation given in Figure 7 of Sampson (1990) showed that the carbonated strength, with one exception, was always lower than the wet/dry UCS after 12 cycles. This was not surprising, as some curing and strength increase could occur during wetting and drying cycles. The results showed that, providing the requirements of the vacuum carbonated seven-day RUCS is met, this will also satisfy a residual wet/dry strength requirement. Thus, it was recommended that only the residual, vacuum-carbonated, seven-day UCS needed to be satisfied.

7.   Conclusions

7.1   Recommendations on the use of chemical stabilisation

1)   The results of the study showed that although all cement and lime stabilised bases investigated were carbonated, the pavements performed well.

2)   Chemical stabilisation of laterite, quartz and other common roadbase materials in the region can be carried out successfully.   Thus, chemical stabilisation still provides a practical and cost-effective option of improving materials for the construction of durable road pavements in many circumstances.

3)   A revised guideline is not required as there are adequate recommendations available. However, consideration should be given to introducing an upper strength limit of 2MPa into specifications (eg ORN 31) for stabilised roadbase materials to avoid excessive cracking and imbalance in the stiffness of the pavement layers.

4)   Chemical stabilisation of calcretes appears to present particular problems which are still not fully explained and need further investigation.

5)   Where materials are encountered which could give potential durability problems, additional testing using mechanical wet/dry brushing and residual UCS (carbonated) are recommended. The ICL or ICC test should be carried out for all stabilised base materials.

7.2   Evaluation of performance relative to pavements design criteria

6)   Correlations between CBR and UCS and also between the strength coefficients of stabilised and unstabilised materials were developed for use in structural number computation. A new relationship was developed for calculating the structural coefficient in terms of UCS which should be more appropriate for stabilised granular materials.

7)   The point load test proved quick and reliable, and the relationship:
   $$UCS = 5.631(Is(50))$$
was developed which enables the UCS of the material to be estimated from the point load index.

8)   Increases in plasticity due to carbonation were not generally observed. It was unclear whether the few high values measured were due to reversal of plasticity or to poor mixing at construction.

7.3   Reasons for disparate performance in the region

9)   The distress effects reported elsewhere in the region, including rutting, shearing, pumping and scabbing of the surfacing were not observed on the sites investigated. Some pot-holing of the surface was observed, but this was primarily due to poor surface maintenance.

10)   Shrinkage, causing block cracking, was prevalent in Zambia. This was probably due to the use of a high percentage of stabiliser, typically between three and six per cent. In Zimbabwe, only a one to two per cent cement content is used, resulting in a lower incidence of observed cracking. Further investigation is required to determine the factors which result in this improved pavement performance with a lower cement content.

11)   Many of the bases in Zambia appear to have lost some strength, assuming that the materials met the original CBR specification of 180 per cent. Over 30 per cent of the bases in Zimbabwe and Zambia showed *in situ* strengths greater than this. However, nearly all the sections had *in situ* CBRs greater than 80 per cent and, from a structural point of view, could carry much higher volumes of traffic than suggested by the original designs.

12)   The importance of recording and archiving field records in a convenient and reliable way, such as in a computerised database, was highlighted during the study. Records which would have proved invaluable to the investigation were destroyed due to poor management and storage.

7.4   Some suggested procedures to reduce the risk of carbonation

Although the strength losses determined on the bases in this study did not lead to premature pavement distress, it is apparent that carbonation can cause a loss in strength in some c ircumstances and it is sensible to take precautions against it during construction. These include:

- Testing the density-strength-delay time relationship in the laboratory and revising the construction procedures accordingly
- Adopting good curing practice, including continuous watering, use of sand blankets and sheets, and avoiding wetting and drying cycles during the curing phase
- Compacting as early as possible after mixing to increase the density and to reduce the permeability
- Compacting at the correct optimum moisture content for the soil-stabiliser mix
- Avoiding over-compaction of the roadbase layer which leads to micro-cracking on the surface and loose material
- Brushing the surface to remove loose material and probably carbonated material which may affect prime penetration and adherence of the seal
- Sealing with a surfacing as soon as possible after compaction to exclude carbon dioxide
- Reducing the possibility of reflection cracking by proper curing during construction
- Where cracking occurs, crack seal as soon as possible, if lower pavement layers are moisture sensitive

8. References

Bagonza, S et al, 1987. Carbonation of stabilised soil-cement mixtures. In: PTRC. *Proceedings of Seminar H, PTRC Transport and Planning Summer Annual Meeting held at the University of Bath*. London: PTRC Education and Research Services, 29-48.

British Standards Institution, 1990 BS 1924. Stabilized materials for civil engineering purposes, Part 2. Methods of test for cement stabilized and lime stabilized materials. British Standards Institution, London.

CEBTP, 1980. Guide pratique de dimensionement des chaussees pour les pays tropicaux. Paris: Centre Experimental de Recherches et d'Etudes du Batiment et des Travaux Publics.

De Wet, L F and Taute, A, 1985. Durability of stabilised materials. *Proc. Annual Transportation Convention, Pretoria, FB*, 1-15.

Department of Transport South Africa, 1986. Cementitious stabilisers in Road Construction. *Draft TRH 13*. Pretoria: Committee of State Road Authorities.

Federal Highway Administration, 1979. Soil stabilisation in pavement structures. A users guide. Volume 2. Washington DC: US Department of Commerce.

Government of the Republic of Zambia, 1973. Standard specification for roads and bridges. Lusaka: Zambia Roads Department.

Hewitt, N et al, 1998. The Zimbabwe road deterioration study. Paper to SFRDP II workshop Harare, Zimbabwe.

Hodges, J W et al 1975. The Kenya road transport cost study:research on road deterioration. Laboratory Report LR 673. Transport Research Laboratory, Crowthorne.

Lionjanga, A V et al, 1987a. The development of specifications for the use of low-grade calcretes in lightly trafficked roads in Botswana. *Transportation Research Record 1106, Vol 1.* Washington DC: National Research Council, pp 281-304.

Lionjanga, A V et al, 1987b. The use of calcrete in paved roads in Botswana, In Soil Mechanics and Foundation Engineering, Ninth Regional Conference Africa, Lagos, Vol 1. AA Balkema 489-502.

MOWS (Malawi), 1978. Standard specification for roads and bridges. Lilongwe: Ministry of Works and Supply.

National Institute for Transport and Road Research of the Council for Scientific and Industrial Research, South Africa (1986). Technical Methods for Highways, TMH1, Standard methods of testing road construction materials.

Netterberg, F, 1971. Calcrete in road construction. *NITRR Bulletin 10*. Pretoria: National Institute for Transport and Road Research. Pretoria.

Netterberg, F, 1984. Rapid field test for carbonation of lime or cement-treated materials. *National Institute for Transport and Road Research Report RS/2/84*. Pretoria: Council for Scientific and Industrial Research.

Netterberg, F, 1991. Durability of lime and cement stabilisation. *South Africa Concrete and Pavement Engineering*. Halfway House: Portland Cement Institute.

Netterberg, F et al, 1987. Prevention of surface carbonation of lime and cement stabilised pavement layers by more appropriate curing techniques. *Proceedings Annual Transportation Convention, Pretoria. Paper 4A/X*, p45.

Paterson, W D O, 1987. Road deterioration and maintenance effects: models for planning and management. *The Highway Design and Maintenance Standards Series*. Washington DC: The International Bank for Reconstruction and Development.

Pinard, M, 1987. Durability aspects of lime-stabilised weathered basalt roadbases - a case study from Botswana. *Proceedings Annual Transportation Convention. Pretoria, Volume 4A*.

Pocock, R G, 1970. The use of cement stabilised chalk in road construction. *RRL Report LR 328*. Crowthorne: Road Research Laboratory.

Sampson L R, 1986. A study of the precision limits of the wet/dry brushing durability test for cement stabilised materials. *NITRR Technical report RP/26*. Pretoria: Council for Scientific and Industrial Research.

Sampson L R, 1988. Development of the mechanical wet/dry/brushing test for the durability testing of stabilised materials. *DRTT Research report DPVT-3.1*. Pretoria: Council for Scientific and Industrial Research.

Sampson L R, 1990. Recommended durability tests and specification limits for base course aggregates for road construction. *DRTT Research report DPVT 131*. Pretoria: Council for Scientific and Industrial Research.

Sherwood, P, 1968. The properties of cement-stabilised materials. *RRL Report LR 205*. Crowthorne: Road Research Laboratory.

Sherwood, P, 1993. Soil stabilisation with cement and lime. *TRRL State of the Art Review*. Her London: H M Stationery Office.

South African Roads Board, 1990. The carbonation of chemically stabilised road construction materials: guide to its avoidance. *Project Report PR/89/146/1*. Pretoria: Department of Transport.

TRL, 1993. A guide to the structural design of bitumen-surfaced roads in tropical and sub-tropical countries. *Overseas Road Note 31*. Crowthorne: Transport Research Laboratory.

TRRL, 1977. A guide to the structural design of bitumen-surfaced roads in tropical and sub-tropical countries. *TRRL Road Note 31*. Crowthorne: H M Stationery Office, third edition.

World Bank, 1988. Road deterioration in developing countries. *Report No.6968*. Washington, DC: World Bank.

9.   Acknowledgements

Appendix A

Table A.1 Characteristics of test sections in Zambia

Site	Chainage	Base material	Surface thickness (mm)	Surface type	Base thickness (mm)	Sub-base thickness (mm)	Road width (m)	Shoulder width (m)	Shoulder type
CALI (Z1)	34+350	LAT	13	SD	205	105	6.4		
CALI (Z2)	34+650	LAT/QZ	12	SD	180	155	6.3		
CALI (Z3	35+700	LAT/QZ	10	SD	150	110	64		
CALI (Z4)	40+850	LAT/QZ	10	SD	155	135	6.4		
KASE (1)	74+960	CT		SD	160	150			
KASE (2)	80+400	SA/CT	30	SD	125	150	6.1		
KASE (3)	91+040	CT	15	SD	160	150	5.4		
KEME (X)	18+250	LAT/QZ	10	SD	155	150	6.3	1.5	Gravel
KEME (Y)	18+650	LAT/QZ	10	SD	135	145	6.3	1.5	Gravel
LAMU (4)	126+990	QZ	25	SD	130	150	6.2		
LAMU (R)	24+510	LAT	19	SD	190	125	6.6		
LAMU (S)	58+300	LAT/QZ	20	SD	130	110	6.25		
LAMU (T)	104+290	LAT/QZ	10	SD	125	125	6.2		
LEZA (X1)	51+400	GR	24	SD	145	150	6.8		
MANE (L)	158+000	LAT/QZ	30	SD	150	100	6.2		
MANE (M)	104+600	LAT/QZ	20	SD	130	85	6.1	1.0	Gravel
MANE (N)	25+000	LAT	13	SD	120	135	6.2	2.4	Gravel
MANE (O)	36+000	LAT	13	SD	120	120	6.2	2.5	Gravel
MANE (X3)	119+000	LAT	19	SD	145	145	6.2	1.0	Gravel
NAKE (P)	15+500	ND		A	150	150			
NAKE (Q)	17+600	ND		A	150	150			
SEMA (X4)	91+450	LAT	20	SD	135	120	6.4	1.0	Gravel
SEMA (X5)	90+000	LAT	13	SD	130	100	6.2	1.0	Gravel

Notes:

CT	Calcrete	SD	Surface Dressing
GR	Decomposed granite	A	Asphalt
LAT	Laterite		
ND	Not determined		
QZ	Quartz		
SA	Sand		
SST	Sandstone		

Table A.2 Summary of construction data for Zimbabwe

Road	Chainage	Pit no.	Pit classification[1]	Triaxial class	Thickness (mm)	Stabiliser	Per cent stabiliser	MDD (kg/m^3)	Compaction (% Mod AASHTO)	OMC (%)	Construction date	Sample date
BOPE (1)	21+360	BOPE13	Q/NP/F/13		120	Lime	3.0	2 145	101.0	7.2	00-12-72	07-12-96
BOPE (2)	21+895	BOPE14	Q/SP/E/21		180	Cement	2.0	2 125	102.0	5.5	00-12-72	08-12-96
BOPE (3)	61+125	BOPE29	LAT/NP/D/20		120	Cement	2.0	2 040	101.0	7.5	00-09-75	09-12-96
BOPE (4)	66+910	BOPE34	LAT/02/D/19		110	Cement	0.0	2 080	102.0	7.0	00-09-75	10-12-96
CUCI (1)	70+010	SAHY4	LAT/NP/45/15	2.8	100	Cement	3.0	2 150	98.7	7.5	31-08-85	26-11-96
MIAS (1)	1+300	CUCI1	Q/NP/46/13	2.8	100	Cement	4.5	2 160	107.6	7.5	02-11-85	21-11-96
RENA (1)	6+010	RENA12	LAT/NP/28/18	2.8	110	Cement	3.0	1 980	100.4	10.0	31-01-92	28-11-96
SIME (1)	38+500	SIME37	Q/NP/40/13	2.8	115	Cement	2.0	2 130	98.3	6.0	15-11-92	05-12-96

Notes:
(1)   Pit classification from the materials inventory
MDD   Maximum dry density
OMC   Optimum moisture content

Table A.3 Results of visual condition survey in Zambia

Site	Lane	Cracking (%)			Crack width (mm)	Cracking type	Other defects
		OWT	IWT	Lane			
CALI (Z1)	LHS		30	30	>3	Block	
CALI (Z1)	RHS			30	>3	Block	PH
CALI (Z2)	LHS	30	30	60	>3	Block	
CALI (Z2)	RHS	30	30	60	>3	Block	
CALI (Z3)	LHS			30	>3	Block	
CALI (Z3)	RHS		10	10	1-3	Longitudinal	
CALI (Z4)	LHS	30	10	45	>3	Transverse	PH
CALI (Z4)	RHS	30	30	95	>3	Block	PH
KASE (1)	LHS	30	30	95	>3	Block	EF
KASE (1)	RHS	30	30	95	>3	Block	EF
KASE (2)	LHS	30	30	95	>3	Block	
KASE (2)	RHS	30	30	95	>3	Block	
KASE (3)	LHS	0	0	0			PH
KASE (3)	RHS	0	0	0			PH
KEME (X)	LHS			30	>3	Block	
KEME (X)	RHS			30	>3	Block	
KEME (Y)	LHS			30	>3	Block	
KEME (Y)	RHS			30	>3	Block	
LAMU (4)	LHS	30	30	95	>3	Longitudinal	PH/EF
LAMU (4)	RHS	20	20	60	>3	Longitudinal	
LAMU (R)	LHS	0	0	0			
LAMU (R)	RHS	10	0	10	1-3	Longitudinal	
LAMU (S)	LHS		20	20	1-3	Block	
LAMU (S)	RHS		20	20	1-3	Block	
LAMU (T)	LHS	0	0	0			
LAMU (T)	RHS	0	0	20	1-3	Longitudinal	
LEZA (1)	LHS	5	0	10	<1	Transverse	
LEZA (1)	RHS	0	0	5	<1	Transverse	
MANE (L)	LHS	0	0	0			PH
MANE (L)	RHS	0	0	0			PH
MANE M)	LHS	0	0	0			PCH
MANE M)	RHS	0	0	0	--	--	PCH
MANE (N)	LHS	0	0	0	--	--	
MANE (N)	RHS	0	0	0	--	--	
MANE (O)	LHS	0	0	0	--	--	
MANE (O)	RHS	0	0	0	--	--	

Site	Lane	Cracking (%)			Crack width (mm)	Cracking type	Other defects
		OWT	IWT	Lane			
MANE (X3)	LHS	0	0	0	--	--	
MANE (X3)	RHS	0	0	0	--	--	
NAKE (P)	LHS	0	0	0	--	--	
NAKE (P)	RHS	0	0	0	--	--	
NAKE (Q)	LHS	0	0	0	--	--	
NAKE (Q)	RHS	0	0	0	--	--	
SEMA (X4)	LHS	0	0	0	--	--	
SEMA (X4)	RHS	0	0	0	--	--	
SEMA (X5)	LHS	10	0	30	>3	Block	
SEMA (X5)	RHS	0	0	30	>3	Block	

Notes:

IWT	Inner wheel-track	PH	Pot-holing
SNC	Modified structural number	EF	Edge Fretting
OWT	Outer wheel-track	PCH	Patching

Table A.4 Results of visual condition survey in Zimbabwe

Site	Lane	Cracking (%)			Crack width (mm)	Cracking type	Other defects
		OWT	IWT	Lane			
BOPE (1)	LHS	0	0	0	--	--	
BOPE (1)	RHS	0	0	0	--	--	
BOPE (2)	LHS	0	0	0	--	--	
BOPE (2)	RHS	0	0	0	--	--	
BOPE (3)	LHS	0	0	0	--	--	
BOPE (3)	RHS	0	0	0	--	--	
BOPE (4)	LHS		0	25	1-3	Longitudinal	
BOPE (4)	RHS		0	25	1-3	Longitudinal	Cracks on shoulder
CUCI (1)	LHS	10	0	10	1-3	Transverse	Cracks on shoulder
CUCI (1)	RHS	10	0	10	1-3	Block	Cracks on shoulder
MIAS (1)	LHS	10	0	10	1-3	Transverse	Cracks on shoulder
MIAS (1)	RHS	25	0	25	Spalling	Block	Cracks on shoulder plus ravelling
RENA (1)	LHS	10	0	10	<1	Transverse	Cracks on shoulder
RENA (1)	RHS	10	0	10	<1	Block	Cracks on shoulder
SIME (1)	LHS	0	0	0	--	--	
SIME (1)	RHS	0	0	0	--	--	

Notes:
OWT   Outer wheel-track
IWT   Inner wheel-track

Table A.5 Results of Zambia carbonation tests

Site	Sample	Chemical reactions				pH	Carb'n
		Phenol	Dil HCl	Universal	Phenol red		
CALI (Z1)	A	Clear	Mod	Orange	Purple/red	8.17	Carb
CALI (Z1)	B	Clear	Mod	Orange	Purple/red	8.17	Carb
CALI (Z1)	C	Clear	Mod	Green	Orange	8.17	Carb
CALI (Z2)	A	Clear	Mod	Orange	Pink/red	8.12	Carb
CALI (Z2)	B	Clear	Mod	Orange	Clear	8.12	Carb
CALI (Z2)	C	Clear	Mod	Orange	Yellow	8.12	NSA
CALI (Z3)	A	Clear	Rapid	Orange	Yellow	8.12	Carb
CALI (Z3)	B	Clear	Rapid	Orange	Yellow	8.12	Carb
CALI (Z3)	C	Clear	Mod	Orange	Yellow	8.12	Carb
CALI (Z3)	D	Clear	Mod	Orange	Yellow	8.12	Carb
CALI (Z4)	A	Clear	Rapid	Orange	Yellow	8.05	Carb
CALI (Z4)	B	Clear	Mod	Green	Yellow/pink	8.05	Carb
CALI (Z4)	C	Clear	Mod	Orange/green	Yellow/clear	8.05	Carb
KASE 1	A	Clear	Rapid	Dark green spots	Yellow	8.91	Carb
KASE 1	B	Clear	Mod	Dark green spots, rest clear	Red/purple to clear	8.91	Carb
KASE 1	C	Clear	Mod	Orange/green	Yellow/orange (purple/red spots)	8.91	Carb
KASE 1	D	Clear	Mod	Yellow/green	Orange turning purple/red	8.91	Carb
KASE 2	A	Clear	Mod	Orange	Yellow	8.19	Carb
KASE 2	B	Clear	Mod	Orange	Yellow pale	8.19	Carb
KASE 2	C	Clear	Mod	Orange/green	Orange/pink	8.19	Carb
KASE 2	D	Clear	Mod	Orange/green	Orange/red	8.19	Carb
KASE 3	A	Clear	Mod	Orange	Yellow	8.09	Carb
KASE 3	B	Clear	Mod	Orange	Yellow	8.09	Carb
KASE 3	C	Clear	Mod	Pale green	Pink	8.09	Carb
KEME (X)	A	Clear	Mod	Green spots	Purple red	9.17	Carb
KEME (X)	B	Clear	Mod	Green	Purple/red	9.17	Carb
KEME (X)	C	Clear	Mod	Dark green	Purple red	9.17	Carb
KEME (X)	D	Clear	Mod	Orange/green	Pink	9.17	Carb
KEME (Y)	A	Clear	Mod	Orange	Clear	8.06	Carb
KEME (Y)	B	Clear	Mod	Clear	Clear	8.06	Carb
KEME (Y)	C	Clear	Mod	Orange	Yellow	8.06	Carb
KEME (Y)	D	Clear	Mod	Orange	Yellow	8.06	Carb
LAMU (4)	A	Clear	Mod	Orange	Yellow	7.99	Carb
LAMU (4)	B	Clear	Mod	Orange	Yellow	7.99	Carb

/continued

Site	Sample	Chemical reactions				pH	Carb'n
		Phenol	Dil HCl	Universal	Phenol red		
LAMU (4)	C	Clear	Mod	Orange	Yellow	7.99	Carb
LAMU (4)	D	Clear	Mod	Green	Yellow	7.99	Carb
LAMU (R)	A	Clear	Mod	Orange	Yellow/orange	7.98	Carb
LAMU (R)	B	Clear	Rapid	Orange	Yellow	7.98	Carb
LAMU (S)	A	Clear	Mod	Green	Purple/red	8.02	Carb
LAMU (S)	B	Clear	Mod	Orange	Yellow/orange	8.02	Carb
LAMU (T)		Clear	Rapid	Clear	Clear		Carb
LEZA (X1)	A	Clear	Mod	Orange	Yellow	8.06	Carb
LEZA (X1)	B	Clear	Mod	Orange	Yellow	8.06	Carb
MANE (0)	A	Clear	Mod	Orange	Yellow to clear	10.91	Carb
MANE (0)	B	Clear	Mod	Orange	Yellow	7.91	Carb
MANE (0)	C	Clear	Mod	Orange/green	Yellow/pale green	7.91	Carb
MANE (0)	D	Clear	Mod	Orange/green	Yellow/orange	7.91	Carb
MANE (M)	A	Clear	Mod	Orange	Yellow	8.09	Carb
MANE (M)	B	Clear	Weak	Orange	Yellow	8.09	PC
MANE (M)	C	Clear	Mod	Orange	Yellow	8.09	Carb
MANE (M)	D	Red	Weak	Green	Purple red	8.09	PC
MANE (M)	D	Clear	Mod	Orange	Yellow	8.09	Carb
MANE (N)	B	Red/Orange	Weak	Dark green	Orange/red	8.23	PC
MANE (N)	C	Clear	Mod	Orange	Yellow	8.23	Carb
MANE (N)	D	Pink/Red	Weak	Orange/pale green	Purple red	8.23	Carb
MANE (X3)	C	Clear	Mod	Orange	Yellow	7.90	Carb
MANE (X3)	D	Clear	Mod	Orange	Yellow	7.90	Carb
MANE (X3)	A	Clear	Mod	Orange	Yellow		Carb
MANE (X3)	B	Clear	Rapid	Orange	Yellow		Carb
SEMA (X4)	A	Clear	Mod	Orange	Yellow	8.07	Carb
SEMA (X4)	B	Clear	Mod	Orange/green	Yellow/orange	8.07	Carb
SEMA (X4)	C	Clear	Mod	Orange/green	Yellow/pink	8.07	Carb
SEMA (X4)	D	Clear	Mod	Pale green	Yellow/pink	8.07	Carb
SEMA (X5)	A	Clear	Mod	Orange	Yellow/clear	8.26	Carb
SEMA (X5)	B	Clear	Mod	Green	Yellow/orange	8.26	Carb
SEMA (X5)	C	Clear	Mod	Orange/green	Yellow to clear	8.26	Carb
SEMA (X5)	D	Clear	Mod	Orange/green	Yellow/orange	8.26	Carb

Notes:

A,B,C   Different samples collected at same chainage

Carb   Carbonated

PC   Partially carbonated

NSA   No stabiliser added

Table A.6 Results of Zimbabwe carbonation tests

Site	Sample	Chemical reactions				pH	Carb'n
		Phenol	Dil HCl	Universal	Phenol red		
BOPE (1)	A	Clear	Rapid	Orange	Yellow	8.12	Carb
BOPE (1)	B	Clear	Rapid	Orange	Yellow	8.12	Carb
BOPE (2)	A	Clear	Mod	Pale yellow	Red-orange	8.10	Carb
BOPE (2)	B	Clear	Mod	Orange	Yellow	8.10	Carb
BOPE (2)	C	Clear	Rapid	Brownish red-orange	Yellow	8.10	Carb
BOPE (3)	A	Clear	Nil	Brownish red	Clear	7.90	NSA
BOPE (3)	B	Clear	Nil	Orange	Clear	7.90	NSA
BOPE (3)	C	Clear	Nil	Orange-brownish red	Clear	7.90	NSA
BOPE (3)	D	Clear	Nil	Orange-brownish red	Yellow	7.90	NSA
BOPE (4)	A	Clear	Mod	Brownish red-orange	Brown-yellow		Carb
BOPE (4)	B	Clear	Mod	Brownish red-orange	Yellow		Carb
BOPE (4)	C	Clear	Mod	Brownish red-orange	Yellow		Carb
CUCI (1)	A	Clear	Rapid	Pale yellow	Reddish-brown	8.23	Carb
CUCI (1)	B	Reddish-brown	Rapid	Orange	Reddish-brown/pale yellow	8.23	Carb
MIAS (1)	A	Clear	Rapid	Orange	Yellow-brown	8.37	Carb
MIAS (1)	B	Clear	Rapid	Orange	Reddish-brown	8.37	Carb
MIAS (1)	C	Clear	Rapid	Orange	Yellow-brown	8.37	Carb
MIAS (1)	D	Clear	Rapid	Orange	Orange-yellow	8.37	Carb
RENA (1)	A	Clear	Mod	Green	Orange-brownish red	8.42	Carb
RENA (1)	B	Clear	Mod	Orange	Yellow	8.42	Carb
RENA (1)	C	Clear	Mod	Orange	Yellow	8.42	Carb
RENA (1)	D	Clear	Mod	Orange-brownish red	Yellow	8.42	Carb
RENA (1)	E	Clear	Rapid	Orange	Clear	8.42	Carb
SIME (1)	A	Clear	Mod	Pale yellow	Orange	8.37	Carb
SIME (1)	B	Clear	Mod	Orange	Yellow	8.37	Carb
SIME (1)	C	Clear	Mod	Orange	Yellow-brown	8.37	Carb
SIME (1)	D	Clear	Mod	Orange	Yellow	8.37	Carb

Notes:

A,B,C   Different samples collected at same chainage
Carb   Carbonated
PC   Partially carbonated
NSA   No stabiliser added

Table A.7 Results of Zambia rut depth survey

Site	Wheel-track	Percentile rut depth (mm) left hand side				Percentile rut depth (mm) right hand side			
		50	70	80	90	50	70	80	90
CALI (Z1)	Outer	24.2	25.7	26.7	27.7	9.7	11.1	12	13.1
CALI (Z1)	Inner	6.8	7	7.1	7.2	5.6	6.9	7.4	7.7
CALI (Z2)	Outer	6.9	7.4	7.9	10.2	5.3	5.8	6.2	6.9
CALI (Z2)	Inner	10.4	11.8	12.3	13	3.3	4.2	4.6	5
CALI (Z3)	Outer	10.8	12.8	13.5	14	7.3	8.7	9.8	11
CALI (Z3)	Inner	5.2	5.6	5.9	6.8	4.4	4.9	5.7	6.9
CALI (Z4)	Outer	9.6	10.5	11.2	11.8	9.2	9.5	9.8	10.8
CALI (Z4)	Inner	0.5	2.4	3.8	5.8	0.5	2	2.7	3.4
KASE (1)	Outer	6.6	8.4	8.6	9				
KASE (1)	Inner	8.4	10.1	10.9	11.8	4.2	5	5.4	6
KASE (2)	Outer	8.5	10.2	10.9	11.9	5.6	6.4	6.7	7
KASE (2)	Inner	2	3.2	4	4.8	1.2	2.2	2.8	3.6
KASE (3)	Outer	5.7	6.8	7.8	9.8	6.2	6.8	7.5	8.6
KASE (3)	Inner	0	5.4	5.9	7.6	0	0	2.3	5.6
KEME (X)	Outer	5.6	5.8	6	6.4	6.7	7.3	7.6	8.1
KEME (X)	Inner	6.7	7.3	7.8	9	5.3	5.8	6.8	7.6
KEME (Y)	Outer	5.3	5.7	6	6.9	6.4	6.6	6.8	7.1
KEME (Y)	Inner	5.2	5.6	5.9	6.7	5.9	6.5	6.9	8
LAMU (4)	Outer	5.3	6.4	7	8	6.1	6.8	7.8	8.6
LAMU (4)	Inner	5.9	6.3	6.6	7.2	4.3	4.6	4.9	4.4
LAMU (R)	Inner	2.7	4.2	4.5	4.8	4.2	5.1	5.3	5.6
LAMU (S)	Outer	5	6.5	7.1	8.1	7.7	8.3	8.9	9.9
LAMU (S)	Inner	2.6	4.2	4.5	4.8	0	0	0	0
LAMU (T)	Outer	5.3	6.4	7	7.9	3.3	3.6	4	5.6
LAMU (T)	Inner	5.5	6.2	6.6	7	3.3	4.2	4.5	4.8
LEZA (1)	Outer	6.9	7.8	8.4	9	4.1	4.6	4.9	5.8
LEZA (1)	Inner	6.5	7.2	7.3	7.6	5.3	5.8	6.6	8.8
MANE (L)	Outer	5.6	6.4	6.8	7.1	7	8	8.6	9
MANE (L)	Inner	8.2	8.7	10.6	12.1	4.7	5.4	6	7
MANE (M)	Outer	3.9	4.9	5.4	6	3.9	4.5	4.7	5.2

Site	Wheel-track	Percentile rut depth (mm) left hand side				Percentile rut depth (mm) right hand side			
		50	70	80	90	50	70	80	90
MANE (M)	Inner	4.3	4.8	5.2	5.4	5.9	4.8	7.8	8.9
MANE (N)	Outer	4.2	4.9	5.9	6.6	6.1	6.6	6.7	7.0
MANE (N)	Inner	5.4	5.7	5.9	6.4	4.4	5.3	6.0	7.0
MANE (O)	Outer	4.4	4.8	5.0	5.4	5.4	5.8	5.9	6.8
MANE (O)	Inner	4.2	4.9	5.9	6.4	4.1	4.6	4.9	5.3
MANE (X3)	Outer	3.0							
MANE (X3)	Inner	5.9							
NAKE (P)	Outer	2.6	6.7	6.4	6.8	4.3	4.7	5.2	5.4
NAKE (P)	Inner	5.2	3.3	3.6	3.8	4.3	5.0	5.4	5.6
NAKE (Q)	Outer	1.2	2.5	3.3	4.1	3.6	4.0	4.2	4.5
NAKE (Q)	Inner	4.4	5.4	5.9	6.4	0	0.8	2.3	3.8
SEMA (X4)	Outer	5.2	5.4	5.6	5.8	4.2	4.7	5.0	5.4
SEMA (X4)	Inner	3.3	4.3	4.9	6.0	3.4	4.2	4.9	6.0
SEMA (X5)	Outer	3.2	3.6	3.9	4.8	3.3	4.0	4.3	4.6
SEMA (X5)	Inner	3.2	3.5	3.6	3.8	3.4	3.8	4.0	4.2

Table A.8 Results of Zimbabwe rut depth survey

Road	Lane	Percentile, mean and standard deviation of rut depth (mm) outer wheel-track						Percentile, mean and standard deviation of rut depth (mm) inner wheel-track					
		50	70	80	90	Mean	SD	50	70	80	90	Mean	SD
BOPE (1)	LHS	8.8	11.3	11.9	14.0	10.3	2.72	4.3	5.3	5.9	7.0	4.8	1.78
BOPE (1)	RHS	8.7	9.5	9.7	10.0	8.8	1.89	6.6	7.5	8.2	9.0	7.6	1.75
BOPE (2)	LHS	13.2	13.7	14	14.9	13.7	1.56	7.5	8.0	8.4	9.0	8.0	1.10
BOPE (2)	RHS	6.2	7.0	7.0	8.0	6.7	1.42	5.5	6.4	6.9	8.0	6.3	1.62
BOPE (3)	LHS	6.6	7.2	7.6	8.0	6.8	1.66	5.4	6.0	6.4	7.0	6.0	1.34
BOPE (3)	RHS	10.5	12.8	13.5	14.2	11.6	2.50	8.0	8.8	10.5	11.8	8.1	2.43
BOPE (4)	LHS	5.0	5.2	5.5	5.8	5.3	0.79	4.5	5.2	5.5	6.0	5.2	0.98
BOPE (4)	RHS	10.2	11.8	12.4	13.0	10.6	2.66	8.2	9.1	9.6	10.0	8.5	1.69
CUCI (1)	LHS	7.0	7.5	7.6	7.8	7.4	0.81	1.8	2.5	3.0	3.8	3.3	0.65
CUCI (1)	RHS	9.3	10.5	11	11.5	9.2	2.64	2.3	3.6	4.9	5.5	3.8	1.25
MIAS (1)	LHS	4.3	5.0	5.3	5.6	4.7	1.10	3.3	3.7	4.0	4.8	4.0	0.89
MIAS (1)	RHS	4.7	5.5	6.0	6.5	5.2	1.33	3.7	4.6	5.0	5.9	4.6	1.12
RENA (1)	LHS	5.5	6.2	6.5	6.8	6.0	0.89	4.5	4.8	5.0	5.5	5.0	0.63
RENA (1)	RHS	5.0	5.5	6.0	7.0	5.6	1.21	4.1	5.0	5.4	6.0	5.0	1.26
SIME (1)	LHS	8.8	9.3	9.5	10.0	8.4	2.34	4.2	4.7	5.0	5.9	4.7	1.49
SIME (1)	RHS	8.8	9.4	9.6	10.0	9.2	1.29	4.3	4.6	4.8	6.0	4.3	0.90

Table A.9 In situ CBRs and structural numbers from Zambia

Site	No.	Lane	Wheel-track	CBR			Thickness (mm)		ai Base	BSN	SN	SNC	SGC
				Subgrade	Sub-base	Base	Sub-base	Base					
CALI	Z1	RHS	IWT	16	20	80	105	205	0.13	1.01	1.50	3.07	1.56
CALI	Z1	RHS	OWT	16	13	40	105	205	0.09	0.73	1.17	2.73	1.56
CALI	Z2	LHS	IWT	16	29	50	155	180	0.10	0.71	1.45	3.02	1.56
CALI	Z2	LHS	OWT		23	100	155	180	0.14	0.98	1.68		
CALI	Z3	LHS	IWT	28	60	>200	110	150	0.18	1.04	1.69	3.56	1.87
CALI	Z3	LHS	OWT	28	>200	140	110	150	0.16	0.94	1.73	3.60	1.87
CALI	Z4	LHS	IWT	40	60	>200	135	155	0.18	1.08	1.84	3.86	2.01
CALI	Z4	LHS	OWT	40	27	140	135	155	0.16	0.97	1.62	3.63	2.01
KASE	1	RHS	IWT		>200	>200	150	160	0.18	1.11	2.15		
KASE	1	RHS	OWT			>200	150	160	0.18	1.11			
KASE	2	LHS	IWT		55	180	150	125	0.17	0.85	1.67		
KASE	2	LHS	OWT	>200	140	170	150	125	0.17	0.83	1.82	3.96	2.15
KASE	3	RHS	IWT	70	80	100	150	160	0.14	0.87	1.76	3.92	2.15
KASE	3	RHS	OWT	100	110	100	150	160	0.14	0.87	1.82	4.01	2.19
KEME	X	RHS	IWT	30	22	>200	150	155	0.18	1.08	1.75	3.65	1.90
KEME	X	RHS	OWT			170	150	155	0.17	1.03			
KEME	Y	LHS	IWT	30	45	>200	145	135	0.18	0.94	1.71	3.61	1.90
KEME	Y	LHS	OWT	30	45	140	145	135	0.16	0.84	1.62	3.52	1.90
LAMU	R	RHS	IWT	150	>200	160	125	190	0.17	1.24	2.13	4.31	2.18
LAMU	R	RHS	OWT	150	>200	130	125	190	0.15	1.16	2.04	4.22	2.18
LAMU	S	RHS	IWT				110	130	0.22				
LAMU	S	RHS	OWT		90	90	110	130	0.13	0.68	1.37		
LAMU	T	RHS	IWT	55	120	150	125	125	0.16	0.80	1.62	3.72	2.10
LAMU	T	RHS	OWT	27	100	170	125	125	0.17	0.83	1.62	3.47	1.85
LAMU	4	LHS	IWT	70	65	110	150	130	0.14	0.74	1.59	3.75	2.15
LAMU	4	LHS	OWT	120	70	130	150	130	0.15	0.79	1.66	3.85	2.19
LEZA	1	LHS	IWT			>200	150	145	0.18	1.01			
LEZA	1	LHS	OWT			>200	150	145	0.18	1.01			
MANE	L	RHS	IWT			>200	100	150	0.18	1.04			
MANE	L	RHS	OWT	45	130	190	100	150	0.17	1.03	1.71	3.76	2.05
MANE	M	RHS	IWT	110	160	180	85	130	0.17	0.88	1.49	3.69	2.19
MANE	M	RHS	OWT			>200	85	130	0.18	0.90			
MANE	N	LHS	IWT	29	35	>200	135	120	0.18	0.83	1.52	3.41	1.89
MANE	N	LHS	OWT			140	135	120	0.16	0.75			

/continued

Site	No.	Lane	Wheel-track	CBR			Thickness (mm)		ai Base	BSN	SN	SNC	SGC
				Subgrade	Sub-base	Base	Sub-base	Base					
MANE	O	RHS	IWT	65	120	>200	120	120	0.18	0.83	1.62	3.76	2.14
MANE	O	RHS	OWT	120	120	140	120	120	0.16	0.75	1.54	3.73	2.19
MANE	X3	RHS	IWT	65	120	>200	145	145	0.18	0.94	1.72	3.86	2.14
MANE	X3	RHS	OWT			>200	145	145	0.18	0.94			
NAKE	P	LHS	IWT			>200	150	150	0.18	1.04			
NAKE	P	LHS	OWT			>200	150	150	0.18	1.04			
NAKE	Q	RHS	IWT			>200	150	150	0.18	1.04			
NAKE	Q	RHS	OWT			>200	150	150	0.18	1.04			
SEMA	X4	RHS	IWT			>200	120	135	0.18	0.94			
SEMA	X4	RHS	IWT	130	>200	>200	120	135	0.18	0.94	1.79	3.98	2.19
SEMA	X5	RHS	IWT	65	80	>200	100	130	0.18	0.90	1.53	3.67	2.14
SEMA	X5	RHS	OWT			>200	100	130	0.18	0.90			

Notes:
ai	Pavement layer strength coefficient
BSN	Contribution of roadbase to structural number
SNC	Modified structural number
SGC	Contribution of subgrade to structural number
SN	Structural number

Table A.10 In situ CBRs and structural numbers from Zimbabwe

Site	Lane	Wheel-track	CBR			Thickness (mm)		ai Base	BSN	SN	SNC	SGC
			Subgrade	Sub-base	Base	Sub-base	Base					
BOPE (1)	LHS	IWT	70	>200	>200	175	120	0.18	0.83	2.03	4.19	2.15
BOPE (1)	LHS	OWT	40	130	110	175	120	0.14	0.68	1.80	3.81	2.01
BOPE (1)	LHS	IWT	55	130	>200	175	120	0.18	0.83	1.95	4.05	2.10
BOPE (1)	LHS	OWT	90	65	>200	175	120	0.18	0.83	1.81	4.00	2.18
BOPE (1)	LHS	IWT	70	>200	>200	175	120	0.18	0.83	2.03	4.19	2.15
BOPE (1)	LHS	OWT	35	55	130	175	120	0.15	0.73	1.68	3.64	1.96
BOPE (2)	RHS	IWT	19	80	130	190	180	0.15	1.16	1.94	3.61	1.67
BOPE (2)	RHS	OWT	16	65	110	190	180	0.14	1.02	2.08	3.64	1.56
BOPE (2)	RHS	IWT	21	65	100	190	180	0.14	0.98	2.04	3.76	1.72
BOPE (2)	RHS	OWT	27	75	80	190	180	0.13	0.89	1.97	3.83	1.85
BOPE (2)	RHS	IWT	35	80	130	190	180	0.15	1.10	2.20	4.16	1.96
BOPE (2)	RHS	OWT	20	100	100	190	180	0.14	0.98	2.13	3.83	1.70
BOPE (3)	LHS	IWT	50	80	120	140	110	0.15	0.65	1.49	3.57	2.08
BOPE (3)	LHS	OWT	75	75	90	140	110	0.13	0.57	1.40	3.56	2.16
BOPE (3)	LHS	IWT	75	70	110	140	110	0.14	0.63	1.44	3.61	2.16
BOPE (3)	LHS	OWT	55	80	100	140	110	0.14	0.60	1.44	3.54	2.10
BOPE (3)	LHS	IWT	65	65	140	140	110	0.16	0.69	1.49	3.63	2.14
BOPE (3)	LHS	OWT	70	75	110	140	110	0.14	0.63	1.45	3.61	2.15
BOPE (4)	RHS	IWT	140	100	120	120	120	0.15	0.71	1.47	3.66	2.19
BOPE (4)	RHS	OWT	100	90	180	120	120	0.17	0.81	1.56	3.75	2.19
BOPE (4)	RHS	IWT	100	90	130	120	120	0.15	0.73	1.48	3.67	2.19
BOPE (4)	RHS	OWT	100	110	>200	120	120	0.18	0.83	1.61	3.80	2.19
BOPE (4)	RHS	IWT	110	100	120	120	120	0.15	0.71	1.47	3.66	2.19
BOPE (4)	RHS	OWT	120	100	170	120	120	0.17	0.80	1.56	3.75	2.19
CUCI (1)	RHS	IWT	130	160	200	130	100	0.18	0.69	1.58	3.77	2.19
CUCI (1)	RHS	OWT	110	140	180	130	100	0.17	0.68	1.54	3.74	2.19
CUCI (1)	RHS	OWT	110	140	180	130	100	0.17	0.68	1.54	3.74	2.19
CUCI (1)	RHS	IWT	140	170	200	130	100	0.18	0.69	1.59	3.78	2.19
CUCI (1)	RHS	OWT	120	180	170	130	100	0.17	0.67	1.57	3.76	2.19
CUCI (1)	RHS	IWT	100	>200	170	130	100	0.17	0.67	1.58	3.77	2.19
CUCI (1)	RHS	OWT	120	110	150	130	100	0.16	0.64	1.47	3.66	2.19
CUCI (1)	RHS	IWT	130	160	200	130	100	0.18	0.69	1.58	3.77	2.19
MIAS (1)		IWT	80	120	>200	115	100	0.18	0.69	1.45	3.62	2.17
MIAS (1)		OWT	35	100	>200	115	100	0.18	0.69	1.43	3.39	1.96
MIAS (1)	RHS	IWT	80	130	>200	115	100	0.18	0.69	1.46	3.63	2.17

/continued

Site	Lane	Wheel-track	CBR			Thickness (mm)		ai Base	BSN	SN	SNC	SGC
			Subgrade	Sub-base	Base	Sub-base	Base					
MIAS (1)	RHS	OWT	65	150	>200	115	100	0.18	0.69	1.48	3.62	2.14
MIAS (1)	RHS	IWT	100	150	>200	115	100	0.18	0.69	1.48	3.67	2.19
MIAS (1)	RHS	OWT	90	190	>200	115	100	0.18	0.69	1.51	3.69	2.18
RENA (1)	LHS	IWT	65	110	>200	135	110	0.18	0.76	1.62	3.76	2.14
RENA (1)	LHS	OWT	60	70	>200	135	110	0.18	0.76	1.55	3.68	2.12
RENA (1)	LHS	IWT	70	130	>200	135	110	0.18	0.76	1.65	3.80	2.15
RENA (1)	LHS	OWT	50	150	>200	135	110	0.18	0.76	1.67	3.75	2.08
RENA (1)	LHS	IWT	60	190	>200	135	110	0.18	0.76	1.70	3.83	2.12
RENA (1)	LHS	OWT	50	130	>200	135	110	0.18	0.76	1.65	3.73	2.08
SIME (1)	RHS	OWT	65	100	160	115	115	0.17	0.75	1.49	3.63	2.14
SIME (1)	RHS	IWT	140	150	130	115	115	0.15	0.70	1.49	3.67	2.19
SIME (1)	RHS	IWT	120	170	120	115	115	0.15	0.68	1.48	3.67	2.19
SIME (1)	RHS	OWT	90	130	140	115	115	0.16	0.72	1.49	3.67	2.18
SIME (1)	RHS	IWT	130	140	130	115	115	0.15	0.70	1.48	3.67	2.19
SIME (1)	RHS	OWT	100	140	120	115	115	0.15	0.68	1.46	3.65	2.19

Table A.11 Unconfined compressive strength and point load test results from Zambia

Site	Sample	Condition	UCS (MPa)	Condition	Mean IS(50)	SD	Density (kg/m^3)
CALI (Z1)	A	Unsoaked	1.15				2 199
CALI (Z1)	A			Soaked	0.19	0.09	2 199
CALI (Z1)	B	Unsoaked	3.05	Unsoaked	0.23	0.15	2 175
CALI (Z1)	C	Soaked	0.85				
CALI (Z2)	A	Unsoaked	2.46				
CALI (Z2)	A			Soaked	0.10	0.08	2 242
CALI (Z2)	B			Unsoaked	0.36	0.14	2 174
CALI (Z2)	C	Soaked	2.22				
CALI (Z3)	A	Unsoaked	0.66	Unsoaked	0.43	0.28	2 193
CALI (Z3)	B	Unsoaked	0.72				2 244
CALI (Z3)	C	Soaked	0.34				
CALI (Z3)	D	Soaked	0.75				
CALI (Z4)	A	Unsoaked	0.30				1 608
CALI (Z4)	B	Soaked	0.12				2 183
CALI (Z4)	C	Soaked	0.12				2 162
KASE (1)	A	Unsoaked	4.24				2 113
KASE (1)	A			Soaked	0.69	0.23	2 113
KASE (1)	B	Soaked	0.89	Unsoaked	0.69	0.27	2 118
KASE (1)	C	Unsoaked	4.93				
KASE (1)	D	Unsoaked	3.72				
KASE (2)	A	Unsoaked	0.81				1 852
KASE (2)	A			Soaked	0.15	0.09	1 852
KASE (2)	B	Unsoaked	1.26	Unsoaked	0.16	0.08	1 952
KASE (2)	C	Unsoaked	1.51				1 932
KASE (2)	D	Soaked	0.57				
KASE (3)	A	Unsoaked	0.30				1 703
KASE (3)	A			Unsoaked	0.18	0.09	1 703
KASE (3)	B	Unsoaked	0.17	Soaked	0.07	0.03	1 720
KASE (3)	C	Unsoaked	0.30				1 756
KEME (X)	A	Unsoaked	4.53				2 176
KEME (X)	A			Soaked	0.14	0.09	2 176
KEME (X)	B	Unsoaked	4.11	Unsoaked	0.65	0.22	2 108
KEME (X)	C	Soaked	2.05				
KEME (X)	D	Soaked	4.79				
KEME (Y)	A	Unsoaked	0.77				2 054
KEME (Y)	A			Soaked	0.09	0.07	2 054

/continued

Site	Sample	Condition	UCS (MPa)	Condition	Mean IS(50)	SD	Density (kg/m³)
KEME (Y)	B	Unsoaked	0.95	Unsoaked	0.08	0.04	2 088
KEME (Y)	C	Soaked	0.85				
KEME (Y)	C			Unsoaked	0.09	0.07	
KEME (Y)	D	Soaked	0.75				
LAMU (4)	A	Unsoaked	0.77				1 987
LAMU (4)	A			Soaked	0.07	0.05	1 987
LAMU (4)	B	Unsoaked	0.42	Unsoaked	0.03	0.01	1 944
LAMU (4)	C	Unsoaked	0.28				1 910
LAMU (4)	C						1 910
LAMU (4)	D	Soaked	0.32				
LAMU (4)	D						
LAMU (R)	A	Unsoaked	0.59	Unsoaked	0.03	0.01	2 085
LAMU (R)	B	Unsoaked	0.26				2 117
LAMU (S)	A	Unsoaked	0.21	Unsoaked	0.04	0.02	2 017
LAMU (S)	B	Unsoaked	0.51				2 014
LAMU (S)	B						2 014
LAMU (S)	C						2 014
LAMU (T)							
LAMU (T)							
LAMU (T)							
LAMU (T)	A			Unsoaked	0.18	0.08	2 013
LAMU (T)	B			Soaked	0.08	0.09	1 946
LEZA (X1)	A	Unsoaked	0.43				2 050
LEZA (X1)	B	Unsoaked	0.21				2 101
LEZA (X1)	C						2 069
MANE (0)	A	Unsoaked	0.59				1 951
MANE (0)	A			Soaked	0.03	0.00	1 951
MANE (0)	B	Unsoaked	0.60	Unsoaked	0.04	0.02	1 975
MANE (0)	C	Soaked	0.15				
MANE (0)	D	Soaked	0.17				
MANE (M)	A	Unsoaked	1.28				1 983
MANE (M)	A			Unsoaked	0.32	0.12	1 983
MANE (M)	B			Soaked	0.10	0.05	1 955
MANE (M)	C	Soaked	0.78				
MANE (M)	D	Soaked	1.23				
MANE (N)	A	Soaked	0.78				1 970
MANE (N)	A			Soaked	0.45	0.17	1 970

/continued

Site	Sample	Condition	UCS (MPa)	Condition	Mean IS(50)	SD	Density (kg/m^3)
MANE (N)	B	Unsoaked	3.62				2 032
MANE (N)	B			Unsoaked	0.47	0.14	2 032
MANE (N)	C	Soaked	3.09				
MANE (N)	D	Soaked	2.71				
MANE (X3)	A	Unsoaked	1.75				2 189
MANE (X3)	A			Soaked	0.13	0.06	2 189
MANE (X3)	B			Unsoaked	0.32	0.14	2 196
MANE (X3)	C	Soaked	0.74				
MANE (X3)	D	Soaked	0.50				
SEMA (X4)	A	Unsoaked	1.90	Unsoaked	0.04	0.01	1 979
SEMA (X4)	B	Soaked	0.58				1 903
SEMA (X4)	C	Soaked	0.57				
SEMA (X4)	D	Soaked	0.58				
SEMA (X5)	A	Soaked	0.68				2 009
SEMA (X5)	A			Soaked	0.29	0.13	2 009
SEMA (X5)	B	Soaked	0.71				2 034
SEMA (X5)	B			Unsoaked	0.36	0.17	2 034
SEMA (X5)	C	Unsoaked	2.61				
SEMA (X5)	D	Unsoaked	3.49				

Notes:

A-E   Different samples taken from the same chainage

IS(50)   Point load test index

SD   Standard deviation

No samples were recovered from NAKA (P) and (Q)

Table A.12 Unconfined compressive strength (block) and point load test results from Zimbabwe

Site	Sample	Per cent Stabilised	Condition	UCS	Condition	Mean IS(50)	SD	Density (kg/m³)
BOPE (1)	A	3.0-3.5	Unsoaked	0.59	Unsoaked	0.06	0.08	2 376
BOPE (1)	B	3.0-3.5	Unsoaked	0.67				2 262
BOPE (2)	A	1.6-1.8	Unsoaked	0.02	Unsoaked	0.07	0.05	2 618
BOPE (2)	B	1.6-1.8	Unsoaked	0.03				2 194
BOPE (2)	C	1.6-1.8	Unsoaked	0.34				
BOPE (3)	A	0.6-0.8	Unsoaked	0.36	Unsoaked	0.03	0.02	2 250
BOPE (3)	B	0.6-0.8	Unsoaked	0.44				2 339
BOPE (3)	C	0.6-0.8	Unsoaked	0.43				
BOPE (3)	D	0.6-0.8	Unsoaked	0.29				
BOPE (4)	A		Unsoaked	0.17	Unsoaked	0.03	0.01	2 274
BOPE (4)	B		Unsoaked	0.16				2 045
BOPE (4)	C		Unsoaked	0.34				
CUCI (1)	A	1.6	Unsoaked	0.07	Unsoaked			2 511
CUCI (1)	B	1.6	Unsoaked	0.06				2 947
MIAS (1)	A	2.4	Unsoaked	2.22	Unsoaked	0.52	0.18	2 369
MIAS (1)	B	2.5-2.8	Unsoaked	4.84				
MIAS (1)	C	2.5-2.8	Unsoaked	3.44				
MIAS (1)	D	2.5-2.8	Soaked	1.78	Soaked	0.40	0.20	
RENA (1)	A	2.1	Unsoaked	0.13	Unsoaked	0.25	0.05	2 082
RENA (1)	B	3.0-3.2	Unsoaked	0.15				2 169
RENA (1)	C	3.0-3.2	Unsoaked	0.09				
RENA (1)	D	3.0-3.2	Unsoaked	0.60				
RENA (1)	E	3.0-3.2	Soaked	2.02	Soaked	0.17	0.06	
SIME (1)	A	3.0	Unsoaked	0.03	Unsoaked	0.08	0.04	2 300
SIME (1)	B	2.0-2.7	Unsoaked	0.12				
SIME (1)	C	2.0-2.7	Unsoaked	0.88				
SIME (1)	D		Soaked	0.34	Soaked	0.03	0.01	

Notes:
Abbreviations as in Table A.11

Table A.13 Properties of roadbase materials from Zambia

Site no.	Layer	Percentage passing sieves (mm)														WL	Ip	Ls
		50.0	37.5	26.5	19.0	13.2	9.5	6.3	4.75	2.36	1.18	0.425	0.300	0.150	0.075			
CALI (Z1)	Base	100	100	93	92	89	73	68	61	40	26	16	16	16	8	41	SP	1
CALI (Z2)	Base	100	100	86	79	74	61	57	53	39	28	17	14	9	7	51	NP	1
CALI (Z3)	Base	100	100	90	84	81	66	60	54	37	27	18	16	12	10	37	NP	1
CALI (Z4)	Base	100	100	90	84	78	57	50	42	22	13	7	6	5	4	24	NP	0
KASE (1)	Base	100	96	52	37	32	23	20	17	14	8	4	3	2	1	34	NP	0
KASE (2)	Base	100	94	86	75	74	62	59	55	49	46	34	25	14	9	33	SP	1
KASE (3)	Base	100	100	100	100	91	88	88	85	73	63	35	25	12	10	28	NP	0
KEME (X)	Base	100	97	67	64	55	25	19	13	3	1	1	1	1	1	47	NP	0
KEME (Y)	Base	100	100	87	80	78	66	62	57	37	23	10	8	6	5	45	17	7
LAMU (4)	Base	100	100	96	94	91	80	75	71	56	48	42	40	33	26	32	5	2
LAMU (R)	Base	100	100	89	71	64	53	47	41	30	27	25	24	20	14	34	NP	0
LAMU (S)	Base	100	100	96	94	93	83	76	65	40	33	27	26	22	16	33	15	6
LEZA (X1)	Base	100	100	100	100	100	100	100	97	68	46	26	22	18	17	38	NP	0
LAMU (T)	Base	100	100	100	97	96	85	78	69	44	37	31	30	24	22	37	SP	-
MANE (L)	Base	100	100	100	98	97	92	85	73	27	14	9	8	7	6	30	NP	0
MANE (M)	Base	100	100	93	80	68	30	23	18	11	9	7	6	4	3	33	NP	0
MANE (N)	Base	100	100	94	79	74	49	42	35	23	18	13	12	10	9	33	SP	1
MANE (O)	Base	100	100	100	98	95	80	73	65	42	33	24	21	14	12	29	NP	0
MANE (X3)	Base	100	93	68	31	23	8	7	6	5	5	5	5	4	3	30	NP	1
SEMA (X4)	Base	100	100	100	99	97	79	71	60	40	35	30	27	20	17	34	SP	2
SEMA (X5)	Base	100	100	97	78	59	35	27	13	10	5	3	3	2	2	37	NP	-

Notes:
WL   Liquid limit
Ip   Plasticity index
Ls   Linear shrinkage
NP   Non-plastic
SP   Slightly plastic

Table A.14 Properties of sub-base materials from Zambia

Site no.	Layer	Percentage passing sieves (mm)														WL	Ip	Ls
		50.0	37.5	26.5	19.0	13.2	9.5	6.3	4.75	2.36	1.18	0.425	0.300	0.150	0.075			
CALI (Z1)	Sub-base	100	100	100	99	99	97	90		80		53	57	43	36	40	21	4
CALI (Z2)	Sub-base	100	100	93	89	89	87	72		46		30	27	22	20	44	19	7
CALI (Z3)	Sub-base	100	83	80	79	77	74	64		50		40	38	32	27	-	-	7
CALI (Z4)	Sub-base	100	86	86	86	85	83	72		61		51	49	41	34	42	21	7
KASE (1)	Sub-base	100	100	100	100	100		95		89		59	44	21	11	44	NP	0
KASE (2)	Sub-base	100	100	98	96	93	89	86		84		59	44	21	10		NP	0
KASE (3)	Sub-base	100	100	100	100	100	100	97		93		55	45	28	20		NP	0
KEME (X)	Sub-base	100	65	65	65	64	63	58		43		28	26	22	19		NP	6
KEME (Y)	Sub-base	100	85	78	74	74	71	64		46		29	27	23	20	45	22	7
LAMU (4)	Sub-base	100	95	91	89	87	79	73	50	57	36	48	46	39	30	31	15	5
LAMU (R)	Sub-base	100	100	97	93	92	89	74	65	52	48	42	40	34	26	31	16	4
LAMU (S)	Sub-base	100	100	96	86	82	78	65	58	51	46	41	39	33	21	30	18	4
LAMU (T)	Sub-base	100	81	76	73	67	58	45	42	37	36	31	30	26	20	34	19	5
LEZA (X1)	Sub-base	100	100	100	100	100	100	98		80		48	44	36	32	36	NP	0
MANE (L)	Sub-base	100	100	100	100	100	98	86		49		22	20	16	12	30	14	6
MANE (M)	Sub-base	100	80	75	72	68	60	50		44		31	27	17	11	30	NP	0
MANE (N)	Sub-base	100	100	100	100	100	99	93		84		67	64	58	53	-	-	5
MANE (O)	Sub-base	100	100	100	98	91	86	73		60		44	39	27	21	31	11	4
MANE (X3)	Sub-base	100	86	70	65	62	55	42		33		26	24	20	18	27	11	4
SEMA (X4)	Sub-base	100	100	100	100	100	57	80		63		47	41	31	27	36	SP	3
SEMA (X5)	Sub-base	100	100	93	90	82	73	47		32		26	24	20	15	30	13	3

Notes:

Abbreviations as in

Table **A.13**

Table A.15 Comparison of structural designs

Site	Base thickness (mm)	Sub-base thickness (mm)	Subgrade		Base CBR	Design SNC	Max traffic design class (ORN31)	Potential traffic class based on in situ SNC	Average SNC (CBR eqn)
			SG ICBR	SG class					
CALI (Z1)	205	105	16-20	S5	80	3.35	T4	T1	2.90
CALI (Z2)	180	155	16	S5	50	3.35	T4	T1-T2	3.02
CALI (Z3)	150	110	28	S5	>200	2.98	T2	T4	3.58
CALI (Z4)	155	135	40	S6	>200	3.26	T4	T6	3.74
KASE (1)	160	150			>200				
KASE (2)	125	150	>80	S6	180	2.94	<T1	T6	3.96
KASE (3)	160	150	70->80	S6	100	2.94	T2	T6	3.96
KEME (X)	155	150	30	S6	>200	2.94	T2	T6	3.65
KEME (Y)	135	145	30	S6	>200	2.94	<T1	T6	3.56
LAMU (4)	130	150	>80	S6	130	2.94	<T1	T6	3.80
LAMU (R)	190	125	>80	S6	130	3.1	T3	T6	4.27
LAMU (S)	130	110							
LAMU (T)	125	125	27-55	S6	170	2.94	<T1	T6	3.60
LEZA (1)	145	150			>200				
MANE (L)	150	100	45	S6	>200	2.94	T2	T6	3.76
MANE (M)	130	85	>80	S6	>200	2.94	<T1	T6	3.69
MANE (N)	120	135	29	S5	>200	2.94	<T1	T4	3.41
MANE (O)	120	120	>80	S6	>200	2.94	<T1	T6	3.74
MANE (X3)	145	145	65	S6	>200	2.94	T1	T6	3.86
NAKE (P)	150	150			>200				
NAKE (Q)	150	150			>200				
SEMA (X4)	135	120	>80	S6	>200	2.94	<T1	T6	3.98
SEMA (X5)	130	100	65	S6	>200	2.94	<T1	T6	3.67
BOPE (1)	120	175	35-40	S6	>200	2.94	<T1	T6	3.98
BOPE (2)	180	190	16-27	S5	110	3.35	T4	T6	3.81
BOPE (3)	120	120	55-75	S6	90	2.94	<T1	T6	3.59
BOPE (4)	110	140	>80	S6	180	2.94	<T1	T6	3.71
CUCI (1)	100	130	>80	S6	180	2.94	<T1	T6	3.75
MIAS (1)	100	115	35-90	S6	>200	2.94	<T1	T6	3.61
RENA (1)	110	135	50-60	S6	>200	2.94	<T1	T6	3.76
SIME (1)	115	115	>80	S6	130	2.94	<T1	T6	3.66

Notes:
SG ICBR   Subgrade in situ CBR
SG class   Subgrade design class
SNC   Modified structural number