


Guide for the Use of Sand in Road Construction in the SADC Region

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TABLE OF CONTENTS

ANNEXES

LIST OF TABLES

LIST OF FIGURES

LIST OF PHOTOGRAPHS

LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transport Officials
AFCAP	Africa Community Access Programme
ANE	Administração Nacional de Estradas
ASANRA	Association of Southern African National Roads agencies
BRDM	Botswana Road Design Manual
BLS	Bar Linear Shrinkage
BS	British Standard
BSM	Bitumen Sand Mix
CBR	California Bearing Ratio
COLTO	Committee of Land Transport Officials
CSIR	Council for Scientific and Industrial Research
Cu	Uniformity Coefficient
DCP	Dynamic Cone Penetrometer
ESA	Equivalent Standard Axle
ETB	Emulsion Treated Base
FI	Fineness Index
GEMS	Granular Emulsion Mixes
GM	Grading Modulus
ITS	Indirect Tensile Strength
LL	Liquid Limit
LOI	Loss on Ignition
LVR	Low Volume Road
LVSR	Low Volume Sealed Road
MDD	Maximum Dry Density
NAASRA	National Association of Australian State Road Authorities
OMC	Optimum Moisture Content
PI	Plasticity index
PL	Plastic Limit
PM	Plastic Modulus
P425	Percentage Material Passing the 0.425 mm Sieve
P075	Percentage Material Passing the 0.075 mm Sieve
RD	Roads Department
SABITA	South African Bitumen Association
SADC	Southern Africa Development Community
SAMDP	South African Mechanistic Design Procedure
SANS	South African National Standard
STE	Sand Treated with Emulsion
TMH	Technical Methods for Highways
TRL	Transport Research Laboratory
XRD	X-Ray Diffraction
XRF	X-Ray Fluorescence

GLOSSARY OF TERMS

Aeolian	Wind borne; sediments transported and deposited by wind action
Bioturbation	Soil that has been reworked or mixed by living organisms
Colluvial	Weathered material transported by gravity
Deposition	Laying down of material by agents of erosion
Fluvial	Related to streams and rivers, i.e. deposited by a stream or a river
Kurtosis	The sharpness of the peak of a frequency-distribution curve
Lacustrine	Relating to lakes or a lake environment
Lithified	Conversion to rock-like material
Skewness	Degree of slant of the slope of a frequency-distribution curve
Soil suction	A negative pore water pressure within soil due to capillary and salt effects; it is measured in a log scale of pF units (i.e. 1pF = 10 cm head of water).
Transported soil	A soil which has been removed from its place of formation and redeposited

Range of sand colours found in the SADC region.

Source: *SAND: A Journey Through Science and the Imagination,* Michael Welland, Oxford University Press, 2010

EXECUTIVE SUMMARY

Sand is abundantly available in many countries in Africa and elsewhere in the world. However, very few of the pavement and material design guides currently in use worldwide cater for the use of this non-traditional material in its untreated state in the structural pavement layers, despite many instances of its good performance in low volume roads. This is the case largely because of an apparent lack of understanding of the characteristics and properties of sand, coupled with lack of compliance with conventional specifications – factors that have suppressed the more wide-spread use of this ubiquitous material in road pavement construction. This experience provided a strong motivation for the Association of Southern African National Roads Agencies (ASANRA) to initiate the development of a ***Guide on the Use of Sand in Road Construction in the SADC Region.***

In view of the above, wide-ranging investigations were undertaken of a large number of sand samples that were obtained both from existing roads and naturally occurring sand deposits located in Botswana, Malawi, Mozambique, Namibia and South Africa. These samples are considered to be representative of the wide range of sand types that occur in the southern African region and probably elsewhere in Africa.

The objective of the investigations was primarily to ascertain the physical, mineralogical and engineering properties of a wide range of sands that could influence their performance in road pavements. On this basis, those sand properties that discriminate between good and poor performance could be isolated and included in appropriate specifications for selecting specific material types for use in road pavement construction. This, in turn, would lead to more wide-spread use of sand and the lowering of the overall cost of road construction..

The techniques adopted on the project for evaluating the suitability of sands in road construction, in terms of their grading, are not conventional. Instead, they focus on alternative measures of grain size based on the *Φ (phi) sediment size scale,* a size scale which is commonly used in sedimentology. In this regard, based on experience emanating from Australia, the potential performance of sands can be assessed in terms of their mean particle size (in phi units) representing the fineness of the material, and the standard deviation of the grading (in phi units) representing the degree of sorting (grading) of the sand as represented by the so-called Wylde Chart (ref. Figure 3-3).

Other unique aspects of the project include the recognition that the soil constants of the sands that affect their potential performance as a pavement material are best determined on the silt and clay fractions, i.e. on the material passing the 0.075 mm sieve, and not just on the sand fraction, i.e. on the material passing the 0.425 mm sieve, as is traditionally done. Further, a knowledge of the chemistry of the sands, particularly in terms of their free aluminium and iron oxide/hydroxide contents, is also apparently useful in assessing potential performance.

The following is a summary of the main findings and recommendations arising from the investigations:

- The grading of the sand has an impact on its performance in a road pavement, both in terms of providing strength and assisting compactability. In this regard, the presence of a suitable fine fraction is necessary to ensure that a high suction strength is built up in the compacted material when used in its unsaturated state. This appears to be essential and is probably best identified using the mean particle size (Φ units) and standard deviation of the sand (Φ units). Accordingly, the mean particle size (Φ units) and standard deviation of the sand (Φ units) should be used as a preliminary screening test for the identification of potentially suitable sands. An investigation of other test methods that could be used to replace the full grading analysis necessary to determine these parameters has shown no suitable replacements.

- As is suspected to be the case in laterites, the free iron (and aluminium) contents are the major contributor to developing strength with time. In this regard, a minimum iron and aluminium content (i.e. the sesquioxide content) is specified for the use of sands in Western Australia.
- The fact that the sesquioxides content is a combination of iron and aluminium is significant in terms of the colours of the sands. Iron will always stain the sands a yellow, brown, red or dark red, (depending on its content) but aluminium will not. The fact that a number of light brown and grey sands appear to develop high strengths in roads may possibly be attributed to the aluminium content and not necessarily only to the iron content. It may therefore just be coincidence that previous experience with the red sands has been much better than experience with the lighter coloured sands, many of which do not have any fine plastic material in their grading and, in fact, may not even have any aluminium. Thus, colour alone should thus not be used as an important selection criterion.
- In terms of the use of sands in roads, and especially in base courses, it is primarily their in-service strength that determines their suitability for supporting traffic loadings. This should thus be the main selection criterion **but this project has shown that this is a difficult property to characterise consistently.** The materials can be difficult to compact in the laboratory and unusual behaviour has been detected in many of the tests and procedures conventionally used for material characterisation for roads. Despite the more obvious problems such as material variability and the effects of minor changes in sample preparation and testing techniques, **it appears that many of the current standard testing procedures are not appropriate for use with sands as investigated in this project.**
- The **s**tandard compaction and strength testing, even by an accredited and experienced laboratory, have yielded highly variable results. It is expected that testing by smaller, less experienced local laboratories will produce even worse results and a revised testing regime should be employed. This should concentrate on the strength of the compacted sand that can be mobilised at different density and moisture conditions that simulate in-service conditions in the road closely. **Indirect tests such as the CBR could possibly be avoided and the actual strength in terms of, for instance, the DCP penetration rate should be investigated.**
- The project has confirmed the findings of a number of earlier studies regarding the correlation between CBR and DCP penetration rate for sands, which have shown that the commonly used correlations may be invalid for sands. It has also shown that low higher penetration rates are found in the laboratory than in the field under similar conditions. ***New protocols must be urgently developed for laboratory testing of***

sands using the DCP in order to benefit fully from this most cost-effective and appropriate test technique.

- Despite all of the testing criteria and performance specifications it is apparent that the state of the materials in the road is still the major contributor to performance. The degree of densification, the in situ moisture content (related to effective drainage) and probably the effects of traffic moulding appear to all contribute to successful performance of sands.
- The construction of sand pavements needs to be undertaken carefully. Sands tend to be moisture sensitive and proper compaction will only be achieved with the correct type of plant operating at the appropriate moisture content. However, such roads have been successfully constructed before and there is no reason why they should present insurmountable problems in future. In addition, as is the case for all roads constructed of natural occurring materials, **drainage is of paramount importance** as is maintenance.

In summary, the project has provided appropriate guidance on methods of prospecting for, screening and testing of, sands to ascertain their potential suitability for use as neat base course in the construction of low volume sealed road pavements. Guidance has also been provided on how to design low volume road pavements using a simplified method based on the use of the DCP as well as how to construct them using sand as a pavement material.

In terms of the way forward, it is recommended that a gradual, staged approach should be followed in ensuring that the outcome of the investigations are put into practice through the construction of demonstration projects which are comprehensively monitored on a regular basis.

It is important to ensure that effective technology transfer of the use of sands in road construction occurs within the region. For this to be achieved, all the links of the technology transfer chain should be addressed in terms of the typical pathway from research to implementation, as illustrated in the Figure E-1 below.

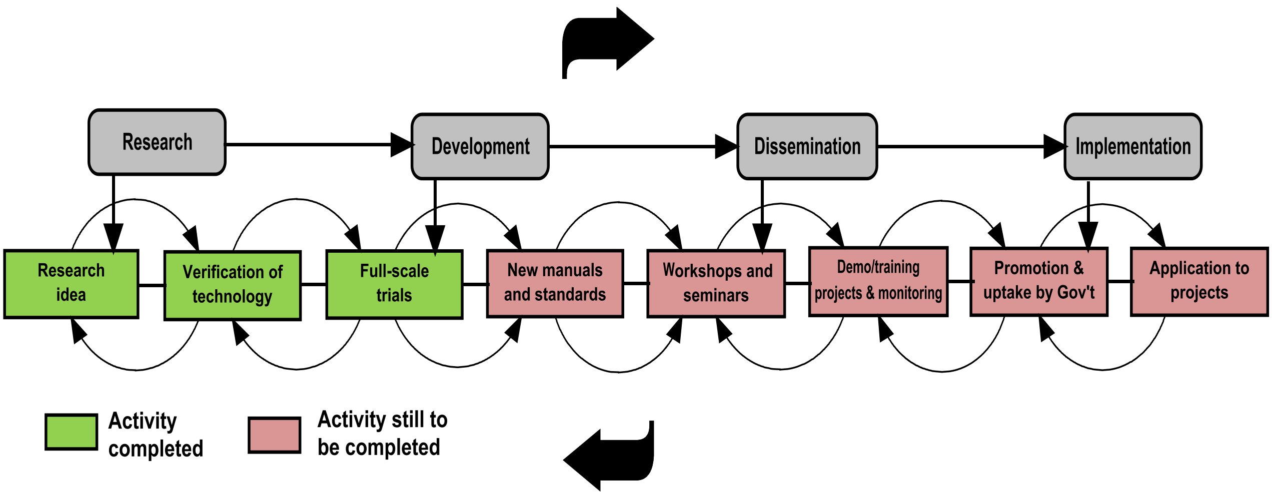


Figure E-1: Pathway from research to implementation
(adapted from TRB Special Report 256, 1999)

The following activities are still required to complete the pathway to full implementation of sands technology in the SADC region:

- **New manuals**: The current ASANRA Guide is a step in the right direction. However, this information still needs to be incorporated in national standards of all SADC countries. Until new, nationally approved manuals are in place, the implementation of sands technology will be impeded.

- **Workshops and seminars:** There is need for promotion of the sands technology to a wider, national audience in all countries so as to obtain understanding and buy-in of the new technology.

- **Demonstration/training projects and monitoring**: This needs to be undertaken in all countries where neat sand is potentially suitable for use in low volume road construction. Monitoring of these demonstration projects is of paramount importance to provide inputs for refinement of the LVR designs and construction.

- **Promotion and uptake by Government:** This is a critical activity which will ultimately manifest itself in the form of national policy.

- **Application to projects:** This final link in the technology transfer chain will only occur when all the preceding activities outlined in Figure 7-1 have been addressed.

Finally, it is recommended that the wealth of information contained in this Guide is distilled to produce a succinct ***Guideline on The Use of Sands in Road Construction*** which focuses specifically on those aspects of the Guide that influence the manner of selecting, specifying, designing and constructing low volume roads using neat sand as the pavement material.

1. INTRODUCTION

1.1 Background

In many parts of the Southern Africa Development Community (SADC) region, good quality road-building materials are becoming increasingly scarce, especially for the construction of all-weather roads in rural areas where traffic volumes are generally relatively low (less than about 300 vehicles per day). The principal road building material for these roads is gravel which, in many areas, has been heavily utilized and depleted through ongoing construction and maintenance activities; and in extreme circumstances, no suitable gravel is now available for road construction and maintenance purposes. In addition, the haulage of good quality gravel from other areas, over long distances, is prohibitively expensive. Hence, the innovative use of locally available materials, which would have been considered marginal or rejected by traditional specifications for road construction, needs to be investigated for use in the construction of low volume roads (LVRs) in the region.

Fortunately, there is an abundance of naturally occurring sand that could be used in the construction of LVRs to provide an appropriate level of access to many areas. One variety of these sands – Kalahari sands - covers large parts of Botswana, Namibia, eastern Angola, western South Africa, western Zimbabwe (see Figure 1-1) and Zambia as well as the south west of the Democratic Republic of the Congo. Other varieties of windblown sand occur in the coastal regions of KwaZulu-Natal in eastern South Africa, western Namibia, Mozambique and Tanzania.

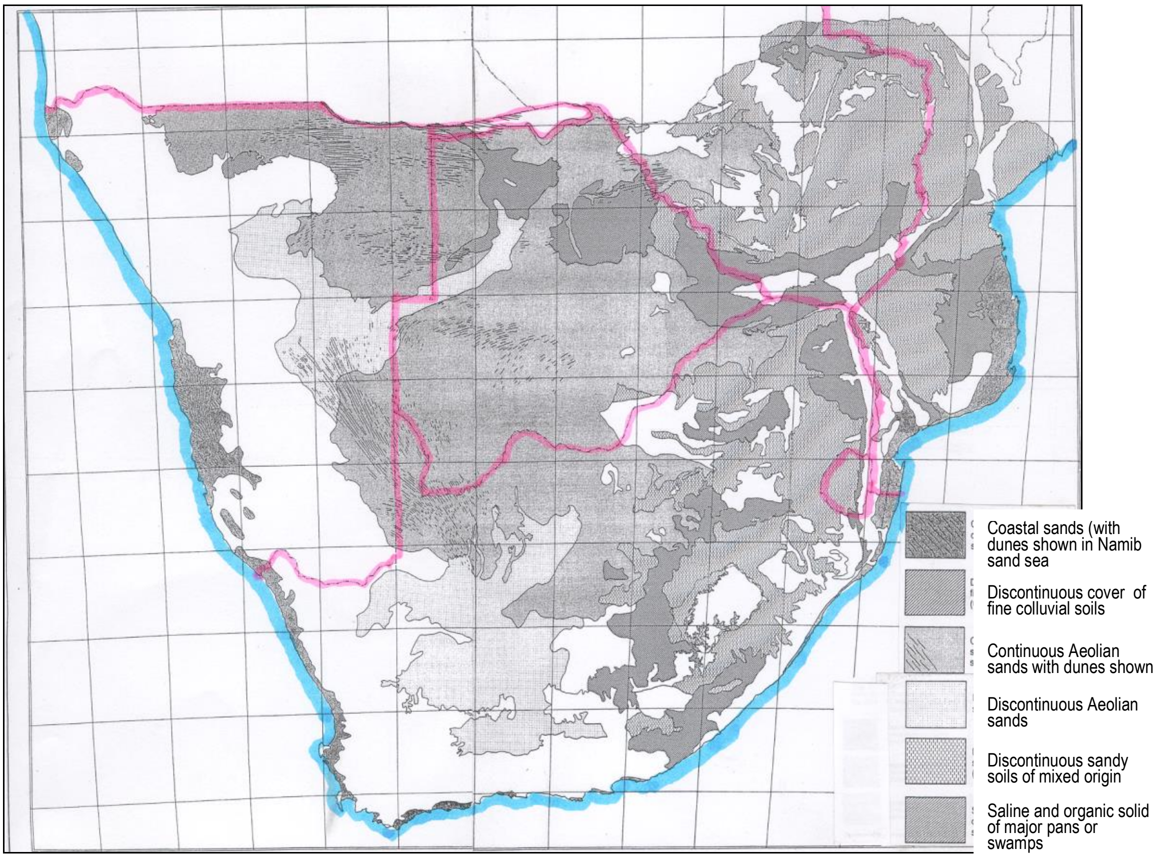


Figure 1-1: Distribution of transported soils in southern Africa (Brink, 1985)

Even though naturally occurring sands seldom satisfy the requirements of traditional specifications for use as a pavement material, especially in their untreated state, they have, nonetheless, been used successfully on a number of road projects at least in Botswana, Namibia, Malawi, South Africa, Zambia and Zimbabwe and elsewhere in the world such as in Western Australia and Brazil.

Technical Guides for the use of untreated Kalahari sand for the construction of low traffic roads have been successfully developed and utilized in Botswana. Also, the use of wind blown and coastal sands has been investigated to a limited extent in Mozambique, South Africa and Zimbabwe. However, the appropriateness and suitability of the technologies used in those countries, along with other appropriate technologies developed and investigated subsequently, need to be extended to all countries within the SADC region.

Successful use of the Kalahari sands in Botswana has led to the development of a Guide document (BRD, 2010) to assist with their wider use as a road construction material. In addition, some information is available on the relatively limited use of windblown and coastal sands in Namibia, Mozambique and South Africa which has been extended and enhanced by further investigations carried out as part of this project.

1.2 Purpose of Guide

The main purpose of this *Guide on the Use of Sand in Road construction in the SADC Region* is to provide practitioners with a good understanding of the properties of one of the most abundant, naturally occurring materials found in the SADC region – sand – so as to facilitate the more wide-spread use of this ubiquitous material in the construction of low volume roads.

The need for the Guide has been identified as a priority by SADC member states and, with the support of AFCAP, the project is being carried out for the Research and Development Standing Committee of the Association of Southern African National Road Authorities (ASANRA). The application of the Guide, especially in areas where traditional road building materials may be scarce, is timely and is likely to be very beneficial to practitioners in the SADC region.

It should be stressed that the most economic use of sands in LVR construction will be achieved when this material is used as a base layer in its neat (i.e. untreated) state. In the past, the use of sand in this manner would generally not have been considered to be feasible largely because of lack of a thorough understanding of the characteristics and properties of this material, coupled with its lack of compliance with conventional specifications. Fortunately, however, research and experience in some SADC countries, notably Botswana and South Africa, as well as in Australia, has demonstrated that, when correctly selected, tested, designed and constructed, sand of appropriate quality can provide a suitable construction material in all layers of a road pavement (see Annex B for such examples).

Unfortunately, no long-term performance of roads constructed with unstabilized sand bases is available from either Namibia or Mozambique. Conclusions related to likely performance can thus only be reached by comparing the properties of the sands from these two countries with the known properties and performance of sands in other countries, namely Botswana and Australia. Thus, the research work carried out previously in these countries has provided the basis by which the sands from Namibia and Mozambique have been evaluated for use in the various layers of a LVSR pavement. The recommendations in the Guide apply to all SADC countries

1.3 Scope of Guide

The focus of the Guide is on the use of neat sand as a pavement material for both paved and unpaved roads. However, the Guide also briefly considers other uses of sand in road construction including:

- proprietary chemicals and polymers to improve performance of the structural layers of both sealed and unsealed roads;
- cement, lime, bitumen and tar stabilization (including foamed bitumen) for use as structural layers in sealed roads;
- blending (mechanical stabilization) with other materials to improve performance of the structural layers of both sealed and unsealed roads;
- sand cushioning and track stabilization procedures aimed at improving unsealed road performance;
- geocells for sand containment, erosion control and slope stability improvement.

The scope of work has been carried out in 2 phases and has been reported upon as follows:

- Phase 1: *The Use of Sand in Road Construction in the SADC Region.* ***May 2013***.
- Phase 2: (a) *The Use of Sand in Road Construction in the SADC Region – Additional Investigations.* ***February 2014.***
  (b) *The Use of Sand in Road Construction in the SADC Region. Final Report,* ***July 2014***.

This final Phase 2 (b) report updates the Phase 1 **May 2013** report with the inclusion of the outcome of the additional investigations carried out under Phase 2 (a). These additional investigations included the testing of samples obtained from existing roads in which sand had been used in the road pavement, either as base or subbase. The analysis of the results of the additional investigations has provided a better understanding of the properties of sands in relation to their suitability for use in road construction, but has still not provided all the answers.

1.4 Structure of Guide

The Guide comprises seven chapters as follows:

Chapter 1 (this chapter): Provides the background to the project including its purpose, scope and structure.

Chapter 2: Presents the general characteristics of including their definition, formation, composition, distribution, geology, origin and classification.

Chapter 3: Outlines the approach to prospecting for sands and discusses the various tests used to evaluate their properties for design purposes.

Chapter 4: Provides information on the locations of the sand sampling sites and the laboratory tests carried out and discusses the results, in terms of the physical, mineralogical, chemical and engineering properties of the sands.

Chapter 5: Discusses the design, specifications and use of sands in road construction.

Chapter 6: Considers various issues related to the construction of roads using sand.

Chapter 7: Summarises the outcome of the project and proposes the way forward.

References: Lists the references used in the development of the Guide.

Annex A: Provides an example of the use of the Phi Scale for the analysis of sands.

Annex B: Presents examples of the use of neat sand as base course of LVSRs.

2. GENERAL CHARACTERISTICS OF SANDS

2.1 Introduction

The sands of the SADC region comprise various types, the major one (in terms of area of distribution) being the Kalahari sands, with smaller surface exposures of more recent aeolian sands in Namibia (Namib Desert), the Western Cape (Cape Flats Sands), various sands along the Cape southern coast (Strandveld and Algoa) and the eastern and northern areas of KwaZulu Natal (Berea Red, Maputoland and other Quaternary sands), which pass into the coastal areas of Mozambique and reach up to Tanzania. In addition, there are some inland sand deposits in Mozambique resulting from the interaction of some of the large east flowing rivers (Limpopo and Zambezi) with the prevailing winds, e.g. at Chibuto.

The term Kalahari (or *Kgalagadi* as used in Botswana) sand is often used in a loose way to describe sands that are found in the Kalahari region of south and south-western Africa (Carney et al, 1994). This general term belies the wide variety of different types of sands that occur in the Kalahari region by virtue of their different modes of formation and agency of transportation. However, useful predictions about the suitability of these materials for use in road construction can often be made once their geology and origins have been identified, their properties classified, their performance characteristics determined and their distribution ascertained.

This section considers the general characteristics of the sands in the southern SADC region, including their definition, distribution, geology and origin, and classification. In order to place these sands in a general world-wide context, reference is also made to the characteristics of similar sand types found outside of this region.

2.2 Definition

Sand is generally defined as a granular material resulting from the weathering of especially siliceous rocks and composed mostly of silica i.e. in the form of quartz, (one of the minerals least susceptible to weathering) with sizes falling in the range 2.0 to 0.06 mm. (Anon, 1974). It should be noted, however, that the commonly used Unified Soil Classification System classifies sand as material between 0.075 and 4.75 mm. For convenience, the Southern African and American roads terminology usually places the minimum size at 0.075 mm (the No. 200 mesh sieve). However, very few sands contain material solely within this range, either having some fines (<0.075 mm) or a small coarse component (> 2 mm).

The above *generic* description of sand is just that – a very broad, non-specific description that does not adequately describe either the differing mineralogy or physical and mechanical properties of this material both of which influence its engineering properties. The *Kalahari* sands are a particular type of sand found in the Kalahari region of southern and south-western Africa and by virtue of their unique formation they differ genetically from those in many other parts of the region and the world although the wind-blown (aeolian) origin of parts of them make them comparable with the widespread sand-clays of the Australian interior.

Many of the sands are not single sized and contain varying amounts of finer material. This can be from almost nothing to significant amounts and has a major influence on the engineering properties of the sand to the extent that in Australia, many of the useful sands are actually classified as sand-clays (this definition would not comply with the Unified Soil Classification System nomenclature (ASTM D 2487) for a clayey sand.

For the purposes of this Guide, the discussion in this report is focused on the typical sands found in the SADC region, including the Kalahari sands, as well as those in the coastal areas of the region.

2.3 Formation

It is generally accepted that there are three conditions necessary for the formation of large bodies of sands (Bagnold, 1941). These are:

- sufficient sources of materials;
- geological forces (wind, water, etc.) for continual transportation of sand material from one place to another and topography conducive for continued deposition;
- favourable climatic conditions.

Once disaggregated from the original source rock, the resulting material is then eroded and transported by either wind, water or ice, often ending up as the deposits of rivers or lakes, as sand dunes or desserts, or ultimately as sediment in the sea.

Figure 2-1: Formation of sand by erosion and weathering of rocks

After deposition, the sands are, over time, reworked by the movement and infiltration of surface water and environmental forces. These tend to alter their mineralogical composition through weathering, leaching and enrichment. In addition, the physical properties of the sand mass and particles are changed through weathering, cementation, consolidation, particle leaching and/or disintegration.

2.4 Composition

The composition of sand varies from place to place and is largely dependent on the nature of the source material as well as the weathering process (physical, chemical or both) acting on rock masses which determine the mineralogical composition of the unconsolidated sands, silts and clays produced. Transportation of these particles by water and wind will further modify the material in terms of its particulate size, shape and mineralogical sorting, physical distribution and the degree and type of secondary consolidation – all of which influence the geotechnical behaviour of sands in road construction.

For example, the sands found in desert areas, such as the Kalahari Desert in Botswana, tend to be aeolian (windblown) deposits which have undergone deposition by wind, while those found on beaches, such as in Cape Town in South Africa, are both commonly dominated by silica (silicon dioxide SiO_2), in the form of quartz (one of the minerals least susceptible to weathering) which has been derived from weathering and erosion of the mountain ranges nearby.

Many of the sands investigated in this project are clearly of mixed origin, containing particles with a wide range of sizes, shapes and angularity.

2.5 Distribution

The arid and semi-arid areas of the world, which together occupy more than a third of the earth's land surface, have a number of characteristics in common, one of which is the occurrence of sands and sandy materials over large areas of their surface. These sands occur in all seven continents of the world and, collectively, contain probably the largest source of a naturally occurring material for possible use in road construction.

As shown in Figure 2-2, there are vast inland deposits of Kalahari sands that cover large areas of southern Africa, south of the equator. It should be noted that this map only indicates thick inland sands and does not include areas of sand that may be between 1 and possibly 5 m thick, that extend well eastwards of the zone indicated. This material forms what is believed to be the largest continuous stretch of sand in the world. Today, they begin north of the Orange River in South Africa, embrace the western two-thirds of Botswana, more than a third of Namibia, and stretch north through eastern Angola, western Zimbabwe and Zambia to the Democratic Republic of the Congo. Figure 2-2 also shows the extensive deposits of coastal aeolian, littoral and fluvial sands which occur sporadically around the coast of Southern African.

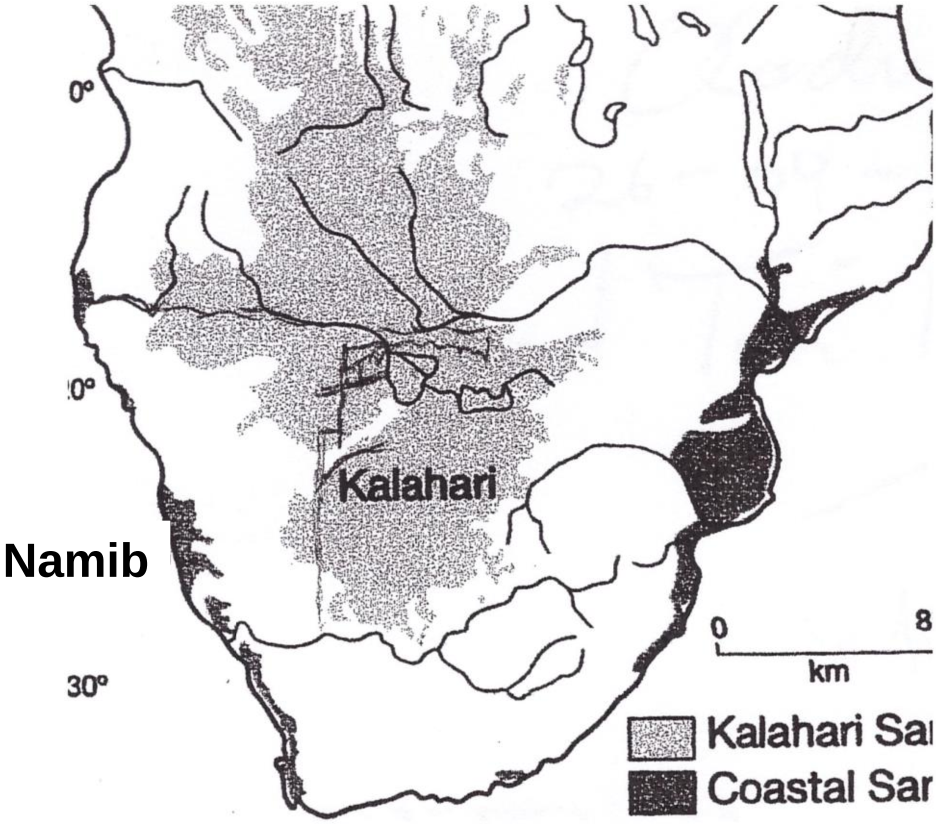


Figure 2-2: Distribution of sands in southern Africa (Cooke, 1964)

The sands in Botswana are part of a continuous mass of sand extending from the Northern Cape in South Africa to north of the equator. In South Africa the equivalent sands are classified as the Gordonia Sand Formation within the Kalahari Group (Brink, 1985; Partridge and Maud, 1987). Kalahari Group deposits have been intensively studied and found to thicken from their southern limits towards Botswana, where they occur in deep palaeovalleys, up to 300 m thick, just south of the Botswana border. The sands of the Gordonia Formation, however, are generally not more than 30 m thick.

More specifically, Baillieul (1975) identified four major sand areas in Botswana, each having distinct types of Kalahari sand depending on their mode of formation. These are shown in Figure 2-3.

Figure 2-3: Map of Botswana showing the four major sand areas (after Baillieul, 1975)

Description of Sand Types

Area I: Fine grained (mean diameter 0.17 to 0.23 mm), well to moderately well sorted, quartz sand with the particles coated with a skin of red iron oxide. Where the particles have been affected by water, (fluctuating water table or adjacent to the Okavango Delta) this coating has been removed and the sands are grey in colour. The sands have been formed in an aeolian environment.

Area II: The sands are slightly finer (mean diameter 0.14 to 0.2 mm) than those from Area I with similar sorting. They comprise particles of two distinct origins - well-rounded polished quartz grains of aeolian origin and finer (0.125 mm), angular feldspathic particles derived from the underlying Ghanzi sandstones.

Area III: These sands are similar to those of Area II but lack the feldspathic component. Their composition reflects that of the underlying Karoo sandstones. They still, however, show evidence of an aeolian origin.

Area IV: The sands in Area IV are thin and are directly related to the underlying bedrock. They are coarser than the other three types and have been derived predominantly by fluvial action and bioturbation.

2.6 Geology and Origin

The geology and origin of the wide range of sand types that occur worldwide varies considerably. However, most of them have been produced by some form of rock weathering in which the constituents have subsequently been transported by wind or water. The following examples illustrate the diversity of geology and origin of sands worldwide:

- *Australia:* the sand-clays in South Australia (Wylde, 1979) are derived from stranded beach ridges and consist of rounded to sub-angular quartz grains, cemented together and containing some clay and iron staining (Sandman et al, 1974). Other Australian sands have been described as river channel to flood plain, deltaic or wind-blown (Wylde, 1982).
- *Brazil:* the Brazilian sands tend to be derived from the weathering and transport of sandstones and consist of sand-size quartz with kaolinite and ferruginous oxides (Aranovich and Heyn, 1984).
- *Fiji:* the coral sands of Fiji are derived from the coastal erosion of limestones and consist predominantly of calcium or magnesium carbonate.
- *Southern Africa:* The original Kalahari sands were derived from the erosion of underlying rock and subsequent transport and redistribution. This was carried out by rivers into lakes and by wind. The surficial sands observed today were deposited primarily by wind.

- *South-eastern South Africa:* The Berea Red Sands have a distinct origin, resulting from the weathering of Mio-Pliocene aeolian calcareous deposits. The feldspathic component of this material produced a significant clay content (3 to 40%) (Maud and Botha, 2000).

- *Namibia: The* Kalahari Sequence is contiguous with and mostly similar to the Kalahari sands of Botswana and South Africa and consists primarily of vegetated static dunes. The Sossus Sand formation of the Namib Sand Sea (desert) is a highly active more recent sand formation consisting of a wide range of dune types (Lancaster, 1986). These sands are predominantly quartz with up to 10% feldspar. Coastal sands also occur extensively in the country. A comprehensive description of the Geology of Namibia has been published which gives more information on the sands in the country (Miller, 2008).

- *Mozambique:* The sands are mostly the extensive coastal dunes, inland dunes in the southern provinces, and the sandy terrigenous deposits of the Mocambique and Rovuma sedimentary basins and Karoo depressions. A number of red and brown sands are listed as Berea-type red sand and calcareous (i.e. carbonate-containing) and non-calcareous sands. A list of the best known building sand deposits is given in the *Geology and Mineral Resources of Mozambique. Republica de Mocambique Minesterio dos Recursos Minerais e Energia, Maputo*, (Lachelt, 2004).

2.7 Classification

Various systems have been developed to classify soils and provide only a *general* guide to their engineering properties of which particle size distribution and plasticity are the principal ones. This enables engineers to understand the general, rather than the particular, properties of the soils of other countries or regions.

2.7.1 AASHTO Classification System

An example of such a system that is commonly used in the region is the AASHO (now AASHTO M-145) system in which soils having approximately the same general load bearing capacity are grouped together to form seven basic groups which are designated from A-1 to A-7 (Table 2-1). In general, the best soils for subgrades are classified A-1 and the poorest A-7. Thus, it may be assumed generally that structural thickness requirements of the pavement progressively increase as the soil classification group increases from A-1 to A-7.

Table 2-1: AASHTO M-145 Classification System for Soils and Soil-Aggregate Mixtures (M145)

General Classification	Granular Materials (35% or less passing No. 200)							Silt-Clay Materials (More than 35% passing No. 200)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5; A-7-6
Sieve Analysis: Percent passing: No. 10	50 Max.										
No 40	30 Max.	50 Max.	51 Min.								
No. 200	15 Max.	25 Max.	10 Max.	35 Max.	35 Max.	35 Max.	35 Max.	36 Min.	36 Min.	36 Min.	36 Min.
Characteristics of fraction passing No. 40: Liquid Limit				40 Max.	41 Min.	40 Max.	41 Min.	40 Max.	41 Min.	40 Max.	41 Min.
Plasticity Index	6 Max.		N.P.	10 Max.	10 Max.	11 Min.	11 Min.	10 Max.	10 Max.	11 Min.	11 Min.
Group Index	0		0	0		4 Max.		8 Max.	12 Max.	16 Max.	20 Max.
Usual Types of Significant Constituent Materials	Stone Fragments Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General Rating as Subgrade	Excellent to Good							Fair to Poor			

The Kalahari and coastal sands typically classify as A-3 or A-2-4 with a few A-1-b materials in both Mozambique and Namibia (about 4 in each country) and no A-2-5 materials with the AASHTO classification and SM in the Unified Soil Classification System (ASTM D 2487-06). By way of comparison, the typical sands in Australia classify similarly (Jewell, 1970; Pederson, 1978; Paige-Green, 1983) those in Brazil classifying predominantly as A-4 but also as A-2-4, although plasticity indices are generally higher (Utiyama et al, 1977). Sands in parts of the Middle East are similar to the Kalahari sands, classifying primarily as A-3 and A-2-4 with some A-4 materials (Bindra, 1987). The French have their own 6-group classification system for soils using grading, plasticity and sand equivalent (Chauvin, undated) – sands are classified as A (1-4) or B (1-6).

The Berea Red Sands of South Africa which grade into southern Mozambique generally classify as A-2-4 but some of them have been classified as A-6 (Bergh et al, 2008).

2.7.2 British Standard Classification System

This system classifies sand on the basis of three grain sizes as follows:

- Coarse: 0.6 – 2.0 mm
- Medium: 0.2 – 0.6 mm
- Fine: 0.06 – 0.2 mm

The AASHTO, Unified and British classification systems for engineering purposes differentiate sand type on the basis of differences between the grain diameters. However, such systems may not be the most useful or convenient scale to use when geological processes which affect the characteristics of sands that are to be examined (Bagnold, 1941). In such cases, it has been found far better to use a log-scale which exhibits the ratios between the grain diameters (McManus, 1982).

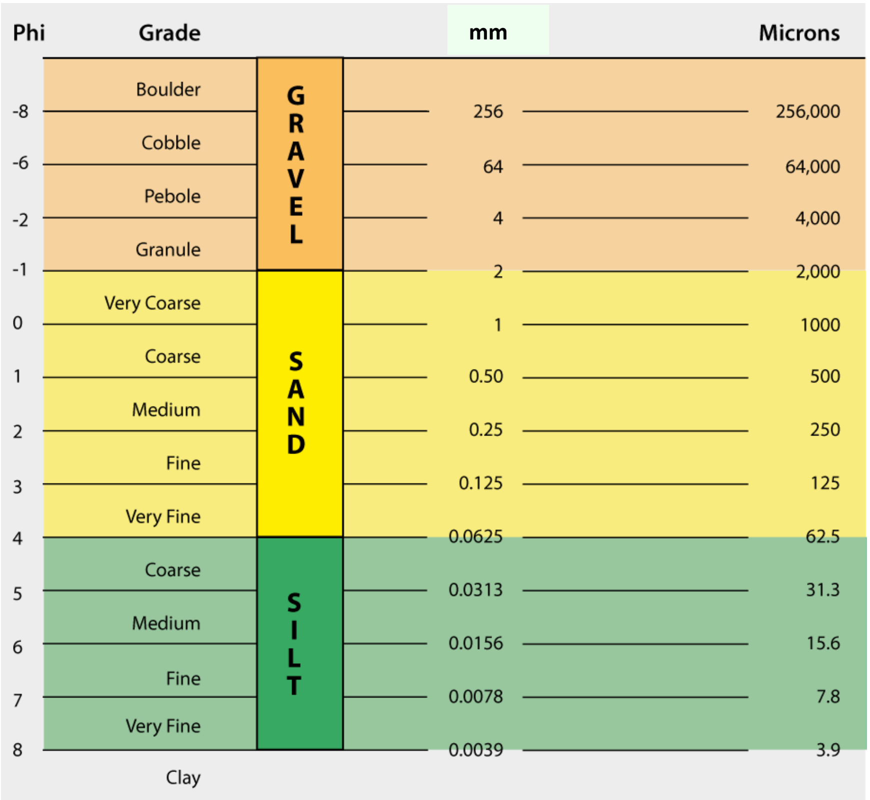
2.7.3 Wentworth Particle Size Classification System (Wentworth, 1922)

The Wentworth scale for measuring grain size uses a geometric interval of ½ to define the limits of each size fraction in which sand is divided into five sub-categories based on size as follows:

- Very coarse sand: 1 – 2 mm
- Coarse sand: 0.5 – 1 mm
- Medium sand: 0.25 – 0.5 mm
- Fine sand: 0.125 – 0.25 mm
- Very fine sand: 0.0625 – 0.125 mm

The above sizes are based on the **"Φ sediment size scale"**, a size scale which is commonly used in sedimentology (Krumbein and Pettijohn,1938; Folk, 1954) and in which Φ (phi) is the negative logarithm to the base 2 of the grain diameter. In this scale the use of Φ, an arbitrary and artificially derived grain size unit, allows the particle size distribution (PSD) to be represented as a single point on a plot of mean particle size against standard deviation in Φ units and thus allows other properties of the sands to be related to that plot – an approach that has been used for improved selection procedures of sand in road construction (Metcalf and Wylde, 1984).

For comparative purposes, Figure 2-4 shows the relationship between grain size data expressed in Φ units, millimetres and microns (micro meters) (Anderson, undated).



The Phi scale
In previous research on sediments, grain size data is given in phi (Φ) intervals rather than in microns or mm as statistical and graphic presentations are much simpler when phi diameters are used. Phi is defined as:

$$\Phi = -\log_2 d$$

where d = particle size in mm.

(A particle size of 0.5 mm = φ of 1 and a particle size of 0.125 mm = φ value of 3).

Figure 2-4: Alternative measures of grain size (Φ, mm and microns)

In general, the major role of international classification systems, such as the AASHTO system, is to enable engineers to understand the general, rather than particular, properties of the soils of other countries and to be able to communicate on a common basis. Thus, a regional system, based on the properties of regional of sands, will generally be preferable and should be used in conjunction with broader classifications. This is one of the major aims of the Guide – *to customise a broad, international classification system for sands in general, to one that reflects the performance characteristics of sands that occur in the SADC region.*

As would be apparent from the various classification systems, there are differences in the particle size range used to define sands with the minimum size varying from 0.06 mm (BS classification) to 0.075 mm (AASHTO classification system). However, from a practical point of view, the difference in particle size distribution is very small, typically < 1%). Hence, in this manual the different sizes used to describe sands are used interchangeably.

3. PROSPECTING FOR AND TESTING OF SANDS

3.1 Introduction

Because of their extensive occurrence in southern and western Africa, sands are of potentially major engineering importance for use in the construction of LVRs. However, in an area that is vast, and has such a variable climate and vegetation as the southern African region, the major challenge is to find a reliable procedure for locating those sands that are likely to be suitable for incorporation in a LVR pavement.

3.2 Prospecting

3.2.1 General procedure

In principle, the procedure for prospecting for sand is not dissimilar to prospecting for any other construction materials and is summarised in Figure 3-1.

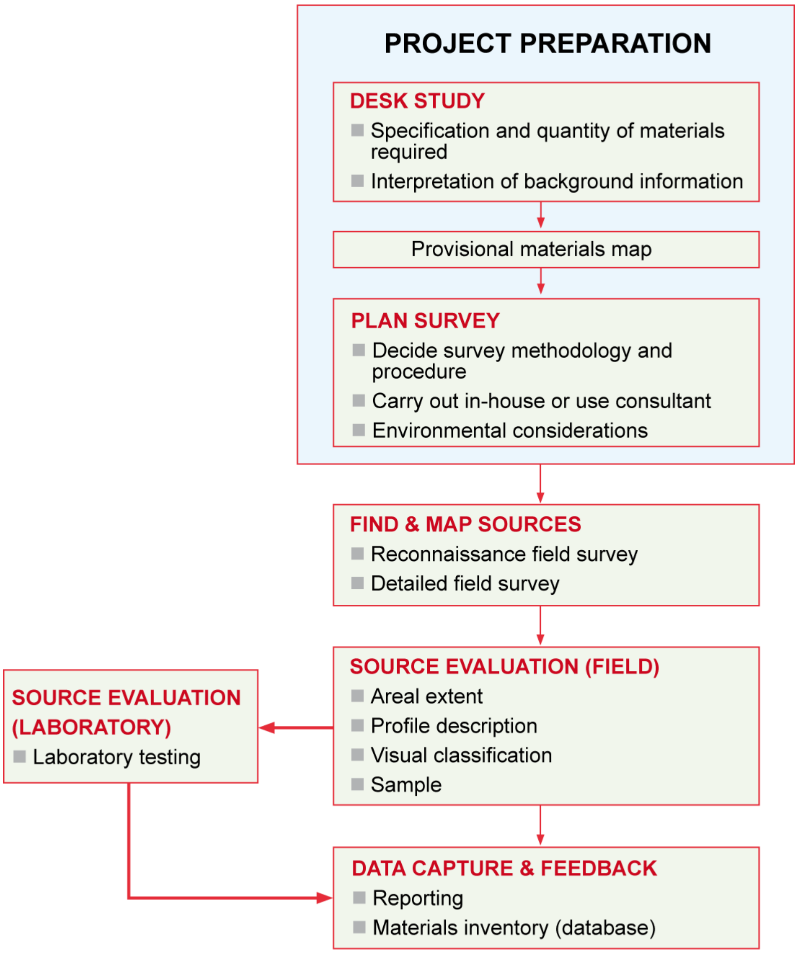


Figure 3-1: Flow diagram stages for sand prospecting

As regards mapping, the 1 in 6 million scale maps of southern Africa in Brink (1985) showing the distribution of transported soils according to origin (Figure 1-1) and the similar scale map showing the pedological classes of soils (Figure 3-2) provide some information on the nature of the sands that may, with some refinement, be used for the location of suitable sands for construction.

Figure 3-2: Pedological classes of soils in southern Africa (Brink, 1985)
(Electronic version of map available from https:/www.afcap.org)

Specific national geological and especially soil maps of particular countries also contain useful information. For example, the Kalahari sands are mostly classified as arenosols. Although there are many subdivisions of arenosol (and mapping units), each has a specific description of the colour, particle size distribution (classification) and nature. The relevant soil profile description and standard soil analysis results provide comprehensive information on various properties of the materials that can potentially be usefully applied to material location.

The following points should, however, be noted when prospecting for sand.

- In many cases sands overlie gravel sources, particularly calcrete, in interdunal hollows and depressions, often known in some areas as streets. Thus, botanical indicators for calcrete are often equally applicable to locating sands. One such botanical indicator for deep red sandy soils, *Acacia haematoxylon* (Photo. 3-1), is endemic to the more arid/semi-arid regions of Southern Africa. Another common tree in southern Africa, the silver cluster leaf (*terminalia sericea*), is only found on sandy, well-drained soils. In the more sub-tropical to tropical areas of Southern Africa, the vegetation is typically far more luxuriant than in the more arid/semi-arid areas, and the underlying materials tend to be masked by thicker organic soil horizons and overlying vegetation – this makes aerial differentiation of surface features more difficult to carry out.

Photo 3-1: Botanical indicator *Acacia haematoxylon*

- Since sands usually occur in the top 2 metres of the subsurface, it is highly beneficial to carry out centreline survey of the road alignment first before material prospecting. The trial pit logs of the centreline should then be studied and areas with potentially good sands identified. These areas should then be marked in the base map for follow up during the material prospecting. The advantage is that the sand borrow pits are then located as near the road as practically and environmentally acceptable and the data from the centreline will provide additional information on the sand.

- It should be noted that it is not necessarily only the vegetation type that indicates the possible sources of materials, but changes in the vegetation type and density. Specific attention should be directed towards assessing the causes of these changes.

- One of the basic field indicators of potentially good quality sands is the firmness of the sand which can be judged by the ease with which a vehicle can traverse the sand when it is dry, without the need to engage 4 wheel drive. Such sands tend to be relatively plastic and exhibit some cohesion, even in the dry season, which facilitates their trafficability.

- The calcrete probe also provides a very easy method of determining the thickness (or minimum thickness) of a sand layer (Netterberg, 1971).

- The effects of leaching in wet climates should be taken into account. Water percolating through the upper few metres of the layer tends to leach out soluble and fine material resulting in an upper soil profile distinctly different to the lower one. As the lower one (particularly in sands) may be more useful for road construction, this needs to be considered during the material location process.

3.2.2 Initial Screening

Field sampling and laboratory testing of potential borrow materials is a costly operation. As the successful use of sand in road pavements appears to be closely related to its particle size distribution based on the sedimentological Φ scale (ref. Section 2.7.3) in which sands are characterised by their mean particle size (in Φ units) representing the fineness of the material, and the standard deviation of the grading (in Φ units) representing the degree of sorting of the sand, these two parameters should be used as a preliminary screening test to identify potentially useful sands.

The use of the relatively simple sedimentological Φ scale, which is illustrated in Figure 3-3 (Wylde, 1979), can be beneficial as a preliminary screening test before carrying out large scale sampling and laboratory testing. Sands suitable for base course would typically plot in Zone B whilst those that that plot in Zones A, C or D would not be expected to perform satisfactorily due to insufficient fines to bind the material (Zone A), or being difficulty to handle during construction (Zones C and D).

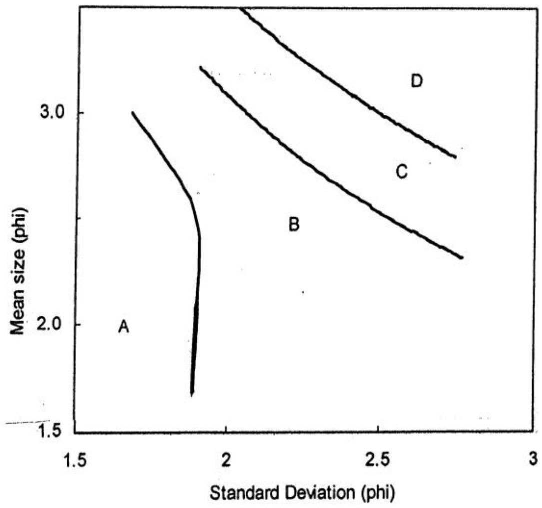


Figure 3-3: Particle size distribution (Phi units) related to performance zones

Figures 3-4, 3-5 and 3-6, 3-7 show the distribution of the mean (Phi units) and standard deviation (Phi units) of the sands from Mozambique and Namibia respectively using a kriging technique. The B indicators on all the maps fall within the B zone of the Wylde Chart

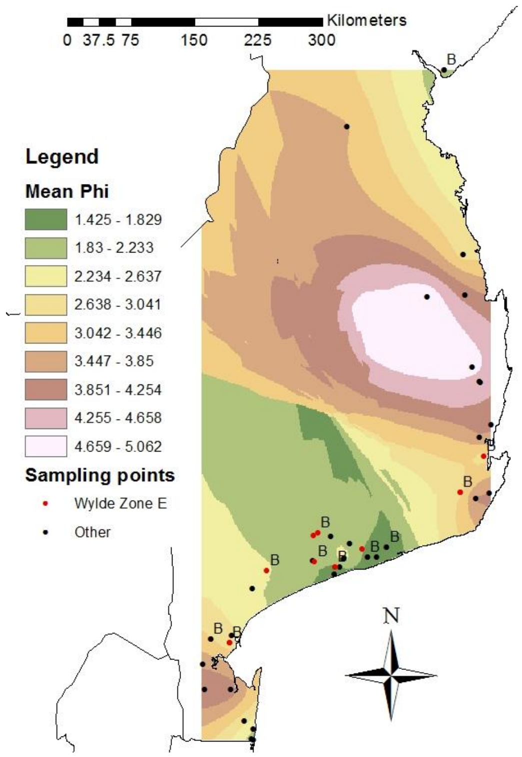


Figure 3-4: Distribution of mean (Phi units) of sands in Mozambique

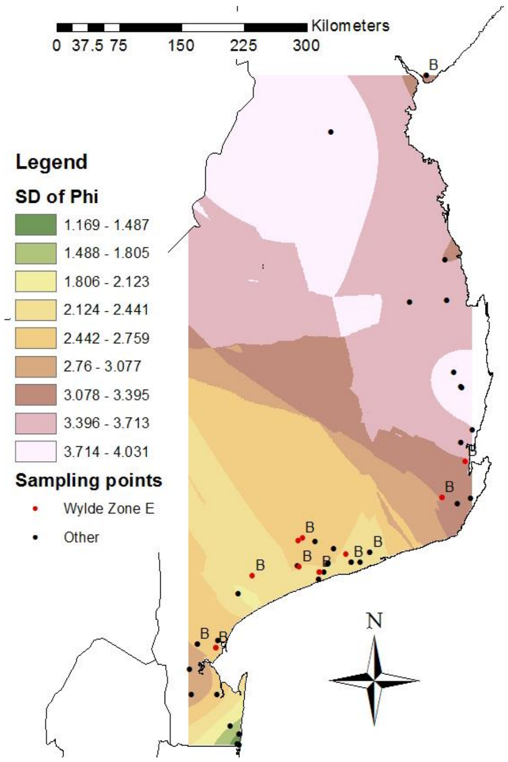
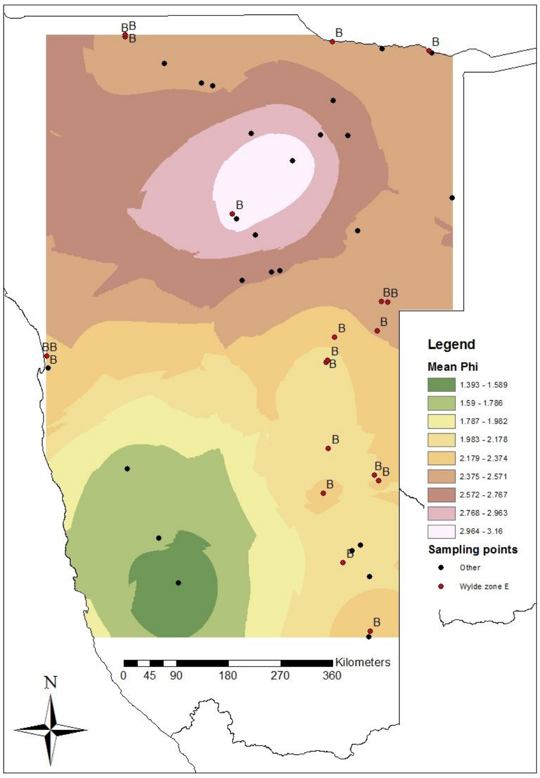
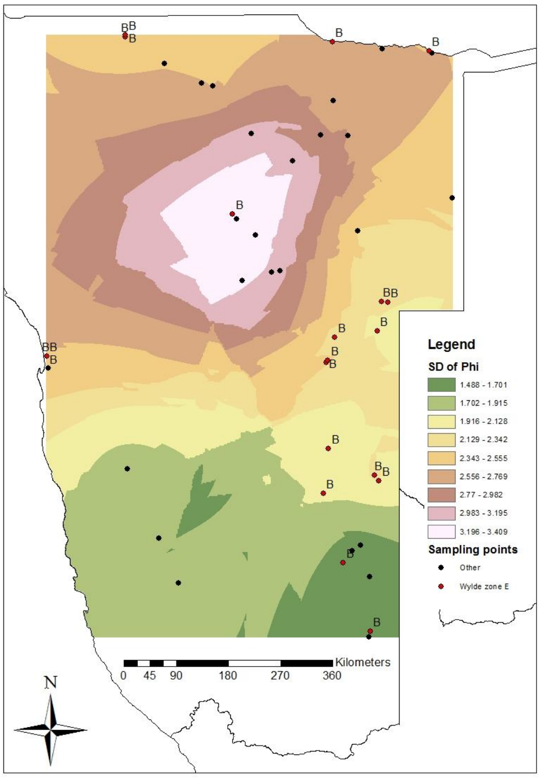
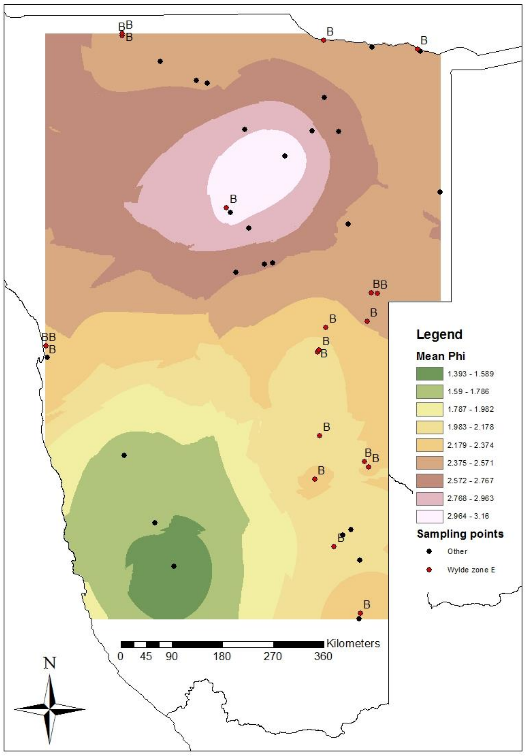


Figure 3-5: Distribution of SD (Phi units) of sands in Mozambique







F… ts)

Figure 3-7: Distribution of SD (Phi units) of sands in Namibia

F… pears that the southern Limpopo river area (and the O… s the west) show a strong trend of coarser and better sc…

F… that the coarser materials occur in the Namib desert a… e east and significantly finer towards the north. The m… s the east and poorly sorted as one moves north.

T… licators during material location exercises, but need a… terial selection for specific projects.

3…

T… ollow standard techniques, although certain inherent c… al consideration as discussed below.

3.…1 …

It should be noted that there are significant differences between certain test methods specified by the BS standards and the TMH methods, particularly with respect to plasticity index and bar linear shrinkage. Moreover, some countries (e.g. Botswana, Namibia and South Africa) use TMH1-type test methods whilst other countries in the SADC region (e.g. Tanzania, Malawi) use BS methods. It is therefore important not to mix test methods because the differences in some test procedures will produce different results. The recently revised South African SANS test methods have been used, where available, to replace the TMH1 test methods, in the testing of the sand samples.

3.3.2 Grading

The grading of sands should always be carried out using a wet preparation method such as SANS 3001: GR1 (equivalent to TMH1 Methods A1(a) and A5 (NITRR, 1986) in order to ensure that all cementing bridges are broken and that the small quantities of plastic fines are released for determination of the Atterberg Limits. The standard range of sieves employed and recommended in TMH1 includes only the 0.425 and 0.075 mm screens that will typically retain any sand material. It is recommended that for these materials, the 0.300 (or (0.250) and 0.150 screens are also included and where the fraction passing 0.075 mm exceeds about 12 per cent, a hydrometer or pipette analysis is also carried out using ASTM, BS or SANS GR3:2012). This will give a better indication of the distribution and sorting of the material.

Studies in Australia (Wylde, 1982) indicate that, despite the relatively fine nature of sands, the process of excavation, mixing and compaction physically damages the sand constituents, particularly releasing the fines. It is thus desirable that grading analyses for design are carried out on material that has been subjected as far as possible to a process or processes that simulate these actions. This, of course, applies to materials complying with any specification, e.g. COLTO, 1998.

3.3.3 Atterberg Limits and Linear Shrinkage (Soil Constants)

Routine testing of the Atterberg Limits following TMH1 Methods A2 and A3 (NITRR, 1986) (SANS: 3001: GR10, GR11 and GR12) usually indicates that the materials are non- or slightly plastic. For sands, it is imperative that the Atterberg Limits are determined on the fraction finer than 0.075 mm, this almost invariably giving a Plasticity Index (PI) in the range 5 to 27 in the samples tested, with more than 9 samples giving a PI in excess of 20%. This property can then be expressed as the Fineness Index (FI_{075}) (Mainwaring, 1968), which is the product of the PI on 0.075 mm sieve and the percentage passing the 0.075 mm sieve. This property is a useful indicator of the compactability of sandy materials as discussed in a following section.

From experience of testing the Kalahari sands in Botswana, it is noteworthy that the use of the Cassagrande bowl type Liquid Limit Device (as adopted in TMH1) proved to be unsuitable for determining the Liquid limit of the relatively less plastic sands. This was because there was a tendency for the groove that was cut in the paste either not to close or to slump on the first blow during the operation of the device. For this reason, it was necessary to resort to the use of the BS 1377 Cone Penetrometer for determining the Liquid Limit of the sand samples. It is important to note that both the BS cone and cup Liquid Limit devices yield a Liquid Limit and therefore also a PI on average 4 units higher on the P425 than the TMH1 ASTM type of LL device (Sampson and Netterberg, 1984). The bar linear shrinkage should always be measured with the Atterberg Limits which should also be done on the minus 75 μm fraction. A method of preparing the 0.075mm fines for the determination of soil constants is given in SANS 3001-GR1.

3.3.4 Compaction characteristics

Laboratory investigations carried out by many practitioners reveal that Kalahari sands do not always exhibit the typical parabolic dry density/moisture content curve exhibited by other soils (Figure 3-8). Instead, the compaction characteristics of some of these sands are such that they can be compacted over a wide range of moisture contents, without a significant change in density.

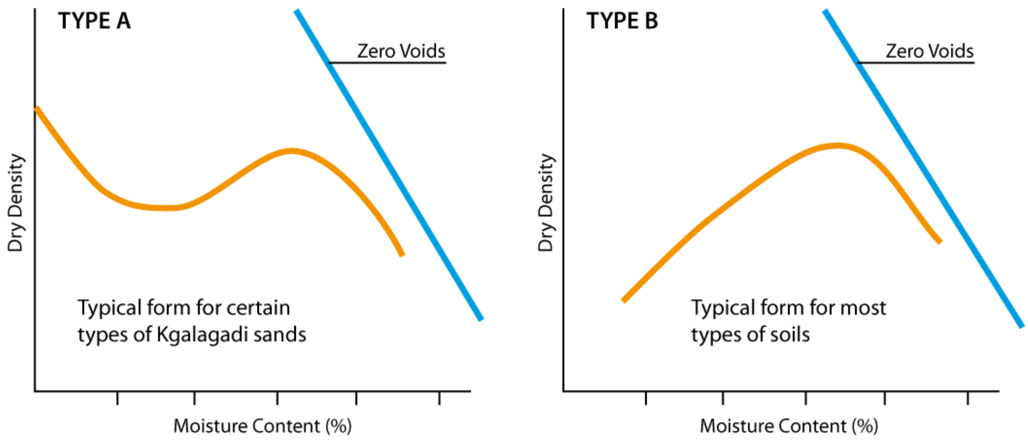


Figure 3-8: Typical forms of compaction curve for sands.

For some Kalahari sands, the density obtained in a dry condition can be markedly greater than that obtained at any finite moisture content. However, as much as there may be a temptation to employ "dry compaction" in practice, other pertinent factors are noteworthy. For example, high air voids and high soil suction values both have a potential for causing post-construction problems (albeit reduced ones in semi-arid climates) such as greater susceptibility to loss of strength, should the degree of saturation increase in service, resulting in deformation of the pavement structure. Materials compacted in their dry state also do not have an inherent strength developed by soil suction as they dry back from the optimum moisture content. It is often necessary to place a thin rubber disk on the sand during laboratory compaction in order to facilitate densification of the upper layer. During the compaction of the red sands investigated in this work, rubber disks were not necessary.The laboratory compaction of fine cohesionless materials using dynamic compaction methods (e.g. TMH1 method A7 (NITRR, 1986) (SANS 3001: GR30) can be difficult, producing unreliable results due to shearing of the material during compaction. The use of the BS 1377 vibrating hammer method (Test 14) (British Standards Institution, 1990) was thus recommended. As indicated from research work carried out in Botswana (Guide No. 11 – The Use of Kgalagadi Sands in Road Construction) and illustrated in Figures 3-8 and 3-9, the benefits of using vibratory over dynamic compaction for a given sand type are: (1) a reduction in air voids, (2) an increase in density and (3) an increase in strength (CBR).

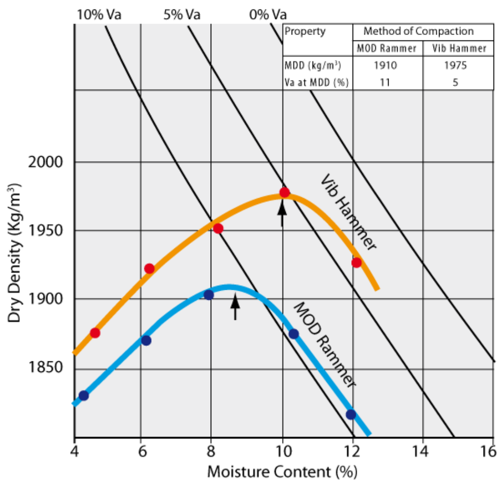


Figure 3-9: Density/voids relationship

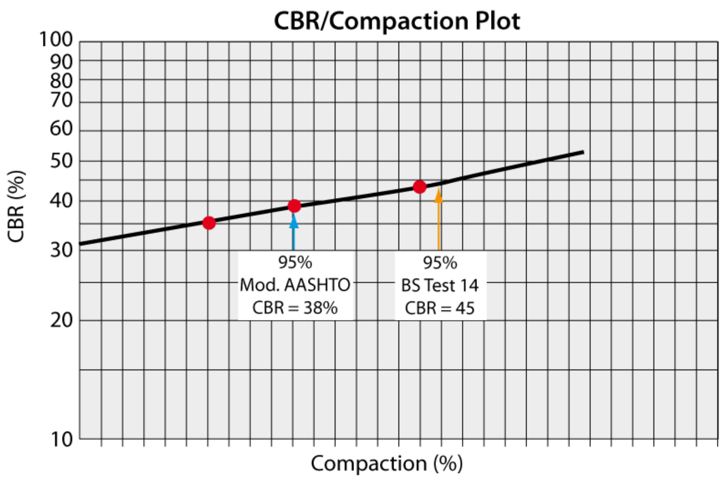


Figure 3-10: CBR/compaction relationship

In practice, therefore, the full strength potential of sand can be better exploited by specifying that the stipulated maximum dry density be assessed by the laboratory vibrating test method rather than the dynamic test method.

Sands with little or no cohesion can be compacted using the traditional TMH1 dynamic compaction technique, but they usually require a rubber mat on the surface to avoid excessive displacement of the sand.

3.3.5 Strength

The California Bearing Ratio (CBR) is traditionally used for the specification and control of road construction materials. This test normally uses a dynamic compaction technique. Where the density of the material is specified to be tested using a vibrating hammer, the CBR should be carried out on specimens prepared in the same manner and to the same density. However, the use of the Dynamic Cone Penetrometer (DCP) for measuring a soils resistance to penetration (DN value), both in the field and the laboratory, is considered to be a simpler and more appropriate approach than the laboratory CBR based methods. The DN values provide a critical input to the DCP pavement design method which is described in Chapter 7.

3.3.6 Salinity

Salinity of the sands can be tested using the electrical conductivity as described in Guide 6 (Botswana Roads Department, 2001). It is suggested that either the quick method provided in Guide 6 or the TMH1 (Method A21) (NITRR, 1986)is used.

3.3.7 Durability

The durability of sand as a construction material is seldom a problem. Materials with high non-quartz contents may include soft feldspar or basalt grains. The effect of these can be assessed by carrying out an extended sand equivalent test. In this test, the standard method is followed and the test is repeated with 10 minutes of agitation. A significant increase in the sand equivalent value for the latter test indicates the potential for softer particles to degrade in service.

3.3.8 Collapse settlement

Where it is suspected that the material may have a collapsible structure, for example, when the material excavated from an inspection pit is insufficient to re-fill the pit, oedometer testing should be carried out to assess the collapse potential. Although the double oedometer test is regarded as the classical means of quantifying collapse, a single oedometer test known as the collapse potential test is simpler and quicker. In this test, a sample is loaded to 200 kN and then saturated (Brink, 1985). The reduction in voids ratio is mathematically expressed as the collapse potential and this related to the thickness of potentially collapsible material can quantify the magnitude of collapse that could occur.

3.3.9 Mineralogy and chemical properties

Specialised techniques such as X-ray diffraction (XRD) and X-ray fluorescence (XRF) can be used to determine the mineralogical and chemical composition of sands respectively. However, these techniques are relatively expensive to use and only in exceptional cases will provide any additional information that will materially affect the use of the sand.

4. PROPERTIES OF SANDS

4.1 Introduction

As may be inferred from the work done by Baillieul (1975) and others (Carney, et al, 1994; Brink, 1985) the sands of the Kalahari region occur either as:

- *aeolian* (windblown) deposits which have undergone redistribution by wind, or as
- *soils of mixed origin* which have resulted through the operation of a number of processes including bioturbation, redistribution by wind during arid periods, and pedogenesis.

Because of their different modes of formation, the sands in the SADC region would be expected to display very different engineering properties which affect their use in road construction. In order to investigate these properties, an extensive sampling and laboratory testing programme was carried out in Namibia and Mozambique as described below.

4.2 Locations of Sands Investigated

4.2.1 General approach

Phase 1: The sand sampling sites were identified on the basis of expected differences in the sand properties and with the objective of achieving as wide a geographical coverage as possible. The sites were selected based on consideration of the following:

- satellite imagery,
- soils and geology maps,
- local information from the roads agencies
- discussions with other individuals familiar with the local terrain

The maps showing the distribution of transported soils in southern Africa provided in Brink (Brink, 1985) proved particularly useful to differentiate between the sand types in Namibia and Mozambique.

Once the different areas had been delimited on the maps, Google Earth was used to identify approximately fifty sites each in Namibia and Mozambique where 10 kg samples were to be obtained for laboratory testing. In order to ensure that the samples were representative of those likely to be used in practice, existing (old) borrow pits that could be located on Google Earth were used as far as possible. However, during the sample collection, minor changes were made as necessary to locate suitable materials. Some of the expected borrow pits were no longer visible or accessible. In many cases, samples were taken at two depths in order to obtain both leached and unleached materials.

Phase 2: Investigations were also undertaken to locate existing roads in Botswana, Malawi and Mozambique where sand had been used as neat base course. The occurrence of such roads has provided an excellent opportunity to evaluate their performance and to extract samples for appropriate laboratory testing to compare the properties of the sands used in their construction with the sand in Botswana (Serowe-Orapa experimental section) that had performed successfully as neat base course over an extended period of time.

4.2.2 Site locations - Mozambique

Phase 1: The majority of sands in Mozambique occur in the southern half of the country, although the coastal sands re-appear in the extreme north eastern areas running into Tanzania. Sampling was thus mainly centred in the coastal areas of the south and eastern regions and the interior fluvio-aeolian sands.

The sites chosen for sampling are pin-pointed on the Google Earth map and are presented in Figure 4-1. The co-ordinates of the sampling locations are presented in the Sampling, Laboratory Testing and Analysis of Results report (InfraAfrica et al, 2012). These coordinates were not meant to precisely define the sand sample location but, rather, to provide guidance on the approximate location of the sample which, in the field, depended on the prevailing ground conditions and other related considerations.

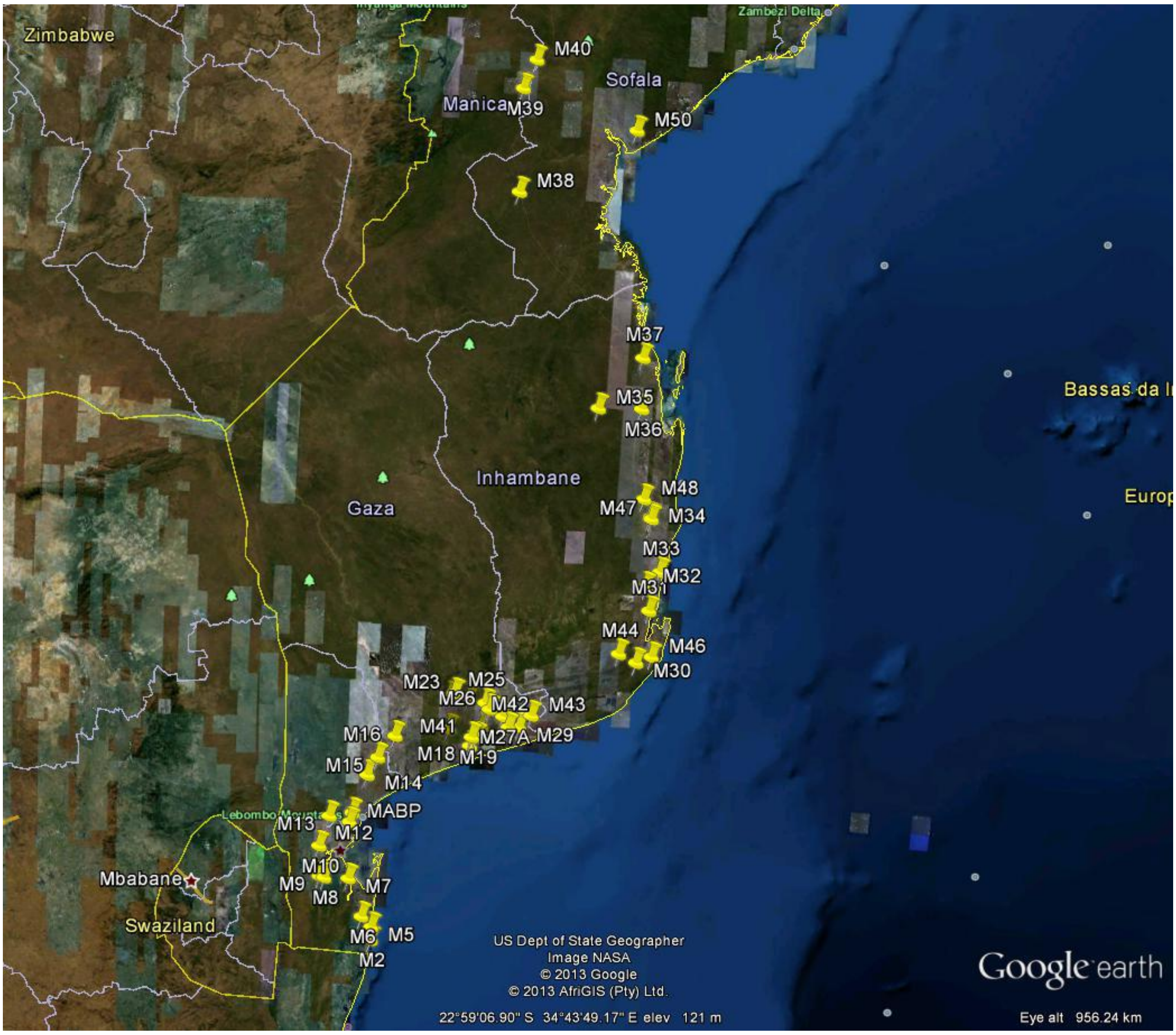


Figure 4-1: Location of sand samples in Mozambique

One site was located on an experimental section on the Cuabama-Chacane road in Inhambane Province where a 2 km section of neat, red sand was used as base course. This experimental section was constructed in May 2011 as part of an AFCAP supported project – Targeted Interventions on Low Volume Rural Roads in Mozambique. Bulk samples were extracted for appropriate laboratory testing.

Phase 2: The sand sampling programme for Mozambique was undertaken as follows, with the facilitation of the roads agencies in the country:

2 No. road sections with a total of 4 No. bulk samples as follows:

- Nametil-Angoche road
  - grey sand – subgrade
  - grey sand – borrow pit
- Marracuene-Macaneta (ETB trial site)
  - cream sand – base
  - cream sand – borrow pit

4.2.3 Site locations - Namibia

Phase 1: The sites chosen for sampling are pin-pointed on the Google Earth map and are presented in Figure 4-2. Sands in the eastern Caprivi were not sampled for logistical and cost reasons as well as the likelihood that the potential degree of development in this highly elongated area does not warrant their investigation.

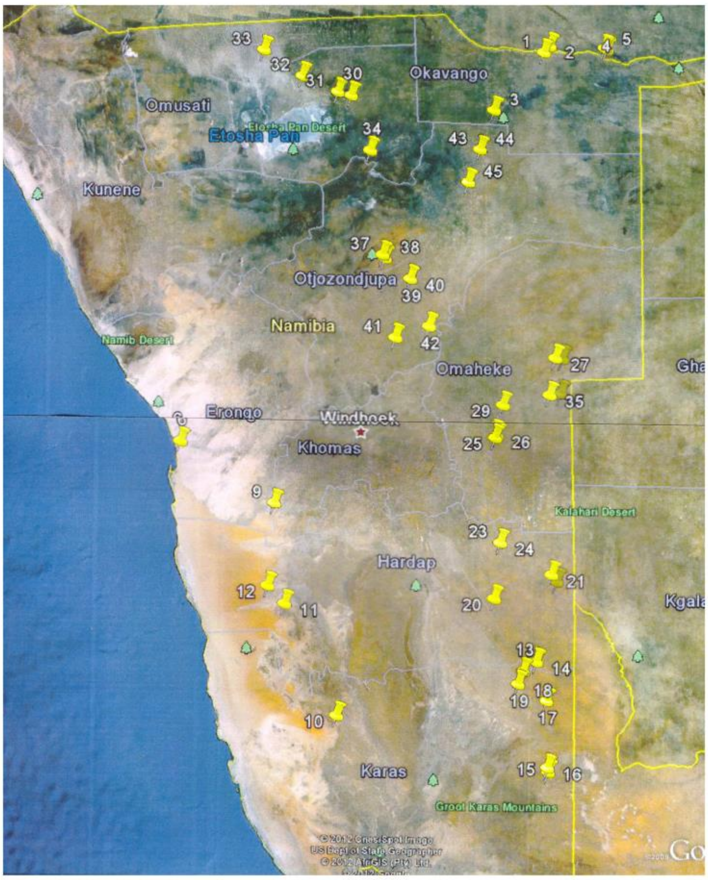


Figure 4-2: Location of sand samples in Namibia

Phase 2: Despite extensive discussions with the Roads Authority in Namibia, no roads were found where neat sand had been used as base course in Namibia.

4.2.4 Site locations – Botswana

Phase 1: Because of the prior investigation of off-road sand samples from Botswana, as reported in the Botswana Roads Department Guide No. The Use of Kgalagadi Sands in Road Construction (2010), further sampling was not undertaken as part of the Phase 1 project.

Phase 2: The sand sampling programme for Botswana was undertaken as follows, with the facilitation of the roads agencies in the country.

3 No. road sections with a total of 5 No. bulk samples as follows:

- Maun-Shorobe road
  - grey sand – subbase
  - grey sand – borrow pit
- Kang-Hukuntsi road
  - orange sand - borrow pit
- Serowe-Orapa road
  - Red sand – base

4.2.5 Site locations – Malawi

Phase 1: The focus of Phase 1 of the project did not include the collection of sand samples from Malawi.

Phase 2: The sand sampling programme for Malawi was undertaken as follows, with the facilitation of the roads agencies in the country.

1 No. road section with 1 No. bulk sample as follows:

- Lukini-Malingunde road
  - Dark red sand - wearing course

4.2.6 Site locations – South Africa

Phase 1: One site was located in the Free State Province in South Africa along the Hoopstad-Bultfontein road where a neat, 90m long road section with a yellowish-brown sand base course was constructed in 1962 as part of a soil stabilisation experimental project. Bulk samples were extracted from the section for laboratory testing similar to that carried out for the on-road sections from Mozambique.

Phase 2: Because of the exceptional value of the almost 50 year old experimental section along the Hoopstad-Bultfontein road, more extensive investigations were undertaken as part of a related project. The nature and outcome of these investigations have been reported separately, and are not part of this report.

4.3 Laboratory Testing

Phase 1: A wide-ranging laboratory testing programme was developed to fully characterise the properties of the sands. The following tests were specified:

4.3.1 Sand samples

(1) Particle size distribution

- Sieve analysis – Standard SANS 3001: GR1:2011 test with extra sieves (0.3 mm, 0.6 mm, 0.85 mm and 1.18 mm).
- Hydrometer analysis - SANS 3001:GR3 (oven-dried preparation)

(2) Atterberg limits (on fraction passing 0.425 mm and 0.075 mm sieves)

- Liquid Limit (SANS GR12:2010)
- Plastic Limit and Bar Linear shrinkage (SANS 3001: GR10:2011)
- Liquid Limit using BS 1377-2 1990.

(3) Description

- Soil classification (AASHTO M145 and ASTM D2487)
- Soil colour (Munsell Colour (air dried and at the liquid limit (on minus 0.425 mm fraction used for bar linear shrinkage test before placing in oven to dry)

Note: In the event, the SANS methods were not followed by the laboratory and, instead, the TMH1 procedures were used. However, the differences between the two methods are very small and no significant differences in results are likely.

4.3.2 Selected representative samples

- Field Moisture Equivalent (FME) (AASHTO T93-86). - to determine particle shape and texture.
- Pedological description and origin
- Free Fe and Al contents – CBD method (Loock, 1990), XRD and XRF methods
- Organic matter content

4.3.3 Bulk samples (from road sections)

- All tests listed in Section 4.3.1
- MDD/OMC/CBR
  - 3 compactive efforts
  - CBR at varying moisture contents (soaked, OMC (after 4 days equilibriation), and dried back after 4 days equilibriation to 75, 50 and 25% OMC.

Phase 2: The laboratory testing programme for the Phase 2 samples obtained form the roads sections in Botswana, Malawi and Mozambique was similar to Phase 1, but with the additional wet/dry cycled CBRs as well as laboratory DCP testing to obtain an alternative measure of the material's strength in terms of the DN value (resistance to penetration in mm/blow) as compared to the more traditional CBR value..

- DN at varying moisture contents (soaked, OMC (after 4 days equilibration), and dried back after 4 days equilibration to 75, 50 and 25% OMC.
- Wet/dry cycled CBRs 1, 5 and 10 day cycles).

4.4 Physical Properties

4.4.1 Colour

The colour of each sample collected was classified according to a simplified soil colour chart prepared by the Soil and Irrigation Research Institute (now the Institute for Soil, Climate and Water, Agricultural Research Council, South Africa). This is based on the Munsell classification system and consists of various *hues* (red, yellow, blue, green, purple (with intermediate combination values between each). Each hue has a modifier indicating the *value* (or lightness) from a value of 0 (black) to 10 (white). The *chroma* is then classified as the "purity" (or saturation) of the colour with low classifications indicating less pure (more pastel) and high numbers being more vivid.

On this basis, a soil could be for example classified as a 5YR 3/4, which indicates that the hue is in the middle of the Yellow red sector (Figure 4-3(a), the value 3 indicates that the sample is dark (3 or 30% up the black to white scale – Figure 4-3(b) simplified) and the chroma is 4, i.e. more towards the pastel (left) side of Figure 4-3(c) than the vivid (right) colour side.

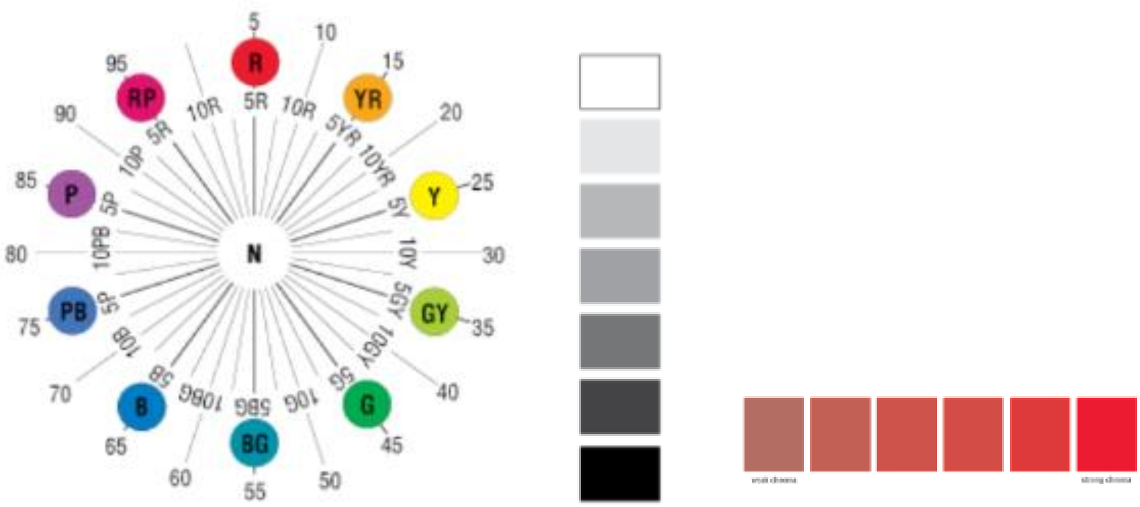


Figure 4-3: (a) Definition of hue; (b) Definition of value; (c) Illustration of chroma

Each of these Munsell values has a colour narrative description, and although these are used in conventional soil profiling, they are rather vague, e.g. dark grey brown or light olive brown.

For this project the actual Munsell classification of each sand has been determined in the air dry state as well as near the liquid limit. However, typically when using sands, the darker materials loosely described as red or brown are generally thought to perform better than those lighter materials described as grey or white.

The materials collected had a wide range of colours illustrated in Fig 4-4 with their Munsell descriptions in the accompanying diagram.

Figure 4-4: Typical range of colours of sands in Mozambique and Namibia

The reddish colouration of the soils is derived from the mobilisation of iron oxides (Cooke, 1964) during in situ weathering of mainly ferromagnesian minerals. In contrast, where the sands have been affected at some stage of their development by a fluctuating water table, the skin of iron oxide has been removed and the sands tend to be grey in colour.

4.4.2 Grading

Phase 1: Sand samples from Mozambique and Namibia

Knowledge of the grading of sand provides a critical insight into their engineering behaviour. Sands derived from different origins tend to classify differently and to exhibit gradings that reflect their mode of formation (aeolian versus mixed origin). As indicated in Figures 4-5 and 4-6, many of the sands investigated in Mozambique and Namibia are clearly of mixed origin. These sands exhibit a typical S-shaped grading curve and comprise a wide range of particle sizes.

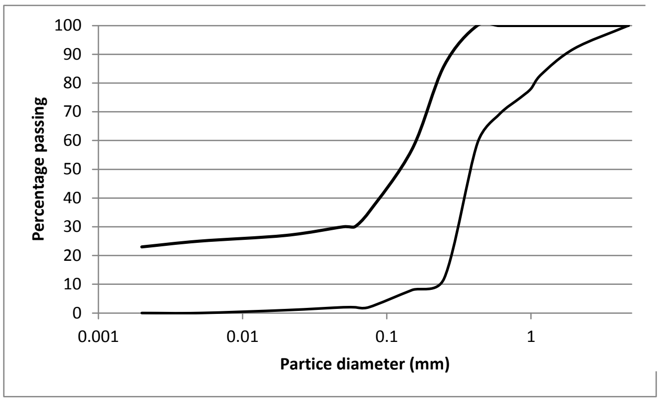


Figure 4-5: Grading envelope – Namibia all sands

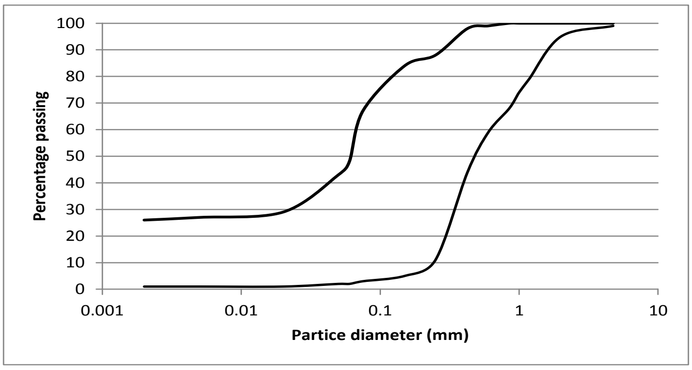


Figure 4-6: Grading envelope – Mozambique all sands

The classification of the sands based on their grading in terms of Phi units for both Mozambique and Namibia are shown in Figures 4.7 and 4.8 respectively.

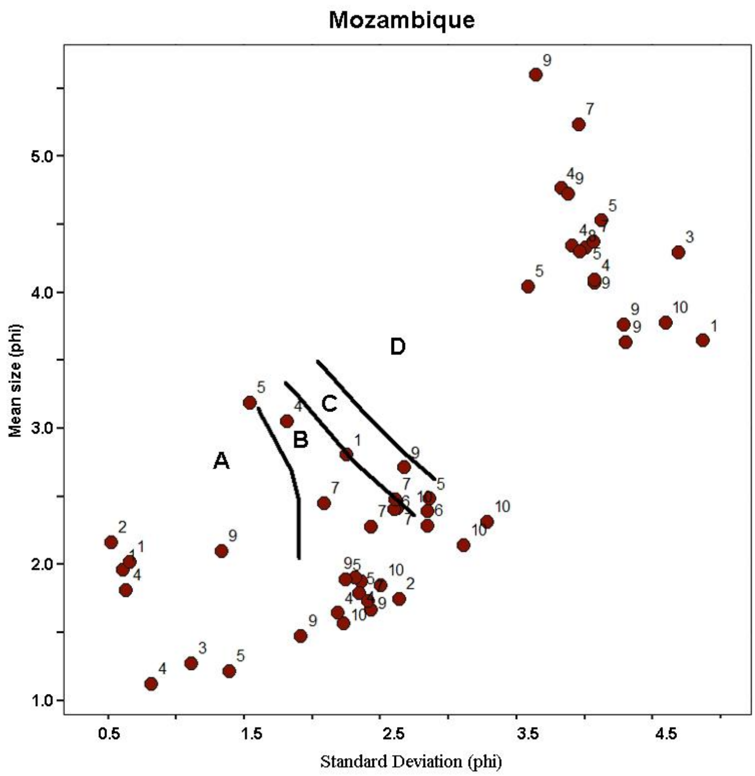


Figure 4-7: Classification of Mozambique sands based on grading (Phi units)

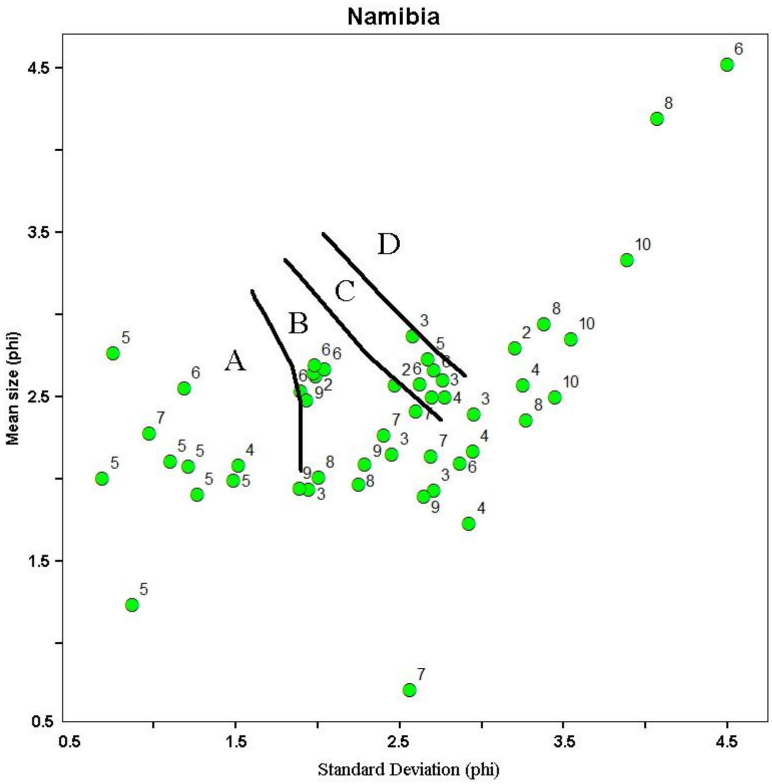


Figure 4-8: Classification of Namibia sands based on grading (Phi units)

The colour code for the various sands from Mozambique and Namibia is based on a scale of 1 to 10 which ranks the sands on their apparent darkness, irrespective of whether this darkness is a function of its redness or browness, both due to different forms of iron. This scale is shown in Photo 4-1.

Photo 4-1: Colour scale used for ranking of sands by darkness

Of the 99 samples collected and tested (the results of 3 were disregarded as they were classified as gravels and not sands) 36 of them (36.4% of the total) plot within the Wylde chart B zone and, based on experience in Australia and Botswana, are potentially suitable for use as neat base course in a low volume road pavement. These 36 samples were subjected to detailed examination which revealed the following observations.

- Sands from Mozambique and Namibia plotting in Zone B exhibit a wide range of colours – not just red/dark red (Code 4, 5, 7, 9 and 10 in Figure 5) as was the case in the experimental road section in Botswana where red sand was used successfully as neat base course.
- Only 9 samples (24.0%) were red coloured sands whilst 17 samples (47.2%) were dark sands (colour code > 6) and the remaining 10 samples (27.8%) were light coloured sands (colour code < 3).
- Significantly, the red sands used as neat base course in the ANE sections in Mozambique plot in Zone B of the Wylde Chart, as was the case with the red sands that were used successfully as experimental neat base course in Botswana.

An analysis of the particle size distribution of the sands from Mozambique and Namibia that plot in Zone B is shown in Figures 4-9 and 4-10 respectively.

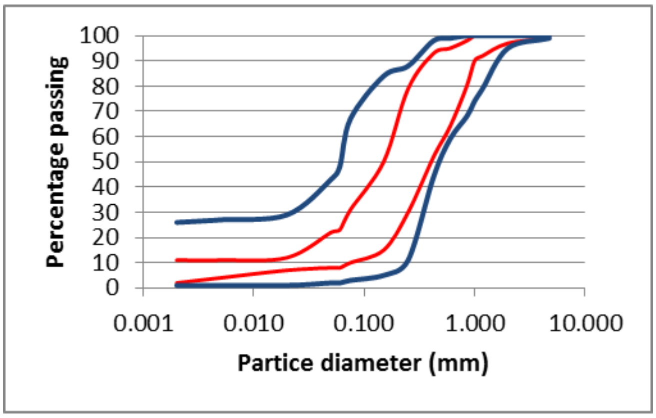


Figure 4-9: Mozambique: Grading envelope for sands in B Zone (red) of the Wylde Chart

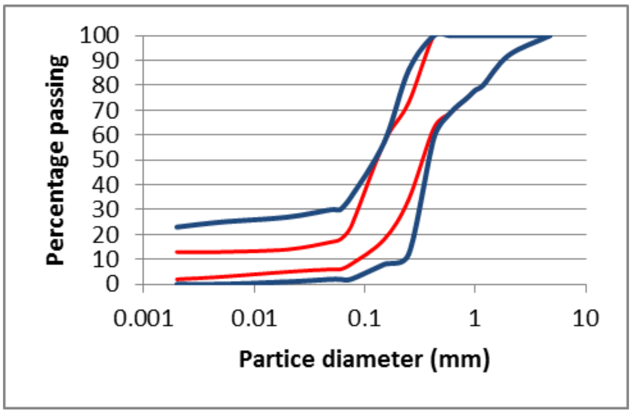


Figure 4-10: Namibia: Grading envelope for sands in B Zone (red) of the Wylde Chart

From analysis of the sands that plot in Zone B of the Wylde Chart, the following findings emerge:

- The grading moduli of the Namibian sands in Zone B ranged between 0.82 and 1.34 and those from Mozambique between 0.83 and 1.24, a relatively wide range for sands. This indicates the coarseness of some of these sands, comparable with the range for selected red and yellow sands from Mozambique described by de Vos (2007). It also indicates that there are no significant differences in the gradings of the different coloured materials that plot within the B zone of the chart.
- Within the red sands alone from Namibia, the GM varied between 0.91 and 1.24 and from Mozambique between 0.65 and 1.34

As would be apparent from Figures 4-9 and 4-10, the grading envelopes for both the Mozambique and Namibia sands are somewhat narrower (P075 of 2-12%) than the envelope for the entire Mozambican and Namibian data as presented in Figures 4-5 and 4-6 (P075 of 2 – 25%). It seems likely, therefore, that the shear strength of the sands reduces when fines exceed about 12%. Thus, it would appear that grading is the dominant feature affecting the potential performance of sands as structural materials, with aspects such as the colour and angularity probably having a secondary influence. As a result, grading in Phi units should be considered as the primary indicator for identifying potentially useful sands.

Sand samples from experimental sections

In addition to the sand samples collected from Mozambique and Namibia, samples were also collected from the base course of two experimental sections where neat sand had been used as base course - one in Botswana from the Serowa-Orapa experimental section and the other in South Africa from the Hoopstad-Bultfontein experimental section. These experimental sections have performed satisfactorily for 23 and 50 years respectively and have both carried in the region of ± 0.5 MESA.

Plotting the results obtained from the Serowe-Orapa and Hoopstad – Bultfontein experimental sections, indicates that these materials do not fall into Zone B but all fall in Zone A (ref. Figure 4-11). It would thus appear that although the materials do not have as much fines (as wide a grading) as those in Zone B, they are generally a little finer and the upper portion of the line separating Zones A and B could perhaps be moved towards the left to include these materials as indicated in Figure 4-11. This can only be checked against more performance-related samples and further comparison of these materials based, for example, on DCP/DN tests as discussed in Section 5-2.

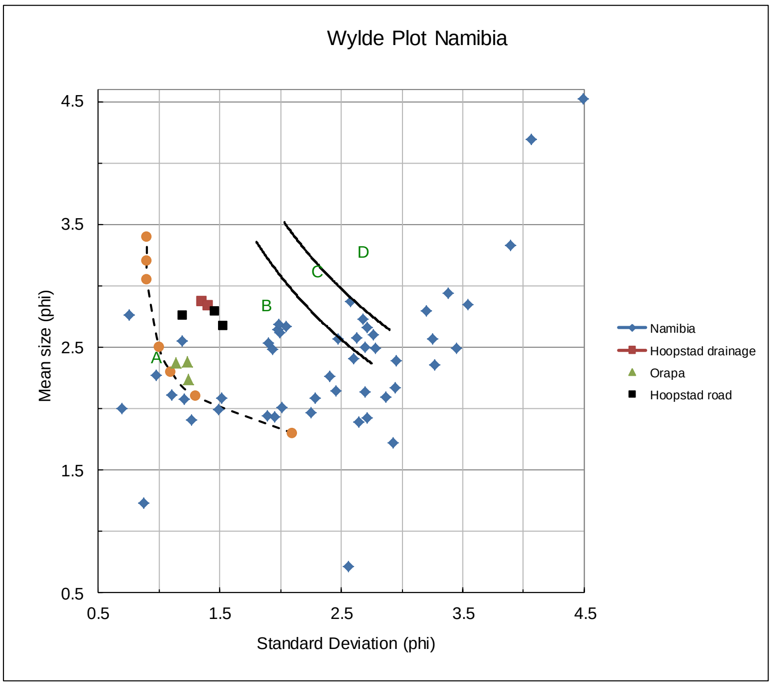


Figure 4-11: Classification of all sands based on grading (Phi units)

It should also be noted that the Hoopstad-Bultfontein samples were brown and not red as many of the other successful samples have been.

Phase 2: The results obtained from the additional investigations are, in general, similar to those obtained from the Phase 1 investigations and are not included in this report. They may be found in the companion report: The Use of Sand in Road construction in the SADC Region – Additional Investigations. Final Report. July 2014.

4.4.3 Plasticity

Sands exhibit some variation in plasticity depending primarily on their mode of formation and the consequent presence or not of either clay minerals or, possibly in some cases, salts deposited by the evaporation of salt-laden ground water. Determination of plasticity in the traditional manner, i.e. on material passing the 0.425 mm sieve (P425), invariably tends to mask the plasticity that can be mobilised in the silt and clay fraction of the sand (P075). Many sands show non- or only slight plasticity on the P425 fraction but significant plasticity on the P075 fraction. *This characteristic is of critical significance in differentiating between the engineering properties of the different kinds of sands found I the SADC region.*

The relationships between plasticity and soil colour are shown in Table 4-1. Note that this classification is based on the 2 and 6 scale (ref. Section 2.7.2) and does not follow, for example the normal classification of silt as material between 0.002 and 0.075 mm (the upper limit is 0.06 mm).

Colour	Sand (%)			Sand Total (%)	Silt Total (%)	Clay Total (%)	Atterberg limits and shrinkage ((P075)			
	Coarse	Medium	Fine				LL (%)	PL (%)	PI (%)	LS (%)
White	0	42	50	92	0	8	NP	-	-	-
Lt brown	36	34	17	87	4	9	47	28	19	9.0
Yellow br	8	70	18	96	1	2	-	-	-	-
Dark br	14	25	30	69	7	23	52	28	24	10.0
V dk gr br	11	39	31	81	3	16	48	21	27	9.0
Red	21	20	37	78	8	5	22	15	7	3.0
Red	6	44	37	87	2	11	NP	-	-	-
Brown	30	32	29	91	6	2	NP	-	-	-
Dk red	20	24	37	81	4	14	33	21	12	7.5
Dk red br	18	22	36	76	8	14	40	22	18	10.0

Table 4-1: Average sand, silt and clay content and PI of samples in relation to colour

As indicated in Table 4-1 which applies to all the samples from Mozambique and Namibia, there appears to be a correlation between sand colour (influenced by mode of formation and mineralogy) and plasticity. Thus, in general, the whitish aeolian sands tend to exhibit little, or no, plasticity (even on the P075 fraction) compared with those dark red/brown/grey brown sands of mixed origin which tend to exhibit higher plasticities on the P075 fractions. Unfortunately, there was insufficient material to determine the PI on the P075 of the Hoopstad-Bultfontein samples – a shortcoming that should be rectified in future.

4.4.4 Particle shape

The shape of sand particles has a significant influence on its compactability, density and stability and, hence, the overall engineering behaviour of the material. For a specific grading, this parameter controls the manner and degree of particle interlock upon which is dependant such components as shearing resistance, crushing resistance and flexural or tensile strengths. Thus, smooth rounded particles would offer less resistance to rearrangement than angular and/or elongated particles with rough surfaces and, therefore, other things being equal, the latter would be expected to give lower compacted densities but higher strength.

The definition and measurement of particle shape is a complex subject. However, an elementary definition of shape is shown in Figure 4.12 (Lees, 1964). Based on the simple measurement of longest, intermediate and shortest dimension of a sand particle, its *form* can be classified into one of four classes:

- *Oblate:* Particles which are tabular or shaped like a disk.
- *Equant:* Particles which are equi-dimensional.
- *Bladed*: Particles which are elongated and somewhat flattened.
- *Prolate:* Particles which are rod-shaped.

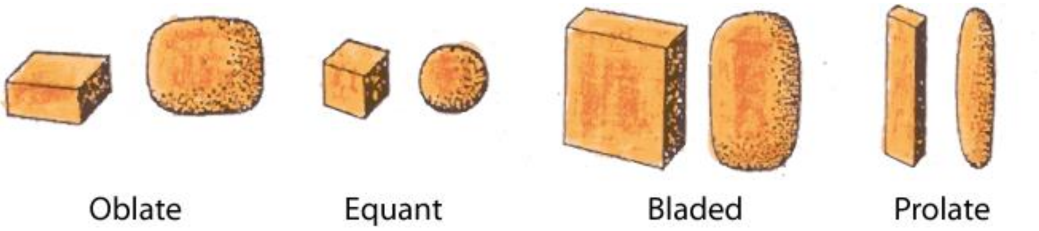


Figure 4-12: Particle shape

The above classification is a broad one and shape can be further described in terms of different degrees of *roundness* as illustrated in shown in Figure 4-13.

Figure 4-13: Degree of particle roundness

The particle shape of representative samples was examined visually using a low magnification binocular microscope. As indicated in Photos 4-2 and 4-3, the sands exhibit a wide range in particle shape, which is influenced by their mode of formation. In this regard, aeolian sands tend to exhibit a more rounded shape (Photo 4-2) than sands of mixed origin (Photo 4-3). This is a factor that significantly influences their compactability characteristics and related strength and bearing capacity as discussed above (Semmelink, 1992).

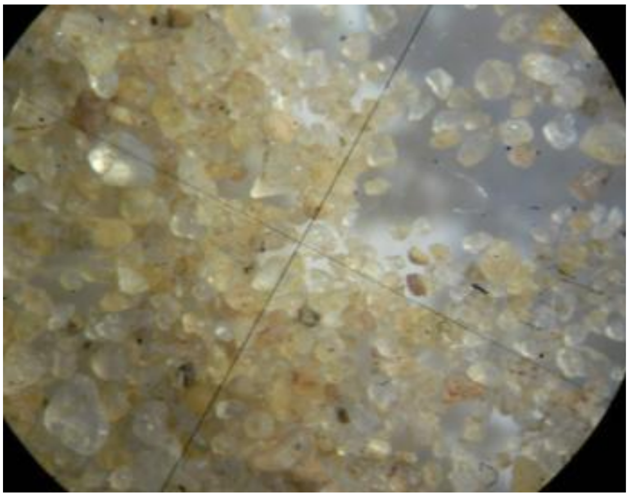


Photo 4-2: Example of well rounded, well sorted sand (circled particle = 0.5 mm dia.)

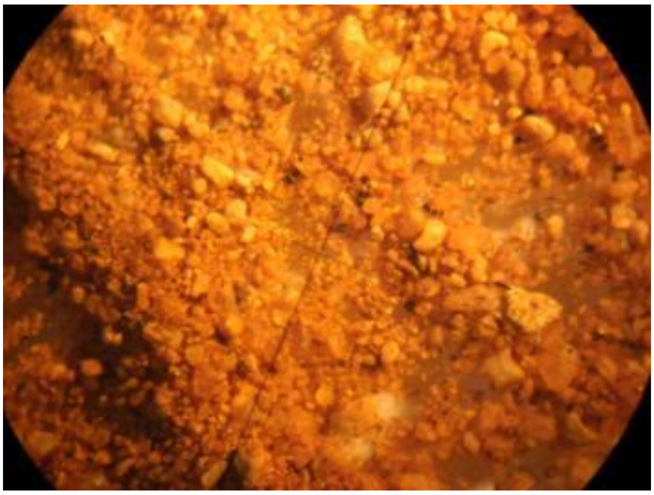


Photo 4-3: Example of poorly sorted, angular to sub-rounded sand (circled particle = 1 mm in dia.)

4.4.5 Particle density

Testing of the apparent and bulk relative density (ARD and BRD) and water absorption (WA) was carried out. The results are summarised in the Table 4-2.

Table 4-2: Results of particle density tests

Test	No of tests	Mean	Max	Min	Standard deviation
BRD	24	2.60	2.65	2.52	0.03
ARD	24	2.62	2.67	2.33	0.07
WA (%)	24	0.45	0.90	0.10	0.23
FME (%)	24	10.00	14.90	6.20	2.22

The table also shows the Field Moisture Equivalent. This property has been shown to indicate the approximate surface texture/friction angle of the materials when plotted against the LL. A plot of the field moisture equivalent against roundness is shown in Figure 4-14. The roundness was given a value based on angular being 1, well rounded being 9 and the intermediate values being allocated proportionally. Although there is some scatter, a definite trend is shown with increasing field moisture equivalent as the angularity of the particles increases.

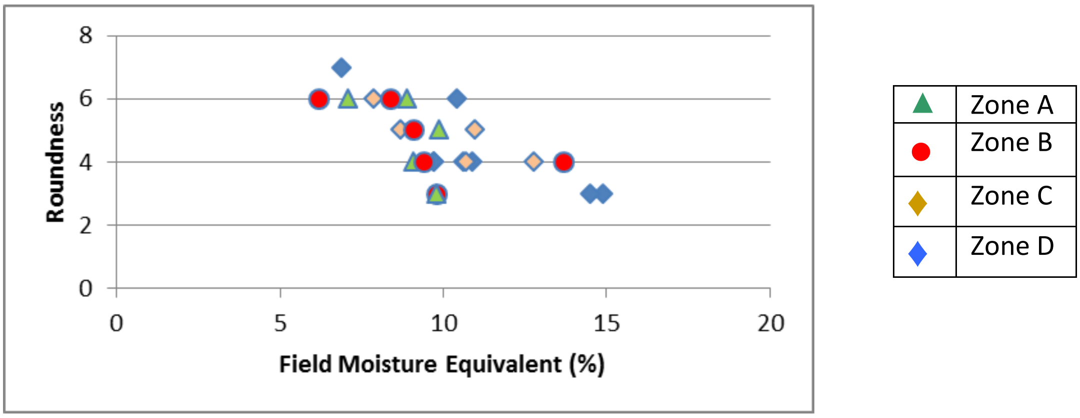


Figure 4-14: Field moisture equivalent versus roundness

4.4.6 Texture

The texture of sand is influenced by its grain size, sorting, rounding and maturity (see Figure 4-15).

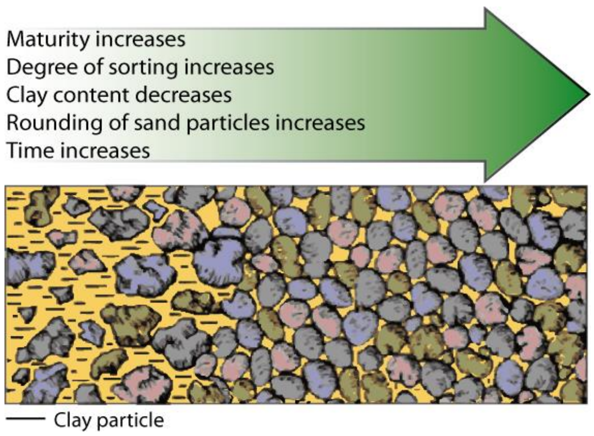


Figure 4.15 – Factors influencing sand texture

Grain size: Is a measure of the diameter of a sand particle and is a result of several factors, including composition, durability, severity of weathering conditions, transport distance from its site of origin, and physical sorting by wind and/or water currents.

The distribution of grain size in a sand sample is generally measured by sieving which essentially measures the maximum diameter of a sand grain in the size range 0.06mm to 2mm (equivalent to 62.5 to 2,000 microns or, on the Phi scale, 4Φ to -1Φ.

Material in the silt range (material finer than 0.06mm) or the clay range (material fines than 0.002mm) is normally determined by undertaking a hydrometer analysis. ***The determination of these fractions is very important as they affect the engineering characteristics and performance of sands as pavement layers in terms of plasticity, strength and bearing capacity.***

As the percentage of soil finer than 0.075 mm increases, terms such as silty sands and clayey sands are used. The majority of Kalahari sands would be classified typically as fine to medium or coarse sands.

Sorting: Is the range of sizes in a sand. Well graded (poorly sorted) sands show a range of grain diameters, while poorly graded (well-sorted) sands have a similar size (see Figure 4-16).

Figure 4.16 – Degree of sorting

An important distinction must be made between the definitions of sorting used by sedimentologists and civil engineers. A well-graded material in geology is one that has a relatively small range of particle sizes whereas engineers refer to well-graded materials as those with a wide range of particle sizes.

- ***Rounding:*** The presence or absence of corners and sharp edges on the sand particle. Particles with many edges are "angular". Particles lacking edges are "rounded" (see Figure 4.15). Roundness is not the same as spherical. An oblong particle can also be highly rounded.

  Generally speaking, the more well rounded the individual grain, the greater the energy involved in transport or the longer the duration of transport. Aeolian sands tend to be well rounded by high energy collisions and abrasions.

- ***Maturity:*** Is a relative measure of how extensively and thoroughly a sediment (sand size and larger) has been weathered, transported and reworked towards its ultimate end product, quartz sand. As a general rule, the greater the transport distance and the greater the length of time in the transport medium the more mature a resultant sediment becomes. Maturity is gauged largely in terms of grain size, grain sorting and grain roundness in terms of *immature*, *submature*, *mature*, and *supermature.*

An attempt to classify the maturity of each sand according to Folk (1980) was carried out with the results presented in Table 4-3.

Table 4-3: Classification of sand maturity (Folk, 1980)

	Immature	Sub-mature	Mature	Super-mature
Sorting	Extremely poorly to very poorly sorted. $\sigma < 2.00\varphi$	Poorly to moderately sorted $2.00 < \sigma < 1.00\varphi$	Moderately to well sorted $1.00 < \sigma < 0.5\varphi$	Well to very well sorted > 0.5φ
Grain size	Very coarse > 1 mm (0φ)	Coarse 0.5 – 1.0 mm (0 - 1φ)	Medium 0.25 – 0.5 mm (1 - 2φ)	Medium 0.25 – 0.5 mm (1 - 2φ)
Roundness	Angular	Sub-angular	Rounded	Well rounded

Unfortunately, the Folk classification is one that incorporates the mutual exclusion of properties within each class. It was found in this work that, although the principle of maturity makes sense on the basis of common expectation, the actual data recorded were not mutually exclusive. In other words, it was for example, possible to get a coarse, well sorted and rounded material. In most cases, however, the degree of roundness covered a wide range of classifications, probably indicative of a mixed origin for the sand. In general, however, one would expect beach and littoral sands to be mature and aeolian and fluvial sands to be less mature.

Both shape and texture play an important role in the performance of sand as a pavement material. Although the influence of shape and texture is not very critical in terms of the compactability of sand, they are very critical in terms of strength (DN) and bearing capacity and most likely in terms of elastic properties as well (Semmelink, 1992). As the sand grading changes from a well-graded material to a uniformly or poorly graded one the influence of shape and texture on strength and bearing capacity is reduced.

4.5 Mineralogical and Chemical Properties

4.5.1 Mineralogy

Knowledge of the mineralogy can assist in assessing the expected performance of the sands. This is particularly applicable to the fine or clay component and whether the grains are quartz or less durable feldspars, basalt or other rock and/or mineral types. Similarly, the chemical properties may also be useful indicators of the behaviour of the materials.

The presence of iron and/or aluminum oxides in the sand clays of Western Australia certainly appears to facilitate “self cementation” as shown by Emery et al (2007) where indirect tensile strengths (ITS) between 250 and 570 kPa were measured on materials extracted from roads as well as after drying back in the laboratory (Main Roads, 2002).

In order to determine the iron oxide (Fe_2O_3) and aluminium oxide (Al_2O_3) contents of the sand samples, as part of the full elemental analysis, Citrate-Bicarbonate-Dithionite (CBD), X-Ray Fluorescence (XRF) and X-Ray Diffraction (XRD) testing were carried out. The latter testing techniques produced more meaningful results than those determined using the CBD method. The results of the XRF tests are presented in Table 4-4.

Table 4-4: Results of oxide content determinations (XRF test)

Sample	Orapa	Orapa	1402	4020	4021	4036	4037	4042	4048	4049	4065	4073	4081	4092	4117	Hoopstad
SiO_2	93.24	93.71	94.06	83.92	89.37	84.29	82.68	89.70	94.56	93.44	97.04	91.25	93.07	92.22	91.90	91.89
TiO_2	0.21	0.22	0.21	0.26	0.18	0.56	0.57	0.35	0.26	0.29	0.11	0.26	0.16	0.42	0.32	0.33
Al_2O_3	1.83	1.79	1.80	6.87	5.64	5.89	7.08	3.87	2.81	2.80	1.57	4.99	3.33	4.91	4.19	2.85
Fe_2O_3	3.02	1.78	1.90	1.92	1.08	3.50	3.77	1.73	1.23	1.11	0.71	1.53	0.96	1.56	1.35	1.69
MnO	0.019	0.026	0.018	0.022	0.024	0.048	0.054	0.030	0.014	0.027	0.007	0.016	0.013	0.016	0.018	0.027
MgO	0.08	0.11	0.07	0.65	0.16	0.37	0.44	0.17	0.08	0.38	0.01	0.04	0.04	0.02	0.06	0.16
CaO	0.11	0.28	0.08	0.29	0.33	0.36	0.40	0.18	0.07	0.24	0.05	0.02	0.16	0.02	0.68	0.42
Na_2O	<0.01	0.02	<0.01	0.34	<0.01	0.24	0.22	0.24	0.10	0.29	0.08	0.02	0.32	0.03	0.03	0.07
K_2O	0.29	0.30	0.31	2.38	0.20	1.08	1.09	0.97	0.67	0.80	0.67	0.69	1.03	0.82	0.56	0.74
P_2O_5	0.014	0.014	0.011	0.030	0.016	0.037	0.035	0.026	0.015	0.014	0.010	0.015	0.013	0.013	0.017	0.023
Cr_2O_3	0.008	0.004	0.004	0.001	<0.001	0.007	0.009	0.002	0.003	<0.001	<0.001	0.004	0.013	0.024	0.003	0.006
LOI	0.85	0.82	0.75	2.14	1.53	2.11	2.52	1.17	0.78	0.81	0.15	1.56	1.42	0.48	1.33	1.42
Total	99.68	99.08	99.23	98.84	98.50	98.50	98.87	98.44	100.59	100.21	100.40	100.41	100.52	100.53	100.47	99.63
H_2O-	0.54	0.54	0.53	2.32	1.28	1.04	1.73	0.42	0.36	0.62	0.10	0.40	0.29	0.17	0.38	0.64
LOI–H_2O-	0.31	0.28	0.22	0	0.25	1.07	0.79	0.75	0.42	0.19	0.05	1.16	1.13	0.31	0.95	0.78
Al_2O_3 + Fe_2O_3	4.85	3.57	3.70	8.80	6.72	9.40	10.85	5.60	4.04	3.91	2.28	6.52	4.29	6.47	5.54	4.54

The iron content varied between 0.71 and 3.77% and the aluminium between 1.57 and 7.08 %. The sum of the two varied between 2.28 and 10.86%. It is notable that only 3 of the samples exceed the Australian minimum requirement of 8% combined Aluminium and Iron oxides. Of these three, none are the materials known to perform well, all are from Namibia and they all plot in Zone D of the Wylde chart. One of the materials is dark brown, one red and one dark red. The origin of the Australian specification requirement and manner of determining the iron and aluminium oxide contents is uncertain and still requires to be verified. Moreover, the Australian requirement appears to be too high for the sands tested. In fact the proven, neat base course sands at the Serowe-Orapa and Hoopstad-Bultfontein experimental sections had combined Al and Fe oxide contents of > 3.5% and in only one instance was the iron content significantly greater than the aluminium. Thus, subject to further confirmatory testing, this interim value, based on the XRF analysis method used, serves as a positive indicator of potentially suitable base course sands.

It should also be noted that the iron and aluminium analysed using XRF techniques include these Al and Fe that are part of clays and other minerals (ores, feldspars, etc) and may be over-estimated in these analyses in terms of their availability for "self-cementation". X-Ray diffraction analyses were also carried out to supplement the XRF analyses. The results show that the iron and aluminium quantities in the XRF analyses include contributions of these elements that are not available to self-cementation reactions as they are incorporated in clays, feldspars and micas.

The loss on ignition (LOI) indicates "volatile" materials that may evaporate or be broken down at temperatures above about 850°C. These include calcite, organic matter and included water in clays. The range of LOI measured is between 0.15 and 2.52% with the water component being between 0.1 and 1.73%. The difference between the two ranges between 0 and 1.13% indicating the maximum combined quantity of organics, CO2 from calcite, and other volatile components.

The LOI test does indicate the presence of materials that break down on heating, which could give clues to possible problematic constituents, such as organic materials, but also indicates the presence of potentially beneficial components such as calcite. These data do need some experience with their interpretation.

The various colours of sands also show a very striking and clear relationship between iron, (as Fe_2O_3) content and clay fraction (Cf) (Figure 4-17). There is also a strong, logical connection between the colour of the sand and its iron oxide content (Botswana sands).

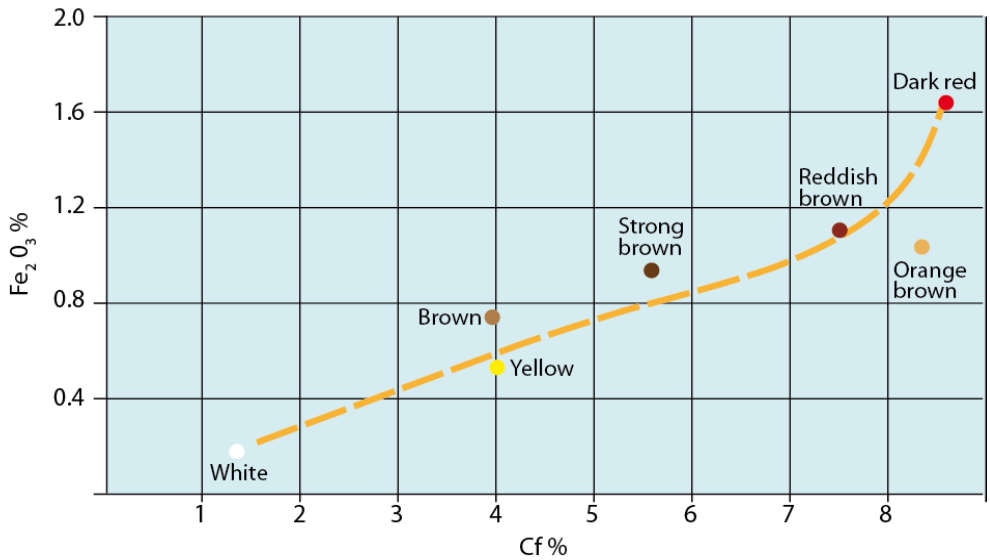


Figure 4-17: Iron oxide content versus clay fraction (Botswana sands)

4.5.2 Organic content

The presence of organic matter in the sands was identified using a simplified version of the standard test for determination of organic matter in concrete sands (SANS 5832:2006) in which a 3% solution of sodium hydroxide is added to a small sample of the sand. If there is NaOH –soluble organic matter present, the solution turns dark and a qualitative estimate of the organic matter can be obtained. The results of this testing are summarised in the addendum to this Guide – Summary of Laboratory Testing Results whilst Figure 4-20 illustrates the manner of interpreting the test in terms of the discolouration of sodium hydroxide (NaOH) solution with organic content.

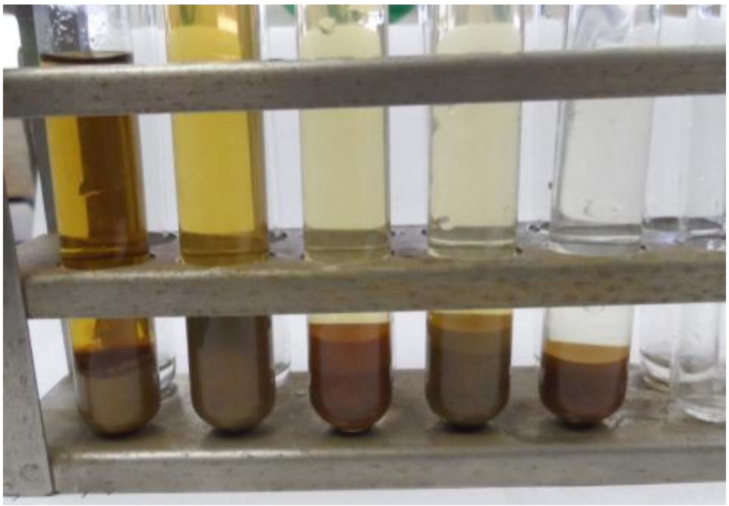


Photo: 4-4: Example of discolouration of NaOH solution with organic content (From left to right – Strong, moderate, weak, trace, none).

In general, the organic content is restricted to the upper portions of the soil profile. When using sands, this material would normally be stripped off and retained for rehabilitation of the borrow pit, as the humus, nutrients and seeds would be included in the material. The effect of excessive organic material in a neat pavement layer would essentially be restricted to the potential for decomposition of the organics resulting in small voids and openings in the layer and the possible germination of seeds incorporated into the pavement layer. This could lead to an increased porosity, potential settlement and rutting, increased permeability and a lower shear strength. These effects would in most cases probably be rather marginal. However, if the material was to be stabilized with lime or cement, organic matter could have more serious consequences, with deleterious reactions and possibly even failure of the stabilizer to react properly being possible.

4.6 Engineering Properties

4.6.1 Collapsible Sands

Certain types of Kalahari sands exhibit a *collapsible fabric.* This results from the generation of clays and iron oxides within the soil mass during weathering of susceptible particles. As a result, these sands possess an open structure, their void ratio is high and there is very little interlock between the sand grains - the structure is held together by the clay and iron oxide/hydroxide bridges. As long as the sand is not disturbed and is kept dry, these bridges provide considerable bearing strength. However, if the sand is wetted, the bonding bridges between the grains soften. Under load above a certain limit, the bridges break and collapse occurs as illustrated in Figure 4-18.

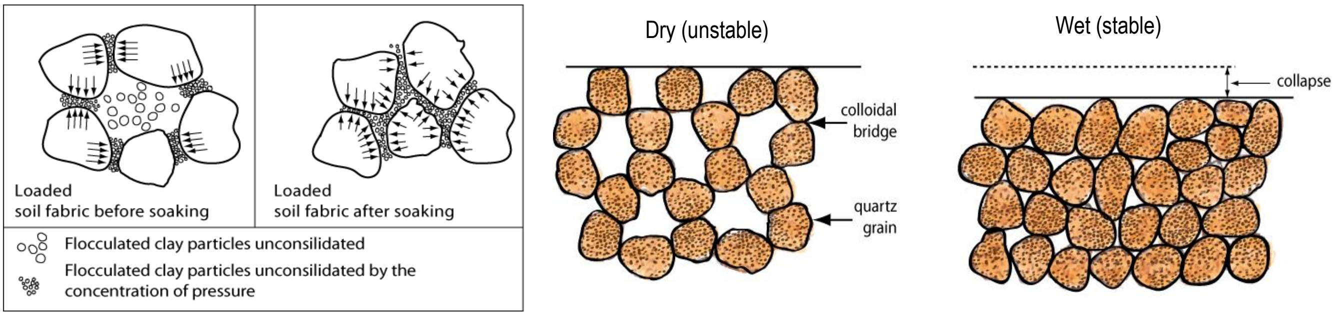


Figure 4-18: Graphical illustration of collapsible sand fabric (Brink, 1985) and mechanism of collapse

When sands collapse, they can spontaneously lose up to 20% of their original volume. This can be of particular significance to roads, airfields and railways, where the subgrade is subjected to increased load and increased moisture content. To reduce the chances of such collapse occurring in practice, current specifications often require that subgrade soils with a collapsible fabric be densified to a depth of about 1 m (Weston, 1984). However, this can be a very costly requirement to meet in practice, particularly in arid or semi-arid areas where water is often scarce and expensive. Options for dealing with this problem are discussed in Chapter 6.

The description and classification for collapse potential (CP) is shown in Table 4-5 which was originally developed for gauging the severity of differential settlement of structures, but can be a useful guide for roads. In this regard, the collapse potential would be determined at a loading equivalent to the overburden pressure of the pavement (typically of the order of 36 kPa for a road on a low embankment) and at saturation moisture content.

Table 4-5: Collapse potential related to severity of problem (Jennings and Knight, 1975)

Collapse Potential	Severity of Problem
0 – 1%	No problem
1 – 5%	Moderate trouble
5 – 10%	Trouble
10 – 20%	Severe trouble
> 20%	Very severe trouble

An example of the properties of a typical collapsible Kalahari sand in Botswana and the effect of application of compaction using a high energy impact compactor are shown in Table 4-6.

Table 4-6: Typical properties of collapsible reddish brown sand (Serowe-Orapa Road, Botswana)

Property	BRDM Guide for CP	Average results
Dry density (kg/m^3)	< 1600	1432.0
% passing 2.0 mm and retained on 0.075 mm	>60%	85.0%
% passing 0.075 mm	< 20%	17.5%
Relative density	< 85%	75.4%

Depth below surface (mm)	Collapse Potential	
	(200 kPa)*	(36 kPa)**
200 - 350	11.5%	6.3%
450 – 600	11.2%	7.1%
700 – 850	9.3%	5.0%
1000 - 1150	7.5%	3.6%
Based on a guide to the potential severity of the collapse problem proposed by Jennings and Knight (1975), the sand subgrade would fall in the "trouble" to "severe trouble" category.		

* as specified by Jennings and Knight
** estimated at in-service loading

Photo 4-5 – Collapse settlement of sand subgrade: In excess of 150 mm was obtained along a section of the Serowe-Orapa road in Botswana after compaction to near refusal by a 25 kJ high energy impact compactor.

4.6.2 Permeability

The permeability of sands is most influenced by compaction as well as the size, shape and connectivity of the water passages which are themselves largely affected by the grading and segregation of the soil. In general, an increase in plasticity will reduce permeability whereas segregation of particle sizes will increase permeability.

Typical permeability coefficients for other sands are shown in Table 4.7 (Lee, White and Ingles, 1983).

Table 4-7: Typical permeability coefficients for sands

Sand Type	Coeff. Perm. (cm/sec)	Qualitative Description
Clean coarse sand	$1 – 10^{-2}$	Medium
Graded sand	$10^{-2} – 5 \times 10^{-3}$	Medium
Fine sand	$5 \times 10^{-3} - 10^{-3}$	Medium to low
Silty sand	$2 \times 10^{-3} - 10^{-4}$	Low
Very fine uniform sand	$6 \times 10^{-3} – 10^{-4}$	Low
Dune sand	0.1 – 0.3	High

There is a perception that sandy materials are free draining and water-related problems affecting roads are minimal in sandy areas. This is not necessarily true (Brink, 1985) and has important implications for the use of Kalahari sands in road pavements, particularly as regards the important factor of avoiding permeability inversion within the pavement structure. The finer and less well-sorted sands are, the lower their permeability (Masch and Denny, 1965). This can be observed by the long periods during which rainwater stands on the sides of roads in, particularly, red sand areas. It is also common for the fines in the sand to be "washed" to the surface of exposed sands (for example in drains and adjacent areas) where they decrease the permeability (or infiltration rate) and lead to ponding of water over long periods.

4.6.3 Suction

Soil suction is primarily a function of the soil grading and inter-particle voids, which are indirectly indicated by plasticity through the fineness of clay minerals. A typical equilibrium curve showing the relationship between the moisture content and suction (pF units) for wetting and drying cycles is shown in Figure 4-19 which indicates that (1) the suction pressure increases with decreasing moisture content and (2) for any given suction pressure, the moisture content increases with increase in clay content.

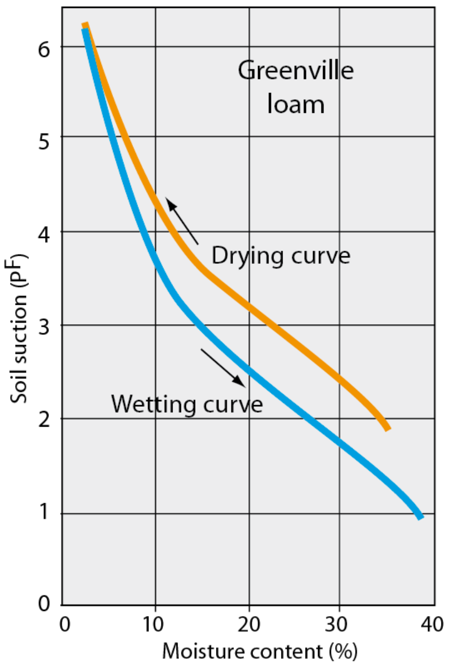


Figure 4-19: Soil suction/moisture content relationship (Wooltorton, 1954)

The development of soil suction strength allows some sands (particularly those with an appreciable fines content) to generate relatively high strengths in service as they dry back from their moisture content at compaction processing to their equilibrium moisture content – typically 0.6 – 0.7 of OMC (Emery, 1992). The soil suction generated over this range of moisture content will be considerably higher than those existing under soaked conditions. However, if the sand is to retain its suction generated strength then the soil suction must be maintained by ensuring that the moisture content in the pavement layers does not increase above the OMC of the material. This can only be achieved if the road surfacing is impermeable and effective drainage is in place.

4.6.4 Strength (CBR and DN)

Phase 1: The strength of the red sands used as neat base course in the ANE experimental sections and at various densities and moisture contents is shown in Figure 4-20 which shows the variation in conventional CBR strength of a sand at various compaction densities as well as at various moisture contents. In the soaked condition, the CBR measures mainly the frictional strength component of shear strength. As the moisture content of the material decreases, the cohesion component of the shear strength increases contributing to an increase in total shear strength, particularly as the density increases.

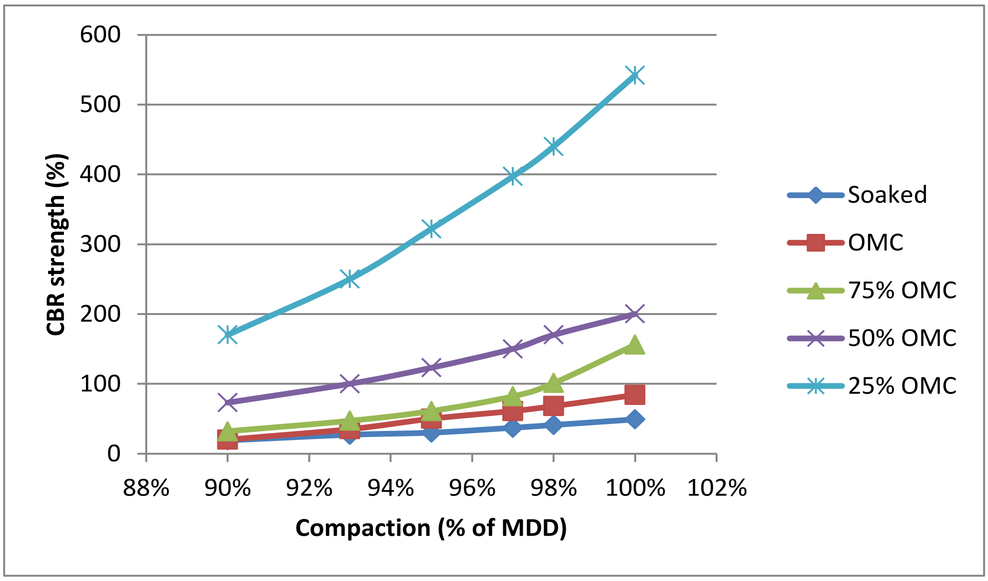


Figure 4-20: Variation in CBR strength at various densities and moisture contents

If these data are assessed in term of their corresponding Dynamic Cone Penetration (DCP) rate (in mm/blow) a similar pattern is observed (Figure 4-21), although the exponential relationship between the CBR and DN masks the large increases in strength as the material dries out.

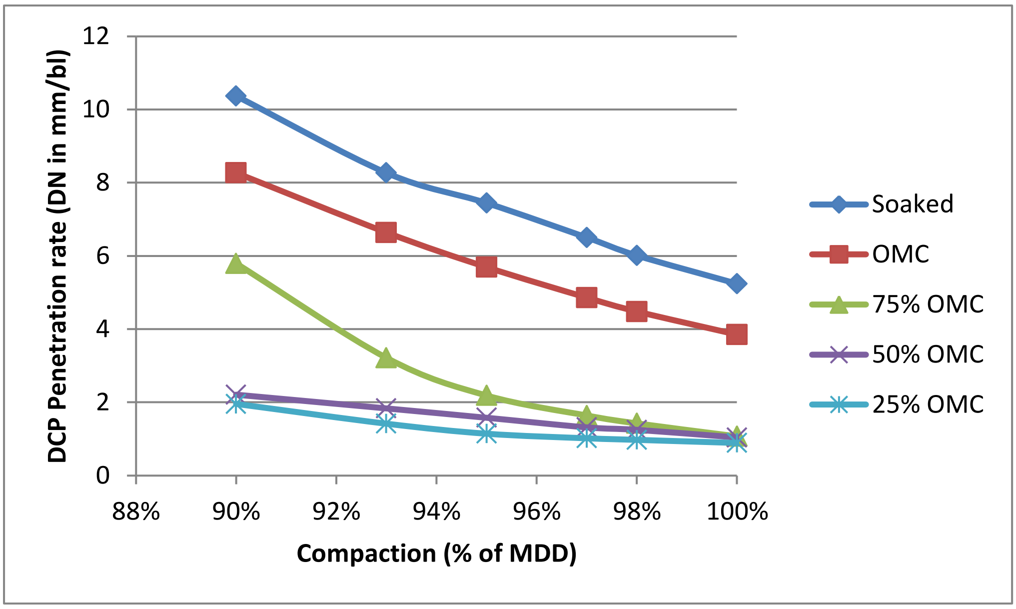


Figure 4-21 - Variation in equivalent DN value at various densities and moisture contents

The above figures illustrate the importance of ensuring that, in service, the moisture in the road pavement, particularly in the outer wheel path, is maintained at or below OMC. This can be achieved by a waterproof surfacing that is properly maintained in service as well as by proper drainage, as discussed in Section 5.4.8.

CBR results from 3 samples from the neat base course of the Hoopstad-Bultfontein experimental section exhibited values at 98% Mod. AASHTO between 47 and 57% and OMC CBRs of 71 to 90% - much lower than conventional requirements but, nonetheless, performed remarkably well.

Phase 2: Greater attention was paid in Phase 2 to the CBR/DN relationship of the sands, the outcome of which is discussed below.

Standard CBR tests were carried out on materials compacted using both the dynamic (SANS 3001:GR40) and vibrating hammer (BS 1377) compaction methods. Testing was done at the standard soaked condition as well as at OMC and after drying back to different percentages of OMC. DCP penetrations were also carried out on various moulds and are described in the next section. The OMC and dried back samples were allowed to equilibrate for at least 4 days in a sealed plastic bag before the CBR penetration was carried out. Although the results for penetrations to depths of 2.54, 5.08 and 7.62 mm when tested according to SANS 3001: GR40 were provided, only those for the 2.54 mm test are shown graphically in Figure 4.13.

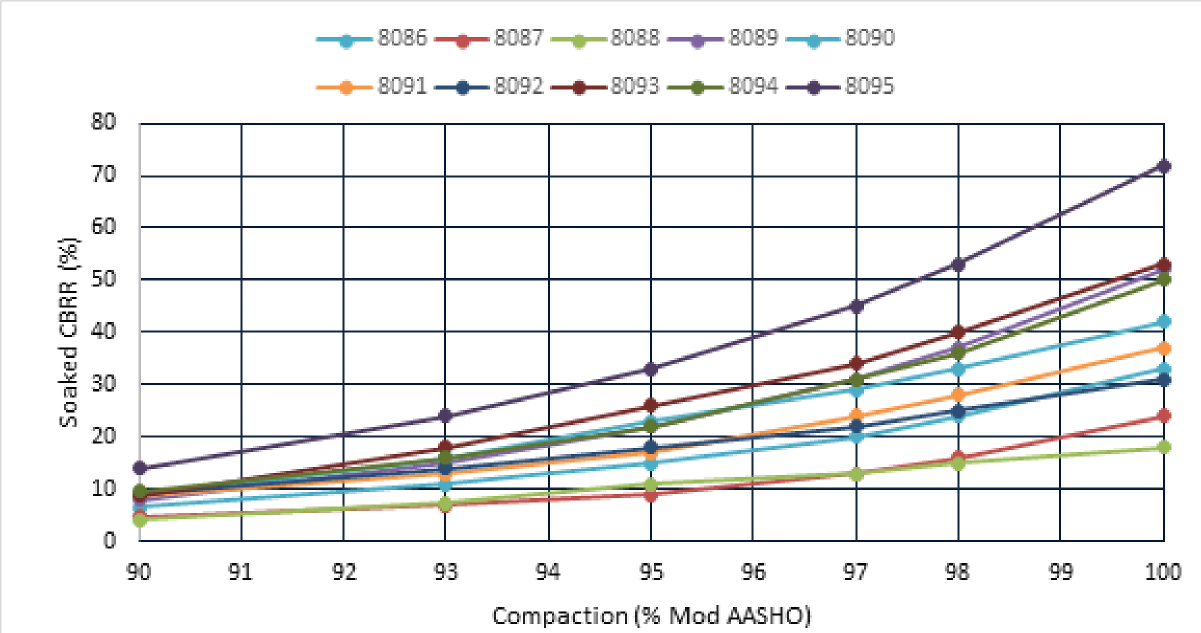


Figure 4-22: Plots of strength (soaked CBR) versus density

The soaked CBR strengths at 98 % Mod AASHO compaction vary between 15 and 53%, none of these complying with the specification for conventional base course materials. Despite this, samples 8086 and 8091 (CBR 24 and 28%) have provided excellent service in a 25 year old road in Botswana. These are two of the weakest samples tested and there appears to be no reason why the remaining materials with CBRs higher than this should not theoretically perform well in practice.

The CBR values are generally quite low compared with many sands that have been tested. The plot clearly shows how the sensitivity of the strength to density varies from sand to sand. This is an important aspect that needs to be considered during construction, specifically the impact in terms of not achieving the required density.

CBR testing was also carried out on samples compacted using the vibrating hammer at various moisture contents. Apart from the normal soaking procedure, all specimens were equilibrated in plastic bags for 4 days after reaching their required moisture contents.

It was found, however, that the majority of the specimens actually underwent plastic deformation and complete failure before a penetration of 2.5 mm was reached. In these cases, the CBR was estimated from the peak load prior to the deformation and failure. The reason for this type of failure is unknown but is possibly associated with segregation of the sands during vibration in the mould. As it affected both the soaked and dried back samples similarly, it is unlikely to be a moisture related problem. The results are summarised in Table 4.14, with a number of the results still outstanding.

Table 4.8: Soaked CBRs of samples compacted by vibrating hammer at various moisture contents

Sample	Source	CBR (%) at various moisture contents					
		Vibrating hammer					Conventional
		Soaked	OMC	75% OMC	50% OMC	25% OMC	Soaked
8086	Orapa	21	49	6?	81	108	33
8087	Kang	44	48	99	111	256	24
8088	Lukini	1.1	7	2	16	26	18
8089	Nametil	40	79	161	135	135	52
8090	Marracuene	13	15	75	120	165	42
8091	Orapa	35	28	91			37
8092	Maun	15	44		64	37	31
8093	Maun	39	24			102	53
8094	TRL/ANE	4	16				50
8095	Nametil	90	112				72

Notes: All CBRs were determined on soaked specimens after different curing/drying periods. All materials were compacted using a vibrating hammer. The last column is the standard CBR determined at 100% Mod AASHO density (see Table 5.13).

Although there are general trends of increasing strength with reduced moisture content there are many anomalies indicating the fundamentally poor reproducibility of the CBR test. Of greater concern is the poor correlation between the conventional dynamically compacted CBRs and the vibrating hammer results. The ratio of the soaked CBR determined using the vibrating hammer to the impact hammer varies between 6 and 180 per cent with a mean of 71%.

As there is a move away from CBR and traditional classification testing for the specification of sands (AFCAP, 2013) towards using the DCP design method for low volume roads, all of the compacted moulds have been tested using a Dynamic Cone Penetrometer (DCP) as well. Each of the compaction specimens was allowed to equilibrate in a sealed plastic bag for at least 4 days before being tested with the DCP (Figure 4.14). The sample was clamped to the mould holder over the base plate and the DCP driven in from the top of the specimen as compacted. The penetrations were carried out next to the CBR plunger depression at two points 180° apart. Typical plots of the data are shown in Figure 4.15.

Photo 4.6: DCP test being undertaken in CBR moulds

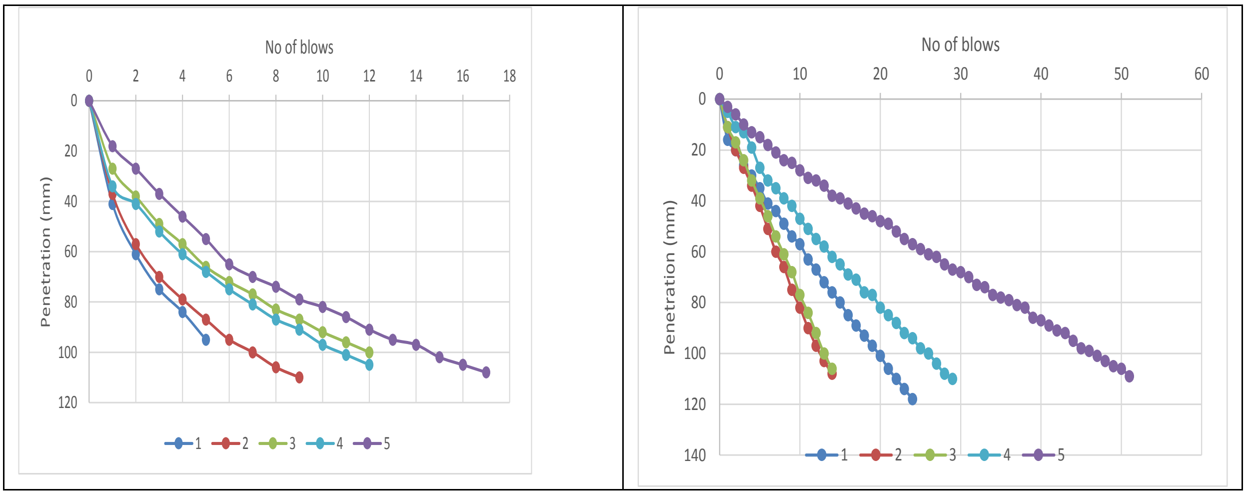


Figure 4-23: Plots of DCP penetration depth against number of blows

Most of the results show the pattern in the left hand plot with a rapid penetration through the upper portion of the mould and then a more linear penetration rate beneath this. This is far less noticeable in the much stronger material in the right hand plot. The quicker penetration rate in the upper portion of the mould could be due the method of equilibrating the samples (upside down), which allows the water in the sandy materials to accumulate at the bottom of the inverted specimens.

Standard practice for carrying out DCP testing in CBR moulds appears to be to knock the DCP cone into the sample in the depression made by the CBR plunder, with the annular weight in place on the mould (Netterberg and Kleyn, Pers comm, 2014). Attempts to penetrate the material through the centre of the annular surcharge weight used for the CBR penetration proved unsuccessful as the material bulged under the weight lifting it (Figure 4.16) and squeezing sand into the central ring. This was thus abandoned and the material was penetrated with no weight.

Photo 4-7: Lifting of the annular surcharge weight during a central DCP test

To calculate the representative DN value (mm/blow) for the material, a study of the results indicates that for most samples the average penetration rate below a depth of 60 mm appears to be the best value. An example of this is shown below for a typical material.

DN values measured on these unsoaked moulds ranged between 1.9 and 17.5, approximately equivalent to CBR values of between 12 and 69% although in the upper 60 mm of the moulds values as low as 5% were measured.

Figure 4.17 shows a plot of the laboratory determined soaked CBR against the DN value obtained on the samples in the laboratory, immediately after penetration for the CBR test and adjacent to the penetration mark between the mark and the side of the mould. The correlation is poor with an R^2 value of only 0.59 and a prediction model of

$CBR = 180.2\ DN^{-0.549}$

compared with the Kleyn model of

$CBR = 410\ DN^{-1.27}$.

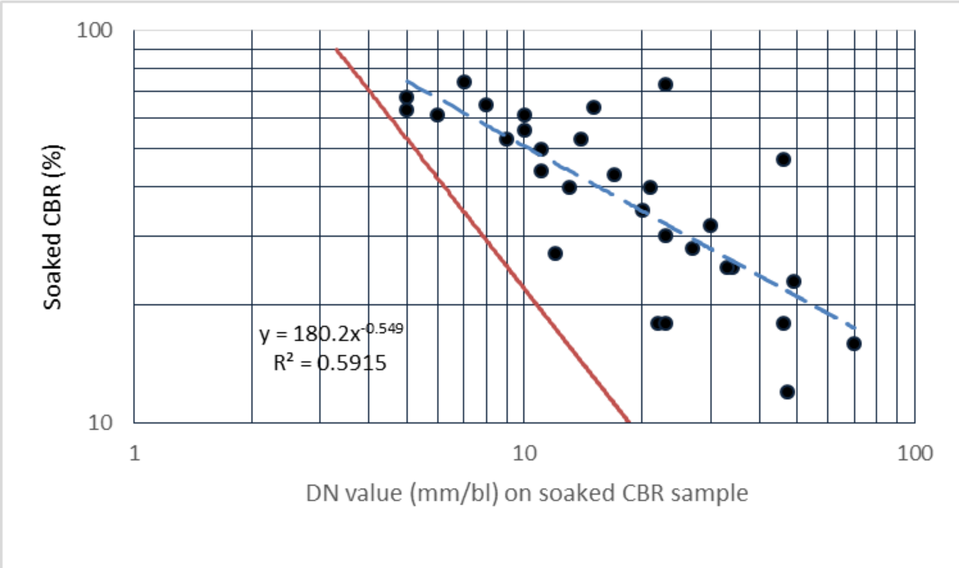


Figure 4.17: Plot of CBR versus DN (dashed blue line is regression model of actual data and orange line is plot of Kleyn prediction model)

This supports the conclusion of Sampson (1984) that the standard DCP CBR prediction models that have been developed can be highly material dependent. At high strength the difference in the models is small (almost zero at a CBR of about 100% but at low strength it is large (an order of magnitude at strengths less than 10%).

Sampson (1984) found that for non-plastic materials (which were probably mostly G1 and G3 aggregates but included sands) the CBR at a DN of 10 mm/bl was about 70% compared with 23% for all materials, much closer to the 50% determined on the sand samples tested. *New correlation models and penetration rates should thus be developed for the use of DCP design methods with sands.*

In situ DCP test results were available for some of the sites (Orapa and Maun-Shorobe) and a comparison of these with the laboratory values discussed above was attempted. The average DN values obtained at these two sites were 2.36 to 3.19 mm/blow for Orapa and 5.25 to 5.43 mm/blow for Maun-Shorobe. These are considerably lower than the laboratory determined values at about OMC (6 to 10 mm/blow for Orapa and 21 to 27 mm/blow for Maun-Shorobe). This indicates the possibility of a number of factors:

1. The materials are much drier than OMC;
2. The materials are at much higher densities than those tested in the laboratory;
3. The materials have undergone some sort of “self-cementation” or traffic moulding that increases their in situ strengths;
4. A high suction strength is created in the compacted material as a result of the greater surface tension effects associated with finer materials.

It is difficult to identify which, if any or even all, of these prevail at this stage.

The DCP penetrations at both of these sites were taken through the underlying layers, which were also constructed of local sands. In these cases the in situ DCP penetration rates were even lower than the upper layers ranging between 1.21 and 2.41 mm/blow. During testing of some of the roads in Mozambique, it was also found that underlying layers were almost or in some cases, impenetrable with the DCP. It would thus appear that the confinement of the sands may have an important influence on their behaviour.

This shows that sands in general can be expected to perform significantly better in service than their laboratory results would indicate.

5. DESIGN, SPECIFICATION AND USE OF SANDS

5.1 Introduction

Sandy materials do not comply with the requirements of conventional road material specifications, especially for structural pavement layers. They are typically excluded because of their particle size distributions, although their plasticity and strength characteristics may fulfil the specified requirements. However, when appropriately designed, specified and constructed, sands can be, and have been, successfully used in practice in Australia and Brazil for all layers in a LVR pavement and, in Botswana, as subbase and, experimentally, as a base course.

5.2 Design

5.2.1 Dynamic Cone Penetrometer (DCP) method

The DCP design method is an alternative to the more traditional laboratory CBR based method of design. The main features of this method are its relative simplicity and ease of use and its ability to undertake a large number of in situ strength measurements quickly and cost-effectively. The method makes use of optimising the material properties in terms of the required strength at the designed moisture and density condition based on material properties obtained from DCP testing. This involves both field and laboratory testing following the procedure summarised in Figure 5-1.

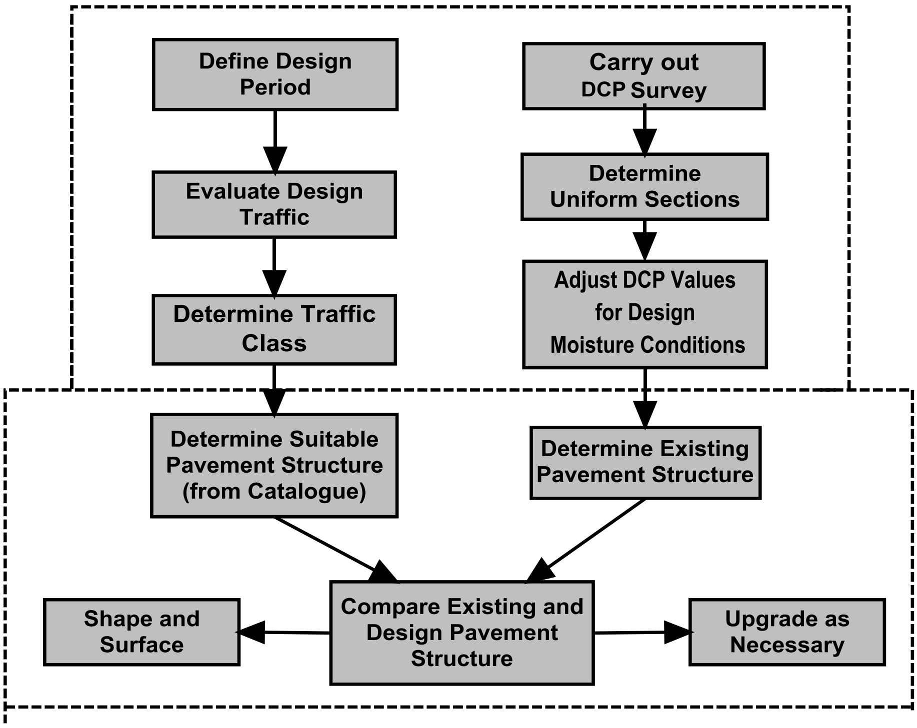


Figure 5-1: Flow diagram of DCP design procedure

In the field, the existing pavement support/conditions are determined using the DCP and these are compared with DCP design catalogues (layer strength diagrams) (Figure 5-2) that relate to various traffic categories and the prevailing local climatic conditions. This data is augmented with laboratory determined DCP penetration values obtained on the specific sands (selected on the basis of the Wylde chart classification) at similar moisture and density conditions (Figure 4-25).

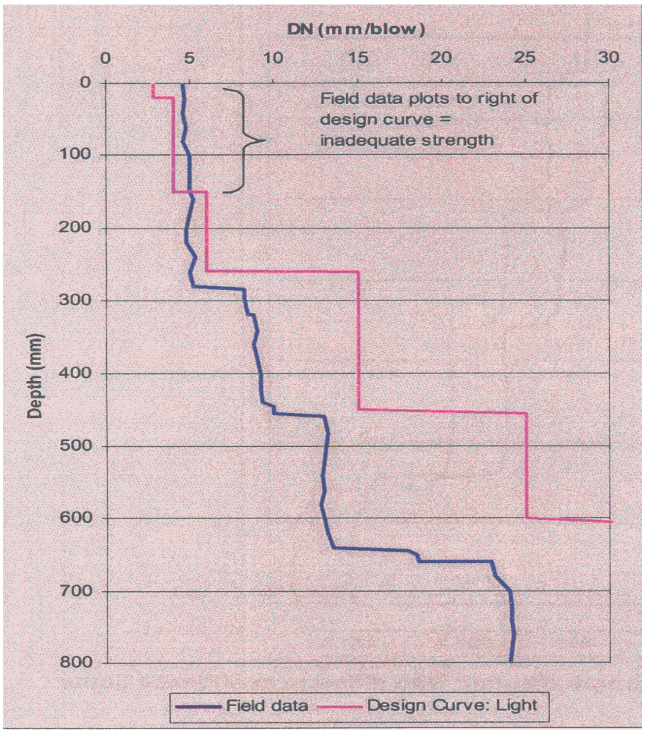


Figure 5-2: Comparison of DCP design profile and in situ strength profiles

5.2.2 Pavement design catalogue

The DCP design catalogue is presented in Table 5-1 and the equivalent layer-strength diagrams for the different traffic classes are illustrated in Figure 5-5 for different traffic categories.

Table 5-1: DCP design catalogue for different traffic classes*

Traffic Class E80 x 10^6	LE 0.01 0.003 – 0.010	LE 0.03 0.010 – 0.030	LE 0.1 0.030 – 0.100	LE 0.3 0.100 – 0.300	LE 0.7 0.300 – 0.700	LE 1.0 0.700 – 1.0
150mm Base ≥ 98% MAASHTO	DN ≤ 8	DN ≤ 5.9	DN ≤ 4	DN ≤ 3.2	DN ≤ 2.6	DN ≤ 2.5
150-300 mm Subbase ≥ 95% MAASHTO	DN ≤ 19	DN ≤ 14	DN ≤ 9	DN ≤ 6	DN ≤ 4.6	DN ≤ 4.0
300-450 mm subgrade 93% MAASHTO	DN ≤ 33	DN ≤ 25	DN ≤ 19	DN ≤ 12	DN ≤ 8	DN ≤ 6
450-600 mm In situ material	DN ≤ 40	DN ≤ 33	DN ≤ 25	DN ≤ 19	DN ≤ 14	DN ≤ 13
600-800 mm In situ material	DN ≤ 50	DN ≤ 40	DN ≤ 39	DN ≤ 25	DN ≤ 24	DN ≤ 23

The design catalogue for this traffic category is only valid if low moisture conditions can be guaranteed in the pavement in service. If there is a risk of moisture ingress the pavement design should be based on the soaked condition.

* Full details of the DCP design method may be found in Ministry of Transport and Public Works, Malawi. 2013. *Design Manual for Low Volume Sealed Roads using the DCP Design Method.*

5.2.3 Layer strength diagram

Figure 5-3 shows the layer strength diagram (LSD) for various traffic categories in terms of the DN values at a specified density and moisture content. The LSD for a particular traffic category indicates the required material properties, as represented by the DN values for each 150 mm layer to a depth of 800 mm. As illustrated in Figure 5-2, the comparison between the in situ strength profile of the existing pavement structure and that of the designed pavement is used to determine the appropriate design requirements.

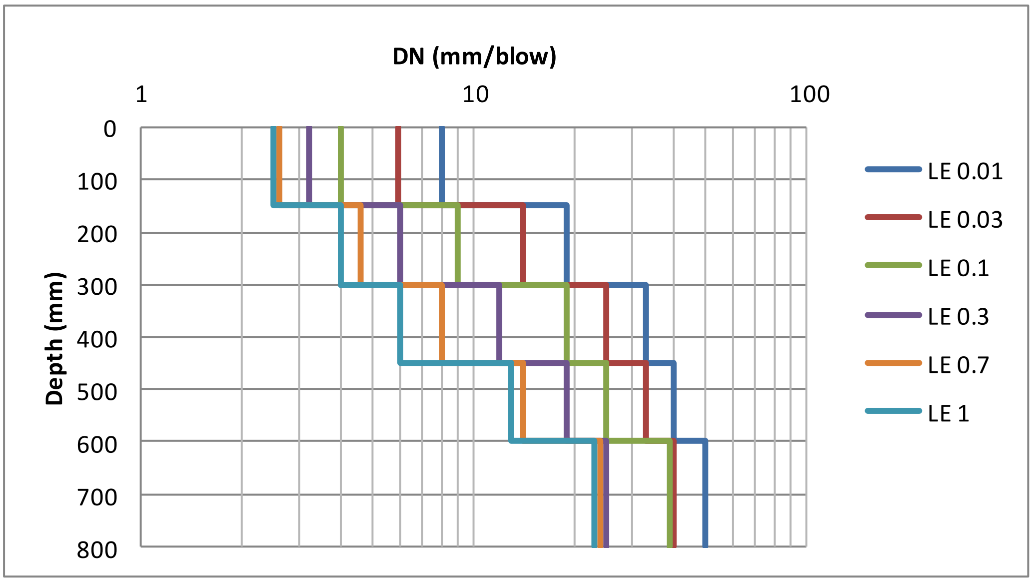


Figure 5-3: LSD for various traffic classes

Full details of the DCP design method are presented in the Malawi *Design Manual for Low Volume Sealed Roads Using the DCP Design Method* (AFCAP, 2013)

5.2.4 Risk factors

It is essential that the following main risks associated with the design of pavements using sand are fully appreciated and that appropriate measures are taken in practice to minimise them.

- Drainage (crown height, cross-section design, sealed shoulders, permeability (internal drainage) (see Section 6-4).
- Materials quality (strength)
- Construction control (supervision, thickness control, compaction standards).
- Maintenance (routine, periodic)
- Traffic (overloading – axle load control)

If any <u>one</u> of the above factors is relaxed, the risks assumed may be acceptable if strict control is exercised over the other factors. However, if more than one of the above factors is not adhered to, there is an increased risk of failure.

5.3 Specification of sands

Apart from the recently constructed trial section in Mozambique, no other sections of road have been in constructed in that country or in Namibia using neat sands as base course so as to be able to estimate or predict their performance in service. For the purposes of this project, the sands in Mozambique and Namibia have been compared with the Australian and Zimbabwean specifications and the limited data obtained from road experimental sections in Botswana and South Africa.

5.3.1 Australian specifications

The specification for red clayey or silty sand used successfully in Western Australia as base course on well drained sealed roads is set out in Table 5-2:

Table 5-2: Australian specification for red clayey or silty sand for use as base course

Design Traffic (ESA) Rainfall Deficit (1) (mm)		< 0.1 MESA > 2500	< 0.03 MESA > 2500
	Sieve size (mm)	**% Passing**	**% Passing**
	4.75	100	100
	2.36	70-100	70-100
	1.18	50-79	50-100
	0.60	36-63	36-100
Grading	0.425	30-56	30-84
	0.300	25-50	25-71
	0.150	18-40	18-50
	0.075	13-31	13-35
	0.0135	5-15	5-15
Dust ratio (2)		0.2 – 0.6	0.2 – 0.6
LL (%) PI (%) LS		≤ 20 ≤ 8 1-3	≤ 20 ≤ 8 1-3
CBR (3)	Unsoaked	≥ 80	≥ 80
WAACT (4) (5)	- Class No.	≤ 2.0	≤ 2.3
	- Cohesion kPa	≤ 85	≤ 85
	- Tensile strength (kPa)	≥ 55	≥ 55
	- Horizontal separation of Class No. Contours (%)	≥ 1.3	≥ 1.3
MDD (6) (kg/m^3) OMC		≥ 2100 5-7	≥ 2100 5-7
Al_2O_3 + Fe_2O_3		> 8?	> 8?

Notes:

(1) Rainfall deficit = potential evaporation – annual rainfall

(2) Dust ratio = P075/P425

(3) CBR carried out on specimens compacted at 90-100% OMC to 95% MDD (modified)

(4) Western Australian Confined Compression Test (WACCT). Class number, cohesion and tensile strength assessed at a dry density of 96% (modified compaction) and moisture content of 60% OMC.

(5) The criteria of the assessment by WACCT may not be met by testing specimens immediately after compaction. In these cases the specimens should be compacted at 100% OMC, dried to the design moisture content and cured for three weeks without further loss of moisture prior to testing.

(6) Maximum dry density. Test method WA 133.1, similar to Modified AASHTO.

(7) Al_2O_3 + Fe_2O_3 determined by ICP on the P425 fraction

5.3.2 Zimbabwean specifications

The Zimbabwean specifications for the use of sand as base course are contained in the Zimbabwe Standards for Base and Subbase Material in Stockpile (1979) and are presented in Table 5-3. These specifications apply to single lane (3.5 m) roads with a minimum base and subbase thickness of 120 mm.

Table 5-3: Zimbabwean specification for sand base course (stockpile material)

Design Traffic (ESA)		**< 0.1 MESA**	
		Base	**Subbase**
Grading	**Sieve size (mm)**	**% Passing**	**% Passing**
	2.36	100	100
	1.18	70-90	70-90
	0.425	35-65	35-65
	0.150	18-38	18-38
	0.075	8-25	8-25
Coarseness Index (I_C)(1)		0-30	0-25
Max. Reject Index (I_r)(2)		10	15
Max. PI (%)		S.P (2)	6
Max. Pp (3)		60	150
Texas Triaxial (4)	Class No.	≤ 3.3	≤ 3.6
CBR (5)	Soaked	45-55	35-45

Notes: (1) Coarseness Index = [(P2.36 to P37.5) * 100]/P37.5
(2) Reject Index = (P250 to P37.5)/P250
(3) Plasticity Product = PI_{P425} x P_{075}; when NP, Pp = P075
(4) Triaxial test values are lower quartile
(5) Approximate equivalent CBR values

5.3.3 Botswana specifications

The most recent specifications for the use of neat sand as base course have been developed from research work in Botswana (Botswana Roads Department, 2010) where, on the basis of one limited 100m test section, the following performance-based requirements have been derived for the prevailing road environment:

- Atterberg Limits: $5 < BLS_{0.075}/\varphi_{mean} < 10$
- Min soaked CBR @ 100% Vib Hammer 60%
- $Al_2O_3 + Fe_2O_3$ (%) > 8 (XRF method)

This Atterberg Limits are derived from the Zone B materials in the Wylde chart. As discussed in Section 3.2.2, these materials were found to perform satisfactorily as neat base course, as was the case for the recently constructed, but as yet unproven, ANE section in Mozambique. As discussed earlier, it is suggested that this requirement is used as a preliminary screening mechanism in order to identify potentially useful sands for construction.

5.3.4 Discussion of specifications

None of the specifications presented above are directly comparable. Nonetheless, certain similarities are evident as summarised below:

(1) **Grading:** The grading envelopes for the Australian and Zimbabwean specifications are presented in Figure 5.4 on which are superimposed the grading envelopes for the Zone B sands for Mozambique and Namibia.

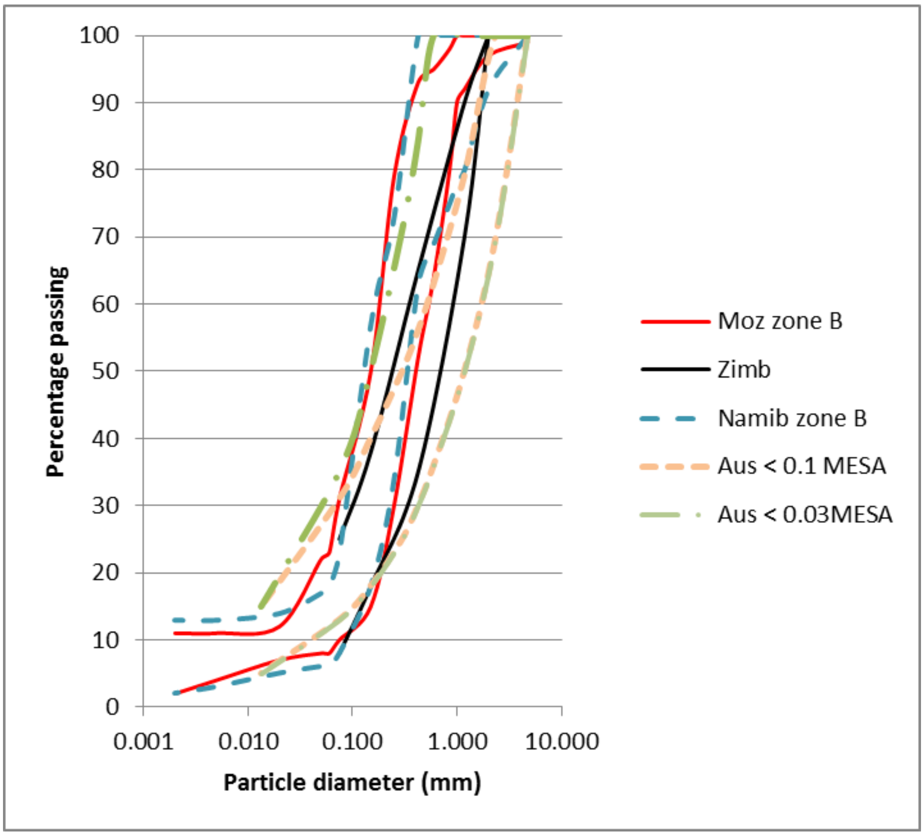


Figure 5-4: Grading envelopes for sands

As indicated in Figure 5-4, the finer fractions (P0.150 to P0.075) of the Mozambique and Namibia Zone B sands are reasonably similar to the Australia and Zimbabwean specifications while for the coarser fractions they generally straddle them. It is noteworthy, however, that neither the Australia nor Zimbabwe specifications consider the grading of the silt and clay fractions of the sands as determined by a hydrometer analysis. Importantly, the properties of the sands in these finer fractions could be very significant in terms of affecting their performance. Thus, the particle size distribution of sand based on the sedimentological Phi scale, as used in this project, is a much more discriminating factor than the conventional wet sieve analysis down to the sand fraction only and on which the Australian and Zimbabwean specifications are based.

Plasticity: The PI on the P425 for the Zone B sands for Mozambique and Namibia are all compliant with the Australian and Zimbabwean specifications in that they are all < 8. However, the determination of the plasticity in the conventional manner (PI measured on the P425), as adopted in the Australian and Zimbabwean specifications, will mask the plasticity that can be mobilised in the silt and clay fraction of the P075 materials.

Thus, the PI on the P075, as used in this project, is of much more significance when sands are being considered for use as pavement materials.

(2) **Strength**: The CBR values of the Zone B sands for both Botswana and Mozambique (no CBR testing on the Namibia sands as none were used neat as base course) are compliant with the Australian and Zimbabwean specifications in that they are > 80 at OMC.

Of the 100 sand samples tested, 14 of them from Mozambique and about 21 of them from Namibia plot within Zone B (or an extrapolation of it). However, only one of the Namibian sands and 4 of the Mozambique sands comply with the bar linear shrinkage:mean size ratio on the P075 and the mean particle diameter (in terms of the Φ-scale) included in the Botswana specification. This ratio should be between 5 and 10.

Non-compliance of the Mozambique and Namibia sands with the bar linear shrinkage:mean size ratio on the P075 of the preliminary Botswana specification is not necessarily an indication that the sands are not "fit for purpose". In fact, numerous investigations in the region have shown that, while traditional design approaches specify a number of laboratory derived engineering properties of the materials, such as grading, PI and CBR, these properties often do not correlate significantly with the performance of LVSRs. It is for this reason that the preferred approach is to express the required quality of the sands in terms of its DCP resistance to penetration (DN value in mm/blow) which is an indicator of overall shear strength that is sensitive to density, moisture content, particle strength, grading and plasticity. Thus, the DN value must be specified at the compaction density and expected in service moisture condition.

The use of the DN value as the primary materials selection criterion for sands does not detract from the importance of grading and PI on the performance of the material in service but, rather, it places secondary importance on these parameters. Thus, a poorly graded, highly plastic material would not be expected to provide the relatively low DN value that might be specified for the base course layer of a low volume sealed road.

In view of the above, the two materials parameters that need to be specified for the sand pavement layers are as follows

(a) **Density**: The density to which the material in the upper/base layer must be compacted should be the highest that is practicable, i.e. "compaction to refusal" such that adverse break down of the material does not occur.

(b) **DN value:** The DN value of the materials to be used in the upper/base layer of the pavement at a specified density and moisture content for a particular design traffic loading (ref. Table 5-1 and Figure 5-3).

A proper evaluation of the suitability of the materials for incorporation in the pavement will require a knowledge of the DN/moisture/density relationship as illustrated in Figure 4-21.

5.4 Use of untreated sands

5.4.1 Subgrade

Many roads constructed in the SADC region will have predominantly sandy subgrades for which the DN requirements are indicated in Table 5-1. Most sands when compacted to the appropriate density will easily comply with the specified requirements. However, there may specific requirements pertaining to materials with potentially collapsible grain structures (Clause 5-102.5.2 (Ministry of Works, Transport and Communications, 1982).

Pavement subgrades usually fail through being too weak to support the applied loads, through excessive volumetric movement or as a result of insufficient compaction. Provided sand subgrades are compacted to the required standard, their chance of failure is relatively low.

Where subgrade compaction problems are encountered, particularly with the more uniform sized sands and manifested as a tendency for shearing of the material during compaction, particularly when excessive vibration is used (Chauvin, undated; Murphy, 1981) a method specification based on controlled compaction trials should be considered.

Recent work in Botswana has shown that roads with Kalahari sand subgrades resulted in deep pavements with n-exponent values (for carrying out traffic equivalency calculations) of less than 2 (Paige-Green and Overby, 2010). This has a significant impact on the determination of equivalent standard axles in pavement design over sand subgrades.

5.4.2 Fill and selected layers

Materials for fill generally require similar quality as subgrade, the only difference being that they are imported to provide the required pavement shape and levels. Discussion pertaining to in situ subgrade is generally relevant to fill materials, although most collapse potential of the material would generally be lost during extraction from borrow, haulage and compaction.

The non-cohesive nature of the sands, however, can result in compaction difficulties because of non-confinement. This may require construction of a wider fill in order to provide a greater degree of confinement in the central pavement area and rolling from the outside inwards only, as should be standard practice on all layers.

Most sands will comply with the standard fill requirements but good supervision and control of compaction is essential. It is recommended that the materials be compacted to refusal, rather than the 90 or 93 per cent of BS 1377 Test 14 that is specified, provided that refusal density is in excess of the specified limit. The South African requirement for sand (with less than 20 per cent finer than 0.075 mm) in roadbed and fill is a minimum compaction of 100 per cent Mod AASHTO density (COLTO, 1998).

Low densities in fills will result in settlement if the fill becomes submerged and traffic continues to use the road. The presence of perched water tables can also lead to this problem.

5.4.3 Subbase

Many sands will provide the specified DN values for subbase when compacted to an appropriate density. Such sands have been successfully used on a number of projects in the region.

5.4.4 Base

It is not uncommon for certain untreated sands to exhibit relatively high DN values at the equilibrium moisture content that typically prevails under paved roads in the southern African region (Emery, 1992), and these materials when adequately compacted in the field are likely to perform satisfactorily. Figure 5-5 illustrates the variation in laboratory CBR strength (reasonably well correlated with DCP CBR value) with moisture content for a red Kalahari sand that was used successfully as neat base on an experimental section of a low volume road pavement in Botswana (the Serowe-Orapa experimental section). The sand sample was compacted at OMC and dried back to various moisture content ratios before its strength was determined.

The reason for the good performance of the red sand has been attributed to the high suctions that the material exhibits (due to its fine grading and the relatively high plasticity of the minus 75µm fraction) and the resulting high strengths when dry.

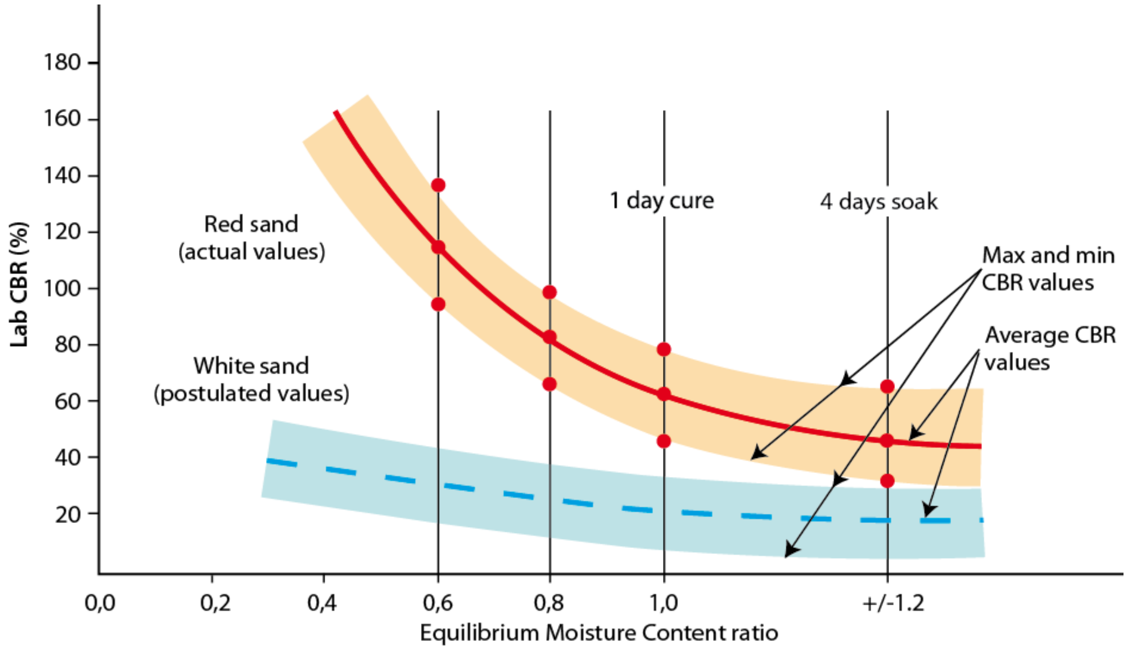


Figure 5-5: Lab CBR of Kalahari sand sample at various moisture content ratios

5.4.5 Surfacing

Sand is generally too fine and uniform for chip seals and is unsuitable for use, even for conventional sand seals. They have been used locally in Zimbabwe as fine aggregate for bituminous surfacings, but no details regarding this have been located. Experience in Botswana indicates that Kalahari sand seals without a surface dressing or graded gravel seal beneath them deteriorate within a few months due to cattle damage (Overby, 1982).

Sand has, however, been used regularly as a cover seal on single (and occasionally on double) Otta seals (Botswana Roads Department, 1999). Typically a 60 % Emulsion or an MC3000 binder is sprayed at between 0.7 and 1.0 ℓ/m^2 and immediately covered with Kalahari Sand at a spread rate of about 0.014 m^3/m^2. This must be rolled with a pneumatic tyred roller, ensuring that the sand is evenly spread and rolled. A light drag must be used to remove any unevenness. The road should be immediately opened to traffic. Sand dislodged by traffic should be broomed back onto the road until the balance between binder and sand is correct and no further sand is lost or bleeding occurs.

The use of Kalahari sands as a sand seal has also been described in Namibia (Marais and Freeme, 1977) where a single sand seal of windblown sand was placed on a cutback binder (sprayed at 1.37 to 1.76 ℓ/m^2) after allowing it to penetrate the bitumen stabilized sand base and thicken. No traffic was allowed on the seal for about 2 months and the seal gave good service for about two years.

The use of primer seals in Australia is almost standard practice in rural areas – these often consist of local sand placed on a heavy bituminous binder primer, which is provided as an initial waterproof wearing course until a conventional chip seal is placed during the hot and dry part of the year (Pederson, 1978). Local river and dune sands have also been successfully used as aggregate for seals (Pederson, 1978).

5.4.6 Wearing courses for unsealed Roads

Sands are not generally suitable as wearing courses for unsealed roads. Although the fines of some sands, particularly the red ones which contain iron oxide, also have some plasticity (cohesion), it is generally insufficient to resist the abrasive forces of moving wheels. Moreover, sands tend to erode easily with rain and to corrugate rapidly, particularly in dry conditions, and to deteriorate to thick, loose layers that are impassable to most vehicles.

The sands are, however, very effective for sand cushioning (Jones, 1995). In this process, a well-compacted gravel wearing course (usually of marginal quality) is covered with a thin layer (about 40 mm) of relatively single-sized sand. This upper sand layer is maintained on a regular basis, using a towed grader and drag, such that vehicles do not traffic the base material.

5.4.7 Shoulders and side slopes

The use of sands on shoulders and side slopes can result in severe erosion by rainfall, road surface runoff and wind. A reduction of the pavement and shoulder crossfall to between 1.5 and 2 per cent reduces the velocity of runoff and minimises the potential for erosion. Sideslopes should be constructed to a maximum batter of 1: 4 or flatter. Although providing structurally sound shoulders, the problem of erosion of sand-clay shoulders by vehicles and wind generated by vehicles is severe in Australia (Jewell, 1970; Paige-Green, 1983).

Significant problems with wind erosion of sand shoulders in Australia have been observed. The wind-shear generated by rapidly moving vehicles, particularly large goods vehicles, results in detachment of sand grains from the shoulder and loss of shoulder shape. This is exacerbated on roads with reduced widths, where a tendency for vehicles to move their outside wheels onto the shoulder when overtaking results in wear of the shoulder.

The application of a bitumen seal to the shoulders, although increasing the cost of the road, significantly reduces problems related to erosion of the sand.

The establishment of vegetation to protect sand slopes on fill is usually not practical or reliable enough in the short term. Erosion can be minimised by placing a gravel cladding layer on the slopes where such material is available. Poor quality calcretes and silcretes that are not suitable for use in pavement layers will usually perform adequately as side slope cladding in the short term while vegetation establishes itself.

5.4.8 Saline sands

Sands are not usually saline and will therefore not usually cause salt damage. Their fine nature, which is likely to encourage capillary rise of saline water, and their potential to contain high air voids, however, make them susceptible to damage by salt crystallisation (Botswana Roads Department, 2001). Their use in saline environments (e.g. fill over saline materials or when saline water is present or used for compaction) should therefore be treated with caution (Netterberg and Bennet, 1993).

Due to the cushioning effect of fine-grained sands, they are well suited for use as a thin cushioning layer below or above a salt blocking membrane (e.g. plastic sheeting or bitumen used to prevent the upward migration of saline water to a pavement layer).

5.5 Stabilization of Sands

5.5.1 General

Various types of soil stabilization have been undertaken in the Southern African region for over 50 years in a variety of circumstances and for a variety of reasons using a variety of stabilizing agents as summarized in Table 5-4.

Table 5-4: Soil stabiliser and related type of stabilisation

Stabilizer	Type of stabilization
• Granular materials	Mechanical stabilization
• Portland cement	Chemical stabilization
• Lime (quicklime and hydrated lime)	
• Pozzolans (fly ash)	
• Bitumen and tar	
• Proprietary chemicals	

The choice of stabilizer is influenced by the properties of the material to be stabilized and the cost of stabilization. The traditional guidance to the choice of stabilizer is summarized in Figure 5-6. More recently, combinations of both bitumen and cement have been used, prompted by the evidence that suggests that adequate strengths can be achieved while cracking is substantially reduced due to a much lower cement content.

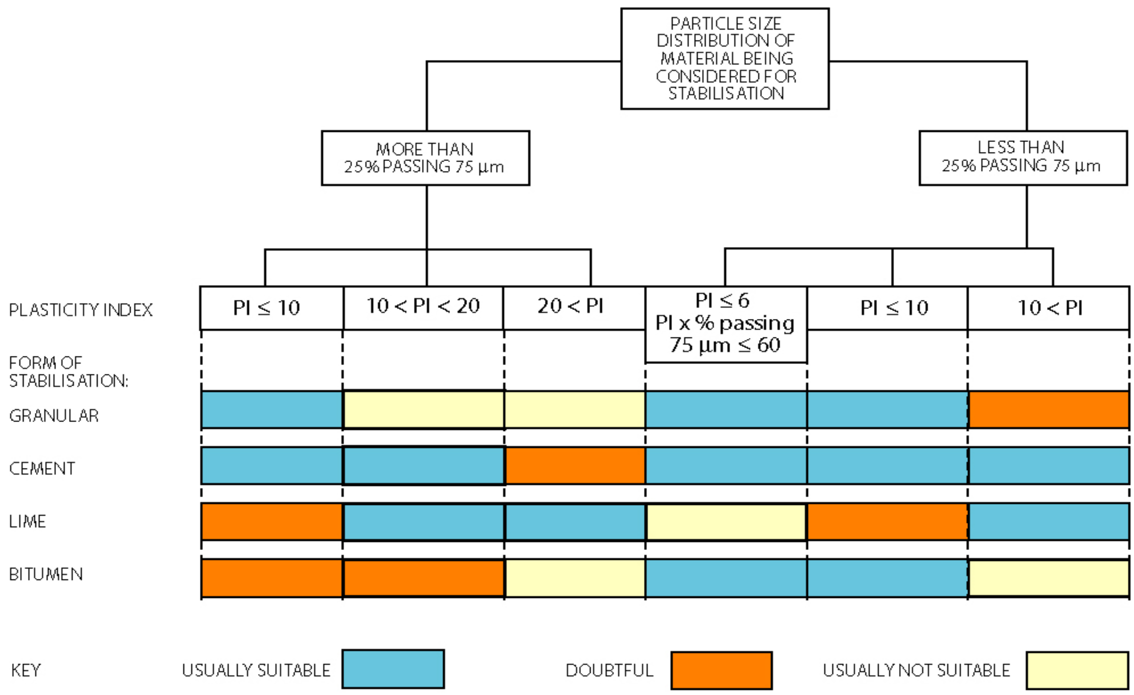


Figure 5-6: Guide to the method of stabilization (Austroads, 1998)

5.5.2 Mechanical stabilisation

Mechanical stabilization is one of the cheaper means of improving the quality of sands. Such stabilization can be achieved by blending sand with other materials with the following beneficial results:

- improved CBR
- lowering of PI
- lowering of OMC
- improved workability
- lowering of soluble salt content where salts are present

For example, blending of windblown sands with between 20 and 40 per cent (typically about 25 per cent) clayey silt or clayey sand has been successfully used in Australia (Leach, 1960). In Botswana, Kalahari sands have been blended with marginal quality calcretes to improve the quality of the calcrete gravel material. Material with a plasticity index of about 9 per cent increased in CBR by about 30 percentage points to in excess of 80 per cent after blending in 20 per cent Kalahari sand –see Figure 5-7 (Overby, 1981; Strauss and Hugo, 1979).

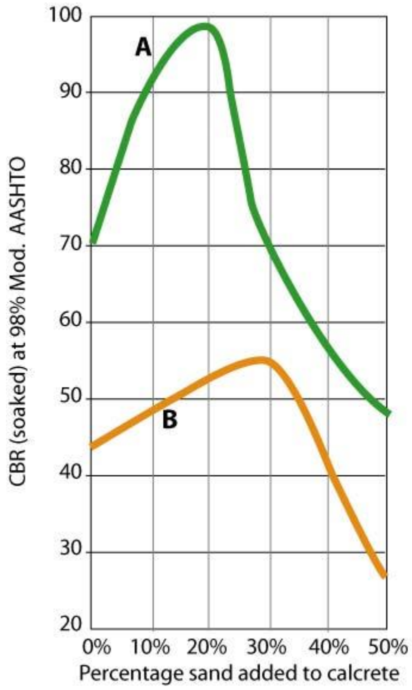


Photo 5-1: Blending of nodular calcrete and sand

Figure 5-7: Results of mechanical blending of calcrete with sand

5.5.3 Lime stabilization

Most sands do not have sufficient clay to react adequately with lime, and cement is the usual choice for stabilization. However, should there be adequate quantities of amorphous silica in the sand it will react with the lime to form a strong material adequate for use in a LVR pavement. It is possible (and often environmentally more friendly) when the lime does not react with the sand to add a pozzolan such as flyash, ground granulated blastfurnace slag (GGBS) or even such products as rice husks or bagasse as a stabiliser. In this case all of the reactive components are provided.

Lime can be obtained from commercial sources as well as by burning limestone, shells or other calcareous materials in small kilns. Success using such lime has been obtained, but the variability of these needs to be carefully controlled.

There has been mixed success with the use of lime in the Southern African region for which some examples are presented below.

Namibia: In the 1980s, lime was used on a sandy calcrete section of the Grootfontein-Rundu road in which both the base and subbase were stabilized (3% and 2% respectively). However, some time after construction, shallow base and surfacing failures were reported and attributed to the degradation of the stabilizing agents and their cemented products during exposure before sealing through the process known as ”carbonation” which can both inhibit the effective formation of, and destroy the cementitious products in soil-cement and soil-lime reactions (Netterberg and Paige-Green, 1984). In spite of this, after such areas were repaired the road performed satisfactorily and has only recently been reconstructed. Many other roads in Namibia with lime stabilised abses have performed well (Netterberg, 2012, pers. communication).

South Africa: Bergh et al (2008) describe the effect of lime treatment on the Berea Red Sands found in South Africa for which CBR values of untreated material were between 18 and 28% at 98% Mod AASHTO and increased to in excess of 90% with up to 4% lime. Samples of the stabilised sands that were collected from these roads indicated that they were still in relatively good condition after more than 30 years in service and they still exhibited significant strength. In contrast, at least one section of road with a lime stabilized Berea red sand is known on which scabbing of the surfacing occurred due to a weak carbonated upper basecourse (Netterberg, personal communication, 2012).

5.5.4 Cement stabilisation

Cement stabilization provides a high strength, relatively stiff and inflexible pavement. This type of structure requires good support to limit the total pavement deflection to low values and to ensure that the layers do not fail due to excessive flexing. It is suggested that n- values in excess of 4 (5 or even up to 6) be used in the calculation of cumulative standard axles for design purposes as suggested in CSRA (1996) and shown by Paige-Green (2010).

Cement stabilised layers usually produce marked block cracking which can be the result of traffic loads as well as shrinkage stresses (Bofinger, 1971). Shrinkage is an unavoidable problem but it should be controlled as much as possible to reduce the damage it causes. Unless sands have a high clay content, compacted sand-cement mixtures will normally expand if they are sealed to prevent any drying. If the cement content is increased the expansion will also increase. As soon as moisture is allowed to escape from the material it will start to shrink. If the material remains sealed it will continue to increase in strength and the potential total shrinkage will decrease as will the severity of cracking. If pavements are properly cured for one or two months the drying shrinkage is likely to be reduced by 50%. However, in practice, this is unrealistic. It is also recommended that sand cement mixtures are compacted at a moisture content no more than 80% of the saturation moisture content of the untreated material. Although cement contents as high as 10% have been used in sand-cement layers (this is usually uneconomical) the cracking has not necessarily been unacceptable.

The rate at which a sand-cement layer dries out depends on the method used to cure it. Most of the moisture lost from a newly constructed sand-cement layer dries out through the surface so the curing method becomes critical. For many years the most common curing methods have been to spray water on the top of the layer two or three times a day or to spray a cut-back bitumen curing membrane on top of the layer. Both of these methods are completely ineffective for sealing moisture in the layer and they also encourage carbonation from the top of the layer. If 30 to 40mm of untreated sand is placed on top of the sand-cement layer and then sprayed three times each day the loss of moisture is reduced to almost zero. Another excellent curing system is to apply a heavy spray of slow-breaking emulsion on top of the freshly constructed layer. However, this method can cause problems with bleeding in a thin bituminous surface placed on top at a later stage.

The surface of roads with sand-cement bases and the cracks which form in them must be carefully sealed to prevent the ingress of water and the pumping of fines from the base and lower layers. Care should also be taken to ensure that sands for stabilisation with cement do not contain excessive deleterious organic material which can interfere with the cementitious reactions.

There has been mixed success with the use of cement in stabilised sand bases in the Southern African region for which some examples are presented below.

Mozambique: Cement stabilization has been used extensively since the mid-1960s on a number of trunk roads in Mozambique, such as the EN1 road south of the Save river where very high cement contents (typically 8%) were required to achieve the required strength. However, such high contents of stabilizer have resulted in significant shrinkage cracking, leading to reflective cracking propagating through the surfacing and consequent high maintenance requirements (Carvalho, undated).

Examples have also been reported of failures in the top few millimeters immediately beneath the surfacing layer of sands stabilized with 2% cement (Carvalho, undated). This had caused the surfacing to become loose, followed by a tendency to strip off. Although not diagnosed as such, this type of failure has the hallmarks of degradation of the pavement layer due to carbonation as reported by Netterberg, 1987, 1991 and Paige-green et al, 1990.

More recently, cement stabilization has been the subject of research which aimed to characterize cement stabilized sand bases under Accelerated Pavement Testing (de Vos, 2007). The main objective of such work is to develop more appropriate pavement design methods for Mozambique which take account of the mostly single sized sands found in the coastal plains and dunes in the country. The development of these new designs is still a work in progress.

Botswana: Cement stabilization of sand has been relatively little used in Botswana. There is one reported example of a 25km section of the AI highway between Artesia and Dibete which was stabilized with 3% cement and which has apparently performed satisfactorily for more than 25 years. However, there was scabbing of the surfacing in areas due to a weak, carbonated upper basecourse (Netterberg, personal communication, 2012).

5.5.5 Bitumen stabilization

Bitumen is used commonly as a stabilizing agent for many materials. However, it is noteworthy that neither of the older Guides to Bitumen Stabilized Materials (GEMS) (Sabita, 1993) or bitumen modified bases (ETB) (Sabita, 1999) include the possible use of sands, probably more through a lack of information than by design. However, the most recent Guide (Asphalt Academy, 2009) includes a material class (BSM3) that caters for the use of sands.

Successful stabilization of sands has been carried out in a number of countries in Southern Africa and the use of foamed bitumen is increasing in popularity and success with this technique using sands for low volume roads having been reported (Paige-Green and Gerryts, 1998). Some examples of the use of bitumen for stabilization of sands are presented below.

Mozambique: In Mozambique hot sand asphalt was used up to the early 1970s before the war of Independence (van Wijk and Carvalho, 2003). More recently, cold bitumen sand mixes (BSM) also referred to as sand treated with emulsion (STE) were constructed as a demonstration project at Marracuene (Guiamba et al, 2010) with the aim of producing a relatively flexible bitumen emulsion/sand admixture (5.6% emulsion) that would be more forgiving to axle overloading and less susceptible to shrinkage cracking commonly associated with cement stabilization.

Experimental trials of emulsion treated base design in accordance with the Technical Guide: Bitumen Stabilised Materials (Asphalt Academy, 2009) and comprising 4% - 8% emulsion and up to 1% cement have also been constructed as part of the AFCAP project - Targeted Interventions on Low Volume Rural Roads Project – see Photo 5-2). These trials are being monitored and, since construction in 2012, have performed satisfactorily.

Photo 5-2: ETB construction-plant based approach

Namibia: Various types of sand bitumen stabilization have been carried out in Namibia over many decades using cutback tar and bitumen (emulsion and foam) (Dierks, 1992). For example, on trunk road 8/6 from Kongola to Katima Mulilo (Trans-Caprivi Highway) a mixture of Kalahari sand and calcrete nodules was stabilized with 8% bitumen (Strauss and Hugo, 1979) and the road is reported to have performed satisfactorily over its design life.

Botswana: Bitumen stabilization has been undertaken only in the form of experimental sections. Such trials were undertaken at Jwaneng in 1980 where both in situ cold mix and foamed asphalt was used and on the Serowe-Orapa road where various percentages of bitumen emulsion were used with different types of sand. The lessons learnt from the monitoring of these sections indicated the scope for using this type of stabilization on full scale projects (Netterberg, 1998).

South Africa: Bergh et al (2008) describe the effect of bitumen emulsion treatment on the Berea Red Sands. The materials tested had CBR values of 30 to 60 % in the untreated form which increased to between 58 and 70% on treatment with between 1 and 2% anionic bitumen emulsion with 1% cement. It was notable that by adding some -9.5 mm graded aggregate, both the natural and emulsion treated CBR strengths increased significantly (73 to 96 and 112 to > 120% respectively). Emulsion treated sands have also been used successfully in South Africa (Henderson, 1988).

5.5.6 Bitumen and cement:

In recent years, bitumen emulsion has become more readily available and is increasingly being used together with cement for stabilization of sands. International evidence suggests that adequate strengths can be achieved, and cracking can be substantially reduced, due to a much lower cement content. However, there is still insufficient experience to determine whether pavements constructed in this way will be sufficiently durable over the intended life of the road, and to establish the optimal proportion of cement and bitumen binder.

Mozambique: Bitumen/cement stabilization was undertaken on the section of the ENI between Chicumbane and Xai Xai where 2% cement and 3% bitumen were initially used but subsequently changed to a more conservative 3% cement and 3% bitumen due to uncertainties over the long term durability of the cement/bitumen emulsion combination of stabilizing agents. Significant sections of the road were surfaced with a chip seal and other section with hotmix asphalt.

Disconcertingly, a number of failures occurred along the recently completed Xai Xai Chizabuka section of the road in Gaza to complete failure of the Gorongoza-Caia section in Sofala and the Macomia-Oasse road in Cabo Delgado. The reasons for such problems have still not been ascertained and are the subject of on-going investigations.

Namibia: In the upgrading of the Trans-Caprivi highway, both cement and bitumen emulsion were used together for stabilization of the sandy-calcrete reclaimed old sand asphalt basecourse-Kalahari sand pavement. However, even during construction, shallow base failures occurred for a variety of reasons including inadequate mixing of the stabilizers, carbonation of the upper base layer, etc. (Netterberg, personal communication).

5.5.7 Other

(1) Polymers: There have been other instances where the quality of sands have been improved using different forms of stabilization. These include the use of proprietary chemical soil stabilizers and polymers which were investigated in Namibia (Mgangira, 2007) and from which it was concluded from laboratory results that some Kalahari sands have the potential to improve their performance when treated with synthetic polymers. However, the use of such polymers in the field and their long term performance and cost-effectiveness are yet to be ascertained.

(2) Tar: Although there have been many cases of the use of coke-oven tar being used as a stabiliser for road materials including sands (1970-1990), its use is not discussed further in this document owing to recent limitations on the use of tar due to its potentially detrimental health effects.

(3) Geocells: These are essentially a form of cellular confinement comprising a geo-synthetic material designed to confine soil or other cohesionless material. After tensioning, granular material, coarse aggregate or lean concrete is vibrated into the cells to produce a load distributing pavement layer.

Geocells have been used in a variety of applications in Southern Africa for which a number of case studies have been reported (Visser, AT and Hall, S. 2003). Geocells also find application in situations where very steep grades preclude the use of either gravel or bituminous surfacings. Although geocells are potentially suitable for use with sand for low volume roads, there are no reported full-scale examples of this application in the SADC region. However, interestingly, the US Corps of Engineers report in a sand road case study (Presto Products Company, 1991) the successful use of sand as the infill material for their Geoweb Cellular Confinement System (a type of geocell) in the construction of roads in the Algerian Sahara Desert.

Photo 5-3: Sand filling of geocell by front end loader

Photo 5-4: Use of sand-filled geocell by 40 tonne truck

5.5.8 Summary of experience with stabilization

Experience with the use of chemical stabilization as a means of soil improvement in Southern Africa has been very mixed. As a result of a number of reported failures in the region, studies were carried out by the CSIR (Netterberg, 1987, 1991; Paige-green, Sampson and Netterberg, 1990). These studies indicated that road failures could be attributed partly to the degradation of the stabilising agents and their cemented products through a process known as "carbonation" (Netterberg and Paige-green, 1984) in which distress manifests itself in roads constructed with stabilized bases as follows:

- 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

The above are inter-related effects since some lead on to others. The effects suggest that carbonation can contribute to premature distress in <u>some</u> circumstances.

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 stabilization process and some countries in the region discontinued the use of chemical stabilization in road projects.

Further studies carried out by the UK Transport Research Laboratory on the *Performance of Chemically Stabilised Road Bases* by the UK Transport Research Laboratory (Gourley and Greening, 1999) have indicated that road failures could be attributed partly to the degradation of the stabilising agents and their cemented products through 'carbonation'. However, they also emphasized that there are also many examples within Southern Africa where the use of chemical stabilization has been very successful, even on roads that have received little maintenance.

Experience has shown that, provided the initial consumption of stabilizer determined in the laboratory is exceeded, the potential for detrimental carbonation is minimal (Paige-Green, 2008). This is probably why there is conflicting evidence on the deleterious affects of carbonation. Only in cases where insufficient stabilizer has been added or the road has been insufficiently trafficked have significant problems been encountered. Nonetheless, this conflicting evidence has resulted in considerable uncertainty about the use of stabilization as an option for road projects.

More recently, a literature survey undertaken as part of a major project on the *Performance Characterisation of Cement Treated Sand Base Material of Mozambique* (de Vos, 2007). concluded that:

- A number of pavement design methods used worldwide are being used in Mozambique. Some are not necessarily applicable to Mozambican conditions resulting in the implementation of inappropriate designs and construction techniques leading to poor performance.
- The South African Mechanistic Design Procedure (SAMDP) is extensively used in Mozambique. However, the SAMDP is based on transfer functions developed for South African materials and conditions that have been found inappropriate for Mozambican conditions.

- The lack of knowledge about the appropriate engineering properties of locally available road building materials, mostly sands but also finely graded materials, is the main cause of unsuccessful projects. Main contributors to the premature failures of pavements appear to be:
  - specification of incorrect type of stablizer or incorrect stabilizer content;
  - inappropriate construction practices;
  - lack of understanding of behaviour and the long term performance of the materials;
  - very high axle loads and extreme climatic conditions;
  - inability to maintain roads due to lack of funds.

In view of the very mixed experience with the use of various stabilizers for use with sands and other materials in Southern Africa, it is apparent that there is an urgent need to undertake a study with the following objectives:

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

b) Evaluate the performance of chemically stabilised sand road bases in relation to current pavement design criteria

c) Make recommendations and provide Guides for the chemical stabilization of sands/sandy materials which are based on performance data, so that confidence is restored in this method of improving sands for use in road bases in the region for both low volume and high volume roads.

The above study would be most beneficial to a country such as Mozambique where sand is generally the only type of material that can be used for road construction in much of the country.

6. CONSTRUCTION ISSUES

6.1 Introduction

Construction of roads using sand should be viewed as a holistic activity that brings together all the factors that may affect both the short- and long-term performance of the road. Unless the specific aims and purposes of each factor, i.e. design, selection of materials, limits for moisture control, etc., are fully understood and achieved during construction, the end product will be unsatisfactory and a less than acceptable performance is likely to be obtained. The various factors that should be considered during construction are highlighted below.

6.2 General Guidance

General guidance for construction in Kalahari sand is provided by the Zimbabwe Manual – Part F: Construction (Ministry of Roads and Road Traffic, 1979) and may be summarized as follows:

- Due to the unstable grading and non-plastic properties of many sands, it is not considered practical or economical to attempt earthworks operations except during or soon after rains.
- As soon as the natural moisture content to a depth of at least 1 m below ground level reaches the limit at which compaction may be attempted, then the road bed must be compacted using the heaviest plant available. This will generally cause the road bed level to drop at least 100 – 150 mm.
- Fills should be kept as low as possible and formed with borrow material. Side drains will only be cut where necessitated by natural cross-fall. It will be sufficient if the lowest edge of the subgrade is kept 75 mm above original ground level. High fills may be compacted in large lifts or more using the appropriate (impact) compaction equipment.
- Particular care should be taken on all cuts and fills to avoid erosion damage and reinstatement of natural grass cover should be encouraged. Fills greater than 1.5 m in height should be clad with suitable cohesive material as soon as possible after construction.
- The sand subgrade cuts easily under traffic, and it is, therefore, necessary to keep the sand wet and graded to shape immediately in front of lorries dumping.
- Fill or shoulder slopes constructed of the lighter soils, unless cladded will usually require protection from road water, particularly on curves. This can be achieved by means of kerbed and channel shoulder drains discharging at intervals into paved flumes which carry the water beyond the toe of the slope. The whole slope must be planted with a suitable grass, a practice which is desirable on all fills, of whatever material.
- The face of every cut must be protected from surface water by means of cut-off drains.
- Where cuts have a large surface area in plan, it may be desirable to provide surface water drainage, by means of benches at a suitable slope, to take water to the extremities of the cut. Great care is required to ensure that these drains will neither silt up nor erode, otherwise, they may do more damage than they are designed to prevent.
- Erosion control in side drains having steepish grades is best achieved by paving or cement stabilizing the drain.

6.3 Compaction Aspects

6.3.1 Achieving maximum density

One of the critical aspects of using sands in road construction is to maximise their strength and increase their stiffness and bearing capacity by producing the highest possible density in the subgrade and pavement layers. This can be achieved, not necessarily by compacting to a pre-determined relative compaction level as is traditionally done but, rather, by compacting to the highest uniform level of density ("compaction to refusal" where possible (i.e. where there is no significant breakdown of the soil particles which is unlikely with sands as they are composed mostly of silica (quartz) – one of the minerals least likely to breakdown under compaction). In so doing, as illustrated in Figures 6.1 and 6.2, there is a significant gain in density, strength and stiffness, the benefits of which usually outweigh the costs of a few additional passes of the roller.

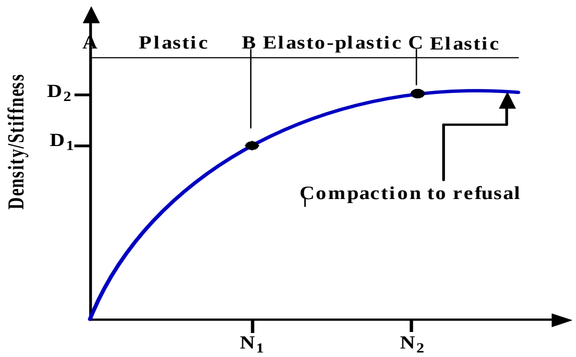


Figure 6.1 – Illustration of concept of "compaction to refusal"

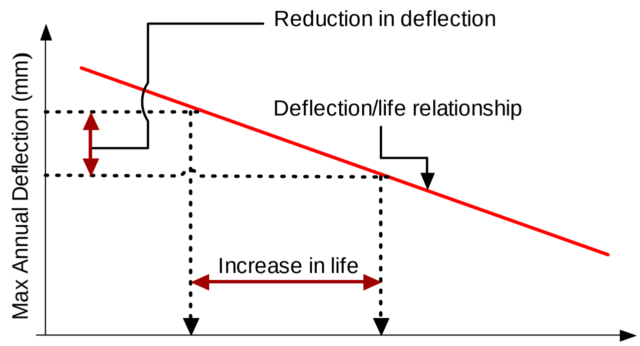


Figure 6.2 – Deflection/life relationship

The use of *dry compaction* (i.e., at natural moisture content, which could be between 3 and 4 percent) can produce the same density as conventional compaction at OMC. The air voids are, however, significantly higher in this material and the benefit of suction forces developed during drying back of moist compacted materials is lost. Moreover, should wetting up of the relatively high voided material occur in service, this is likely to lead to loss of strength and consequent deformation of the pavement. Thus, the implications of this should always be considered when dry compaction is a possible option.

6.3.2 Compaction moisture

The compaction moisture content is critical to the successful compaction of Kalahari sands. It is therefore imperative that strict control of moisture is exercised during compaction. This may be complicated by the fact the OMC differs for different compaction efforts and that the OMC determined in the laboratory may or may not necessarily be applicable for the plant actually used on site. Compaction trials are the best way of determining the OMC for any combination of plant. The recommended technique for this is to prepare strips of material at various moisture contents, partly compact these with the same number of roller passes and find the one with the highest density (i.e. OMC for the plant used). Another test strip is then compacted at the identified OMC to determine the optimum number of passes to achieve maximum density (Wylde, 1979).

It is also necessary to ensure that premature sealing does not lock in construction moisture. Allowing the material to dry back from its OMC before priming, can minimise the incidence of long term shrinkage cracks developing through the seal (Sandman, Wall and Wilson, 1974) as well as allow the sand to develop a large part of its suction strength (Pederson, 1988). It should be noted, however, that some sands tend to lose density on drying out but are quite firm while still wet. It would be advisable therefore to cover such sands as soon as possible after compaction.

6.3.3 Surface shear

A frequent problem observed during the compaction of essentially uniform sized sandy materials is the tendency for shearing of the material during compaction, particularly when excessive vibration is used (Chauvin, undated; Murphy, 1981). The use of heavy static rollers at slow speeds can overcome this but generally, it is necessary to carry out compaction trials using combinations of padfoot or grid rollers, pneumatic tyred rollers and vibratory compactors of different sizes to determine the optimum combination. Generally high amplitude, low frequency vibrating roller operations are more likely than low amplitude (< 1.1 mm), high frequency (> 35 Hz) operations to cause shearing (Murphy, 1981).

6.3.4 Density and strength

In order to maintain its density and strength, sand needs to be confined. This can be achieved by keeping the pavement levels as low as possible, but this is not always desirable, as the pavement drainage characteristics are reduced. Widening the layer to be compacted and using flat side slopes can achieve a similar effect whilst improving the pavement drainage and overall structural capacity. It may, however, be difficult in many circumstances to achieve high densities with some fine materials (Murphy, 1981) but as high a density as possible should be achieved. Frequently, compaction of fine materials can result in the creation of poorly bonded lenses of material, compaction planes and layers of fines on the surface – in these cases, the top 10 to 20 mm should be cut to waste (Murphy, 1981) and provision for this should be made in determining the compaction lifts. A weak interlayer between the base and the seal must be avoided at all costs, even at the expense of surface finish (Netterberg and de Beer, 2012).

6.3.5 Collapsible sands

The mitigation of collapse of sand requires a high degree of compaction. Using conventional plant (i.e., heavy vibrating rollers), the application of large quantities of water is necessary. Where compaction water is scarce or expensive, high-energy impact compaction to pre-collapse the Kgalagadi sands can be a cost-effective solution compared with more conventional vibratory rollers. This type of compaction produces a much deeper effect than conventional plant and relies on high energy to collapse the materials instead of moisture (see Figure 7.3). The use of high-energy impact compaction to pre-collapse Kgalagadi sands has produced good results (Pinard, 1989) and significant economies in terms of water usage can be achieved.

Photo 6-1: Compaction of subgrade/fill by impact compactor

Where collapsible sands exist, the bulk of the collapse potential should be removed from the upper 1 m of the subgrade to ensure stability of the pavement structure (Weston, 1980; Jones and van Alphen, 1980). This is normally difficult to achieve with conventional plant, although good results using such plant have been reported from Zimbabwe (Mainwaring, 1968) This approach typically requires that the sand is brought to a relatively high moisture content prior to compaction – often a costly and time consuming exercise in hot, arid environments. In these cases, construction should be programmed to ensure that subgrade compaction is carried out primarily during the rainy season.

Alternatively, impact compaction can be considered for the reasons stated above for compaction at depth. Because of the undulations left by the impact compactor (see photo opposite), the uppermost 100 mm of the compacted layer has to be finished off with a vibratory or static roller. Prior to such rolling, the top 50 – 100 mm of the impacted layer shall be cut off to remove all surface irregularities which would otherwise be reflected in the surfacing after some time.

Where the sands have such poor traction (e.g. the more single size white sands) that the impact compactor is not able to operate efficiently, it is necessary to use a 100 – 150 mm capping layer of gravel or better quality (more cohesive) sand. The pre-collapsing of the subgrade can then be applied from the top of the capping layer.

6.4 Construction Aspects

6.4.1 Bush clearing, grubbing and topsoil removal

Bush clearing, grubbing and topsoil should be undertaken in an environmentally and socially sensitive manner. Damage to the vegetation cover should be minimised and shifting of soil and associated damage due to erosion avoided. All topsoil that is stripped should be stockpiled for use in areas that are being reinstated for farming purposes or to promote vegetation. Any vegetation being removed should be disposed of in a manner that is to the benefit of the community, e.g. for fuel wood. Labour-based methods should be considered where possible.

6.4.2 Winning of sand for construction

Sand for use as fill can generally be obtained either from side excavation using self-elevating scrapers or graders to side-cast in situ materials to raise the road bed to the appropriate level above the surrounding country and at the same time provide a longitudinal drainage system for the road. However, sand characteristics usually vary with depth and it is not unusual to find that the material in a lower horizon is suitable but becomes unsuitable if mixed with the overlying horizon. Blading techniques can then be used to avoid contamination between the two horizons as shown in Figure 6.3.

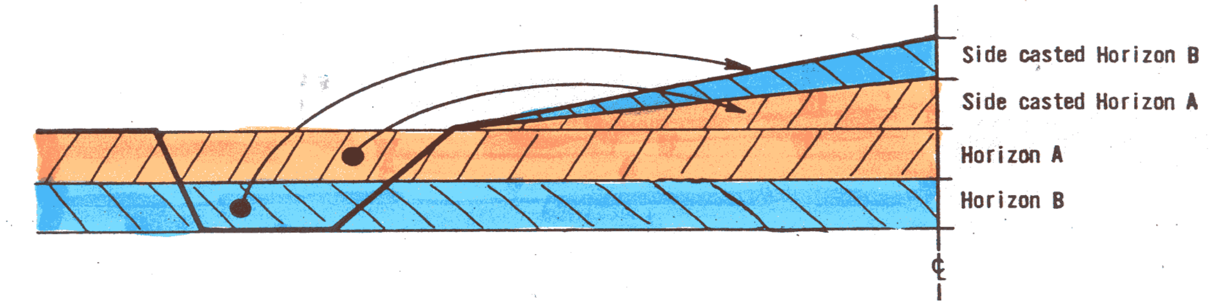


Figure 6.3 – Grader side-casting of in situ material

6.4.3 Compaction of pavement layers

Compaction of the subbase and base layers should follow the guidance given in Section 6.3 above. This guidance is based on experience derived from construction of numerous projects in Botswana in which sand has been used as subgrade/fill and subbase. This includes the entire 213 km length of the Serowe-Orapa road where an experimental section utilising sand as base course was also constructed.

In terms of the method of working, the following restrictions shall apply:

(a) Trafficking of the subbase and base should be kept to a minimum necessary to respectively the distribution of the base layer and the spraying of the bituminous prime.

(b) The combination of compaction equipment should be selected to avoid shearing or lamination of the surface layers.

It is worth repeating that the sands recommended for use as base possess a certain amount of cohesion and that this effect is enhanced if, after compacting at OMC, the material is allowed to dry out before priming and surfacing thus causing an increase in strength through soil suction. Until such time as this is allowed to happen, the surface should not be trafficked. However, this gain in strength is reversible and during the rains relatively small amounts of rain can prevent the subbase or base from drying out. This in turn precludes the Contractor being able to dump base on the subbase or to surface the base. In addition, when the surface of the sand is exposed, heavy rain can cause serious erosion. It is thus far more efficient to process the sand during the dry winter months.

6.4.4 Surfacing

Bituminous surfacing of a sand base is carried out generally in a similar manner as for other types of surfacing. Minor amendments include the use of a relatively lower viscosity prime (e.g. MC 30) to allow good penetration of the uppermost 20 mm of the base. The relatively hard surface so obtained will reduce surfacing aggregate embedment. The use of an “inverted” seal can also be employed beneficially in that the smaller size first application aggregate is less susceptible to punching into the surface of the sand base. However, experiments on the Serowe-Orapa neat base section with different seals showed that an inverted seal was not necessary and that the standard 19 + 9 mm seal performed just as well.

6.4.5 Drainage

Drainage is undoubtedly the single, most influential factor affecting the performance of a road pavement. Water entry into the road structure will weaken the pavement and make it much more susceptible to damage by traffic. Water can also have a harmful effect on shoulders, slopes, ditches and other features. It is therefore essential that appropriate measures are adopted for protecting the road from surface or ground water.

It has been suggested (Overby, 1990) that side-drains should be avoided where possible to reduce the likelihood of water ponding. Depressions within the road reserve should be sloped away from the road. Whether side-drains are constructed or not, it is imperative that a crown height of at least 0.75 m is achieved as illustrated in Figure 6-4. This height has been found to correlate well with the in-service performance of pavements constructed from naturally occurring materials.

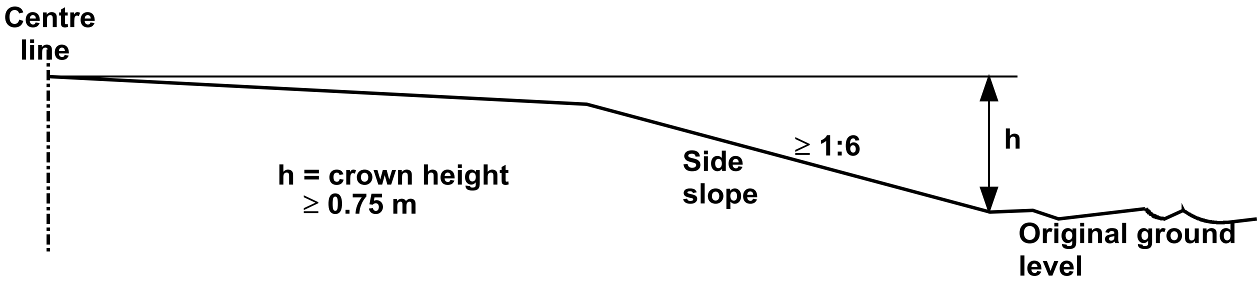


Figure 6-4: Minimum crown height for low volume road

The recommended minimum crown height of 0.75 m applies to flat ground. For longitudinal gradients > 1%, the crown height can be reduced to 0.65 m.

Because of the critical importance of observing the minimum crown height along the entire length of the road, the measurement of this parameter should form an important part of the drainage assessment carried out during the preliminary road evaluation. This is to ensure that any existing drainage problems associated with depressed pavement construction, often observed on unpaved roads in the region that have evolved over time and subsequently upgraded to a bitumen surface standard with no strict adherence to observing minimum crown heights is avoided (Figure 6-4).

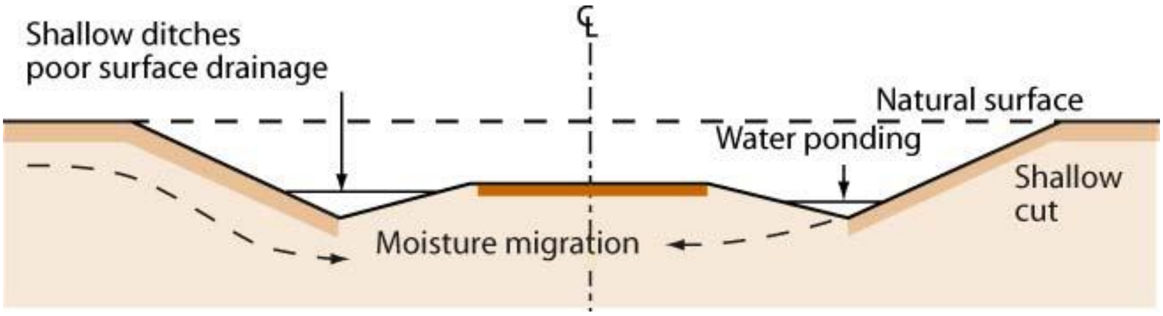


Figure 6-5: Potential drainage problems associated with depressed pavement

Where rock or hardpan layers, such as of calcrete or silcrete occur at shallow depths beneath surficial sand, the possibility of perched water tables is high. Under these situations, precipitation frequently lies at the surface for weeks or even months and can result in severe softening and/or collapse of poorly compacted sand layers. The importance of appropriate drainage cannot be overemphasised. Attempts should be made to fracture hardpan calcrete or silcretes layers forming perched water tables by ripping with a bull dozer or possibly fracturing with an impact compactor, in order to reduce the possibility of their stopping the infiltration of water.

It is important to note that, in Zimbabwe, where there is a possibility that the water table rises within 600 mm of the finished road level, the use of untreated sands in the pavement or underlying Zimbabwe is not permitted in the base (Mitchell, 1982).

7. SUMMARY AND WAY FORWARD

7.1 General Summary

The development of this ***Guide on the Use of Sands in Road Construction in the SADC Region*** is an extension of previous research undertaken in Botswana where neat (unstabilised) sand has been used successfully in all layers up to subbase level as well as in the base course of a 100 m experimental section. This section has performed excellently over a period of more than 20 years during which time it has carried approximately 0.5 MESA. This experience provided a strong motivation to investigate sands in other countries of the SADC region, namely Botswana, Malawi, Mozambique and Namibia and, to a lesser extent, South Africa, to ascertain whether they could also be used as neat base course in the construction of low volume roads.

The techniques adopted on the project for evaluating the suitability of sands in road construction are not conventional. Instead, they focus on alternative measures of grain size based on the *Φ (phi) sediment size scale,* a size scale which is commonly used in sedimentology. Moreover, based on experience emanating from Australia, the potential performance of sands was assessed in terms of their mean particle size (in phi units) representing the fineness of the material, and the standard deviation of the grading (in phi units) representing the degree of sorting (grading) of the sand as represented by the Wylde Chart (ref. Figure 3-3). Materials with a higher standard deviation of the grading (i.e. with a wider grading) tend to perform better in service compared with those that display a lower standard deviation in grading.

Other unique aspects of the project include the recognition that the soil constants of the sands that affect their potential performance as a pavement material are best determined on the silt and clay fractions, i.e. on the material passing the 0.075 mm sieve, and not just on the sand fraction, i.e. on the material passing the 0.425 mm sieve, as is traditionally done.

A knowledge of the mineralogy of the sands, in terms of their aluminium and iron oxide content, is also useful in assessing potential performance. In fact, a minimum iron content is included in some specifications (Western Australia). The requirement for a combination of both iron and aluminium is significant in terms of the colours of the sands, as iron will stain the sands a red or dark red colour (depending on the iron content), but aluminium will not. Thus, the fact that a number of light brown and grey sands appear to develop high strengths in roads may be attributed to the aluminium content and not the iron content. Thus, colour alone should not be used as an indicative selection criterion.

On the basis of the above approaches, it has been possible to develop a method for screening of sands for use as untreated base course in the construction of LVSRs in which the mean particle size (in phi units) and the standard deviation of the grading (in phi units) of the sands allows the potentially suitable ones (those that plot in Zone B of the Wylde Chart) to be selected for further testing in terms of their strength properties.

In terms of the use of sands in road pavements, it is obviously their in-service strength that determines their suitability for supporting traffic loadings. This should thus be the main selection criterion but this project has shown that this is a difficult property to characterise consistently. The materials can be difficult to compact in the laboratory and unusual behaviour has been detected in many of the tests and procedures conventionally used for material characterisation for roads. Despite the more obvious problems such as material variability and the effects of minor changes in sample preparation and testing techniques, it appears that many of the current testing techniques are not appropriate for use with sands as investigated in this project.

Standard compaction and strength testing, even by an accredited and experienced laboratory, have yielded highly variable results. It is expected that testing by smaller, less experienced local laboratories will produce even worse results and a revised testing regime should be employed. This should concentrate on the strength of the compacted sand that can be mobilised at different density and moisture conditions that simulate in-service conditions in the road closely. Indirect tests such as the CBR should be avoided and the actual strength in terms of, for instance, the DCP penetration rate should be investigated.

This project has confirmed the findings of a number of earlier studies regarding the correlation between CBR and DCP penetration rate for sands, which have shown that the commonly used correlations are probably invalid for sands. It has also shown that low higher penetration rates are found in the laboratory than in the field under similar conditions. ***New protocols must be urgently developed for laboratory testing of sands using the DCP in order to benefit fully from this most appropriate test technique***. This approach entails the laboratory determination of the materials resistance to penetration (its DN value in mm/blow) against the required design DN value at a particular moisture and density condition for various traffic categories.

Despite all of the testing criteria and performance specifications it is apparent that the state of the materials in the road is still a major contributor to performance. The degree of densification, the in situ moisture content and probably the effects of traffic moulding appear to all contribute to successful performance of sands.

The construction of sand pavements needs to be undertaken carefully. Sands tend to be moisture sensitive and proper compaction will only be achieved with the correct type of plant operating at the appropriate moisture content. However, such roads have been successfully constructed before and there is no reason why they should present insurmountable problems in future. In addition, as is the case for all roads constructed of natural occurring materials, drainage is of paramount importance as is maintenance.

In summary, the project has provided appropriate guidance on methods of prospecting for, screening (use of the Wylde Chart), and testing of sands to ascertain their suitability for use as neat base course in the construction of low volume sealed road pavements. The focus has been on the use of neat sand as a pavement layer, more so as base course, rather than on the use of relatively expensive stabilisation which can be avoided in a number of cases.

Because of the much larger number and variety of sand samples tested, compared with the previous research work undertaken in Botswana, the project has provided a more complete understanding of the properties of sands that make them potentially suitable for use as neat base in LVSR construction. However, as discussed below, further work is still required to facilitate the more wide-spread use of sands in LVSR construction.

Guidance has also been provided on how to design low volume road pavements using a simplified method based on the use of the DCP as well as how to construct them use sand as a pavement material. The stage has therefore been set for considering the more widespread use of sands in road construction in the SADC region.

7.2 Way Forward

A gradual, staged approach should be followed in ensuring that the outcome of the investigations carried out in the region are put into practice through the construction of demonstration projects. This has already begun in Mozambique where ANE has used red coloured sand as neat base course in an experimental section This type of sand was chosen on the basis of engineering judgement before the outcome of this project. However, as it happens, the sand is dark red in colour and its properties plot in the desirable Zone B of the Wylde Chart. The performance of the section needs to be carefully monitored so as to provide the necessary feedback to the design and construction phases of such roads.

The Hoopstad-Bultfontein experimental section contains a wealth of information which has only been partially exploited in terms of the relatively limited standard laboratory testing that was carried out. There is a need to undertake a more comprehensive investigation and write up of this section including more extensive laboratory DN testing.

There is also scope for undertaking trials in which sands that do not plot in Zone B of the Wylde Chart, but are nonetheless believed to be suitable for use as neat base course, are investigated. This will allow the Wylde Chart to be gradually refined on the basis of more extensive regional research.

It is important to ensure effective technology transfer of the use of sands in road construction occurs within the region. For this to be achieved, all the links of the technology transfer chain should be addressed. Figure 7-1 illustrates the typical pathway from research to implementation of any new technology, such as the use of sands in road construction that should be followed in the SADC region if it is to be implemented in a sustainable manner.

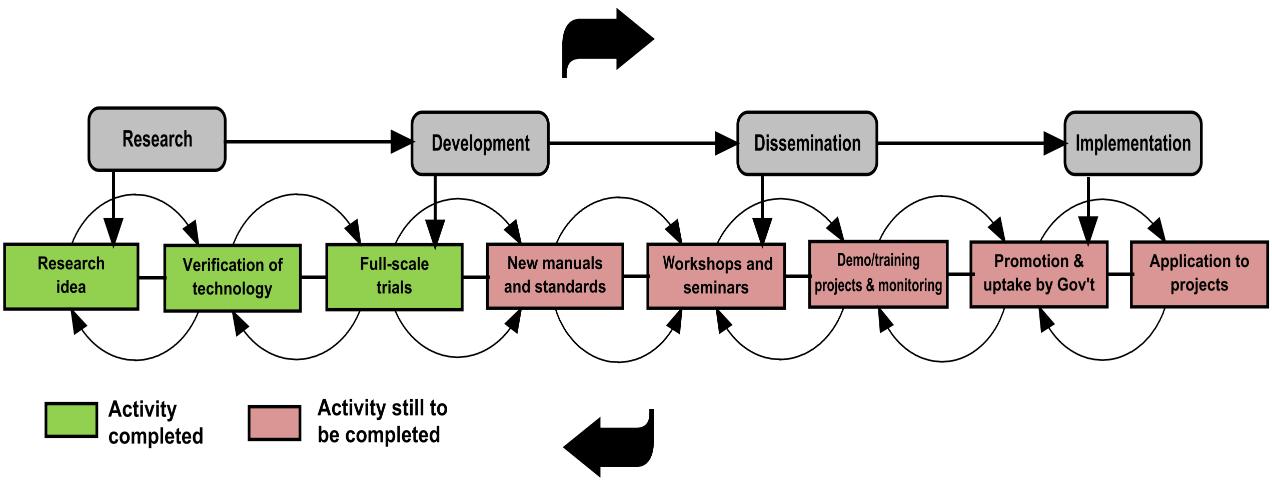


Figure 7-1: Pathway from research to implementation
(adapted from TRB Special Report 256, 1999)

As indicated in Figure 7-1, the following activities in the technology transfer chain have already been completed in the region:

- **Research idea**: initiated by Roads Department, Botswana, in 1989.
- **Verification of technology –** already proven in Australia and Botswana.
- **Full-scale trials –** already undertaken in Botswana and Mozambique but scope for further trials in other SADC countries.

The following activities are still required to complete the pathway to full implementation of sands technology:

- **New manuals**: The current ASANRA Guide is a step in the right direction. However, this information still needs to be incorporated in national standards of all SADC countries. Until new, nationally approved manuals are in place, the implementation of sands technology will be impeded.
- **Workshops and seminars:** There is need for promotion of the sands technology to a wider, national audience in all countries so as to obtain understanding and buy-in of the new technology.
- **Demonstration/training projects and monitoring**: This needs to be undertaken in all countries where neat sand is potentially suitable for use in low volume road construction. Monitoring of these demonstration projects is of paramount importance to provide inputs for refinement of the LVR designs and construction.
- **Promotion and uptake by Government:** This is a critical activity which will ultimately manifest itself in the form of national policy.
- **Application to projects:** This final link in the technology transfer chain will only occur when all the preceding activities outlined in Figure 7-1 have been addressed.

Thus, the activities highlighted above will need to be pro-actively addressed if the potential economic benefits of using selected sands in the construction of LVSRs is to be realised in practice.

Finally, it is recommended that the wealth of information contained in this Guide is distilled to produce a succinct ***Guideline on The Use of Sands in Road Construction*** which focuses specifically on those aspects of the Guide that influence the manner of specifying, designing and construction low volume roads using neat sand as the pavement material.

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ANNEX A

Use of the Phi scale for analysis of sands

In many sedimentological studies of fine sandy materials the phi (Ø) scale is used to represent the sieve sizes instead of the more usual logarithmic scale to base 10. The phi (Ø) scale is also a logarithmic scale but is to base 2 instead of 10 and is negative as follows.

$Ø = - \log_2$ (particle size in mm) Equation A-1

This means that each standard phi unit represents half the size of the previous unit. The cumulative percentage passing (or retained) each sieve can then be plotted against the phi scale as for any particle size distribution, making small differences in particle size distributions of fine materials more easily discernible (Figure 1).

The determination of the particle mean size and standard deviation can also be easily calculated from the particle size distribution on a phi scale.

It should be noted that this type of analysis as applied to the Kgalagadi sands in this document **uses the percentage retained on each sieve** (or remaining in suspension in the case of the hydrometer test) as opposed to the percentage passing used more commonly in civil engineering analyses. Hydrometer analyses are required when the upper percentiles (84 and 95) include material finer than 0.075 mm.

The phi scale for standard sieves is provided below and can be calculated from Equation 1 or in an Excel spreadsheet cell as "log (*sieve size*, 2)" (Table 1).

Table 1: Relationship between particle size and phi units

Sieve size (mm)	53	37.5	26.5	19	13.2	9.5	4.75	2	1.18
Phi value	-5.7	-5.2	-4.7	-4.2	-3.7	-3.2	-2.2	-1.0	-0.2
Sieve size (mm)	0.425	0.25	0.15	0.075	0.048	0.029	0.012	0.006	0.002
Phi value	1.2	2.0	2.7	3.7	4.4	5.1	6.4	7.4	9.0

An example of a typical plot of Kgalagadi sand data (Table 2) is given in Figure 1.

The statistical parameters of the grading analysis can be determined by reading various percentile values off the cumulative particle size distribution, in terms of percentage retained (Figure 1).

The **mean** particle size (M) is determined using the following formula:

$$M = \frac{\phi 16 + \phi 50 + \phi 84}{3}$$

where Ø16 = the 16th percentile, Ø50 = 50th percentile, etc.

The **standard deviation (σ)** is calculated from:

$$\sigma = \frac{\phi 84 - \phi 16}{4} + \frac{\phi 95 - \phi 5}{6.6}$$

These calculations can all be done simply using an Excel spreadsheet although it is generally simpler to read the percentiles off the cumulative distribution curve as shown in Figure 1.

Table 2: Sand data properties

Sieve Size (mm)	Sieve size (φ)	Cumulative percentage passing	retained
26.5	-4.7	100.0	0.0
19	-4.2	100.0	0.0
13.2	-3.7	100.0	0.0
9.5	-3.2	99.8	0.2
4.75	-2.2	99.0	1.0
2	-1.0	98.2	1.8
1.18	-0.2	97.9	2.1
0.425	1.2	50.3	49.7
0.25	2.0	40.4	59.6
0.15	2.7	27.8	72.2
0.075	3.7	6.7	93.3
0.07	3.8	6.3	93.7
0.048	4.4	6.0	94.0
0.029	5.1	5.8	94.2
0.019	5.7	5.4	94.6
0.012	6.4	5.0	95.0
0.008	7.0	4.7	95.3
0.006	7.4	4.5	95.5
0.003	8.4	4.0	96.0
0.002	9.0	3.8	96.2

Figure 1: Particle size distribution curve

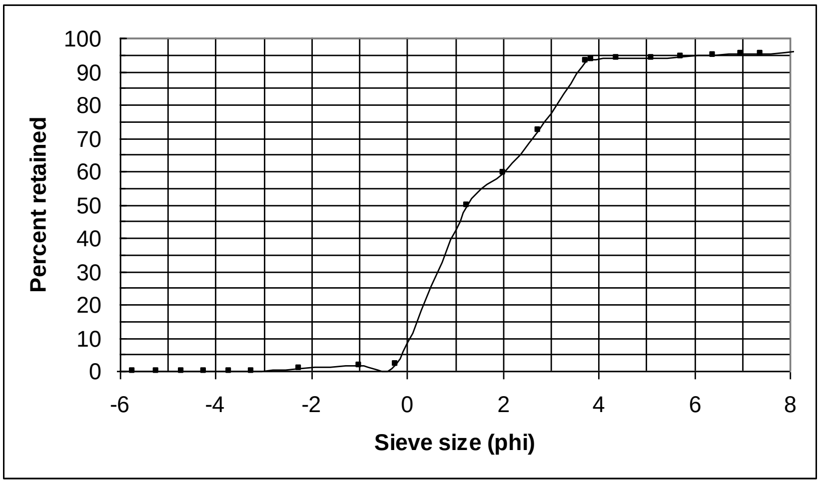


Table 3: Extract of calculation of mean and standard deviation from Excel spreadsheet. Percentiles are read off Figure 1.

Percentile	Phi unit
5	-0.2
16	0.25
50	1.2
84	3.3
95	6.9
Mean	1.58
SD	1.84

ANNEX B

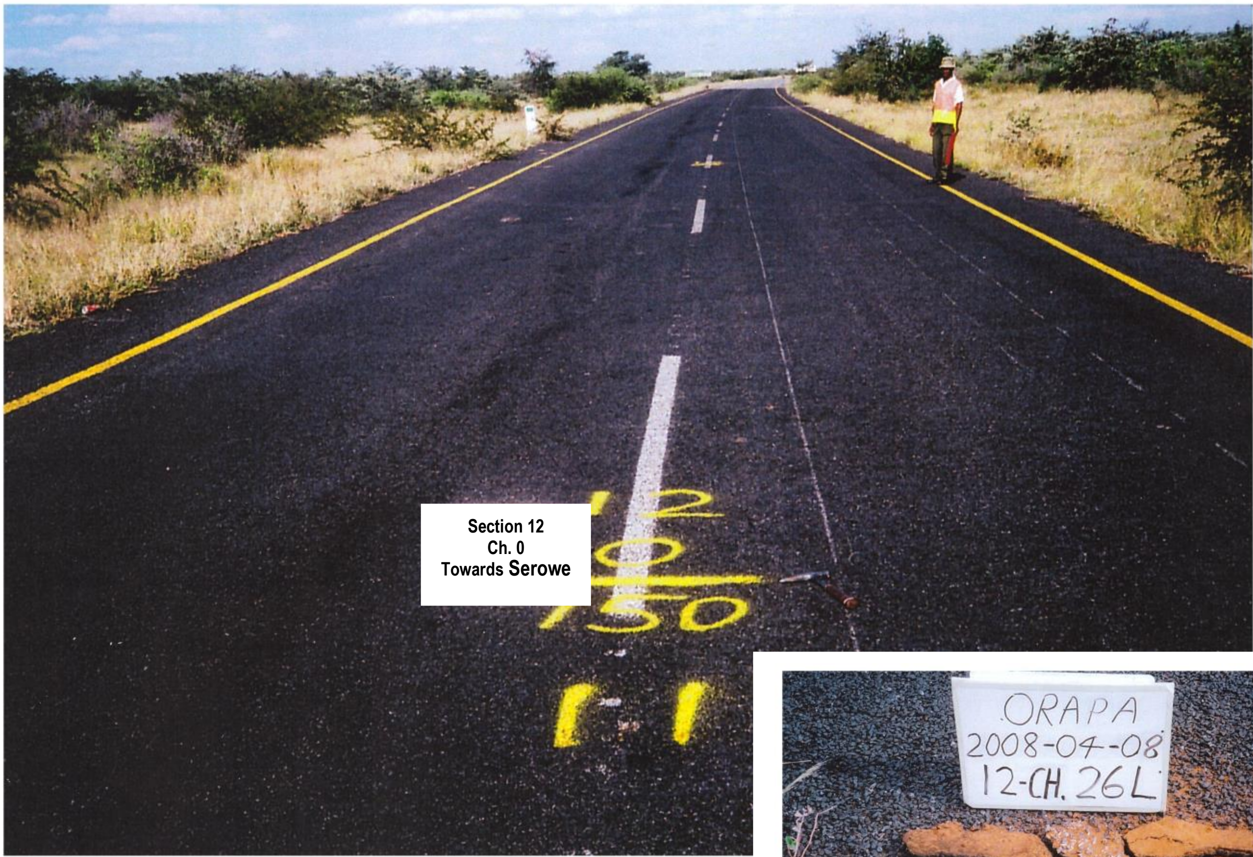
Examples of use of neat sand as base course

Hoopstad-Bultfontein Experimental Section, South Africa. Constructed in 1962.

January 2013 ADT = 667
Total traffic carried to date over 50 years +/- 1.0 MESA

Note: There was no great difference in the condition of the neat sand section (rated poor) and the adjacent cement-stabilized sections of the experimental section (rated fair). This suggests that it is not necessary to go to the expense of stabilization under these conditions – a significant cost saving in the construction of low volume sealed roads using sand of appropriate quality. This evidence is also exemplified by a similar section of neat sand base used in the construction of an experimental neat sand base section of road in Botswana as illustrated below.

Serowe-Orapa Experimental Section, Botswana. Constructed in 1989.



Traffic carried to date (July 2014) = ± 0.5 MESA