SARF COURSE

PRACTICAL ROAD PAVEMENT ENGINEERING

CONTENTS

SOUTH AFRICAN ROAD FEDERATION

COURSE

PRACTICAL ROAD PAVEMENT ENGINEERING

WEBINAR

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COURSE LEADER: **DAVE WRIGHT**

SARF COURSE

PRACTICAL ROAD PAVEMENT ENGINGEERING

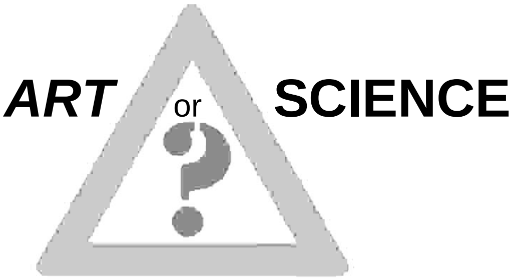
1. INTRODUCTION

PRACTICAL ROAD PAVEMENT ENGINEERING

NEW ROADS AND REHABILITATION

INTRODUCTION

Practical road pavement engineering involves the location, testing, utilization, design, maintenance and re-use of road building materials. The purpose of this exercise is to provide a road which will carry a range and volume of traffic, for a specific design period, at an acceptable level of performance.



Practical pavement engineering is in fact a combination of both.

- Science : A good theoretical knowledge of materials properties, characteristics, performance; the mechanisms of loading, the transference of load and modes of failure; and the interaction of the two.

- Art : Practical experience and engineering common sense.

  - ❖ Conditions are often very similar but never the same.

But why bother? The analogy of a house illustrates the need for appropriate materials usage, sensible and economic design, construction control and subsequent (ongoing) maintenance.

Pavement design is thus largely an exercise in sensible quality control.

Maintenance does not form a part of this course. However, the designer needs to understand that maintenance is a vital part of the life of a road and must take this into account in all phases of road works.

MAINTENANCE

There is an old English saying : "A stitch in time saves nine".

South Africa has an extensive road network which until fairly recently was in relatively good condition. With the curtailment of road funding it is important to look after our current investment :-

FLEXIBLE ROADS IN SOUTH AFRICA

Road Type	Kilometres
Rural – Freeways (paved)	**3 000**
- Single carriageways (paved)	**35 000**
- Single carriageways (gravel)	**135 000**
Urban – Paved	**35 000**
- Gravel	**10 000**

A very rough estimate of Replacement Cost (year 2021) is of the order of three trillion Rands [R 3 X 10^{12}]. So just like with a house it is vital that we maintain our investment. Regrettably because deterioration takes place over a period of time (i.e. years) the short-term view, particularly of politicians, is that in difficult financial times we can either reduce or stop maintenance funding "for a while". This premise is both false and dangerous.

Maintenance encompasses :-

- Routine works such as grass cutting, cleaning side drains and culverts, rubbish removal
- Repairs to signs, fencing, the road surface (patching), and
- More extensive work such as resealing, minor reconstruction/rehabilitation and regravelling.

CRITICAL COMMENT

Often **too little, too late.** Maintenance needs to be systematically and well done. The concept that maintenance is a task of little importance to be carried out by those of **limited** ability is both **foolish** and definitely will be **costly!**

One of the biggest problems in a large road network is to decide where and how to spend one's limited maintenance budget. This is where a pavement management system (PMS) can play a key role.

PMS FUNCTIONS

1. **OBJECTIVELY:** measure the Type, degree and Extent of distress.

2. **COMPARE:** all roads and street conditions and PRIORITISE in order of need for attention.

3. **GUIDELINE:** provide a guide in the form of a set of calculated treatments for each road condition. The calculations are based on a set of algorithms. The outcome can be used for estimating costs and hence preparing a budget for approval by senior management.

The PMS is a management tool and does not take the place of detailed design for each specific situation.

The PMS is :-

- Not a magical black box which provides perfect answers because it has been calculated by a computer which can never go wrong, never go wrong, never go ………………

- A tool to be used by the engineering profession to plan and budget for orderly and timely maintenance.

- To be used with intelligence and common sense.

- To be amended/adapted to suit local conditions.

In this way a PMS will provide a **systematic, objective, logical** approach to road maintenance based on **sound budgeting.** This should ensure that pavements are kept at an **acceptable level of service** with optimal money usage.

REFERENCES

GENERAL

NOTE: Always check for the latest published version.
Some of the TRH /TMH documents which are out of print are available on line as pdf documents.

TMH / TRH:- www.nra.co.za Technical Specifications

Bitumen and asphalt:- www.sabita.co.za /Publications/Manuals

SAPEM (2014) "South African Pavement Engineering Manual" SANRAL

SITE INVESTIGATION

BRINK ABA, PARTRIDGE YC and WILLIAMS AAB (1981)
"Soil Survey for Engineering" Oxford Press

TRH 6 (1985) "Nomenclature and Methods for describing the Condition of Asphalt Pavements" Department of Transport, Pretoria

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SABITA Manual 37 / TMH 5 (2021) " Sampling Methods for Road Construction Materials"

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MATERIALS and TESTING

WEINERT HH (1980) "The Natural Road Construction Materials of Southern Africa" CSIR, Pretoria

ROAD RESEARCH LABORATORY (1951) "Soil Mechanics for Road Engineers" HMSO, London

TRH 14 (1985) "Guidelines for Road Construction Materials" Department of Transport, Pretoria

SANS 3001 "Civil engineering test methods", SABS

SANS 4001 "Civil engineering specifications, bituminous materials for roads

REFERENCES Continued

SABITA Manual 2 (2007) “Bituminous binders for road construction and maintenance”, Cape Town

TG1 (2019) “The use of modified bituminous binders in road construction”, Asphalt Academy

TG2 (2020) "Bitumen Stabilised Materials", SABITA

TG4 (2020) "Water Quality for Use in Civil Engineering Testing Laboratories", SABITA

DESIGN

SABITA Manual 10 (1992) “Appropriate Standards for Bituminous Surfacing” SABITA, Cape Town

SABITA Manual 21 (1999) “ETB, The Design and Use of Emulsion Treated Bases” SABITA, Cape Town

UTG 3 (1988) “Structural Design for Urban Roads” CSIR, Pretoria

SABITA Manual 40 / TRH 3 (2021) “Design and Construction of Surface Treatments”

Draft TRH 4 (1996) “Structural Design of Flexible Pavements for Interurban and Rural Roads Pavements” Department of Transport

PMG (2000) “Interim Guidelines for the Design of Hot-Mix Asphalt in South Africa” B Verhaege, Transportek, Pretoria

SABITA Manual 35 / TRH 8 (2021) "Design and Use of Asphalt in Road Pavements"

Draft TRH 12 (1997) “Flexible Pavement Rehabilitation, Investigation and Design” Department of Transport, Pretoria

TRH 20 (1990) “The Structural Design, Construction and Maintenance of Unpaved Roads” Department of Transport, Pretoria

SPECIFICATIONS

COTO (2020 DS) "Standard Specifications for Road and Bridge Works ...."

MAINTENANCE

SANRAL (2008) “Routine Road Maintenance Guidance Manual”

Produced by DF Wright and AO Bergh, and revised by BH Alexander in 2008

LIST OF TRH DOCUMENTS FOR PAVEMENT ENGINEERING

Some documents have been amended/upgraded as SABITA Manuals
eg TRH3 is now SABITA Manual 40 / TRH3

TRH 1
Prime coat and bituminous curing membranes

NOTE TRH 2 is out of print

TRH 3
Surfacing seals for rural and urban roads

TRH 4
Structural design of flexible pavements for interurban and rural roads

TRH 5
Statistical concepts of quality control and their application in road construction

TRH 6
Nomenclature and methods for describing the condition of asphalt pavements

TRH 7
Use of bitumen emulsions in the construction and maintenance of roads

TRH 8
Selection and design of hot-mix asphalt surfacing for highways

TRH 9
Construction of road embankments

TRH 10
Design of road embankments

TRH 11
Guidelines for the conveyance of abnormal loads

TRH 12
Bituminous pavement rehabilitation design

TRH 13
Cementations materials in road construction

TRH 14
Guidelines for road construction materials

TRH 15
Subsurface drainage for roads

TRH 16
Traffic loading for pavement and rehabilitation design

TRH 17
Geometric design of rural roads

TRH 18
The investigation, design, construction and maintenance of road cuttings

TRH 19
Standard nomenclature and methods for describing the condition of jointed concrete pavements

TRH 20
The structural design, construction and maintenance of unpaved roads

TRH 21
Hot-mix recycling

TRH 22
Pavement Management Systems

CATEGORISATION OF MATERIALS USED IN THE CONSTRUCTION OF ROAD PAVEMENTS

G = Granular (Unbound) **C = Cemented (Bound)** **BSM = Bitumen treated (Bound)**

CAT	DESCRIPTION GRANULAR MATERIALS	USE
G1	Top quality crushed stone, high strength, low plasticity and good grading (max PI NP for > 15 million E80s and ≤ 4 for < 15 million)	Base
G2	As for G1 but small amount of non-parent rock material permitted	Base
G3	Crushed stone, high strength, moderate plasticity (max PI 6, Calcrete PI 8) and moderate grading	Base
G4*	Crushed stone or Natural gravel (may be crushed), CBR > 80 at 100% MDD and max PI 6	Base
G5*	Natural Gravel, CBR > 45, max PI 10 and some grading controls	Sub base
G6	Natural Gravel, CBR > 25, max PI 2 x GM + 10	Sub base
G7	Natural Gravel, CBR > 15 (CBR > 18 when stabilized) and max PI 3 x Grading Modulus +10	SSG
G8	Natural Soil or Gravel, CBR > 10 and max PI 3 x GM + 10	SSG
G9	Natural Soil or Gravel, CBR > 7 and max PI 3 x GM + 10	SSG
G10	Natural Soil or Gravel, and min CBR 3	Fill
Clay	Soil with CBR < 3 – very poor material	Dams!

* Divided into classes: A, crushed stone or boulders and B, gravel with cobbles

CAT	DESCRIPTION CEMENTED MATERIALS	USE
C1	Chem. treated gravel (G2 or better) – UCS > 6MPa *Not in COTO*	Base
C2	Chem. Treated gravel (G4A or better before treat.) – UCS 3 to 5 MPa ITS 300 to 600 kPa	Base
C3	Chem. Treated gravel (G6 or better before treat.) – UCS 1,5 to 3 MPa ITS 250 to 500 kPa	Sub base
C4	Chem. Treated gravel – UCS 0,75 to 3 MPa - ITS > 200 kPa	Sub base

UCS and ITS for compliance with Specifications at 7 day curing

CAT	DESCRIPTION BITUMEN TREATED MATERIALS	USE
BSM1	High Shear Strength for Design Traffic : ≥ 6 Million E80s	Base
BSM2	Moderately High Shear Strength for Design Traffic : < 6 Million E80s	Base

CAT	DESCRIPTION WATER-BOUND MACADAM	USE
WM	Single-sized crushed-stone (max 75mm) with sand (max PI 6)	Base

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2. PAVEMENT MATERIALS

PAVEMENT MATERIALS

1 INTRODUCTION

Materials used to build roads include soil, sand, gravel, and crushed stone. They may be used unmodified (unbound) such as a natural gravel sub-base, or they may be used bound (stabilized with cement, lime or bitumen) such as C4 cemented natural gravel. This paper describes soils, sands, gravels and crushed stone, both unbound and bound.

Naturally occurring soil, is a mass of weathered mineral particles of various shapes and sizes, between which there is water and some air. Near the ground surface, it may contain organic matter from decomposition. The type of soil is defined by the size and shape of the soil particles, but its engineering properties are defined by the interaction between the particles, and the air and water in the soil. The relative state of the particles, air and water can be used to explain much of the behaviour of soils, as well as natural gravels and crushed stone.

Examples: a soil with much water in it (i.e. wet) will perform differently to a soil with little water which will usually be weaker and will shear or fail when loaded. A soil with much air in it (i.e. uncompacted) will perform differently to a soil with little air which will densify (or compact) under traffic and cause-ruts.

1.1 Particle size distribution

The particle size distribution is a common measure of a material:

- ❖ well graded distribution will enable each particle to fit into the voids created by antiparticle contact of large sizes; it gives maximum density and minimum voids,
- ❖ poorly graded material is limited in compaction since it cannot get a close packed geometric arrangement,
- ❖ an excess of fines reduces mechanical interlock reducing strength and presents a slippery surface when wet, is dusty, less stiff, often sensitive to water,
- ❖ internal friction, voids content, wear resistance and permeability depend on distribution sizes.

The behaviour of soils depend on the relative sizes of particles and in turn on their surface areas. The importance of the smallest particles (silts and clays) in determining the engineering properties of materials is shown by their relatively larger surface areas in Table 1.

PAVEMENT MATERIALS

Table 1 Surface areas of various particle sizes

Material		Number of particles in 2mm² cube	Total surface area (mm²)
Type	Diam		
Gravel	2 mm	1	12,5
Sand	1 mm	8	25
Silt	60 µm	37 000	420
Clay	2 µm	1 billion	12 500

1.2 Plasticity

Plasticity is a measure of the influence of clay on the performance of a material. Because there are many microscopic water films associated with the small flat clay particles, any soil with clay content > 15% exhibits plasticity and cohesion. Over certain moisture range (between the plastic limit and the liquid limit), particles will slide over each other – the action being to shear rather than break. At high moisture, soil will flow under its own weight i.e. when you do an Atterberg Limit test and turn the handle, the soil flows (the liquid limit is the moisture content at which it flows together over 25 blows). The plastic limit is drier than the liquid limit and is the moisture content at which it stops shearing and it breaks.

1.3 Strength

Moisture content and density affect the strength of a material. An approximate relationship has been developed between CBR, moisture content and density, which can be used to explain this variation.

Table 2 Variation of CBR with moisture content

Material class (TRH 14)	Soaked CBR	Ratio of moist CBR versus soaked CBR		
		Moisture content		
		OMC	75% OMC	50% OMC
G4	80	1.3	1.9	2.5
G5	45	1.8	2.6	3.6
G6	25	2.4	3.6	5.4
G7	15	3.0	4.7	7.6
G8	10	3.7	5.9	10.0
G9	7	4.3	7.2	12.6
G10	3	6.5	11.4	22.3

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2. SELECTION AND USE OF MATERIALS

The selection and use of materials is important to ensure that the required engineering properties are met. This is generally governed by TRH 14 (1985) Guidelines for Road Construction Materials. Materials are classified in TRH 14 on the basis of a number of properties into various categories e.g. G1 to G10 (now specified as such in COTO and SABS).

This specification can be met by bringing in suitable materials from quarries or borrow pits, or by the careful use of in situ materials. It is often found that the properties of the in situ materials are adequate for the engineering requirements even if they do not meet the specification of THR 14 exactly, and considerable savings can be had by using in situ materials.

2.1 Use of in situ materials

The base materials are a costly and important component of pavement materials, and the most important base material parameter is strength or as is commonly used for simplification, bearing capacity. This leads to the four aspects which must be satisfied with regard to the selection of materials:

- adequate bearing capacity under any individual applied load;
- adequate bearing capacity to resist progressive failure under repeated individual loads;
- the ability to retain that bearing capacity with time (durability); and
- the ability to retain bearing capacity under various environmental influences (which relates to material moisture content and in turn to climate, drainage, and moisture regime).

The control of moisture is the most important goal in ensuring a satisfactory performance, and in this respect it is more important than even the quality of the material. Accordingly where relaxation of material requirements is possible for low volume roads it is on condition that the drainage and moisture regime are suitable.

> *Example: Field investigations into the performance of materials for low volume roads found sections of road with unstabilised base materials with Plasticity Indices of 3 which had failed badly after 4 4000 E80s, whilst others with Pl's as high as 17 had performed well after 70 000 E80s. Obviously there were other differences between the two materials, but to have rejected the high Pl material on the basis of Pl alone would have been wrong. This example illustrates the need to use engineering judgment in selecting materials.*

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Many of the standard engineering tests (grading modulus, Atterberg limits, laboratory soaked CBR) do not correlate well with the actual performance of the materials in pavements. However, characteristics which may make compaction or finishing difficult e.g. whether there are any large stones or the plasticity is high, should be considered.

It is recommended that the in situ materials selection should be primarily based on:

- strength
- the strength/moisture/density relationships and
- long term strength (durability)

3. SUBGRADE MATERIALS

The subgrade material is that which occurs naturally beneath the proposed pavement and thus become an integral part of the pavement. Southern Africa is in the fortunate position of having particularly good subgrade materials over much of the region thanks to the relatively arid recent geological history. This has, however, had the result that occasional deficiencies in the subgrade are often overlooked.

The cost of the road is integrally linked with the subgrade conditions. The poorer the conditions, the greater the cover thickness required to support the design loads. Highly problematic or very weak materials need to be replaced or preferably improved through modification in order to minimize importation of borrow materials.

> *Hint: the layers in existing roads (either paved or unpaved) which have developed strength and density over time should be used as far as possible with* ***minimal disruption of the acquired structure****.*

It is imperative that the subgrade conditions for any proposed road are fully investigated by an experienced person and problem areas are identified and delineated for further investigation or testing. Those subgrade conditions which require particular attention possibly even below material depth are inter alia, soft materials, expansive clays, dispersive soils, collapsible soils and areas of potential drainage problems. Each of these is briefly discussed in the following sections.

3.1 Soft soils

Certain materials may be extremely soft in their natural state or become extremely soft on soaking. These occur particularly in vlei and estuarine areas. They are easy to identify either in situ during site inspections or during laboratory testing of their soaked strengths. Materials with a soaked CBR strength of less than 3 can be considered as having low shear strengths and being susceptible to high settlements under loading and special treatment is necessary. This treatment will depend on the pavement structure and design but will typically require the importation of additional layers of selected materials, with or without (preferably) the removal of the weak material, depending on the cross profile of the pavement.

Where possible pre-loading can allow much of the settlement to occur prior to construction, this being particularly important when structures are involved.

3.2 Expansive clays

Soils containing an expansive clay component in adequate quantities (typically montmorillonite or smectite clays but also possibly vermiculites) may potentially result in significant volumetric changes associated with fluctuations in moisture content or stress levels. Expansive materials are most easily identified from their plasticity indices and clay-sized component using the van der Merwe (1975) plasticity chart or the Weston method (Weston, 1980). The Kantey-Brink limits are a quick guide, and a soils is potentially expansive if:

LL > 30 <u>and</u> PI > 12 <u>and</u> LS > 8

One precaution often recommended is to replace the active clay to a depth of 600 mm over the pavement width with a more stable material. This is, however, costly and the acceptability of tolerating surface unevenness of the road should be investigated. If the expected differential movements within the pavement are likely to cause cracking of the surfacing, appropriate action should be taken. This includes the use of modified bitumens in the surfacing for more expansive materials. The most successful treatment in South Africa has been to wet up the subgrade to > 90% saturation and place an impermeable membrane above it over the width of the pavement formation.

3.3 Dispersive soils

Dispersive soils are typically fine silty clays which contain a high percentage of exchangeable sodium or, less frequently, lithium. These materials have the ability to disperse in a moist environment and the fine particles can then be ‘washed’ out of the soil resulting in tunnelling and formation of cavities. Dispersive materials are typically difficult to positively identify, even in the laboratory, requiring a range of chemical and physical tests. However, any field evidence of excessive erosion channelling or tunnelling should arouse suspicion and warrant additional testing or specialist advice.

Dispersive soils are difficult to treat requiring that the exchangeable sodium or lithium cations are replaced with calcium ions (from added gypsum typically). Movement of moisture within the dispersive materials should be minimized. Removal of the material to a depth of 600mm is another alternative but is costly. Without precautions, dispersive soils will invariably lead to significant distress. Dispersive soils are undesirable as fill materials for roads.

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3.4 Collapsible soils

Collapsible soils are typically low density sandy materials which may densify under load at high moisture contents. This can result in differential movement within the road structure and general unevenness and loss of riding quality. Collapsible materials are difficult to identify without specialist laboratory testing (e.g. double oedometer) but an initial indication can be obtained from backfilling and excavation. A negative bulking factor is typical of collapsible soils. Should this indicate a possible collapse potential, specialist assistance should be obtained bearing in mind the risk of deterioration and the level of serviceability required.

The collapse potential of a soil can usually be reduced by high energy impact rolling or ripping and recompacting to an appropriate depth (600mm recommended). For very lightly trafficked roads, the consequences of differential collapse are often tolerable, resulting in some degree of surface unevenness at worst.

3.5 Poorly drained areas

During the site inspections and centre-line sampling, areas of potential drainage problems and high water table should be identified. This is best done by experienced personnel who use topography, soil type and vegetation variations to identify these areas.

> *Hint: when assessing an existing sealed road, centre-line failures are often indicative of high water table and edge failures may be indicative of edge drainage problems. This knowledge can be used to identify the source of water and the possible corrective action.*

The pavement should be built up on an appropriate fill in order to minimize the effect of excessive moisture on the pavement. It may be necessary for some soils with high capillary suction potentials (fine sands and silts) to incorporate a drainage blanket layer within the fill or lower pavement layers to ensure that the upper layers do not become too moist.

4. PAVEMENT LAYERS

4.1 Untreated materials

The COTO requirements for untreated pavement materials include grading, Atterberg limits, crushing strength, and bearing strength. For low volume roads some slight latitude is permitted in classifying materials according to the COTO classification. Broadly speaking, if a material meets the bearing strength (CBR) requirements but is marginally outside the other requirements then it will still be acceptable.

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(a) **Grading**

The particle size distributions of the materials should lie within the recommended envelopes provided in COTO. These envelopes are based on Fuller-type curves and theoretically result in the maximum densities for the relative maximum sizes defined. Blending of materials may be necessary to improve the grading in an attempt to get closer to the envelopes.

(b) **Atterberg limits**

The Atterberg limits given in COTO apply to soil fines (< 0,425 mm) of graded crushed stone and natural gravels (G4 and G5) after modification, if required. For G1 to G3 materials Atterberg limits are also determined on < 0,075 mm material. Relaxation of Atterberg limits is permitted for low volume roads in drier moisture conditions, and also for calcrete materials, provided that the material meets the appropriate bearing strength and durability requirements. No relaxation is permitted in the wet moisture environment unless the soaked CBR exceeds the specified limits by at least 10 per cent and the materials have low moisture sensitivity and the pavement is well drained.

(c) **Crushing strength**

Aggregate crushing strength requirements are specified for graded crushed stone (G1 to G3) in COTO. The durability of crushed stone should meet the requirements below (see 4.3).

(d) **Bearing strength and swell**

G3 and G4 Materials should have a CBR after soaking of not less than 80 per cent at 100 per cent maximum dry density (MDD) and a maximum swell of 0,2 per cent at 100 per cent MDD.
G5 Material should have a CBR after soaking of not less than 45% at 95% MDD and a maximum swell of 0,5 per cent at 100 per cent MDD. In dry areas (Weinert N > 10) and traffic < 3 million E80s with 150 mm base the CBR may be relaxed to 25% at 95% MDD.
Materials G6, G7, G8 and G10 should have the CBR and swell properties given in Table 3.

Table 3 CBR and swell requirements for G6, G7, G8, G9, G10

Requirements: Soaked CBR test	**G6**	**G7**	**G8**	**G9**	**G10**
Minimum CBR at 93% maximum dry density (%)	25	15			
Minimum CBR at in-situ density (%)			10	7	3
Maximum swell at 100% maximum dry density (%)	0.5	1.0	1.5	1.5	1.5

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4.2 Bound (or stabilized) materials

If no suitable materials are available for base or subbase layers, stabilization with bitumen, lime, cement, lime/slag or any other pozzuolanic stabilizers or combinations may be used to improve local materials. Cemented crushed stone or gravel (C2) are selected materials equivalent to G2 or G4 materials equivalent to G2 or G4 material except for grading, with the addition of stabilizer. Cemented natural gravel (C3, C4) is a selected natural material equivalent to G5 or G6 material with the addition of stabilizer.

The COTO requirements to be met by bound materials are grading, crushing strength, flakiness index, sand equivalent, Atterberg limits (after treatment), strength, and durability. The durability of stabilized materials is critical and the latest knowledge is discussed below.

4.3 Durability

4.3.1. <u>Unbound Materials</u>

Durability is an issue for unbound crushed stone materials used in the base- and for surfacing aggregate. The problems are whether rocks will alter chemically (decompose), or whether existing alteration products (formed by natural weathering processes through geological time) are mobilized and freed (degradation). The end result is that the performance of the material during the life of the road is reduced, and the life of the road is also reduced. This can occur within as short a time as a couple of years. If the affected material is in the base, the mode of failure is shear in the leading to rutting, crocodile cracking and potholing. This can be due to:

- disintegration of the top 10-15 mm in the base forming excessive fines and loss of adhesion of the surface seal;
- breakdown of the base aggregate in service from the repeated action of traffic in the presence of excess moisture with the resultant generation of plastic fines (generally expansive smecitite clays) which damage the nature of the materials and significantly reduce the bearing capacity.

The basic igneous (dolerite, basalt) and acid igneous (granite) classes of rocks (given in COTO) are considered the most likely to decompose in a wetter environment, which can be either climatic or due to excess moisture in the pavement. The disintegrating rocks are the high silica (quartzites, quartz gravels and sandstone), arenaceous (mainly Karoo sandstones), argillaceous (mudrocks, shales) rocks and carbonates. Durability is rarely a problem in the layers beneath the base.

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The durability of a material can be tested by any of several tests:

- Durability Mill test,
- 10% Fine Aggregate Crushing test (10% FACT),
- Aggregate Impact Value (AIV) test
- Secondary Mineral Count

While no single test has proved adequate for defining the durability of all different material types, either the Durability Mill test or the modified AIV test provides the most suitable assessment. Accordingly the specifications for durability are given here in terms of these test results. Testing should be performed on all new material sources for which there is no history of performance, and on all suspected materials.

The durability Mill test shows the durability of a material in terms of the fineness product (FP) which is the product of plasticity index (PI) and percentage material passing the 0,425mm sieve (P425) (Sampson, 1988). The recommended limits and specification are given in Table 4 (Sampson, 1992a).

Table 4 Material related durability limits for unbound materials

<table>
<tr><td>Material type</td><td colspan="2">Modified AIV</td><td colspan="2">Durability Mill Test</td></tr>
<tr><td></td><td>Dry</td><td>Wet/Dry ratio</td><td>Max FP</td><td>Max P425</td></tr>
<tr><td>Basic crystalline</td><td rowspan="3">< 39</td><td rowspan="4">< 1,14</td><td>125</td><td rowspan="7">35</td></tr>
<tr><td>Acidic crystalline</td><td rowspan="2">420</td></tr>
<tr><td>High silica[a]</td></tr>
<tr><td>Arenaceous</td><td>< 31</td><td rowspan="4">125</td></tr>
<tr><td>Argillaceous</td><td>< 24</td><td>< 1,08</td></tr>
<tr><td>Carbonates[b]</td><td>< 39</td><td>< 1,14</td></tr>
<tr><td>Diamictites</td><td>< 22</td><td>< 1,15</td></tr>
<tr><td>Metalliferous[b]</td><td colspan="4">Not required</td></tr>
<tr><td>Pedogenic[c]</td><td>< 39</td><td>< 1,20 calcrete
< 1,14 concrete</td><td>480</td><td>55</td></tr>
</table>

Notes a: applicable if the clay mineral present is kaolinite
b: applicable if soil binder is added to create fines
c: these are tentative limits

4.3.2 Bound materials

Durability is an important issue for lime or cement stabilized layers. Research into low volume roads (Paige-Green, 1992a) has found that many bound layers have carbonated (in which the cementation strength is lost due to interaction with CO_2 in the atmosphere and the soil), and thus have little residual stabilization.

As partial or complete carbonation of the treated layer occurs, this can lead to a large decrease in strength. If the traffic volume is light relative to the structural design and the moisture regime is dry or optimum, this may not lead to significant problems. However, if the traffic volume is at the limit for the particular design or the pavement moisture regime is wet, then severe rutting, cracking and shearing can occur. Durability testing should be conducted for all bound materials used where the strength of the bound layer is likely to be critical for the performance of the road. Three tests have been developed to measure the durability:

- gravel initial consumption of lime or cement test (ICL or ICC);
- wet/dry brushing test; and
- unconfined compressive strength test on cycled or carbonated specimen.

The test limits are shown in Table 5 (Sampson, 1992b).

Table 5 Material related durability limits for bound materials

Test Method	Specification
Wet / dry brushing test	Hand test: Stabilised base < 25% loss after 12 cycles Stabilised subbase < 40% loss after 12 cycles
	Mechanical test: Stabilised base <8% loss after 12 cycles; Stabilised subbase < 13% loss after 12 cycles
Gravel ICL or ICC	Stabiliser content 1% higher than ICL or ICC[a]
Vacuum carbonated UCS	Same TRH 14 limits as for normal UCS test

Note: (a) Only if carbonated UCS values are not sufficient

The tests may show the need to increase the stabilizer content. However, there are both economic and engineering limits to the amount of stabilizer that should be added, and for practical purposes this is about 4% to 5%. Any more than this and cracking of the bound layer can reflect through the surface.

At the lower end, if only 1,5% to 2% stabilizer is added, it is often such that it is more likely to "modify" the material than stabilize it. This practice has value however in improving material workability, even though the quantity is too low to prevent carbonation.

Due to practical constraints with mixing in of the stabilizer limits are usually set at 1,5% to 5% of stabilizer. In many cases this can result in very high layer strengths (in excess of that which is required), with the result that even after carbonation sufficient strength exists in the layer.

Example: Required strength for base:	*CBR 80%*
	UCS 750 kPa
Available material:	*CBR 65%*
	ICC 3%
Uncarbonated:	*UCS 1200kPa (2,5% cement)*
Carbonated:	*UCS 800 kPa*
Uncarbonated:	*UCS 1800 kPa (4% cement)*
Carbonated:	*UCS 1500 kPa*

Although the ICC is not satisfied, stabilization with 2,5% cement is therefore acceptable.

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Low-cost local road materials in Southern Africa

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Summary

Southern African natural gravel and soil road materials are mostly residual soils, residual weathered rocks or pedocretes which are inferior in quality to materials found in northern Europe and North America. Although they can be used to effect substantial savings in costs, the conditions under which marginal and non-standard materials can be successfully used are not yet well-understood, and further research is necessary. Greater political and engineering acceptance of appropriate standards and risk would probably enable more unpaved roads to be surfaced

Keywords: Local materials; roads; aggregate, residual soils, calcretes.

Introduction

As materials make up some 70% of the cost of a typical rural road (Mitchell et al..1979) it is essential that the optimum use be made of local, 'low-cost' materials. For this to be achieved the designer must be aware of what materials are actually available as well as of their limitations in terms of potential problems and performance. Neither the location (that is the finding) of materials nor their performance is an exact science and both are still subject to research.

This paper provides a short review and guide to further local information rather than a detailed review of the materials used locally or specifications for their use; for such a review, see Netterberg and Paige-Green (1988a, 1988b).

'Low-cost' materials

The term *low-cost,* and especially *low-cost road,* is regarded by some engineers as a misnomer because all roads should be engineered to provide the desired performance at the lowest possible cost. Terms such as *low-volume road* or *light-duty pavement* are preferred in this respect. However, the term *low-cost material* can be taken to mean natural gravels with CBRs of 25-80 or more (G6-G4 materials as defined further in TRH 14, (Committee of State Road Authorities (CSRA), 1985a) and soils and gravel-soil mixtures with CBRs of 3-24 (G10-G7).

Little or no processing is implied other than, possibly, loosening of the *in-situ* material by ripping and breaking down (usually with a grid roller) or removing oversize particles. Such materials are typically about one-quarter of the price of graded crushed stone G1-G3 in TRH 14). These low-cost materials may be used raw or modified with sand or, more often, with 2-4% lime or cement. Crushing may occasionally be required. Waste materials such as mine dump rocks and slag are also useful sources of low-cost materials, some of which can be processed to make high-cost, but high-quality materials such as G1 graded crushed stone and macadams suit for heavy-duty applications.

Depending on quality and other factors, the materials discussed in this paper can be used for base courses under thin surfacings for roads with a structural capacity ranging from 0,2 million to 80 kN standard axles (E80 (or less) up to about a cumulative maximum of about 3 million standard axles over a design life of 10 or 20 years (Netterberg, 1988).

Standard materials

Standard materials may be defined as those materials which meet, for example, AASHTO, ASTM, or British or French Ministry of Transport specifications or, locally, those of TRH 14 (CSRA, 1985a) or CSRA (1987). Such materials are tolerant of construction mishandling and adverse environmental conditions and will probably perform well in most cases (Metcalf, 1991). Such materials are conservative in their performance or classification parameters when used as intended, for example, as in TRH 4 (CSRA, 1985b).

An essential feature of all specifications for standard materials is a requirement for strict compliance with limitations on particle size distribution (grading), plasticity index (PI) and aggregate strength. However, material specifications differ from authority to authority, and often a standard material of one authority may be unacceptable to another.

Non-standard materials

Non-standard materials can be regarded as any materials which do not accord with one or more of the requirements for a standard material, for example, grading or PI. Such materials are also called *marginal* or *substandard.* It has become increasingly recognized worldwide (for example, Brunschwig, 1989; Gidigasu, 1991; Metcalf, 1991; Netterberg, 1988; Netterberg and Paige-Green, 1988a, 1988b) that under favorable circumstances many such materials can be used successfully. However, special consideration may have to be given to aspects such as durability of the aggregate (Weinert, 1980; Pinard et al., 1987; Sampson and Netterberg, 1989; Sampson, 1991), durability of lime and cement modification (Netterberg, 1991), damage due to soluble salts and acids (Netterberg, 1979; Hoehler and Rimmer, 1989; Obika and Freer-Hewish, 1990), scabbing and /or punching of the surfacing, and a variety of construction problems (for example, Netterberg and Paige-Green, 1988a; Netterberg et al., 1989). Although further research is needed in order to make the optimum use of local low-cost materials, many of these problems can be avoided on the basis of present knowledge.

Political and engineering acceptance of risk

Whilst many successful examples of the use of non-standard materials can be quoted, few are well-documented, and the conditions necessary for successful performance have not yet been defined. There is therefore an understandable reluctance to utilize non-standard materials because of an undoubtedly greater *risk* of problems or even failure.

On the other hand, in Australia, a country six times the size of South Africa, greater political, public and engineering acceptance of such risks has apparently enabled the surfacing of practically all roads carrying more than about 50 vehicles per day (v.p.d.). In contrast, in South Africa, some 5% of the unpaved roads (that is, about 7000 km) carry traffic in excess of 200 v.p.d. (Visser and Van Niekerk, 1987). Greater local acceptance of such risk should enable these roads and others to be economically surfaced, provided that it can be shown that this will not lead to an unacceptable increase in maintenance costs. More long-term performance studies – such as those reported by Meireles (1967), Laboratoria Nacional de Engenharia Civil et al., (1967), Laboratoria Nacional de Engenharia Civil et al., (1969), and Grace (1991) for laterites, Netterberg (1982), Overby (1983, 1990), Lionjanga et al., (1987) and Netterberg and Pinard (1991) for calcretes, and Overby (1990) for some other materials – are sorely needed.

Whilst not wishing to denigrate in any way the advances in this field made by the road authorities, it is at present perhaps easier for a private organization to take such risks. One such example is a 65 km private road in the diamond area on the arid west coast (Spottiswoode and Graham, 1982). The existing unpaved road was upgraded to the standard of a low-volume paved road at one-quarter of the cost of a road built to normal provincial standards. Local materials which were non-standard in most respects were used, and seawater was used for compaction. This road represents the first full-scale application of the author's research findings with respect to the utilization of calcretes (Netterberg, 1982) and saline materials (unpublished). Along with several other roads and experimental sections, this road is being monitored by the author as part of a DOT-funded long-term road-performance study in dry areas.

South African materials

Because only negligible areas of southern Africa were subjected to the glaciations of the last few million years there are no deposits of clean, durable, fluvioglacial gravels such as are used over much of northern Europe and North America. In contrast, the local scene is one of considerable depths of weathering and pedogenesis. The materials used are therefore mostly residual weathered rocks such as dolerite, granite, mudrock and quartzite, and pedocretes such as calcrete and ferricrete (laterite). Transported materials are generally talus (scree) gravels, sandy hillwash, acolian sands and, occasionally, alluvial gravels. Even nominally fresh igneous rock used for surfacing chippings, crushed rock bases and concrete often contains significant amounts of secondary minerals. Waste materials are generally only of localized importance.

Owing to the strong dependence of engineering performance on the geological and chemical nature of the material (Buckle et al., 1987) and climate (Brink, 1979; Weinert, 1980), the materials used can be usefully classified into the following nine groups (Weinert, 1980), arranged in order of the arcal extent of their use in South Africa:

basic crystalline rocks (e.g. dolerite, basalt or andesite), 67%
perdocretes (e.g. calcrete, ferricrete or silcrete), 66%
high-silica rocks (e.g. quartzite, hornfels or chert), 55%
arenaceous rocks (e.g. sandstone or conglomerate), 48%
argillaccous rocks (e.g. mudstone, shale or schist), 47%;
acid crystalline rocks (e.g. granite or gneiss), 24%;
carbonate rocks (e.g. limestone or dolomite), 16%
metalliferous rocks (e.g. ironstone), 9%;
diamictites (e.g. tillite), 5%.

On a subcontinental basis, calcrete is the most widely used material. Each group of materials has a characteristic range of properties and problems, and test methods and specifications must therefore take this into account (Netterberg and Paige-Green, 1988a). For example, whilst a PI of 15 may prove satisfactory in a calcrete base, anything more than about five is risky in a dolerite base, even if stabilized with cement or lime.

A general account of the engineering geology of southern Africa has been given by Wink (1979, 1981, 1983, 1985) whilst the natural roadbuilding materials have been described by Weinert (1980, 1990) and the roadbed problems have been described by Netterberg (1992). Local pavement materials are generally weaker than those of northern Europe and North America, but roadbeds are generally stronger, and climatic and traffic conditions are more favourable. In these respects southern Africa is probably most similar to Australia.

Material specification and pavement design

Paved roads

Specifications for materials must be coupled to a particular pavement- design method. Thus TRH 14 should be used with TRH 4 or some other suitable departmental pavement-design method *and* the appropriate test methods (TMH 1) (CSRA, 1986a) must be used. Road Note 31 (Transport and Road Research Laboratory, 1977) is also used in some southern African countries, but it may be risky to mix its specifications with TMH 1 test methods. The French guide to tropical pavement design (Centre Experiméntal de Recherches et d'Étudés du Batiment et des Travaux Publics, 1984) does not so far appear to have been used locally.

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Evaluation of material quality by the CBR method and its utilization in thickness design has so far invariably been undertaken after 4 days soaking.. However, as the equilibrium moisture content in most roads is less than MAASHO optimum (Emery, 1992) there appears to be scope for design on an unsaturated basis. CSRA (1987) and TRH 14 and to a lesser extent SABS 1083 and 1200 material specifications are most used. Further guidance in the case of rural roads is given in TRH 14, in bofinger et al.(1990), in Netterberg and Paige-Green (1988a) and in roads department handbooks. Whilst the same principles apply to urban streets, the generally poorer drainage and greater consequences of distress of failure probably necessitate more conservative designs. Guidance is given in Horak et al. (1988), UTG 3 (Committee of Urban Transport Authorities, 1988) and Paige-Green and Sampson (1990).

Materials practice is conservative in comparison with Australia, and many materials are used after modification with 2 – 4% cement of lime. The current stabilization practice is described in TRH 13 (CSRA, 1986b). As there have been a number of cases of surface disintegration during construction (Netterberg et al., 1987) and loss of cementation in service (Pinard, 1987; Sampson et al., 1987 Netterberg, 1991), increased attention during design and construction to durability aspects – especially carbonation (Netterberg and Paige-Green, 1984 – is required (Netterberg, 1991). The extra over costs of modification are substantial – usually approximately trembling the cost of the untreated layer – and it can be cheaper to haul good base material for 15 km in preference to modifying it with even 3% of road lime. Controlled experimental sections have shown some materials have an improved performance when used *unmodified.*

The quality of single-sized stone for surfacing chippings has seldom been relaxed except in the case of sand, primer and Otta (graded aggregate) seals (Wolff and Visser, 1991). Whilst substantial relaxations are possible (Netterberg and Paige-Green, 1988a; Woodbridge et al.,1991) savings on cost are not substantial – about 7% of the cost of the surfacing (presumably unless very long haulages are involved) – and must be weighed against the increased risk of failure and shorter life (Southern African Bitumen and Tar Association (SABITA), 1992). Surfacing are usually designed according to one of the methods in TRH 3 (CSRA, 1986c). Guidance on appropriate surfacings for low-volume roads has been given by SABITA (1992). The bituminous products usually used in southern Africa have been described by SABITA (1878).

Unpaved Roads

Unpaved gravel or soil roads still make up about three-quarters of the rural road network in South Africa as a whole, and they make up perhaps 90% in developing areas and in some adjacent countries. The materials used are similar to those used for paved roads except that considerable relaxation of material quality is permitted (it is often only judged visually), processing is limited to removal, breaking down or pushing down of oversize particles, and haulage is kept to a minimum. The requirements and specifications for wearing courses have recently been reviewed (Netterberg and Paige-Green, (1999b), new performance-related specifications have been developed (Paige-Green, 1989) and a draft guide on structural design, construction and maintenance has been produced (CSRA, 1990). Guidance as to when it is economic to surface an unpaved road is also available (e.g. SABITA, 1989).

Prospecting and proving

Prospecting for (location of finding of) materials is as much an art as a science. It does not always receive the attention it deserves (Mitchell et al., 1979). It cannot be emphasized too much that it is impossible to make the best use of low-cost materials unless the designer is fully aware of at least the approximate quality, extent and location of *all* the materials within an economic haul distance of the proposed road. Ideally, this involves the mapping of all soils and potential materials in the landscape according to TRH 2 (CSRA, 1978), but this is not always necessary or feasible.

The methods used include local knowledge, aerial photography and satellite-imagery interpretation, botanical and other indicators, geological and pedological maps, and probing and pitting (Brink et al., 1982; Netterberg, 1985). The methods available and in use are currently under review (Netterberg, in preparation).

Deposits of natural gravel are often extremely variable, and careful proving is necessary. It is best to approve a deposit only for stockpiling, then to retest the stockpile before approving it for a particular use.

Conclusions

1. Substantial savings in construction costs (and probably in overall costs) are possible if the best use is made of local 'low-cost' materials. At the same time care must be taken to avoid significantly increased maintenance costs.
2. Such materials are, however, often deficient in some respect, and political, public and engineering awareness and acceptance of increased risk is necessary for their optimum use.
3. Further research is necessary in order to define the conditions under which such materials can be used and the overall cost (that is, construction and maintenance costs and salvage value) savings possible.
4. Southern Africa materials are different to most northern European and North American materials and appropriate local guidance and specifications are necessary for their optimum use. In general, local pavement materials are weaker, roadbeds are stronger, and climatic and traffic conditions are more favourable in southern Africa.
5. A large proportion of the existing road network consists of unpaved gravel and soil roads and low-volume paved roads. Much scope therefore exists for the upgrading of unpaved roads and streets to light-duty-paved-road standards.

6. The designer must be fully aware of what materials are actually available if the best use is to be made of local low-cost materials. This may necessitate increased attention to methods of material prospecting.
7. The optimum use of local low-cost materials requires more rather than less engineering input.

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SARF COURSE

PRACTICAL ROAD PAVEMENT ENGINEERING

3. SAMPLING AND TESTING

SAMPLING AND TESTING

ENVIRONMENTAL

BASIC REQUIREMENTS FOR ROADS

BORROW PITS AND QUARRIES

- Appoint an independent environmental consultant to draw up an EMP which would cover making excavations safe, general re-establishment of the source and revegetation

  Typical Approach

  a) Draw up draft EMP and submit to Client for approval.

  b) Submit EMP to Regional Office of Mineral and Energy Affairs for their approval.

    - When M & E approval gained write EMP into contract documentation making allowance for any necessary work in the Bill of Quantities.
    - During construction liaise with both environmental and safety offices from M & E,
    - At completion of contract hold final inspection with and obtain Closure Certificate from M & E.

REHABILITATION/UPGRADE AND ROUTINE MAINTENANCE

Usually covers work within the existing road. However consult Provincial Environmental Authority and confirm that there are no major environmental problems and thus that no further input is required. Typically thereafter draw up an EMP for any construction works required.

SAMPLING AND TESTING

ENVIRONMENTAL

BASIC REQUIREMENTS FOR ROADS

NEW ALIGNMENT (outside Urban areas)

- Apply to the relevant Provincial Environmental Authority. They will advise on the level of environmental input required.

Typical Approach

1) Scoping Report

which would contain: Description of activity (what and why)
Affects on the environment - Social and Biophysical
Assessment of the activity's impact on the Social and Biophysical.

a) Produce draft report and make available to the public and interested/affected parties for feedback.
b) Submit report with comments and feedback to the Environmental Authority (EA).
c) EA take decision – if Approved, usually with conditions.
d) Advertised with an appeal period.

2) On approval of Scoping Report an Environmental Management Plan (EMP) is prepared to address environmental requirements during construction and operation.

For new alignment within an urban situation a similar procedure would be followed. However, the situation is usually more complex in that there are a lot more affected/interested parties. Time to reach consensus can take years and this should be recognized at preliminary planning stage when drawing up a programme for the project.

SAMPLING AND TESTING

1. TRIAL PITS

These are most commonly excavated by hand but mechanical means such as a backactor, power auger or dozer may be required.

The trial pits must be of adequate size for the depth being investigated and sensibly distributed to facilitate estimates of quantities.

A profile for each pit is required and information such as:

- Moisture.
- Colour.
- Consistency.
- Soil type.
- Structure.
- Origin (residual, transported).

As soon as test pits are no longer required for further inspection they shall be filled in to normal ground level immediately.

Where test pits are required for further inspection they need to properly covered with purpose made timber or corrugated iron covers fixed firmly in place, or suitably fenced. The covering of pits with branches is ineffective and should not be used.

In the case of borrow pits, these should be located with due regard to their impact on the environment and the reinstatement proposals required by regulations promulgated in terms of the Minerals Act (Act 50 of 1991), The Mine Health and Safety Act (Act 29 of 1996), and The National Environmental Act (Act 107 of 1998).

Borrow pits should be located as near as practicable to points on the road where material will be required. Trial pits and / or trenches, cuts, or boreholes should establish the depth and quantity of materials. Mechanical means will generally be required to obtain representative samples. The full depth of the borrow pits should be sampled.

For each trial pit the grading, soil mortar test and Atterberg Limits shall be carried out on each type of material encountered, i.e. the different horizons.

CBR tests shall be carried out at a frequency of one test per trial pit per type of material with a minimum of five CBR tests for each type of material encountered in the borrow pit.

SAMPLING AND TESTING

2. SAMPLING

Should be carried out in accordance with SABITA Manual 37 / TMH 5 (2021).

G^2 : Garbage in = Garbage out

Sampling is very important to obtain information on the properties of the various materials that will be encountered on a project.

The properties of materials in the area in which a road is constructed or rehabilitated have a tremendous impact on the economy, structural and functional performance of the pavement design.

The main objective, therefore, is to obtain a representative sample of each type of material for testing.

However, to obtain a representative sample is complicated by the following factors:

- At a single location there can be a greater change in material character with increase in depth than over the length of the project, which leads to the taking of more than one sample in order to make a meaningful decision on quality and quantity of each material.
- Materials encountered on the project can vary from solid, hard material to a soft, loose state, which requires different excavation techniques from hand digging to core drilling.
- Variation in the particle size, the number of samples to be taken and the tests to be performed.

The need, therefore, for accurate sampling is essential. If the samples are not truly representative, the testing is a complete waste of time and the viability of a project can be jeopardized.

Avoid sampling from an already completed stockpile, especially in the case of coarse graded material, crushed or natural, when the sampling is done for the purpose of determining characteristics that may depend upon the grading of the material.

In the sampling of such stockpiles, it is very difficult to ensure representative samples due to segregation, which often occurs when coarse particles roll to the base of the stockpile. This becomes complicated with stockpiles scraped together from natural deposits with bulldozers.

Normally the sampling of material during processing at the plant or borrow pit is the responsibility of the supplier or contractor for the purposes of process control.

SAMPLING AND TESTING

However, if circumstances make it necessary to obtain samples from a completed stockpile, a sampling plan should be designed in liaison with the supplier, or contractor, which will give results, which are representative of the properties of the stockpile.

The sampling plan should define the minimum number of samples, normally not less than five. Where one sample represents a quantity of not more than 200 m^3 in the case of coarse graded materials (crushed or natural). In the case of single sized or fine aggregate for concrete or asphalt surfacing, one sample shall represent not more than 30 m^3.

Duplicate Samples

To ensure proper quality control it is important to verify the quality of testing as carried out by laboratories. Samples, which have been randomly selected, should be obtained and tested by an approved, reputable, and impartial laboratory.

A duplicate sample, prepared as described in the test method, is a one-to-one division of a representative sample into two or more equal sized portions. These portions shall be identical in all aspects, which have an effect on the quality of the material.

All samples must be adequately labelled. Labels must be attached to the sample as well as one inside the sample bag and such labels should reflect the following:

- The project.
- The location.
- The borrow pit or layer.

3. TESTING

IMPORTANT: Most test methods have been converted from TMH1 to SANS (SABS) standards - Series 3001 and 4001. Check the status of each method.

INDICATOR TESTS

Grading analysis and hydrometer analysis: SANS 3001-GR1, GR2 and GR5

Grain sizes in soil samples are tested in two ways. The *sieve analysis* is used for sands and gravels, and the *hydrometer test,* for silts and clays. If significant quantities of both coarse- and fine-grained soils are in the sample, the results of both tests may have to be combined to plot the grain size distribution curve.

The grading analysis provides a general description of a material in terms of its material properties and compactability.

SAMPLING AND TESTING

Sieve analysis: Soil samples are sieved through a nest of standard sieves - two methods are used (dry and wet) and the percentages passing through each sieve calculated. Care must be taken not to overload the sieves as inaccuracies may occur.

Hydrometer test: A soil specimen is mixed with a mixture of distilled water, sodium oxalate solution and sodium silicate solution, allowed to stand overnight. It is the dispersed, placed in a graduated cylinder flask and a hydrometer inserted. The flask is topped up to 1205 mL and the hydrometer removed. The mixture is then agitated to ensure even temperature distribution and placed in a water bath at 20ºC for 1 hour. During this time it is agitated occasionally to ensure even soil particle distribution. The hydrometer is inserted and an initial reading taken. Further hydrometer and temperature readings are taken at predetermined time intervals. Particle sizes are determined by equations based on Stokes Law

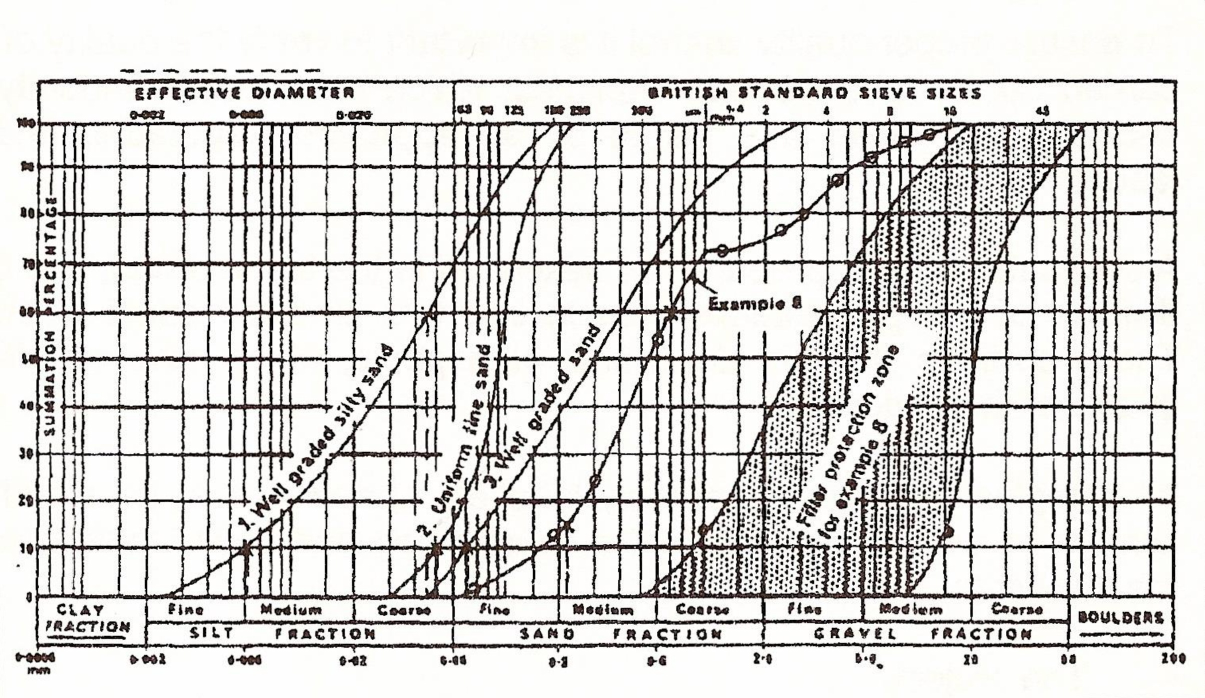

Figure 1: Particle size distribution curves

Terms used to describe materials can be based on the following:

%Passing 0,425mm Mesh	Description
> 85 50 – 85 < 50	Soil Soil plus sand and gravel Gravel

SAMPLING AND TESTING

Sand is defined as material that conforms to the following:

Percentage passing 4,75mm sieve	:	95% minimums
Percentage passing 0,425mm sieve	:	50% minimums
Percentage passing 0,075mm sieve	:	10% maximum
Plasticity Index	:	Non-plastic

Clay is is that material passing a 0,002 mm sieve – usually determined by means of a hydrometer – after 60 minutes. In the case of silt it is the percentage passing the 0,060 mm sieve minus the clay, i.e. the 40 second reading minus the 1 hour reading.

The *Grading Modulus* (GM) of a material is an index which provides an indication of the coarseness or fineness of the materials relative to the last three standard sieves used in the sieve analysis. It is defined as the cumulative percentage of material which is retained on the 2.0mm, 0.425mm and the 0.075mm sieves divided by 100. The coarser the material the higher the GM and visa versa. The maximum value is 3 and the minimum is 0. It can be expressed as follows:

$$\boxed{\text{Grading modulus} = [\ 300 - (P2{,}00 + P0{,}425 = P0{,}075)\ ] / 100}$$

Where:

P2,00 is the percentage passing the 2,00mm sieve.

P0,425 is the percentage passing the 0,0425mm sieve.

P0,075 is the percentage passing the 0,075mm sieve.

Plasticity

Fine-grained soils with similar particle size distributions may exhibit behavioural patterns which are markedly different. In order to classify fine-grained soils, a criterion other than grain size distribution is necessary and the criterion most widely used for this is plasticity.

Plasticity refers to the soil's ability to undergo permanent changes of shapes without showing signs of rupture or undergoing volume change.

Plasticity is a property exclusive to soils in which clay mineral particles are present. Clay particles attract one another as well as water to their surfaces. The thickness of such absorbed water films determine the ease with which the particles may slide relative to one another and for that reason the water content of a soil which contains a significant proportion of clay particles materially affect the behaviour of the soil.

SAMPLING AND TESTING

The amount of plasticity a soil possesses is measured by its Plasticity Index or PI.

The plasticity index is the magnitude of the moisture content range over which the soil is in the plastic state. In other words, it is the difference between the liquid and plastic limits of that soil:

$$PI \quad = \quad LL \quad - \quad PL$$

Since the liquid limit LL, and the Plasticity Index, PI, is also shown as a percentage.

The Liquid limit and Plastic limit are also termed the Atterberg Limits.

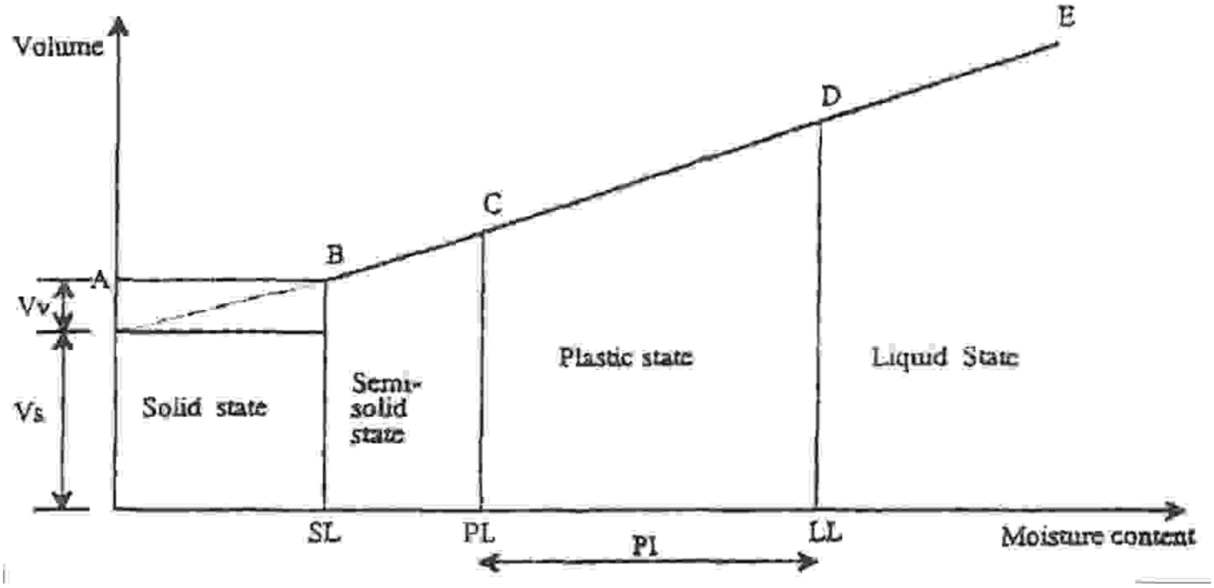

Figure 2: Graph of volume versus moisture content

Atterberg Limits *(SANS 3001-GR10)*

Determined on the soil fines of a material, i.e. finer than 0,425 mm.

The *Liquid Limit* is that moisture content of oven dried soil at the boundary of liquid and plastic states.

The Liquid Limit is determined by means of a special device.

Water is added to soil fines and mixed for 10 minutes.

The wet material is grooved. The grooved material is normally given 3 ranges of taps @ 2 taps / second until the soil meets across a distance of 10mm – a flow curve is then drawn and the moisture content corresponding to 25 taps is the Liquid Limit. There is also a none-point method, which will be described this afternoon.

The *Plastic Limit* is then determined.

This is the moisture content at the boundary between plastic and semi – solid states. Basically, it involves drying of the material and rolling it until crumbling occurs when the thread has a diameter of slightly greater than 3 mm.

The Plasticity Index, P.I. = L.L. - P.L.

Application of the Atterberg Limits:

- Classifying soils for both agricultural and construction purposes
- Used in specifications for
- The test can indicate expansiveness, compatibility, strength, permeability and sensitivity to water

Linear Shrinkage this test of the shrinkage of a soil bar is useful as a check of P.I. particularly in the case of certain materials, eg. Calcretes.

The grading, P.I. and type of material together, can give a very good indication of their engineering characteristics, e.g. if you have a well graded, hard shattered dolerite or sandstone with a P. I. of less than say 8 you can virtually be sure that this material will have CBRs of at least sub base quality. On the other hand one would be less sure in the case of shales or mudrocks.

Reasonably graded decomposed dolerites with coarse sand fraction < 50 and P.I. <10 will probably yield CBRs of 35 – 40.

In the case of fine-grained soils, when the PI exceeds 15, the material is referred to as clayey. When it exceeds 15 and has a CBR value of less tan 3 at 100% MDD compaction it is referred to as clay.

4. MAXIMUM DENSITY TESTS

<u>The Maximum Dry Density. (SANS 3001-GR30 natural and GR31 for stabilized material)</u>

A series of specimens of the same material but with varying moisture contents is compacted into cylindrical moulds in five layers, each of which is compacted by dropping a 4.5kg rammer, 55 times from a height of 457.2mm before adding the next layer. After compaction, each sample is weighted over to determine its wet density, while a small sample is taken of the left over material to determine the moisture content and hence the dry density of each moulding. The dry densities are then plotted against moisture contents. A parabolic curve is then obtained from increasing density/moisture relationships and the Maximum Dry Density (MDD), i.e. at 100%, and Optimum Moisture Content (OMC) so obtained.

SAMPLING AND TESTING

The Maximum Dry Density (MDD) has a twofold purpose:

1. It provides a Moisture Content to aim for when compacting both in the lab and in the field.
2. It provides a 'standard' density against which to check densities actually achieved.

Note: In the field different compaction plant can achieve the required compaction at Moisture Contents other than the MDD OMC.

The Modified Proctor test is similar to the foregoing test but is carried out using a smaller mould with less effort (lighter tamper).

The Maximum Dry Density (MDD) and Optimum moisture Content (OMC) of material changes with change in compactive effort, the higher the compactive effort, the lower the OMC and the higher the MDD. Thus the MDD and OMC represent only a point on the graph as depicted in Figure 3. It is, however, a useful tool against which to judge laboratory and field compaction, i.e. as a standard density in a specification.

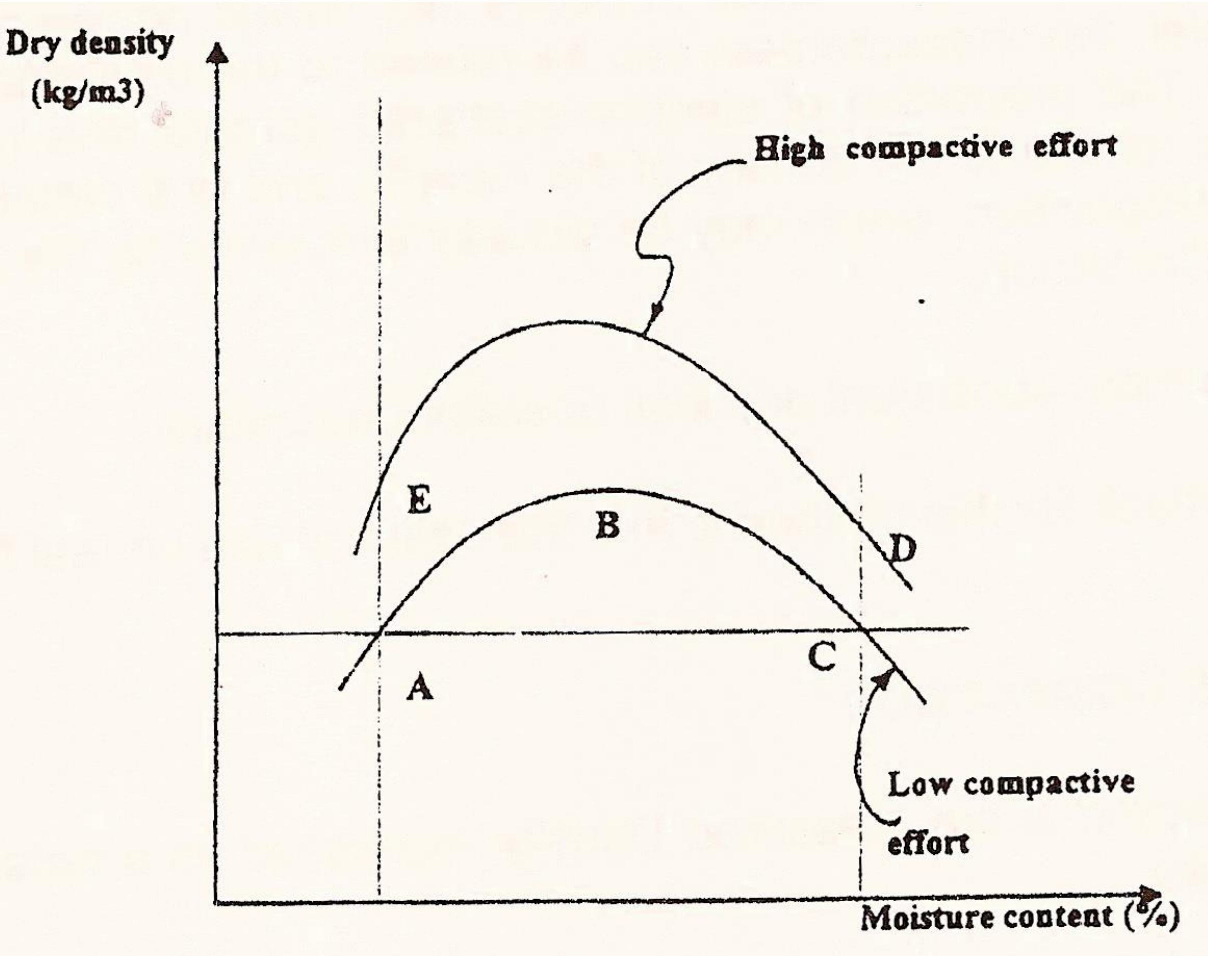

Figure 3 Effect of varying the compaction effort

SAMPLING AND TESTING

Apparent (Relative Density) (AD) SANS 3001-AG22

Cohesionless materials, such as crushed stone, does not have a clearly defined moisture density curve and hence it is difficult to determine the maximum density and optimum moisture content in a similar manner to that of natural gravels. Therefore, instead of using the maximum dry density, crushed stone is controlled using another reference density called the apparent density. This apparent density is that density which would theoretically be achieved if the material were compacted to such an extent that no air or moisture remains between aggregate particles.

This is also used as a 'standard' density (% of solid) and depends solely on physical properties of individual particles (similar to the old S. G.).

Bulk (Relative Density) (BD) SANS 3001-AG20 and AG21

ARD does not take water permeable voids into consideration whereas BRD does and hence the slightly lower value of the latter for the same aggregate (in the case of BRD the aggregate is soaked for 24 hours resulting in water penetrating the water permeable voids).

5. IN SITU DENSITY

In South Africa basically only two types of tests are used:

Sand Replacement SANS 3001-GR35

This method is best known to most 'older' technicians / engineers.

It consists of excavating material from the roadway – calibrating the volume of the material so removed and determining its density. In order to do this one makes use of a density ring – funnel – hammer and chisel and calibrated 'density sand'.

This method was used extensively in the past but has lost favour due to its variableness (especially with coarser materials) and the need for a good experienced consistent tester.

Nucleonic Gauge SANS 3001-NG1 to NG5

The nucleonic gauge enables the engineer to make quick, non-destructive in situ measurements of the density and moisture contents of the soils and other pavement materials before, during and after compaction. The materials are irradiated with gamma rays and high energy neutrons from a radio-active source within the apparatus. This is placed on the surface of the material being tested. The fast neutrons are slowed down by collisions with light nuclei in the material and the instrument measures the flux of back-scattered, slow neutrons at the surface, by means of a slow neutron detector which is insensitive to fast neutrons. Since neutrons are slowed down principally by hydrogen nuclei, this measurement can be related to the moisture content of the material. The proportion electron-scattered gamma rays received by its chemical composition may be virtually eliminated by the application of a Moisture Correction. From practical experience it has been determined that the gauge moisture content should be checked against gravimetric moisture contents taken directly from the compacted layer.

This method is more consistent and less operator dependant.
The main drawback for this method is that the operator has no real feel for the data output.

6. RELATIVE COMPACTION

Relative Compaction is the Measured Density expressed as a percentage of 'Standard' density.

e.g.	Field Density	=	2065	kg/m^3
	MDD	=	2131	kg/m^3
	AD	=	2575	kg/m^3

Relative Compaction (MDD)	=	96,9%	NB. Similar packing but different standards.
Relative Compaction (AD)	=	80,2%	

7. CALIFORNIA BEARING RATIO SANS 3001-GR40

This is a crude strength test on saturated samples prepared at three densities.

The MMD and OMC of a test sample is determined. Three separate compactive efforts at 100% MMD (A), about 97% MMD (B) and about 93% MMD (C) are compacted and soaked. The samples are then penetrated using a standard plunger at a rate of 1.27mm/minute. Readings are taken of both load and penetration for the 2,54 mm, 5,08 mm and 7,62 mm penetrations at regular intervals, generally 30 seconds, and results plotted against each other on a graph.

These forces are expressed as a percentage of a standard force (originally from work by O.J. Porter of California) = CBR.

The CBR values at 2,54 mm penetration versus Relative Compaction (A, B and C mould) are then plotted.

The CBR is usually expressed at a relative compaction equivalent to the minimum requirement for a particular layer.

i.e.	Subgrade	CBR @ 90% MDD =	10
	Subbase	CBR @ 95% MOD =	45
	Base	CBR @ 98% MOD =	80

SAMPLING AND TESTING

Problems: The course material (+ 37 mm) is screened out and this affects the grading of the sample. The test, which is really a bearing test, is very dependant on particle size and grading, and a single static load determines the CBR. Anomalies can arise with lower bearing values being obtained at the greatest compaction effort as a result of aggregate breaking down. The test is carried out in a saturated condition – is this representative of in situ material? The CBR has very poor reproducibility.

Swell is measured during soaking and gives a good indication of moisture sensitivity.

In the case of sand, the bearing strength criterion is the same as for other material but 100% compaction is required instead of 90%. A fair definition of sand is:

- Percentage 0,075 mm < 10% + PI = NP (Sand)
- Percentage 0,075 mm > 10% = PI = SP (Silt)

Obviously moulding moisture, dry density and CBR are interrelated.

8. Dynamic Cone Penetrometer (DCP)

This test was originally developed as a mini SPT (Standard penetration-test) for testing of fine – grained non-cohesive materials.

It has subsequently been used in other materials (without adaptation) by Kleyn (TPA) and others – who developed correlations with CBR & UCS.

It measures the resistance of the in situ materials to penetration of the cone in mm / blows = DN.

Drawbacks:
- It is moisture sensitive.
- It is material type sensitive (depending on gravel content).
- It is controlled by in situ moisture at time of test.
- It is sensitive to overburden pressure.
- It is sensitive to plasticity.

Warning: Correlations with CBR + UCS should be treated with extreme caution. Other data must be carefully considered. But ……… it is better than sticking your thumb in it, and very useful for relative comparisons, i.e. checking uniformity or soft spots.

9. Stabilization

Stabilization is normally confined to subbase and/or selected layers of the pavement structure and is generally subject to the quality of materials available. Limited road funds however, may also dictate stabilization of an existing base as a first stage rehabilitation action to be followed by an overlay at some future stage.

SAMPLING AND TESTING

Stabilization can be defined as the process of improving the engineering properties of material by means of the addition of:

- Calcium, magnesium, or dolomitic lime conforming to SABS 824.
- Portland cement conforming to ENV 197/1.
- A mixture in equal proportions (by mass) of Portland cement and milled Granulated blast furnace slag or lime and milled granulated blast furnace slag.
- A combination of the above.

The process whereby stabilization is achieved by using either cement, cement/slag, lime/slag or lime are briefly as follows:

Cement or Cement / Slag

Cementation due the formation of strongly cementitious hydrates which bond soil particles together. The hydration proceeds largely independently of the aggregate and do not rely on chemical interaction between cement and aggregate for development of strength.

Lime

A change in the state of aggregation of the clay particles as a result of a cation exchange. The physical properties of the material itself are the changed. This process is generally termed modification and takes place fairly rapidly. In addition, normally over long period of time and under favourable conditions oil-lime pozzuolanic reactions may take place during which hydrates similar to those encountered in cement are formed, leading to a cementing action.

It should be appreciated that the processes of cementation and modification are not necessarily distinct and may occur simultaneously.

Apart from instances where it is desirable to stabilise subgrade in order to expedite construction progress, the objectives of stabilisation of pavement layers are:

- Reduction of construction costs by improving the properties of substandard, readily available material where such stabilisation is a cheaper alternative to the procurement of materials complying with the relevant specifications.

- The achievement of tensile strength due to an increase in cohesion as a result of cementation.

Modification

Modification tests are performed on uncured material, irrespective of the type of stabiliser, although it will most likely be lime. Generally, the only material properties under consideration in this respect are Plasticity Index and California Bearing Ratio. In view of the inherent variability of materials and non-uniformity of mixed-in stabiliser, the target values for materials design should be chosen such that this variability is accounted for. The following guidelines are given:

Target CBR (SANS 3001-GR41) = CRB min + 25 Target PI = PI max - 2

The practical minimum amount of stabilizer to ensure adequate distribution is:

Stabiliser	Content % by mass
Lime	1,25% (by mass)
Cement	2,0% (by mass)

Cemented Materials

The development of tensile (cohesive) strength is gauged by means of the cured Unconfined Compressive Strength (UCS) and Indirect Tensile Strength (ITS) as described in SANS 3001-GR53 and GR54. The UCS / ITS of a stabilized material is defined as the load in kilopascal required to crush / shear a cylindrical specimen 127,00 mm high and 152,4 mm in diameter to total failure at a rate of application of load of 140 kPa/sec for UCS and 40 kN/min for ITS. The Specified (COTO) field test curing criteria is ambient temperature for 7 days and compaction at 100% Maximum dry density.

For practical reasons and to obtain test results more rapidly the use of 24 h rapid curing may be considered. This consists of curing the specimens in an oven for 24 h at 70º C (cement) and 45 h at 60º C (lime); prior to them being soaked for 4 hr in water (UCS only not ITS). However, a good number of test results are required in order to assess the relationship (factor) between 24 h and 7 day test results.

Cracking in Cement Treated Layers

Cracks in cement-treated layers cannot be avoided and must be accepted as an essential feature of cement treatment. However, cracking may cause structural and maintenance problems.

There are essentially two types of cracks in cement treaded layers:

- Cracks that are not caused by traffic and these are usually referred to as ‘initial cracks’ due to the percentage and type of stabilizer, type of material, excessive moisture, ineffective curing etc.

- Traffic associated cracks – due to construction vehicles or insufficient cover.

There are several design and construction techniques which have been shown to minimize or eliminate these causes. These techniques include the following:

SAMPLING AND TESTING

α) The compaction moisture content should be limited to 75% of saturation moisture content
β) Thorough mixing of the natural soil and of the soil/stabilizer mix should be done to minimize thermal stresses and to minimize differential water absorption or drying out of the material.
χ) Thorough curing of the mixture, including prevention of wetting/drying cycles with associated thermal gradients.
δ) The quantity of rapid cementing stabilizers should be limited to 3,5%, in the case of lime this could be increased to 5%.

<u>The determination of the unconfined compressive strength of stabilized soils, gravels and sands</u> : SANS 3001-GR50 to GR53.

First the optimum moisture content (OMC) and maximum dry density (MDD) of the stabilized material are determined using. Compaction is delayed for 4 hrs to simulate conditions on the road.

Mixtures of cement, milled blast furnace slag, lime or other additives usually take longer to reach the same strength as that of ordinary Portland cement.

<u>The determination of the indirect tensile strength of stabilized materials :</u>
SANS 3001-GR50, GR51 and GR54.

The indirect tensile strength of a stabilized soil, gravel or crushed stone is determined by measuring the resistance to failure of a cylindrical prepared or cored specimen when a load is applied to the <u>curved sides</u> of the specimen.

Method

Laboratory Compacted Specimens

Make and cure the specimens as described in SANS 3001-GR50 to GR52. After curing, place a specimen on the bottom loading strip and place the top loading strip diametrically opposite the bottom strip on top of the specimen.

Centre the load transfer platten on top of the top-loading strip and place the assembly centrally under the loading ram of the compression-testing machine. Apply a load of 0,1 kN to the seat the loading strips and inspect the assembly for symmetry.

Apply a load to the specimen without shock at a constant rate of 40 kN / min until failure. Record the maximum applied load accurately to 0,1 kN and note the mode of failure.

<u>Stabilization Design</u>

Determine the stabilized MDD and OMC (SANS 3001-GR31) using the middle stabilizer content. Compact and cure specimens, at 3 selected stabilizer contents chosen over a sufficiently broad range to produce a soil-cement mixture conforming to a specified strength (UCS or ITS). The results should be recorded and a graph of stabilizer content against strength should be plotted. For the graph the average strength of the three specimens of each stabilizer content should be used. An obviously incorrect result, due to possible damage to a specimen before testing, should be ignored.

Report the average of the laboratory compacted specimens for each stabilizer content, together with the following:

- The maximum dry density.
- The optimum moisture content.
- The mean density and relative degree of compaction of the specimens for each stabilizer content.
- The date tested.
- The mode of failure, e.g. clean break, crumbling, etc. and the method of curing employed.

Report the strength of cored specimens to the nearest 10 kPa, together with the following data:

- Position at which the core was taken.
- Stabiliser content (control test value if available).
- Age when tested (from date of stabilisation).
- Date tested.
- Field density and maximum density.
- Diameter of core.
- Length of core.

Evaluation of Chemically Treated Layers (DTWC Method)

A method has been developed by the DTWC Roads for the evaluation of mixing uniformity based on a statistical evaluation of the test results for a specified property. The properties, which may be controlled, are the Plasticity, California Bearing Ratio and the Unconfined Compressive Strength.

Provided that sufficient test data is available, the mean and standard deviation are an excellent combination for describing the ‘quality’ of the property. The results obtained are not only affected by mixing variation, but also by the variation of the properties of the natural material (plasticity, grading, strength, etc.).

Outline of Method

In order to deal with the total variation of the material a proof section is constructed for each type or source of material. The Proof Section is constructed under strict supervision with uniform mixing. Sample points are selected on a random, stratified basis. Statistical analysis provides the norm against which construction work is judged. This norm is expressed in terms of the primary variation and is dependant on the type of material, mixing process and chemical treatment.

The following procedures are adopted to control the layer:

- Preliminary tests are carried out on the untreated material in order to decide on an appropriate treatment.
- Laboratory designs are carried out using different quantities and types of stabilizer.
- Construction of a proof section in order to finalise the mixing technique to be adopted and to determine the efficiency of mixing.

The success of chemical treatment is dependant on the quality of the cementations agent and the distributor thereof. Ensuring that the correct quantity is spread over the target area can easily control the content of cementitious agent. The mixing process determines the distribution of the stabilizer. The stabilizer must be evenly distributed throughout the material, i.e. over the width, length and depth of the layer.

The total variation (VT) can be calculated as follows:

$$VT = VL + VW + VD + PV$$

Where

VL is the variation over the length as result of mixing
VW is the variation over the width as result of mixing
VD is the variation over the depth as result of mixing
PV is the primary variation

When the value of PV is large some or all of the following points can be inferred:

- The variation of the properties of the untreated material is large.
- The reaction of the stabilizer with the material is not very good.
- The mixing of stabilizer is poor.

10. AGGREGATE STRENGTH

<u>The Determination of the Treton Impact Value of Aggregate</u> SANS 3001-AG9

The Treton value is a measure of the resistance of aggregate to impact. The aggregate is subjected to the blows of a falling hammer and the resulting <u>disintegration</u> is measured in terms of the quantity passing the 2,0 mm sieve, which is then expressed as a percentage of the test sample. This is called the Trenton value.
A Treton apparatus consisting of a base plate, anvil, cylinder and a hammer weighting 15 kg ± 50 g (Figure B7/1). The base plate should be placed on a firm concrete block.

SAMPLING AND TESTING

Test sieves, complying with SABS 197 (200 mm in diameter) : 19,0 mm, 16,0 mm and 2,0 mm. the bigger sieves must be made of perforated plate and the 2,0 mm sieve of wire mesh.

Method

The test is performed on the -20,0 + 16,0 mm fraction. 15 to 20 of the most cubical pieces, weighting as closely as possible 50 times the relative density of the aggregate in grams are selected (it is not necessary to determine the relative density – an estimate will be satisfactory). The aggregate pieces are as evenly spaced as possible on the anvil in such a manner that their tops are approximately in the same horizontal plane.

The cylinder is placed over the anvil. The hammer is placed in the cylinder so that the top of the hammer is level with the top of the cylinder and let drop ten times from this position.

All the aggregate is sieved through a 2,0 mm sieve.

Calculations

The Treton value is calculated to the first decimal place:

$$\text{Treton} = \frac{(A - B)}{A} \times 100$$

Where

A = the mass of the stone particles before tamping (g).
B = the mass of the stone particles retained on the 2,0 mm sieve after tamping (g).

The value is reported to the nearest whole number.

The Treton value, as reported, must be the average of three determinations if an individual result differs from the others by more than five, further test should be carried out.

Determination of the Aggregate Crushing Value and 10% FACT SANS 3001-AG10

The aggregate crushing value of an aggregate is a measure of the hardness of the aggregate determined by crushing a prepared confined aggregate sample (the fraction passing the 14 mm sieve and retained on the 10 mm sieve) under a gradually applied constant compressive load of 400 kN and determining the percentage of the material crushed finer than a specified size. The higher the ACV, the softer the material. The sample is divided into 3 specimens. The ACV is the first of 3 loads applied to the sample. The test applies 2 further estimated loads to the 2nd and 3rd specimens to generate 10% fines passing the 2 mm sieve.

SAMPLING AND TESTING

The following apparatus is required:

- An open-ended steel cylinder of nominal diameter 150 mm with plunger and base plate.
- A metal tamping rod 16 mm in diameter and 450 to 600 mm long. One end must be hemispherical.
- A compression test machine capable of applying a load of 400 kN and which can be operated at a uniform rate of loading so that this load is reached in 10 minutes.
- A cylindrical measure with an internal diameter of 115 mm and 180 mm deep.

Dry Test per load:-

A sufficient quantity of the fraction passing the 14 mm and retained on the 10 mm sieve is sieved out, i.e. enough to fill the cylindrical measure.

The specimen is oven-dried at a temperature of 105° C.

The cylindrical measure is filled to overflowing with the aggregate in three more or less equal layers, each layer being tamped 25 times with the rounded end of the tamping rod. The measure is then levelled off with the tamping rod used as a straightedge and the mass of the aggregate in the measure is determined. The open-ended cylinder is placed on the base plate and the test sample added in thirds, each third being tamped 25 times with the tamping rod. The surface of the aggregate is levelled and the plunger inserted, making sure that the plunger does not jam in the cylinder.

Crushing and Sieving of the Sample

The test specimen is placed between the platens of the testing machine and load is applied at a uniform rate of 40 kN/min ± 5 kN/min. When 400 kN is reached, the load is released, the sample removed from the cylinder, placed in a suitable pan and sieved on a 2 mm sieve. The fraction passing the sieve is weighed.

Two further specimens are crushed at loads estimated to generate 10% fines passing the 2 mm sieve.

Wet Test:-

The procedure described above is followed after the mass of the aggregate has been determined and the aggregate is immersed in water for 24 hours.

After soaking, the aggregate is allowed to drain for 5 minutes and then surface-dry by rolling it in a damp cloth. The test is then carried out as for the dry test except that before sieving the material taken from the cylinder, it should be oven-dried at 105°C for at least 16 hours.

SAMPLING AND TESTING

The dry or wet aggregate crushing value to the nearest 0,1 per cent is then calculated and report to the nearest 0,1 percent.

Aggregate crushing value (wet or dry) percentage (m / m)

$$ACV = \frac{B}{A} \times 100$$

Where:

A = mass of test sample before test (g)
B = mass of fraction passing the 2,36 mm sieve (g).

The 10 percent Fines Aggregate Crushing Value (10% FACT) is also a hardness measurement and is determined by measuring the load required to crush a prepared aggregate sample to give 10 percent material passing a specified sieve after crushing. The 10 percent Fines Aggregate Crushing Value is the force in kN required to crush a sample of – 14 + 10 mm aggregate so that 10 percent of the total test sample will pass a 2 mm sieve.

The higher the force required, and the lower the ratio between dry and wet 10% FACT, the better the quality aggregate.

The determination of the Durability Mill Index of Unstabilized Material for Base

This method provides a measure of the ability of the material to withstand degradation both during construction and under various service conditions. The method also furnishes additional data pertaining to the quality of the material and the possible change in index properties likely to occur in the road and be detrimental to its performance.

The Durability Mill Index (DMI) is taken as the product of the maximum percentage passing the 0,425 mm sieve and the maximum Plasticity Index (PI after treatment of the material under different abrasion conditions.

The Durability Mill consists of a cable driven watertight steel cylinder, capable of rotating at a uniform speed of 60 revolutions per minute. The cylinder must be a watertight, non-corrosive steel drum, closed at one end with internal dimensions of 250 ± 1 mm diameter and 264 ± 1 mm in length.

A steel baffle, with thickness of 5 ± 1 mm, projecting 80 ± 1 mm into the cylinder and 264 mm in length, is welded along one element of the interior surface of the cylinder.

The cylinder is fitted with a removable cover and watertight gasket. The cylinder (more than one can be used) must be mounted in such a way that it may be rotated about a central axis in a horizontal locked position. When not in operation it must be able to be unlocked to tilt in an upright position, otherwise it must be removable from the apparatus.

The total abrasive charge of the six balls must be 2600g ± 50 g.

Basically, a sample of base material complying with the specification is to be obtained (±16 kg).

This sample is divided into 4 portions.

Two of the portions, each weighting 3 500 g are soaked for 1 hour in 2,5L of water in the cylinder. The third sample is not soaked.

Thereafter, the first sample plus 6 steel balls is rotated for 10 minutes at a speed of 60 revolutions per minute (600 revolutions).

The second sample is treated in the same way without the steel balls.

The samples where necessary are dried and the percentage passing the 0,425 mm sieve determined. The PI's are then determined on the soil fines (<0,425 mm).

The Durability Mill Index (DMI) is determined as follows:

DMI = PI max - P425

Where:

Pi max	=	highest plasticity index obtained on any of the four test portions
P425	=	maximum percentage passing the 0,425 mm sieve obtained on any of the four test portions.

The highest value obtained on any of the three test portions B, C or D is taken as the GMI.

11. ASPHALT TESTS

<u>Marshall Briquettes</u> SANS 3001-AS1

<u>Marshall Stability, Flow and Quotient</u> SANS 3001-AS2

<u>Marshall Bulk Density and Void Content</u> SANS 3001-AS10

<u>Maximum Void-less Density and Binder Absorption</u> SANS 30011-AS11

<u>Binder Content and particle size Analysis of Asphalt Mix</u> SANS 3001-AS20

<u>Bitumen Content of a Mix by Ignition</u> SANS 3001-AS21

SAMPLING AND TESTING

Binder Content of Slurry Seals SANS 3001-AS22

Moisture Content of Asphalt SANS 3001-AS23

The Marshall method was commonly used for Asphalt Design and the Marshall tests are still in use for routine Construction Control of Asphalt Mixes.

The same principles as for Maximum Dry Density apply except that a bituminous binder is used instead of water.

Other parameters also apply for asphalt, e.g.:

Marshall Stability i.e. its resistance to permanent deformation (usually at high temperature and long times of loading)

Marshall Flow i.e. its resistance to flow

Maximum Void-less Density is the relative density of the voidless mixture.

$$\text{Voids In Mix (VIMs)} = \frac{(\text{Max. Void-less Density} - \text{Max Bulk Density})}{\text{Max. Void-less Density}} \times 100$$

To enhance adhesion and long term durability, the addition of lime (typically ±1.5% by mass) may be added.

For acceptance control purposes we determine:

- Binder Content
- Grading
- Marshall Properties
- Max Theoretical Density

This data is evaluated statistically. The statistical model also makes provision for reduced payments at predetermined risks to both producer and consumer.

Tests for Asphalt Design

Tests for asphalt design are given in paragraph 7.6 of SABITA Manual 35/TRH8 and are dependent on the level of traffic.

SARF COURSE

PRACTICAL ROAD PAVEMENT ENGINEERING

4. GRAVEL ROADS

GRAVEL ROADS

INTRODUCTION

Unpaved roads form the major part of our South African rural road network and are the lifelines to most small rural communities. With the ever present financial constraints it is most unlikely that this situation will change significantly.

Dr. Paige-Green (Transportek, CSIR) carried out an excellent in depth investigation into unpaved roads the results of which appeared in TRH 20. The principles contained in this document are endorsed and recommended to you. While there are three categories of unpaved roads namely earth tracks, earth roads and gravel roads only the latter is handled in this course.

FUNDAMENTALS

The requirement of a gravel road is to carry traffic in all conditions as safely as possible while providing an acceptable ride. It is therefore essential that the basics of good road design are applied.

Geometrics: While by nature of the road (low volume) expensive measures such as high fills and deep cuts are generally not possible sound geometric design must be applied accompanied by appropriate measures to deal with stormwater. In very dry areas it may be acceptable to allow sheet flow across the road during infrequent heavy rainfall. In the wetter regions the provision of drainage structures will be required.

Subgrade Conditions: Through areas of poor and/or wet subgrades special measures will be needed. However before embarking on very costly measures it is recommended that initially only modest protective measures be taken. The condition of the road can be monitored for the first one or two wet seasons and further improvements made where problems are experience.

GRAVEL LAYER DESIGN

Common problems experienced on gravel roads are dust, potholes, stoniness (rough ride), corrugations (sinkplaat), ravelling (loose gravel), erosion, slipperiness and gravel loss. Paige-Green developed a diagnostic plot to predict performance of gravels based on the shrinkage product (Sp) and the grading coefficient (Gc) – see Table 1 and Figure 3, where

Sp = Linear Shrinkage X percent passing (P) 0,425mm sieve

Gc = (P 26,5mm – P 2,0mm) X 4,75mm ÷ 100

In a number of regions it is almost impossible to locate gravels at an economic haul distance that meet all the requirements. However, with a diligent approach, basic laboratory testing and blending of two sources significant improvements can often be made. Do not use material (which after processing) has material larger than

37,5mm. While a minimum CBR of 15 at 95% MDD is recommended higher CBR material is less likely to break down under traffic.

Unless the gravel road is a stage construction project where upgrading to a surfaced road is next phase, lower pavement layers (SSG) on subgrades with a minimum field CBR of 5% are not required. While a thickness formula is given based on traffic volumes and annual estimated gravel loss the reality is that most roads are constructed with a compacted gravel thickness of between 100mm and 150mm.

CONSTRUCTION

The subgrade should be cleared, shaped, watered and compacted to provide a sound platform where appropriate drainage should be installed.

The gravel should be placed in a uniform thickness layer, watered and compacted to at least 93% of MDD. Typically (depending on traffic volumes) the gravel width should be 8m and have a camber of at least 3%. Oversize material (>37,5mm) should be broken down by gridding, primary crushing (expensive) or hand knapping. If this cannot be achieved find another source!

MAINTENANCE

Maintenance of the gravel surface is the major cost factor in the operation of the gravel roads. The major items are grading, broom/tyre dragging and regravelling. Grading may be used to manage loose gravel or to reshape the road particularly where erosion or corrugation has occurred. Frequency of grading depends on traffic, climate, gravel type and weather. Where reshaping is needed it is recommended that this should only be done during periods when the gravel is moist (i.e. not too dry). In a number of places in Africa it has been found that a drag consisting either of bristle brooms or old tyres towed behind a tractor can improve the ride significantly at a much cheaper rate than by grading. This permits more frequent treatments and delivers a higher standard of ride.

Regravelling is essential and a regular annual check should be made of existing gravel thickness. By tracking thickness verses time a desired regravelling programme can be set up. During regravelling local problem areas and minor improvements should be made.

GRAVEL ROADS

MAINTENANCE OF GRAVEL ROADS
Adrian Bergh

1. INTRODUCTION

Over 70% of Road network in South Africa is unsurfaced and due to the limited funds available, the maintenance of these unsurfaced roads will assume greater importance.

I have decided to concentrate on the Maintenance of Unsurfaced Roads and the strategy that can be used to implement a maintenance programme.

There are certain principles involved in maintenance work which are common in the field of efficiently maintaining public services, whether it be water supply, electricity supply, sewerage systems or roads and streets.

(a) Maintenance must not be done on an ad hoc basis – what could be referred to as crisis maintenance.
(b) Maintenance must be effected on an on-going consistent programme.
(c) Cycle maintenance is positive maintenance and protects capital investment as well as saves the road user significant costs.
(d) The size, extent and implication of the total problem must be clearly understood and defined.
(e) Each category of maintenance work must be correctly and clearly defined.

DETERMINING THE SIZE OF THE PROBLEM

2.1 *Inventory*

Until the size and extent of the problem is known, the planning and organization required cannot be determined.

GRAVEL ROADS

AO Bergh *Gravel road maintenance*

2.1.1 A detailed road map of the area or district is required with the length, width and type of surfacing of each road. The traffic counts and description of the traffic is required.

2.1.2 A strip map of each road is required as well as strip maps indicating the km points, size of pipe and box culverts and bridges. Again, the traffic count and date of the strip map must be recorded.

2.2 A simple method of preparing the strip map and classifying the condition of the road between control and section points are as follows:-

1 **being very good:** shape and riding qualities are very good;

2 **being good:** shape and riding qualities are good, minor corrugations;

3 **being fair:** shape is fair but badly corrugated coarse material;

4 **being poor:** shape poor, erosion, tracking of traffic;

5 **being very poor:** rutted, potholed, erosion channels in the prism and road poorly drained.

This classification can be taken further by adding the following letter to the numerals:

G **=** gravel surface, e.g. 1G 2G 3G etc.

E **=** earth road, i.e. subgrade material has been shaped and is in good condition, e.g. 2E

C **=** clay section, e.g. black turf section, e.g. 5C.

This section obviously gives trouble in the wet season.

GRAVEL ROADS

AO Bergh Gravel road maintenance

Let us just spend some time on the strip map which is basic to the Road Inspector knowing the extent of the problem and developing a practical understanding of the materials and their characteristics in his area:

(a) Know and study the materials in your area of activity as well as the drainage problems.

(i) Subgrades vary within an area – some subgrades give problems in wet weather, other subgrades give sound “all-weather” subgrades.

(ii) Sources of gravel – some gravels are coarse and corrugate and give problems in maintaining a good riding surface. Other gravels tend to be finely graded and could become slippery in wet weather, while there are sources of gravel which may break down with time and become sound gravel surfaces.

It is more economical to overhaul sound gravel than use the closest source.

a. Please note it is not necessary or economical to regravel sound subgrades – especially on low volume roads, but if they are used for building up a road they should be properly compacted.

(b) An understanding and knowledge of your plant is most important to be able to plan the maintenance organization, e.g. a maintenance grader can maintain 12 – 16 km of gravel road per 8 hr. shift, i.e. 4 passes/km.

(c) Management of maintenance staff plays a very important part in the results achieved in the field:

(i) The capabilities of staff must be considered;

(ii) Requirements of staff must be understood and attended to;

(iii) Proper housing of field staff is essential;

(iv) Training of field staff is most cost effective.

3. MAINTENANCE STRATEGY

3.1 To plan a maintenance strategy the concept of routine cycle maintenance must be clearly understood.

Roads which receive maintenance on a regular routine basis slowly improve in shape and riding qualities. They become easier and more economical to maintain. Ad hoc or crisis maintenance is expensive, ineffective long-term, and more severe on the plant (e.g. ± 40% - 80% more fuel is used for slow heavy maintenance grading).

The following table is a good guide as to what frequency of grader maintenance is required for gravel or earth roads.

	Traffic Volume Vehicles/day	**Recommended Blading**	**Extent of Maint. Section**
A	0 – 50 v.p.d.	3 times per year. Beginning and end of wet Season + 1 in dry season.	See Notes *
B	50 – 100 v.p.d.	1 time/month	240 - 300km
C	100 – 150 v.p.d.	2 times/moth	120 - 160km
D	150 - 250 v.p.d.	4 times/month	60 - 80km

Notes:

1. The extent of a maintenance section would normally be combination of A, B, C and D.
2. Depending on the length of A and whether it is a continuous section or not, it may require a special mobile maintenance unit to be set up.
3. From the road map and traffic counts it is possible to plan the maintenance section for a maintenance unit, bearing in mind that a grades (CAT 112 or equivalent) can maintain 60 – 80km of road/week.

GRAVEL ROADS

AO Bergh   Gravel road maintenance

3.2 Cycle Maintenance of Gravel Roads

If the technique of maintaining a road is done correctly and consistently, it is possible to gradually improve the shape of the road and riding qualities. The maintenance costs will also reduce accordingly.

The following pertinent points of technique should be borne in mind to improve quality of gravel maintenance. If one starts the maintenance of a well shaped, good, well compacted gravel road:

(a) At no stage must the surface be cut – i.e. the grader blade must always be set in the spreading position and not the cutting position. See Fig. 2 and Fig. 3.

(b) The correct blading sequence should be followed (refer to Figure 1).

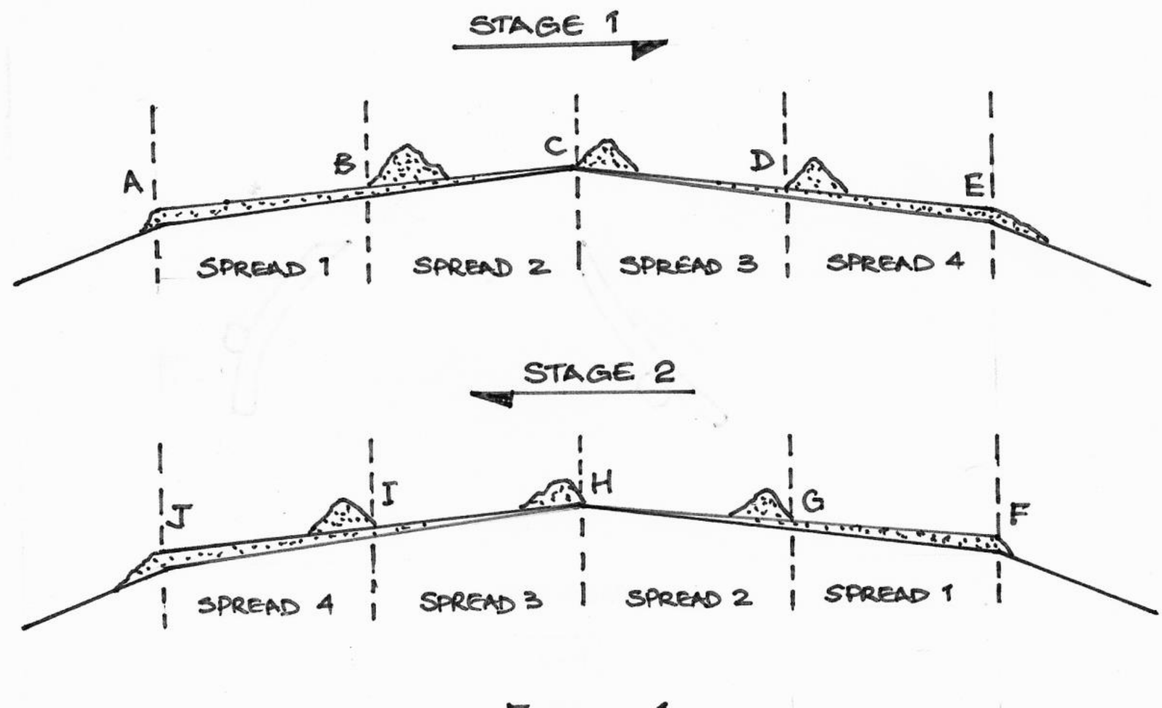

FIGURE 1

A WELL CONSTRUCTED GRAVEL ROAD USING GOOD MATERIAL AND CORRECT CYCLE GRADER MAINTENANCE.

The loose fine material between A & B is swept/spread from left to right and deposited in a small windrow B. The next step is to sweep the windrow between B and C to C, ditto from C to D.

The windrow at D is spread in a "flat windrow" between D and E.

(c) To do the work envisaged in (b) the grader blade must be set in the spreading position – See Fig. 3.

FIGURE 2 FIGURE 3

If it is set in the cutting position then the operator will disturb or rip out any coarse material in the base. If the blade is set in the spreading position will "ride over" or pass over the coarse material without disturbing the surface.

(d) On the second time round, the fine material between E(F) and D(G) will be moved from right to left and deposited at G. This operation will continue until the windrow at I is spread in a "flat windrow" between I and J.

If the grader operator tackles 16km for the day's work, he will finish off at the end of the day where he started, so the next day's work/section will involve some 16km of "dead" km before work starts which means 20 to 30 min before the maintenance commences.

(e) The process described above applies to the **Wet and Dry Seasons**. There is an erroneous concept of "Wet Season" grading and "Dry Season" grading where material is **cut** from the centre to the outer edge of the road in the dry season. This concept of maintenance grading is incorrect.

GENERAL COMMENT

The above system of maintenance applies to a road carrying heavy traffic where the formation width is approximately 10m, i.e. to the shoulder breakpoints.

GRAVEL ROADS

AO Bergh Gravel road maintenance

3.3 Rehabilitation of a gravel road

The rehabilitation of a gravel road basically falls into two categories:

(a) Regravelling, and

(b) Reshaping where at least 30% of the base is still available <u>and</u> where the maintenance grading has been neglected or incorrectly done.

3.4 Regravelling

Before regravelling is done it is advisable to reshape the road to a rolling grade. This will result in a uniform layer of base material to be applied and result in longer life and more economical maintenance. If water bowsers and compaction equipment are available, then compaction of the reshaped subgrade is recommended. If this equipment is not readily available, this work should be done in the rainy season and loaded truck compaction applied.

3.5 Rehabilitation of gravel surface

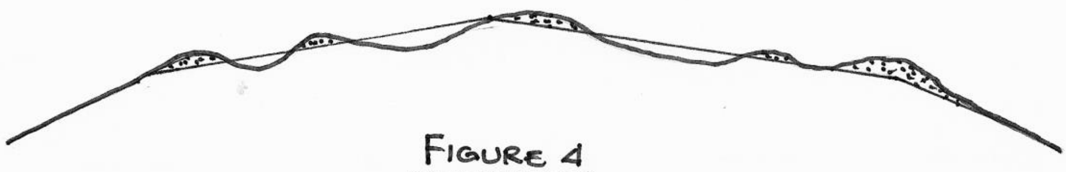

FIGURE 4

THE RESULTS OF SUBSTANDARD GRAVEL AND / OR INADEQUATE CYCLE MAINTENANCE AND / OR GRADER MAINTENANCE DONE ON AN AD-HOC BASIS

The condition depicted in Fig. 4 is essentially caused by poor maintenance grading – the cycle maintenance grading is inadequate or the road has been “grader maintained” on an ad hoc basis.

This condition can be aggravated if the gravel used is below acceptable standard, e.g. coarse and non-plastic and the road is used by truck traffic. The non-plastic gravels are usually suitable for the base course of surfaced roads and should not be wasted on gravel roads.

4. REGRAVELLING PROGRAMMES

The planning of an adequate regravelling programme is often omitted for the following reasons:

(ι) Lack of knowledge of the condition of the gravel road network. It is essential to have available a road plan of the network. It is advisable to set up a pavement management system which can simply describe the condition of the road.

(ιι) Alleged lack of funds for this work – even with relatively limited funds, stage construction/regravelling programmes can be established.

(ιιι) Alleged lack of equipment – it is advisable to have **at least** one road improvement unit in an area/district for emergency work but it is often more practical and economical to let out annual regravelling contracts, even if they are on a small scale.

(ιϖ) Inexperience in budgeting and planning the districts regravelling programme.

(ϖ) Allegedly inadequate gravel. It is essential to carry out quarry surveys well ahead of regravelling contracts.

The following table gives the average gravel loss that can be expected when the roads are subjected to the indicated traffic.

	Estimate of Gravel Loss	
A **Traffic Count**	**B** **Gravel Loss in mm/year**	**C** **Regravelling Cycle**
150 – 250 v.p.d.	20 - 30mm	5 - 8 years
100 – 150 v.p.d.	15 – 20mm	8 – 10 years
50 – 100 v.p.d.	10 – 15mm	10 – 15 years
10 – 50 v.p.d.	7.5 – 10mm	15 – 20 years

The annual loss of gravel (Column B) will depend on the quality of gravel used, the compaction if any that was applied at the time of laying and the type of traffic mix using the road. For budgeting and planning purposes the above figures are eminently suitable. If there are technically trained staff available, the above figures could be refined over a period of time for the different conditions that may be applicable.

The following simplified example is given to render some guidance in preparing the annual regravelling budget. From the road network plan, the kilometres of each gravel road can be determined for the different classes of vehicle count. If it is assumed that a 7,6 m wide, 150mm thick gravel base is to be placed on each road, the following schedule can be prepared for the annual replacement of gravel.

The following is a hypothetical analysis of a road network in a district or area.

A	**B**	**C**	**D**	**E (C ÷ D)**
Traffic Count Class of Road	Total No. of km For each Class	Total Quantity Of Gravel Road Required m^3	Assumed Maintenance Cycle	Annual Gravel Required m^3
150 – 250 vpd 100 – 150 vpd 50 – 100 vpd 10 - 50 vpd	450 1200 1000 600	506 250 1 368 000 1 125 000 675 000	8 years 10 years 15 years 20 years	63 281 136 800 75 000 33 750
		Total Annual Gravel required		308 831

From the above, the total quantity of gravel to be replaced annually is 308 831 cubic meters.

From this total figure it is now possible to decide whether the organization has sufficient capacity to do the work in-house or to let a few small regravelling contracts and so prepare the annual budget.

If the organization has not implemented a systematic cycle maintenance programme previously, a similar exercise can be done and a programme prepared for reshaping and upgrading the formation of the gravel/dirt road network, and by dividing km for each class of road by the assumed maintenance cycle, it will be found that 56km of the heavily trafficked roads will require upgrading/reshaping/year, 120km for the next class etc., and some 30km of the lowest class.

Please note the upgrading/reshaping of a gravel road will require an additional grader unit to assist the existing maintenance unit to expedite the work. This will reduce the interference of the maintenance unit's cycle maintenance to a minimum.

5. ROADS CARRYING HEAVY TRAFFIC

It will be noticed that the roads carrying heavy traffic in excess of 250 v.p.a have not been considered in table No. 1. The reason for this is that in my experience these roads require to be surfaced, and the intensity of maintenance grading that is required is in excess of 4 times per month to keep user costs down to reasonable levels.

5.1 PROBLEMS ENCOUNTERED WITH ROADS CARRYING TRAFFIC >250 V.P.D.

5.1.1 Very little, if any, funds are available to bring geometries and foundation layers up to current Provincial Standard.

5.1.2 Lack of equipment to intensify the maintenance grading e.g. graders and water bowsers.

5.1.3 Incidence of accidents increase substantially as well as the costs involved.

5.1.4. User costs become a substantial feature to the community – and the country, as foreign exchange is involved in replacement of vehicles. This factor has been omitted in the cost/benefit structures.

5.1.5 Wastage of road building material – especially in developing areas where suitable gravels have to be replaced with crusher run. This also applies in areas where intensive tarring occurs.

6. SUGGESTED STRATEGY FOR LOADS WITH TRAFFIC >250V.P.D.

6.1 Stage construction of the subgrade, and drainage. This work is to be done to sound appropriate standards with the view to surfacing the road eventually. (Please note all National roads initially were built to gravel standards before being surfaced).

6.2 Low cost surfacing to be considered even at subbase stage depending on the quality of the 'subbase'. Consider Sand seals using Cationic Emulsions.

6.3 Stabilising the existing material with small percentages of either lime or cement or a combination of both and sealing the surface with a light seal. Note stabilized gravels can be maintained with grader maintenance.

Please note that for light seals to be successful the quality of the finished surface [of the base] must be smooth and the densities must be up to standard.

Before the above work is done D.C.P. tests and physical tests together with selected C.B.R. tests are advisable. The quality and control of the quality together with the quality of the construction will essentially determine the road performance.

A O BERGH

September 1994

SARF COURSE

PRACTICAL ROAD PAVEMENT ENGINEERING

5. UPGRADING

UPGRADING OF GRAVEL ROADS

UPGRADING

There are a number of reasons for upgrading gravel roads. These vary from commercial to aesthetic/environmental to economic. SABITA Manual 10 Section 7 provides a cost benefit type approach via life cycle costing and a break even level of traffic.

Once the decision has been reached that the road should be upgraded the main question to be asked is how can this be cost effectively achieved. In most circumstances it is likely that conventional road design/construction using imported materials will **not** be attractive.

First prize would be to use the existing wearing course as a lower strength base layer, say G5 quality. Where the gravel is not of this quality before resorting to imported materials ways to treat (and improve) the gravel should be examined.

TREATMENT METHODS FOR GRAVEL

Over the years there has been a plethora of treatments all claiming long pavement life for little cost and with traffic riding on the treated surfacing. Kwazulu-Natal is a very good illustration of this – after extensive investigation and use of various products – the Provincial authorities (Howard Bennett) have set a policy of “waterproofing” with a light seal after treating the gravel with a sulphonated oil (Roadbond).

From this reasoning the following set of questions should be asked:

1. What is the quality of the existing gravel/soil?
2. Will it make a satisfactory “base” for the anticipated traffic?
    - Yes: Then no further action is needed other than to select an appropriate seal
    - No: Can the existing gravel/soil be improved to provide an adequate quality base or is suitable gravel available for importation?

UPGRADING OF GRAVEL ROADS

3. What are the cost implications of importing suitable quality gravel versus strengthening/treating existing in situ material?

Assuming that strengthening of in situ material is required what treatments are available?

- Chemical modification
  - Sulphonated oils - some strength, increases compaction and decreases moisture sensitivity
  - Cement/lime, RBi - significant strength gain and decreases moisture sensitivity
  - Emulsion/bitumen - strength gain and decreases moisture sensitivity

- Mechanical
  - Blending with imported Material - some strength

- Surficial Treatments
  - CaCl
  - Dust Palliatives
  - Thin Seals

Note: These surficial treatments do not improve the load carrying capacity of the base but protect its surface against water ingress, erosion and ravelling.

SELECTION OF TREATMENTS

Ease of application, cost and anticipated life are the main considerations. Clearly certain treatments work better with particular gravel and soil types. There is no blanket treatment for all soil and gravel conditions.

- Sulphonated Oils: Relatively cheap – improve compaction, can decrease moisture sensitivity and provide some modest strength gain. Materials with plastic fines are most likely candidates.

UPGRADING OF GRAVEL ROADS

- Cement: Works best with low plasticity (PI > 10) materials but requires some fines (<0,075mm) say 5% min – can provide large increase in strength but can also be subject to block cracking (shrinkage).
- Lime: Works best with moderate plasticity (PI 6 to 20) soils. Can provide a fair increase in strength but can also be subject to block cracking.
- Combinations of Lime, Cement, Slag and PFA: as above.
- RBi: Suited to a fairly wide range of soils/gravels. Finer soils with higher silt and clay contents require more RBi. Provides increased strength (little or no cracking) and reduces moisture sensitivity.
- Emulsion and Foamed Bitumen: With low plasticity granular materials – improves strength moderately to well and reduces moisture sensitivity. Has very good field performance record over the last 20 years.

SURFACING

Where a riding surface improvement is required for a time period of more than two to three years experience indicates that **none** of the gravel treatment methods will on their own provide a solid relatively undamaged surface without further actions such as maintenance or repeated treatments. The conclusion is thus if you are going to spend money on treating the gravel/soil (presumably to strengthen it) you should protect the surface achieved with some form of surfacing.

This can be done in two ways:

- Either provide a relatively expensive **and** durable seal that will last for several years
- Or go for a light treatment on a stage construction basis that will require further action in a three to five year period. In this regard a dust palliative type treatment (self priming) with small aggregate is a good option, but there must be a commitment/understanding that after about 3 years further surfacing will be needed.

UPGRADING OF GRAVEL ROADS

SULPHONATED OIL AGENTS FOR GRAVEL ROADS

Over a number of years various agents have been on the market including Roadbond, ISS, RBi, Roadtreat and abe terrafix. You are cautioned that **no** agent works for all situations/materials and a careful assessment needs to be made both of compatibility and cost. In addition salesmen have been known to tout such agents as a simple treatment for you every gravel road need! These products were used under varying conditions and with a range of in situ materials. The results varied from fair to poor.

Normal road materials tests such as MDD, CBR and plasticity do not give a good indication of whether sulphonated oils are effective. Work by the CSIR Built Environment indicates that pre and post treatment testing of the cation exchange values of an in situ material provides an indication of the response to treatment.

Today, almost world-wide, there is a general shortfall in resources required to provide appropriate road networks. This is mostly the result of cuts in government funding of roads as other facilities gain higher priorities e.g. housing, schools and hospitals. At the same time, the cost of obtaining good road construction materials is increasing and these materials are becoming scarcer as resources are being depleted, necessitating long haul distances. This situation is forcing a re-evaluation of conventional road designs, material standards and construction methods.

Faced with these problems, engineers are having to specify the use of sub-standard materials on many roads. This, however, leads to additional problems, particularly on earth or gravel roads, such as:

* Safety, health and environmental problems related to dust or loose surface material;
* Maintenance problems related to the surface durability under wet and dry conditions;
* Level of service problems related to general surface deterioration such as rutting and pot holing caused by poor materials, high traffic volumes and heavy loads.

Frequent maintenance by experienced and good operators can limit the latter problem to a significant extent but is costly and disruptive to traffic flow with serious road-safety implications.

In an endeavour to overcome some of the problems, road engineers have, internationally, over the past few years embarked on a series of trial evaluations using various chemical additives in gravel wearing courses and pavement layers. These evaluations have fallen into the following groups;

- The use of additives to modify existing gravel wearing courses and in situ and local materials to improve the durability and load carrying capacity of the materials.
- The use of additives to stabilize existing gravel wearing courses which are then treated with conventional thin bituminous surfacings to improve sub-standard pavement layers with a resultant cost benefit.