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**GUIDELINES
FOR ROAD CONSTRUCTION
MATERIALS**

TECHNICAL RECOMMENDATIONS
FOR HIGHWAYS

TRH14

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FOR ROAD
CONSTRUCTION
MATERIALS**

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SYNOPSIS

Recommended standards for road construction materials are given which cover all materials in common use in South Africa, divided into the following categories:

- surfacings and overlays
- material for pavement structures
- Portland cement concrete
- gravel wearing courses

In an appendix the classification (rock type) and properties of road construction materials are discussed.

This document is a companion document to TRH4, "Structural design of inter-urban and rural road pavements".

SINOPSIS

Aanbevole standaarde vir padboumateriale word in hierdie dokument aangegee. Die aanbevelings dek alle materiale wat algemeen in Suid-Afrika gebruik word, en word in die volgende kategorieë verdeel:

- deklae en herdeklae
- materiaal vir plaveiselstrukture
- Portland-sement-beton
- gruisstlytlae

Die klassifikasie (rotstipe) en eienskappe van padboumateriale word in 'n bylae bespreek.

Hierdie dokument hang saam met TRH4 "Strukturele ontwerp van tussenstedelike en plattelandse padplaveisels".

KEYWORDS:

Materials, surfacings, overlays, sand, rock, aggregates, fillers, asphalt, concrete.

PREFACE

TECHNICAL RECOMMENDATIONS FOR HIGHWAYS (TRH) are written for the practising engineer and describe current, recommended practice in selected aspects of highway engineering. They are based on South African experience and the results of research and have the full support of the Committee of State Road Authorities (CSRA).

This document was produced by a subcommittee of the Highway Materials Committee and, to confirm its validity in practice, was circulated in draft form for a trial period before being submitted to the CSRA for final approval, which it has now obtained.

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DEFINITIONS

PAVEMENT STRUCTURE (see Figure 1)

Pavement layers	The combination of material layers constructed over the subgrade in order to provide an acceptable facility on which to operate vehicles.
Fill	Subgrade material placed over the roadbed.
Subgrade	The completed earthworks within the road prism before the construction of the pavement. This comprises the in-situ material of the roadbed and any fill material.
Selected layer(s)	The lowest layer(s) of the pavement consisting of controlled material, either in situ or imported.
Subbase	The layer(s) occurring beneath the base or concrete slab and above the selected layer(s).
Base	The layer(s) occurring immediately beneath the surfacing and above the subbase or, if there is no subbase, above the selected layer(s).
Surfacing	The uppermost pavement layer, which provides the riding surface for vehicles.

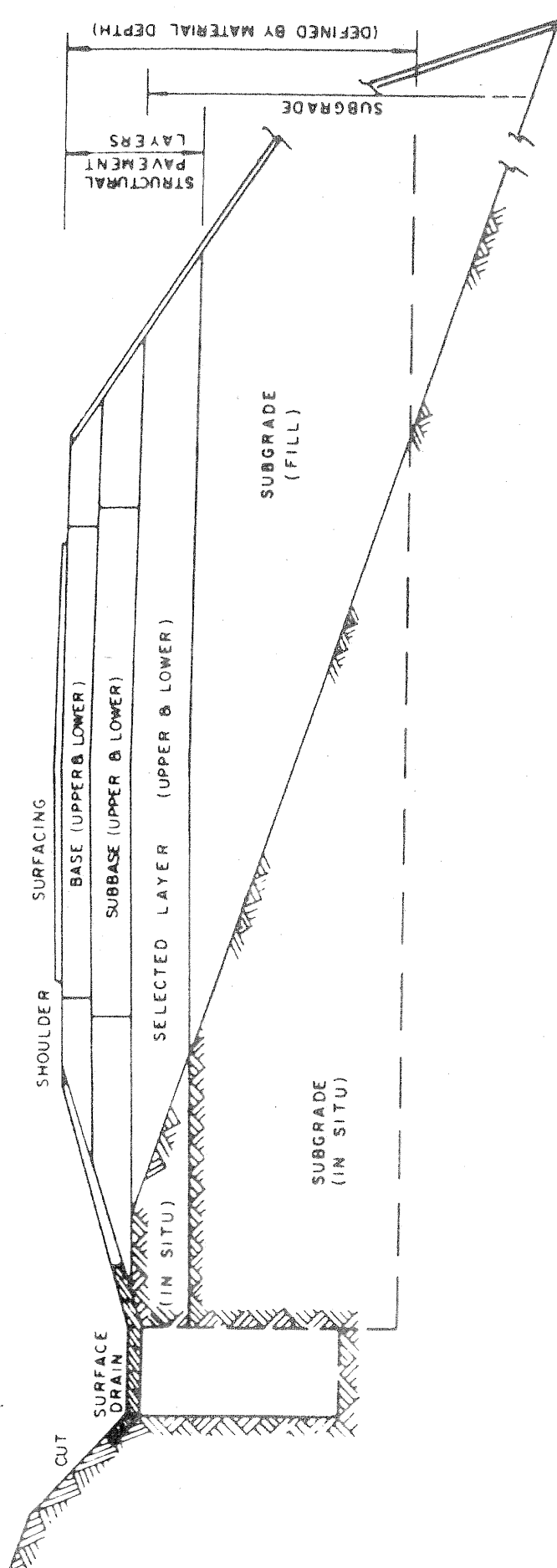


FIGURE 1
PAVEMENT STRUCTURE TERMINOLOGY

1 INTRODUCTION

The structural behaviour of a road pavement during its expected structural life is dependent on the interaction between the strength of the various pavement layers and the repeated traffic stresses imposed on the pavement structure. Moreover, in a given environment the strength of the structural layers of the pavement is dependent on the properties of the materials used in their construction. To ensure that certain minimum strength criteria are satisfied it is therefore necessary to define these physical properties.

The structural design of road pavements is based on a knowledge of both the traffic which will use the pavement during the structural design period and the behaviour of the materials contained in the layered pavement structure. Procedures for estimating the cumulative traffic over the structural design period, the selection of the thickness and material type for the various structural layers in order to obtain a balanced design, planned rehabilitation and economic analysis of alternative designs are covered in TRH4 : "Structural design of interurban and rural road pavements"¹.

This document, which is an essential complement to TRH4, gives recommended standards for materials which may be considered during the structural design of pavements. Materials used in the structural layers of the pavement should be selected according to criteria of availability, economic factors and previous experience. These criteria must be evaluated during the design stage so that materials can be selected which are the most economical and best suited to the prevailing conditions.

All material properties have a certain inherent variability. It is therefore logical that in specifying material properties this variability should be taken into account. This has been the case in practice where products not complying strictly with specifications were often accepted. For most roadbuilding materials a certain percentage of the material population has poorer quality levels than those specified. This percentage is normally ten per cent or less. It is therefore recommended that this be taken into account in specifying the properties of all materials. This implies that the mean quality level for the required property for an acceptable material will be such that not more than ten per cent of the population will have a poorer quality level than the specification limit (refer to TRH5 "Statistical concepts of quality assurance and their application in road construction"² for assessment of results).

In the catalogue of designs presented in TRH4¹, pavement materials are indicated by means of a code, e.g. AG, G1, G5, G10, etc., which refer to materials with certain defined properties. In these recommendations, for the sake of consistency and easy reference, the same material codes and descriptions are used. Table 1 gives a schedule of the material descriptions used in TRH4¹, together with the numbers of the sections of this document where these materials are discussed.

As far as possible all the material types currently used in South Africa have been included, e.g. natural soils and gravels, processed gravels and rock, bituminous and cementitious-treated materials and Portland cement concrete; manufac-

tured materials such as lime, cement, bitumen, tar or milled slag are not included. (Manufactured materials are generally specified in the relevant SABS specification.)

TABLE 1

Schedule of material descriptions used in the Catalogue of designs in TRH4¹ and recommended material standards given in this document (including some materials not given in the Catalogue)

Code	Material		Section
AG	Asphalt surfacing - gap-graded	Surfacings and overlays	2.2
AC	Asphalt surfacing - continuously graded		2.2
AS	Asphalt surfacing - semi-gap-graded		2.2
AO	Asphalt surfacing - open-graded		2.2
S1	Surface treatment - single seal		2.1
S2	Surface treatment - multiple seal		2.1
S3	Sand seal		2.1
S4	Cape seal		2.1
S5	Slurry - fine grading		2.1.3
S6	Slurry - coarse grading		2.1.3
G1	Graded crushed stone	Layered material (untreated)	3.1
G2	Graded crushed stone		3.1
G3	Graded crushed stone		3.1
G4	Natural gravel		3.1
G5	Natural gravel		3.1
G6	Natural gravel		3.1
G7	Gravel-soil		3.1
G8	Gravel-soil		3.1
G9	Gravel-soil		3.1
G10	Gravel-soil		3.1
WM	Waterbound macadam		3.1
DR	Dumprock		3.1
C1	Cemented crushed stone or gravel	Layered material (treated)	3.2
C2	Cemented crushed stone or gravel		3.2
C3	Cemented natural gravel		3.2
C4	Cemented natural gravel		3.2
BC	Bitumen hot-mix, continuously graded		3.2
BS	Bitumen hot-mix, semi-gap-graded		3.2
TC	Tar hot-mix, continuously graded		3.2
TS	Tar hot-mix, semi-gap-graded		3.2
BT1	Bituminous treated crushed stone		3.2
BT2	Bituminous treated natural gravel		3.2
BT3	Bituminous treated cohesionless sand		3.2
PM	Penetration macadam		3.3
PCC	Portland cement concrete	Concrete paving layer	4
GWC	Gravel wearing course	Gravel wearing course	5

Test methods referred to in the text are in accordance with the "Standard Methods of Testing Road Construction Materials", TMH13, unless otherwise indicated.

Attention is drawn to the fact that, for the sake of convenience, material grading is expressed in mass units. Generally this is acceptable except in instances where material fractions of differing apparent relative density are blended to make up the required grading. In such cases it should be ensured that the volume relationship of the blended fractions is maintained, assuming an apparent relative density of say 2,65 for all the fractions making up the grading.

The sampling of materials should be done in accordance with TMH5, "Sampling Methods for Road Construction Materials"⁴.

As a guide to designers the following materials (coded as in Table 1) are shown in the layers of a pavement where they are most often used. The highest quality material which is economically available should always be used.

Layer	Material codes (See Table 1)
Surfacing	AG, AC, AS, AO (in combination with AG, AC or AS) S1, S2, S3, S4, PCC
Base	G1, G2, G3, G4, WM1, WM2 C1, C2, C3, C4 BC, BS, TC, TS, BT1, BT2, BT3, PM
Subbase	G5, G6 C1, C2, C3, C4 BT3
Selected layer	G6, G7, G8, G9
Subgrade	G8, G9, G10

2 SURFACINGS AND OVERLAYS

This section deals with the properties of the following surfacing types: **Single and multiple seals** and **asphalt surfacings**.

2.1 SINGLE AND MULTIPLE SEALS

(a) **Surface treatment (single or multiple) (S1) and (S2)**

(b) **Sand seal (S3)**

(c) **Cape seal (S4)**

This is a surface treatment consisting of a 13,2 or 19,0 mm layer of stone placed shoulder-to-shoulder on a tack coat and covered with one or two applications of slurry so that the tops of the stones are just exposed above the slurry which fills the interstices between the stones.

(d) **Rolled-on chippings** (rolled onto fine-textured asphalt, e.g. gap-graded asphalt, AG)

For details of the design and construction of surface treatments refer to TRH35.

The aggregate for surface treatments (S1 and S2), sand seals (S3), Cape seals (S4) and rolled-on chippings consists of single-sized stone, natural sand and/or crusher sand. The quality required for these materials is of necessity high since the aggregate is exposed to severe handling, environmental and in-service conditions, especially during rolling before it is finally embedded in the bituminous binder.

2.1.1 Properties of single-sized stone

(a) **Grading**

The recommended grading envelopes for the various nominal sizes of stone for surface treatment (S1 and S2), Cape seal (S4) (first layer of stone only) and rolled-on chippings are given in Table 2. The following nominal sizes of stone are generally used for different applications:

Surface treatment (ST) : all indicated sizes

Cape seal (S4) : 19 mm or 13,2 mm

Rolled-on chippings : 19 mm, 16 mm and 13,2 mm.

The nominal gradings are as given above. The final decision, however, should be based on service records wherever possible. These may often show that the fines should be kept as low as possible; this is even more important when an as yet unknown material has to be used.

TABLE 2
Gradings of single-sized stone

Sieve size (mm)	Nominal size (mm) (Percentage passing by mass)				
	19,0	16,0	13,2	9,5	6,7
26,5	100				
19,0	85-100	100	100		
16,0	—	85-100	—		
13,2	0-30	0-30	85-100	100	
9,5	0-5	0-5	0-30	85-100	100
6,7	—	—	0-5	0-30	85-100
4,75	—	—	—	0-5	0-30
3,35	—	—	—	—	0-5
2,36	—	—	—	—	—

(b) Crushing strength

Either the ten per cent Fines Aggregate Crushing Test (10 per cent FACT) or the Aggregate Crushing Value (ACV) can be used on the -13,2 mm +9,5 mm fraction of stone (Table 3). For the larger or smaller nominal sizes of stone difficulty may be experienced in obtaining this size fraction from the surfacing stone. In these cases the supplier should be asked to submit a -13,2 mm +9,5 mm fraction to the crushing strength test. If this is not possible the appropriate test method given in BS 812⁶ should be followed.

TABLE 3
Crushing strength of single-sized stone

Application	10% FACT* (min)	ACV (max)
Surface treatment (ST) Cape seal (SC)	210 kN	21%
Rolled-on chippings		

The above values may not be applicable to certain rock types (see Section 2 of Appendix A).

(c) Flakiness Index

Flakiness is a crucial factor when stone with a high crushing strength is used (see Sections 2 and 4 of Appendix A).

* In certain cases it may be desirable to do the 10 per cent FACT on wet material as well (24 hours' soaking followed by draining). In this case the wet value should be at least 75 per cent of the dry value.

TABLE 4
Permissible flakiness indices

Nominal size (mm)	Flakiness Index (% max)		
	Rolled-on chippings	ST and SC stone	
		Grade N	Grade S
19,0 to 13,2	20	25	30
9,5 and 6,7	—	30	35

Note: Grade N stone is preferred, but Grade S stone may be specified in cases where the lower flakiness index cannot be met.

(d) Polished stone value

The polished stone value (PSV) of the stone is determined according to SABS Method 848⁷.

Even if stone is coated with a bituminous binder, at least part of it will eventually be exposed to the surface of the road, and is likely to polish under the action of traffic. Polishing of stone in surfacing lowers the skid resistance of the surfacing under wet conditions and is of particular importance when the surface texture depth of the surfacing is shallow.

Site condition	PSV (min)
Difficult site	55
Average site	50

(Difficult sites are approaches to intersections, roundabouts, steep grades and tight curves with insufficient superelevation, etc.)

Some rocks that are likely to be used for rolled-on chippings, such as all carbonate rocks, most basic crystalline and certain very dense acid crystalline rocks, e.g. felsite, are likely to give PSV values lower than those stated above (see Sections 1 and 3 of Appendix A).

The suggested PSV requirements for rolled-on chips are important. The PSV should be considered for stone used in single and multiple seals, Cape seals and open-graded asphalt only where heavy traffic conditions are experienced.

(e) Fines

Although there should be no material smaller than a specified size according to the grading, the presence of fines and dust cannot be prevented entirely but should be limited as specified in Table 5.

TABLE 5
Permissible fines and dust contents

Nominal size (mm)	Fines (% max)			
	Fines content (passing 0,425 mm sieve)		Dust content (passing 0,075 mm sieve)	
	Stone		Stone	
	Grade N	Grade S	Grade N	Grade S
19,0 to 9,5	0,5	2,0	–	1,5
6,7	0,5	3,0	–	1,5

(f) **Stone-bitumen adhesion**

Not all types of rock adhere equally well to bitumen. Although there is experience of the different adhesive properties of "acid" and "basic" rocks, it is safer to be guided by the surface texture of the crushed stone. Bitumen, in particular, does not adhere well to "acid" rocks. However, when tar or cationic bitumen emulsions are used, such problems do not usually occur.

Medium to fine-grained rocks

These rocks produce a rough-textured crushing face which is not likely to cause stripping of the binder. Such rocks are almost all carbonate rocks and calcrete, many acid or basic crystalline rocks, tillites (diamictites) and any arenaceous rocks which possess a strong siliceous cementing matrix.

Coarse-grained rocks

Although they also produce a rough-textured crushing face, they possess numerous flat and smooth faces of large minerals, especially feldspars, and are noticeably more inclined to strip than the medium to fine-grained types. Such stone is produced in particular from certain acid crystalline rocks.

Very dense to glassy rocks

These rocks develop a smooth-textured crushing face from which bitumen strips readily. All high-silica rocks and certain volcanic members of the acid and basic crystalline rock groups belong in this category.

The stone-bitumen adhesion should be tested according to Method B11 of TMH13. This test is known as the Riedel and Weber test. Tentative limits have been suggested and are as follows:

<1	unsatisfactory	3	good
1	borderline	4	very good
2	acceptable	5	excellent

2.1.2 Properties of sand (natural)

(a) Grading

All sands used for sand seals (S3), or as a second layer of a multiple surface treatment (S2), should conform to the following grading:

TABLE 6
Grading of natural sands

Sieve size (mm)	Percentage passing by mass
6,7	100
0,300	0-15
0,150	0-2

(b) Sand equivalent value

To limit the degree of contamination of the sand by clayey material the sand equivalent value should not be less than 35.

2.1.3 Properties of slurry aggregate

The aggregate for the manufacture of slurry is generally crusher sand or a blend of crusher sand and natural sand.

(a) Grading

Slurry aggregate should be graded according to the values given in Table 7. Natural sand should not form more than 25 per cent by mass of the aggregate blend unless a cationic bitumen emulsion is used or an adhesion agent is added.

TABLE 7
Grading of slurry aggregate

Sieve size (mm)	Percentage passing by mass	
	Fine (S5)	Coarse (S6)
6,7	100	100
4,75	100	82-100
2,36	90-100	56-95
1,18	65-95	37-75
0,600	42-72	25-50
0,300	23-48	15-37
0,150	10-27	7-20
0,075	4-12	2-8

(b) **Sand equivalent value**

The sand equivalent value of the slurry aggregate should not be less than 35.

2.2 ASPHALT SURFACINGS

This section deals with the properties of the following surfacings:

- (a) **Asphalt surfacing, gap-graded (AG)**
- (b) **Asphalt surfacing, continuously-graded (AC)**
- (c) **Asphalt surfacing, semi-gap-graded (AS)**
- (d) **Asphalt surfacing, open-graded (AO)**

For details of the composition, selection and design of asphalt surfacings refer to TRH8, "Selection and design of hot-mix asphalt surfacings for highways" ⁸.

2.2.1 Properties of aggregate

The aggregate used for asphalt surfacings may be obtained from several sources. The coarse aggregate, i.e. that retained on a 4,75 mm sieve, is in most cases crushed rock, although certain types of blast-furnace slag may be crushed and used provided that they satisfy the strength requirements and are not too water absorbent.

The fine aggregate, i.e. that passing a 4,75 mm sieve and retained on a 0,075 mm sieve, may be crusher sand, clean natural sand, mine sand, selected river gravel or a blend of these. The filler, i.e. material which substantially passes a 0,075 mm sieve, when required, may be flue (cyclone) dust, rock-flour, fly-ash (also known as pulverized fuel ash (PFA)), Portland cement, milled granulated blast-furnace slag, hydrated lime or a combination of these. Hydrated lime in excess of two per cent by mass of dry aggregate should not be added, as large quantities of hydrated lime tend to produce a dry, brittle asphalt mixture. Small quantities of hydrated lime (1 to 1,5 per cent) can improve the anti-stripping properties of the asphalt mixture and are often added for this reason.

(a) **Grading**

The gradings given in Table 8 are recommended in TRH8⁸ for asphalt surfacing for the various mix types shown.

(b) **Crushing strength (coarse aggregate)**

The following strength requirements are recommended:

10 per cent FACT of the dry material (min) 160kN

or

ACV (max) 25 per cent

In the case of open-graded asphalt (AO) it is recommended that the 10 per cent FACT of the dry aggregate should be not less than 210 kN, or that the ACV (max) should be 21 per cent.

TABLE 8
Grading of aggregate used in asphalt surfacings

Sieve size (mm)	Percentage by mass passing sieve								
	Gap-graded (AG)			Continuously graded (AC)			Semi- gap- graded (AS)	Open-graded (AO)	
	Stone content			Coarse	Medium	Fine		Coarse	Fine
	Low	Interm.	High						
19,0	100	100	100	100	100	100	100	100	100
13,2	75-100	75-100	75-100	84-100	100	100	80-100	100	100
9,5	70-90	70-90	64-85	70-92	82-100	100	65-80	75-90	100
4,75	65-75	60-70	50-60	50-70	54-75	64-88	45-60	25-50	30-50
2,36	60-70	53-63	45-55	37-55	40-57	48-70	42-55	5-15	5-15
1,18	60-70	53-63	45-55	26-41	27-42	35-54	40-52	—	—
0,600	55-70	45-63	36-52	18-32	18-32	24-40	35-48	—	—
0,300	45-65	35-55	25-45	12-23	12-23	16-28	25-45	3-8	—
0,150	20-40	15-35	12-32	7-16	7-16	10-20	15-25	—	—
0,075	5-12	5-12	5-12	4-10	4-10	4-12	5-12	2-5	2-5

Note: The maximum size of aggregate should not exceed one half of the compacted thickness of the asphalt.

In certain cases where there is doubt about the durability of the aggregate it may be desirable, in addition, to do the 10 per cent FACT on wet material (24 hours' soaking followed by draining). In this case the wet value should be at least 75 per cent of the dry value.

The above values apply to most groups of natural material. However, the 10 per cent FACT value should be a little higher for tillite (170 kN) and calcrete (180 kN). Argillaceous rocks and ferricrete are unsuitable. Pedogenic materials, calcrete and silcrete, even if they pass all the standard tests, have been found not to perform satisfactorily in surfacing mixtures. If it is necessary to use these materials, a detailed investigation to ensure the satisfactory performance of the surfacing is required.

(c) **Polished stone value**

The polished stone value (PSV) of the aggregate must be determined according to SABS Method 848⁷.

Where the traffic is heavy and where skidding may be a traffic hazard, it is recommended that the coarse aggregate of the asphalt, in particular, should meet the following requirements:

Site condition	PSV (min)
Difficult site	55
Average site	50

(Difficult sites are approaches to intersections, roundabouts, steep grades and tight curves with insufficient superelevation.)

Chips when rolled onto an asphalt layer must meet the above requirements (see Section 3 of Appendix A).

(d) **Sand equivalent**

The sand equivalent of the total fine aggregate should not be less than 35, and sand blended with the mixture should have a sand equivalent of not less than 30.

(e) **Water absorption**

The water absorption of both the coarse and fine aggregates should be limited to a value of not more than 1,5 per cent by mass for fine aggregate and not more than 1 per cent for coarse aggregate. Excessive absorption has an adverse effect on the durability of an asphalt mixture.

If it is necessary to use aggregate of excessively high water absorption a detailed investigation to ensure the satisfactory performance of the asphalt is required.

2.2.2 Properties of filler

Where there is no experience of the satisfactory performance of a filler material, it is recommended that at least 70 per cent by mass of the filler should pass a 0,075 mm sieve and should have a bulk density in toluene of 0,5 to 0,9 g/ml (test according to BS 812⁶), or voids in compacted filler of 0,3 to 0,5 (test according to BS 812⁶).

2.2.3 Properties of asphalt mixture

(a) **Design criteria (refer to TRH8⁸)**

The asphalt mixture should meet the criteria given in Table 9 at 100 per cent Marshall density in order to give satisfactory performance with respect to stability, fatigue resistance and durability. All specimens must be compacted with 75 blows on each end with a standard Marshall hammer using standard equipment.

(b) **Field compaction**

The mixed materials should be compacted to a Marshall density of at least 95 to 98 per cent (2 x 75 blows) depending on the design requirements. Where rolled-on chippings are used with asphalt it is suggested that, because of the difficulty of determining the density of the mixture (excluding the chippings), a trial section should be laid where the method of compaction is tested. Once this has been agreed upon it should be used throughout the construction.

(c) **Resistance to water**

To determine the resistance of the mixture to water, the Immersion Index of the mix should be determined in accordance with Method C5 given in TMH1³. The mixture should meet the immersion index requirement given in Table 9. The addition of small quantities of hydrated lime (<1,5 per cent) often improves the resistance of the mixture to water; otherwise an adhesion agent may also be effective.

TABLE 9

Design criteria for asphalt surfacing mixtures (all traffic categories)

Type of mixture	Continuously graded (AC)		Gap-graded or semi-gap-graded (AG) and (AS)*	
	min	max	min	max
Stability (kN)	4,0	12,5	3,0	12,5
Flow (mm)	2,0	4,0	2,0	6,0
Stability/Flow (kN/mm)	—	—	1,5	—
Air voids (%)	3,0	5,0	2,0	12,0
Immersion index (%)	75	—	75	—
Air permeability (cm ²)	Not specified	—	1,10 ⁸	—
Film thickness (µm)	Not specified	5,0	—	—
Filler/bitumen ratio	Not specified	1,0	1,5	—
Voids in mineral aggregate (5) (min)	Max	VMA (%)	Not specified	
	size			
	agg.			
	19,0 mm	14,0		
	13,2 mm	15,0		
	9,5 mm	16,0		

- * The criteria relating to stability, flow and the stability/flow ratio may not be suitable for very heavy traffic over hilly terrain. Under these circumstances the stability/flow ratio should not be less than 2,0 kN/mm.

3 MATERIAL FOR PAVEMENT STRUCTURE

This section deals with the properties of untreated and treated crushed stone, natural gravels and soils.

3.1 UNTREATED MATERIAL

- Graded crushed stone (G1)
- Graded crushed stone (G2)
- Graded crushed stone (G3)
- Natural gravels (including modified and processed gravel) (G4, G5 and G6)
- Gravel-soil (G7, G8, G9 and G10)
- Waterbound macadam (WM)
- Dumprock (DR)

(a) Graded crushed stone (G1)

This material meets the quality and grading requirements mentioned below and is generally derived from the crushing of solid unweathered quarried rock, clean rock from mine rock dumps or clean boulders. It is placed at near saturation moisture content; the field density is usually specified as a percentage of the apparent density (AD) of the all-in material and expressed as relative compaction (RC) for G1 material. For G2 and G3 materials the field density is expressed as a percentage of Mod. AASHTO density. Slushing is usually carried out to obtain the required density for a G1 material.

(b) Graded crushed stone (G2 and G3)

This material is obtained by crushing rock, boulders or coarse gravel. Provided it conforms to the specified requirements, the crushed material may include natural fines not derived from crushing the parent rock.

It is placed at optimum moisture content for the compaction equipment and compacted to a percentage of Mod. AASHTO density.

(c) Natural gravel (processed and/or modified) (G4, G5 and G6)

This is natural gravel or a mixture of natural gravel and boulders which may require crushing. It differs from G2 in that modification by the addition of small quantities of lime or cement, or mechanical stabilization by the addition of sand, is required to adjust its Atterberg limits. All material passes the 63 mm sieve. The CBR (soaked) and Atterberg limits of the material place it in the correct material category.

(d) Gravel-soil (G7, G8, G9 and G10)

These are natural materials selected to meet the properties mentioned below.

(e) Waterbound macadam (WM)

This material consists of crushed rock with crusher fines or soil filler and meets the quality and grading requirements mentioned below

(f) **Dumprock (DR)**

This is ungraded waste rock from mining activities, blasting or rock excavation.

3.1.1 Properties of materials

(a) **Grading**

In the case of graded crushed stone (G1), a faulty grading may be adjusted only by the addition of crusher sand or other stone fractions obtained from the crushing of the **parent rock**. Only in exceptional cases where the grading is such that the required density cannot be obtained, and fines from the parent rock are not available, may natural fines be added to the material, provided that such fines consist of non-plastic sand and the quantity added does not exceed 10 per cent by mass of material*. G2 and G3 materials may be a blend of crushed stone and other fine aggregate to adjust the grading. The fine aggregate if obtained from a source other than the crushing of the parent rock shall be subject to approval by the purchaser, provided that the supplier furnishes the purchaser with full particulars regarding the exact amount and nature of such fine aggregate.

Materials G1, G2, G3, G4 should comply with the following grading envelopes, and in the case of G2 and G4 at least 50 per cent of the material retained on the 4,75 mm sieve should have at least one fractured face.

TABLE 10

Grading of graded crushed stone (G1), graded crushed stone + soil (G2), (G3), and natural gravel (G4)

Sieve size (mm)	Percentage passing by mass		
	G1, G2 and G3 Nominal maximum size of aggregate (mm)		G4
	37,5	26,5	
53,0	100	100	100
37,5	100	100	85-100
26,5	84-94	100	—
19,0	71-84	85-95	60-90
13,2	59-75	71-84	—
4,75	36-53	42-60	30-65
2,00	23-40	27-45	20-50
0,425	11-24	13-27	10-30
0,075	4-12	5-12	5-15

* It should be ensured that the **volume** relationship of the blended fractions is maintained, assuming an apparent relative density of, say, 2,65 for all the fractions making up the grading.

Natural gravel (G5 and G6) should have a maximum size of 63 mm or two-thirds of the compacted layer thickness, whichever is the smaller. A minimum grading modulus of 1,5 (G5) and 1,2 (G6) should be obtained.

Gravel-soil (G7) should have a maximum size, in place, after compaction, not greater than two-thirds of the compacted thickness of the layer. A minimum grading modulus of 0,75 should be obtained.

Gravel-soil (G8, G9 and G10): no grading requirements.

Waterbound macadam (WM): The grading of the coarse and fine aggregate of waterbound macadam (WM) should comply with the envelopes given in Tables 11 and 12.

TABLE 11

Grading of coarse aggregate (waterbound macadam (WM))

Sieve size (mm)	Percentage passing by mass	
75,0	100	100*
53,0	85-100	85-100
37,5	35-70	or 0-30
26,5	0-15	0-5
19,0	0-5	—

- * This grading conforms to the 53,0 mm nominal size given in SABS 1083⁹ and has been found to give satisfactory performance.

TABLE 12

Grading of fine aggregate or natural filler (waterbound macadam (WM))

Sieve size (mm)	Percentage passing by mass
9,5	100
4,75	85-100
0,075	10-25

Dumprock (DR) should have a maximum size not more than two-thirds of the compacted thickness of the layer and this should not exceed 300 mm per lift.

(b) **Atterberg limits**

The Atterberg limits given in Table 13 apply to the soil fines (<0,425 mm) of graded crushed stone and natural gravels (G4 and G5) after modification, if required, and to the fine aggregate of waterbound macadam (WM). In general, crushed stone (G1) will have non-plastic fines, but in some cases the rock type may produce fines that are slightly plastic. In these cases the limits given in Table 13 should apply.

TABLE 13

Atterberg limits for graded crushed stone (G1, G2 and G3) natural gravel (G4 and G5) and waterbound macadam (fines) (WM)

Property	Material type			
	G1	G2,G3 and G4	G5	(WM) (fine aggregate or natural filler)
Liquid Limit (max)	25	25	30	25
Plasticity Index (max)	4	6	10	6
Linear shrinkage (%) (max)	4	3	5	3
*Linear shrinkage x (%) pass- ing 0,425 mm sieve (max)	—	170 (G4)	170	

* Only applicable to nodular calcrete - for further details see CSIR Research Report 286¹⁰.

In the case of gravel and gravel-soil (G6 and G7) the Plasticity Index (PI) of the material should not exceed 12.

In the case of G6 and G7 material with a large coarse fraction, a higher PI of the soil fines may be acceptable. As a guide the PI may be calculated from the following equation:

$$\text{Maximum PI} = 3 \text{ GM} + 10$$

where the grading modulus, GM, is given by

$$\text{GM} = \frac{P_{2,00 \text{ mm}} + P_{0,425 \text{ mm}} + P_{0,075 \text{ mm}}}{100}$$

where $P_{2,00 \text{ mm}}$, etc., denote the percentage retained on the indicated sieve size.

(c) **Crushing strength**

The following aggregate strength requirements are recommended for graded crushed stone (G1), graded crushed stone (G2), and waterbound macadam (WM).

Property	G1, G2 & WM
10 per cent FACT (min)	110 kN
or	
ACV (max)	29 per cent

Where calcrete is used the minimum 10 per cent FACT should be 80 kN.

In certain cases it may be desirable, in addition, to do the 10 per cent FACT on wet material (24 hours' soaking followed by draining). In this case the wet value should be at least 75 per cent of the dry value.

(d) **Flakiness Index (G1, G2 and WM)**

The weighted average Flakiness Index determined on the -26,5 mm + 19,0 mm and -19,0 mm + 13,2 mm fractions should not exceed 35 per cent.

(e) **Bearing strength and swell**

Materials G2, G3 and G4 should have a CBR after soaking of not less than 80 per cent at 98 per cent Mod. AASHTO density and a maximum swell of 0,2 per cent at 100 per cent Mod. AASHTO density.

Material G5 should have a CBR after soaking of not less than 45 per cent at 95 per cent Mod. AASHTO density and a maximum swell of 0,5 per cent at 100 per cent Mod. AASHTO density.

In the drier areas of the country ($N > 10$; see Figure 2A, Appendix A) and in the case of very lightly trafficked roads ($AADT < 300$ vpd) this requirement may be relaxed if the subbase layer has a cover of 150 mm or greater. In such cases the minimum CBR at 95 per cent Mod. AASHTO density should not be less than 25.

Materials G6, G7, G8, G9 and G10 should have the CBR and swell properties given in Table 14.

TABLE 14

CBR and swell requirements for gravel and gravel-soil (G6, G7, G8, G9 and G10)

Property	Material type				
	G6	G7	G8	G9	G10
Minimum CBR at 93 per cent Mod. AASHTO density (%)	25	15			
Minimum CBR at in-situ density (%)	—	—	10	7	3
Maximum swell at 100 per cent Mod. AASHTO density (%)	1,0	1,5	1,5	1,5	1,5

(f) **Group Index**

The Group Index for gravels should be as given in Table 15.

TABLE 15
Group Indices for natural gravel and gravel-soil (G5, G6 and G7)

Property	Material Type	
	G5	G6 and G7
Maximum Group Index	0	1
Maximum Group Index for calcretes	1	2

(g) **Field compaction**

The normal field compaction requirements for untreated layered materials are given in Table 16.

TABLE 16
Normal field compaction requirements for untreated layered materials

Pavement layer		Relative compaction*	
Base (upper and lower)	Crushed stone	G1	86-88% apparent density
		G2	100-102% Mod. AASHTO
	Crushed stone G3 and natural gravels G4		98% Mod. AASHTO
	Waterbound macadam	WM1	88-90% apparent density
		WM2	86-88% apparent density
Subbase (upper and lower)	Crushed stone	G1	86-88% apparent density
		G2	100-102% Mod. AASHTO
	Crushed stone G3 and natural gravel G4		95% Mod. AASHTO
Selected layers	Natural gravel or gravel-soil		93% Mod. AASHTO
Subgrade	Natural gravel or gravel-soil		90% Mod. AASHTO

* In the case of cohesionless single-sized sand**, 100 per cent AASHTO density. In all cases the compaction moisture content during construction should be the optimum content for the compaction plant employed to ensure that during the compaction process instability or excessive movement of the material is avoided.

** Cohesionless single-sized sand must contain no foreign matter and be closely graded with no cohesion in the dry state.

In the case of dumprock (DR) the density is not normally specified; compaction is continued until the movement of material under the roller becomes negligible.

(h) **Deleterious minerals (G1, G2, G3, G4, G5, G6, G7 and WM (coarse aggregate))**

Sulphide minerals

Crushed rock which is known to contain sulphide minerals (pyrite, marcasite, chalcopyrite, pyrrhotite), should be further investigated for deleterious effects.

Soluble salts

There are a rather large number of salts whose solubility may be deleterious to surfaced roads. Their degree of solubility is not the same, and consequently the level at which they become deleterious changes according to the type of salt.

An easily applicable guide is the electrical conductivity of the fines: the conductivity of saturated paste should not be more than 1,5 mS/cm at 25 °C on the dry-screened -6,7 mm fraction, determined according to Method A21T described in TMH13.

Mica

This material, especially muscovite, if it occurs freely in quantities which can easily be seen, may have a disrupting effect on a compacted base. For more detail see Section 5 of Appendix A.

3.2 TREATED MATERIAL (MIXED)

This section deals with the properties of the following materials:

- Cemented crushed stone or gravel (C1 and C2)
- Cemented natural gravel (C3 and C4)
- Bitumen hot-mix (BC and BS)
- Tar hot-mix (TC and TS)
- Bituminous treated crushed stone (BT1)
- Bituminous treated natural gravel (BT2)
- Bituminous treated cohesionless sand (BT3)

(a) **Cemented crushed stone or gravel (C1)**

This material generally meets at least the requirements for graded crushed stone (G2) before treatment, except for grading (see Table 17).

(b) **Cemented crushed stone or gravel (C2)**

This is a material which generally meets the requirements either for graded crushed stone (G2) or natural gravel (G4) which is untreated except for grading (see Table 17).

(c) **Cemented natural gravel (C3 and C4)**

This is selected natural material equivalent to G5 or G6 material meeting the density and strength requirements given below.

(d) **Bitumen hot-mix (BC and BS)**

This is a high quality aggregate premixed with penetration grade bitumen. It is of either a continuous grading (BC) or semi-gap grading (BS), both of which meet the requirements given below.

(e) **Tar hot-mix (TC and TS)**

This is a high quality aggregate premixed with a hard grade of tar. It is either of a continuous grading (TC) or semi-gap grading (TS), both of which meet the requirements given below.

(f) **Bituminous treated crushed stone (BT1)**

This is a crushed stone which complies at least with the requirements for G2 material, and with which is cold-mixed (in situ) a **very low content** bituminous binder, usually in the form of a bitumen or tar emulsion.

(g) **Bituminous treated natural gravel (BT2)**

This is a selected natural gravel generally equivalent to G4, G5 or G6 material treated with a very low content of cutback tar, bitumen or emulsion (tar or bitumen) meeting the requirements given below.

(h) **Bituminous treated cohesionless sand (BT3)**

This is a cohesionless sand treated with a cutback tar or bitumen or emulsion (tar or bitumen) meeting the requirements given below.

NOTE: From practical experience a wide range of natural materials, from gravel to a sand, can be satisfactorily treated with a Portland cement and lime/slag or cement/slag mix. It is therefore not possible to lay down standards for each material property before treatment. In such cases it is recommended that the properties of the treated material comply with the requirements given to ensure adequate strength and durability. As a guide, the grading modulus of the material before treatment should not be less than 1,75 (base) and 1,5 (subbase). Cement should not be used to stabilize soils that have a Liquid Limit greater than 45. Lime is most suited to clayey materials, with more than 15 per cent passing the 0,425 mm sieve and a Plasticity Index exceeding 10. It is important for sufficient lime to be added to initiate attack on the clay and the formation of cementitious compounds. The amount required is that which will give the required strength properties. In any case the amount should not be less than that indicated by the initial lime consumption of the whole material.

3.2.1 Properties of materials

(a) Grading

The grading of the aggregate of the treated material after compaction, for cemented crushed stone or gravel (C1 and C2) or bitumen/tar crushed stone hot-mix (BC, BS; TC, TS), should be according to the values given in Table 17.

In the case of alluvial or other natural gravel, at least 50 per cent of the material retained on the 4,75 mm sieve should have at least one fractured face.

TABLE 17
Grading of aggregate of treated material

Sieve size (mm)	Percentage passing by mass		
	Nominal maximum size of aggregate (mm)		
	37,5	37,5	26,5
Base type	BS and TS	C1 and C2	BC, TC, C1 and C2
37,5	100	100	100
26,5	90-100	84-94	100
19,0	75-95	71-84	85-95
13,2	65-85	59-75	71-84
4,75	45-60	36-53	42-60
2,00	41-53	23-40	27-45
0,425	30-47	11-24	13-27
0,075	5-12	4-12	5-12

Materials (C3, C4 and BT2)

The maximum size of the material after compaction in place should not exceed two-thirds of the compacted thickness of the layer or 63 mm, whichever is the smaller.

Material (BT3)

The grading of the sand should be such that at least 5 per cent passes the 0,075 mm sieve.

(b) Crushing strength

The following aggregate crushing strength requirements are applicable to the aggregate for cemented crushed stone or gravel (C1 and C2), and for bitumen/tar crushed stone hot-mix (BC, BS; TC, TS):

10 per cent FACT (min)	110 kN
or	
ACV (max)	29 per cent

In certain cases it may also be desirable to do the 10 per cent FACT on wet material (24 hours' soaking followed by draining). In this case the wet value should be at least 75 per cent of the dry value (Section 2, Appendix A1).

(c) **Flakiness Index (C1, C2; BC, BS; TC, TS)**

The weighted average Flakiness Index determined on the -26,5 mm +19,0 mm and -19,0 mm +13,2 mm fractions should not exceed 35 per cent.

(d) **Sand equivalent (C1, C2 and BT1)**

Any sand added to correct the grading of the material should have a sand equivalent of not less than 30 per cent.

(e) **Atterberg limits (C3 and C4)**

After treatment the material should have a Plasticity Index not greater than 6.

(f) **Design strength**

In regard to crushing strength requirements for cemented materials, the laboratory design strength of the cemented material should be in accordance with the values given in Table 18. This strength should be obtained with not more than five per cent by mass of stabilizer at the specified density and at optimum moisture content. In cases where lime stabilizer is used, the above criteria may be satisfied by accelerated cured samples prepared in accordance with the prescribed method.

TABLE 18
Design strength for cemented materials

Property	Cemented material							
	C1		C2		C3*		C4*	
	min	max	min	max	min	max	min	max
Laboratory design, unconfined compressive strength at 7 days (MPa) 100% Mod. AASHTO density	6	12	3	6	1,5	3,0	0,75	1,5
Laboratory design, unconfined compressive strength at 7 days (MPa) 97% Mod. AASHTO density	4	8	2	4	1	2	0,5	1

* These maximum strength requirements are only given as a guide.

Marshall test requirements (hot-mix bituminous materials)

The design criteria at 100 per cent Marshall density (2 x 75 blows) given in Table 19 should apply.

TABLE 19
Design criteria for hot-mix bituminous materials

Property	Continuously graded (BC and TC)		Semi- gap-graded (BS and TS)	
	Min	Max	Min	Max
Marshall stability (kN)	6,0	12,0	5,0	12,0
Marshall flow (mm)	2,0	4,0	2,0	4,0
Voids in mix (%)	2,0	6,0	2,0	8,0
Stability/flow (kN/mm)	2,0	—	2,0	—
Immersion Index (%)	75	—	75	—

To obtain the foregoing values the binder content should be between 3,5 and 5,0 per cent (bitumen) and 4,5 and 6,0 per cent (tar) by mass of mixture.

(g) **Field compaction**

The normal field compaction requirements for treated layered materials are given in Table 20. When the Mod. AASHTO density for cemented material is determined, the specified procedure given in TMH1³ should be followed.

TABLE 20
Normal field compaction requirements for treated layered materials

Pavement layer		Relative compaction
Base (upper and lower)	Bituminous, hot-mix BC, BS; TC, TS	95% 75-blow Marshall or 92% theoretical max.
	Bituminous, cold-mix BT1, BT2 and BT3	100% Mod. AASHTO at field fluid content
	Cemented	97% Mod. AASHTO
Subbase (upper and lower)	Cemented	95% Mod. AASHTO
Selected layers	Cemented	90-93% Mod. AASHTO

(h) **Stability requirements for bituminous-treated natural gravel (BT2) or cohesionless sand (BT3)**

In the case of bituminous-treated fine-grained gravel used as a BT2, or sand, the vane shear strength of the mixed material at 40 °C should not be less than 200 kPa at 100 per cent Mod. AASHTO density (see Method ST7 in TMH6¹¹).

(i) **Grade of bituminous binders for materials (BC, BS; TC, TS) (BT1, BT2 and BT3)**

The following grades of binder conforming to SABS 307¹² (penetration bitumen), SABS 308¹³ (cutback bitumen), SABS 748¹⁴ and 749¹⁵ (tar), and SABS 309¹⁶ or 548¹⁷ (bitumen emulsion) are recommended for use:

TABLE 20(a)
Recommended binder grades

Climate	Hot-mix (BC, BS; TC, TS)	Bituminous-treated gravel or sand (BT1, BT2 and BT3)
Hot summer and moderate winter	40/50, or 60/70 pen. bitumen, or 55/60° or 50/55° EVT tar (RTH or RTL)	MC or RC 250 cutback bitumen; 30/35° or 35/40° EVT tar (RTH or RTL); or bitumen emulsion (stable grade)
Moderate to hot summer and cold winter	60/70, or 80/100 pen. bitumen or 50/55° or 45/50° EVT tar (RTH or RTL)	MC or RC 250 cutback bitumen; 30/35° EVT tar (RTH or RTL); or bitumen emulsion (stable grade)

(j) **Durability of cemented material**

In the case of cemented material, particularly when it is used under a concrete pavement slab, the material should satisfy the durability requirements of the wet-dry durability test for cemented materials, which take precedence over the strength requirements given in Table 18. The percentage loss of material should not exceed 14 per cent after 12 cycles of wetting and drying.

(k) **Deleterious minerals (C1 and C2)**

Sulphide minerals

Crushed rock which is known to contain sulphide minerals (pyrite, marcasite, chalcopyrite, pyrrhotite), should be further investigated for deleterious effects.

Soluble salts

There are a rather large number of salts whose solubility may be deleterious to surfaced roads. Their degree of solubility is not the same, and consequently the level at which they become deleterious changes according to the type of salt. An easily applicable guide is the electrical conductivity of the fines. The conductivity of the saturated paste should not be more than 2,0 mS/cm at 25 °C on the dry-screened -0,425 mm fraction, or not more than 1,5 mS/cm at 25 °C on the dry-screened -6,7 mm fraction. It is also advisable to check the sulphate content and pH of the material for all cases where the conductivity as determined above is greater than 0,4 mS/cm on the -0,425 mm fraction (or 0,2 mS/cm on the -6,7 mm fraction) when tested according to Method A20 described in TMH13. The total sulphate content determined according to BS 1377¹⁸ should not exceed 0,25 per cent, expressed as SO₃, if the Plasticity Index (PI) or the -0,002 mm fraction is greater than 12 per cent before treatment. Up to 1 per cent of SO₃ is permissible if the PI and the -0,002 mm fraction are both less than 12 per cent before treatment. In addition, the pH of the saturated soil fines paste should not be lower than 6,0.

3.3 TREATED MATERIAL (PENETRATION MACADAM (PM))

The aggregate used for penetration macadam should be angular and should have the following properties:

(a) **Crushing strength**

10 per cent FACT (min)	110 kN
or	
ACV (max)	29 per cent

In certain cases it may also be desirable to do the 10 per cent FACT on wet materials (24 hours' soaking followed by draining), and the wet value should then be at least 75 per cent of the dry value.

(b) **Shape of aggregate**

The aggregate should be angular and free from flat spalls and splinters. The weighted average Flakiness Index of the -26,5 +19,0 mm and -19,0 +13,2 mm fractions should not exceed 35.

(c) **Grading**

Coarse aggregate

The grading of the coarse aggregate should conform to the grading envelopes given in Table 21.

TABLE 21
Grading of coarse aggregate (penetration macadam)

Sieve size (mm)	Percentage passing by mass	
	Layers thicker than 75 mm (coarse)	Layers thinner than 75 mm (fine)
53,0	100	100
37,5	85-100	100
26,5	0-50	85-100
19,0	0-25	0-50
13,2	0-5	0-25
9,5	—	0-5

Keystone

The grading of the keystone should conform to the grading envelopes given in Table 22.

TABLE 22
Grading of keystones (penetration macadam)

Sieve size (mm)	Percentage passing by mass	
	19,0 mm nominal size	13,2 mm nominal size
26,5	100	100
19,0	85-100	100
13,2	0-50	85-100
9,5	0-25	0-55
6,7	0-5	0-25
4,75	—	0-10

The 19,0 mm keystone is used with the coarse grading and the 13,2 mm keystone with the fine grading in Table 21.

(d) **Binder**

The following binders, which conform to SABS 307¹² (penetration bitumen), SABS 748¹⁴ and 749¹⁵ (tar), are recommended for use:

Bitumen: 80/100 or 60/70 pen

Tar: 50/55 °C EVT or 55/60 °C EVT (RTH or RTL)

The binder should be applied at the following rates:

Bitumen: 0,8 ℓ/m² per 10 mm compacted thickness

Tar: 1,0 ℓ/m² per 10 mm compacted thickness

4 PORTLAND CEMENT CONCRETE (PCC)

The quality of the material used in concrete pavement construction should comply with the requirements set out below.

4.1 COARSE AGGREGATE

Single-sized coarse aggregate to be used in concrete pavements should comply with the following:

(a) Grading

TABLE 23
Grading of coarse aggregate (concrete pavements)

Sieve size (mm)	Percentage passing sieve						
	Nominal size of stone (mm)						
	53,0	37,5	26,5	19,0	13,2	9,5	6,7
75,0	100						
53,0	85-100	100					
37,5	0-50	85-100	100				
26,5	0-25	0-50	85-100	100			
19,0	0-5	0-25	0-50	85-100	100		
13,2	—	0-5	0-25	0-50	85-100	100	
9,5	—	—	0-5	0-25	0,55	85-100	100
6,7	—	—	—	0-5	0,25	0-55	85-100
4,75	—	—	—	—	0-10	0-10	0-55
2,36	—	—	—	—	—	—	0-10

For economic reasons the largest possible coarse aggregate should be used. In practice, however, this size should not exceed about 25 per cent of the pavement thickness, which means that the maximum nominal size of the coarse aggregate should not exceed 53,0 mm for pavements with a thickness of 200 mm or greater.

(b) Crushing strength

10 per cent FACT (min)	110 kN
or	
ACV (max)	29 per cent

In certain cases it may also be desirable to do the 10 per cent FACT on wet material (24 hours' soaking followed by draining), and the wet value should then be at least 75 per cent of the dry value.

(c) **Flakiness Index**

The weighted average Flakiness Index determined on the -26,5 +19,0 mm and -19,0 +13,2 mm fraction should not exceed 35 per cent.

(d) **Other coarse aggregate characteristics**

These may be determined if necessary under certain circumstances as follows:

Shape

As indicated by voids content, SABS Method 84519.

Water absorbtion

SABS Method 84320.

Content of material of low density

SABS Method 83721.

Abrasion resistance

SABS Method 84622. Limestone, dolomite and any other carbonate rocks may be used as coarse aggregate in concrete pavements only if an approved accelerated wearing test on concrete samples incorporating the coarse and fine aggregate proposed for use indicates that the coarse aggregate is satisfactory.

Any other impurities

4.2 FINE AGGREGATE

The main requirements for sand to be used in concrete pavements are as follows:

(a) **Grading**

TABLE 24

Grading of fine aggregate (concrete pavements)

Sieve size (mm)	Percentage passing sieve	
	Natural sand	Crusher sand
4,75	90-100	90-100
2,36	.	.
1,18	.	.
0,15	0-15	0-20
0,075	0-5	0-10

- * Because of the requirements of the microstructure, texturing and the life of the textured surface, at least 8 per cent of the material should be retained between 4,75 mm and 2,36 mm sieves, and 12 per cent between 2,36 mm and 1,18 mm sieves.

(b) **Dust content**

The material passing the 0,075 mm sieve should not exceed 5,0 per cent for sand derived from the natural disintegration of rock, and 10 per cent for sand derived from the mechanical crushing or milling of rock.

(c) **Fineness Modulus**

The Fineness Modulus of the fine aggregate (determined by Method B13 of TMH1³) should lie between 1,6 and 3,5.

(d) **(Cl⁻) Chloride content**

The chloride content, expressed as Cl⁻ per cent (m/m), should not exceed 0,03 per cent in sands for use in concrete pavements. It should be determined according to SABS Method 830²³.

(e) **Deleterious materials**

The sand should be free from sugar and excessive amounts of organic materials and soluble deleterious impurities.

The strength of mortar made with the sand should not be less than 85 per cent of the strength of mortar made with the sand after it has been washed.

(f) **Siliceous particle content**

Because it is the sand used in concrete pavements, rather than the coarse aggregate, that provides the microstructure required for skid resistance, it is desirable to have a sharp sand of high quality. The siliceous particle content, as determined by the acid insoluble test, should be at least 40 per cent and should consist only of pure quartz unless an approved accelerated wear test indicates that the sand is acceptable.

(g) **Other fine aggregate characteristics**

These may be determined as follows, if necessary under certain circumstances:

Water demand

PCI method. The water demand for fine aggregate (that is the amount of water necessary to produce a concrete slump of 35 mm in one cubic metre of concrete containing an optimum content of nominal 19 mm coarse aggregate) should not exceed 210 l.

Drying shrinkage and wetting expansion

SABS Method 837²¹. The drying shrinkage should be less than 150 per cent of the shrinkage of the reference aggregate to ensure that only a good quality sand is used.

Clay and silt content

BS 812⁶ - sedimentation method.

Content of material of low density
SABS Method 837²¹.

Shell content
SABS Method 840²⁴.

Cement-aggregate reaction

Aggregates containing certain minerals, especially certain types of silica, react adversely with high-alkali cements^{25,26}. Aggregates and cements suspected of adverse reaction should be laboratory checked before use. See Appendix A, Section 5, for further details.

4.3 CEMENT

Ordinary Portland cement (OPC) (complying with SABS Specification 471²⁷) or a blend (usually 50:50) of OPC and milled granulated blast-furnace slag (MGBFS) should be used in concrete pavements.

The cement content of the concrete should not be less than 310 kg per cubic metre.

4.4 WATER

The water used for concrete should be clean and free from deleterious materials, detrimental amounts of acid, alkali, sugar and any other organic substances.

Water suitable for drinking is generally acceptable for use in concrete. If required, the suitability of the water must be tested by an approved laboratory with respect to the following properties:

- pH value
- conductivity
- total dissolved solids
- loss on ignition (at 500 °C)
- total alkalinity (as CaCO_3)
- total hardness (as CaCO_3)
- calcium hardness (as CaCO_3)
- magnesium hardness (as CaCO_3)
- chloride content (Cl^-)
- sulphate content (SO_4^{--})
- suspended solids
- sulphide
- sugar (qualitative).

Water for concrete should not contain chlorides in the form of sodium chloride in excess of 3 000 ppm or sulphates occurring as sodium sulphate in excess of 2 000 ppm.

However, the water may be considered fit for use in concrete if the average 28-day strength of three mortar cubes made with the suspect water is not less than 90 per cent of that of three similar cubes made with distilled water.

4.5 ADMIXTURES TO CONCRETE

The use of an air-entraining admixture in concrete for pavements is highly recommended because it reduces the accumulation of free water on the surface during placing, compaction and finishing.

It also tends to improve the cohesive properties of the freshly placed concrete which makes the finishing-off and texturing much easier.

It may be difficult, however, to obtain the required amount of air (usually between four and six per cent) if the sand being used contains large amounts of fine material, even if much more than the prescribed amount of air-entraining admixture is added. Excessive amounts of admixture may damage the concrete and in this case an air-entraining agent should rather not be used, and an alternative mix proportion or choice of aggregate types should preferably be considered. Also, different types of admixture should be tested because they may react differently to different types of aggregate.

Admixture should be used only in liquid form and should be batched in solution in the mixing water by means of a mechanical batcher capable of dispensing the agent in quantities varying by not more than 5 per cent from the quantities required, and in such a way as to ensure uniform distribution of the agent throughout the batch during mixing.

An air-entraining admixture should preferably be a neutralized vinsol resin, must be of a well-known and approved brand, and should comply with the requirements of ASTM C260²⁸.

The air entrained in the freshly mixed concrete should preferably not be less than 4 per cent and should not exceed 6 per cent by volume. Air content should be determined by the pressure method.

Water-reducing or set-retarding admixtures should comply with the requirements of ASTM C494²⁹, but should not be used without the express approval of the engineer.

4.6 STRENGTH REQUIREMENTS FOR CONCRETE

The minimum 28-day flexural strength should be 3,8 MPa. The required corresponding 28-day compressive strength, meeting the specified flexural strength requirement, should then be determined as follows:

$$\begin{aligned} f_c &= \text{Corresponding compressive strength for 28-day flexural} \\ &\quad \text{strength of 3,8 MPa} \\ f_{cw} &= \text{Working (required) compressive strength for 28-day flexural} \\ &\quad \text{strength of 3,8 MPa} \\ &= 0,85 f_c \end{aligned}$$

In the determination of the relationship between compressive and flexural strength, the tests should be based on not less than six compressive strength specimens and not less than twelve flexural strength specimens for each cement : water ratio. All flexural strength tests should be made in accordance with Chapter 5 of the SABS standard building regulations³⁰ and all cube compression strength tests should be carried out in accordance with TMH1³.

5 GRAVEL WEARING COURSE

The design of gravel roads is not covered in TRH4¹. However, as these unpaved roads remain an important part of the total South African road network, a guide to the selection of suitable gravel wearing course material is necessary. The approved material is usually imported and laid on a prepared (compacted) sub-grade. The compacted thickness of the wearing course normally varies from 100 to 150 mm. The material is invariably a gravel or sand-clay mixture used to carry traffic on lightly trafficked roads which do not warrant an all-weather bituminous surface.

The most common causes of poor performance of gravel roads are slipperiness and potholing when wet, and excessive dust and ravelling when dry. The formation of corrugations is normally the result of inadequate compaction or low plasticity combined with traffic.

Calcretes are often the most readily available roadbuilding material in the drier areas of southern Africa (N-value >5; see Section 6 of Appendix A) and are therefore widely used as gravel and sand-clay wearing courses for unpaved roads.

5.1 GRADING

Experience has shown that a great variety of gradings of natural gravel perform well in practice, and therefore no fixed grading envelopes can be stipulated as essential for good performance.

The performance of the wearing course material is mainly influenced by the properties of the material passing the 2,00 mm sieve although it is generally desirable for the natural gravel to be well graded and not to exceed a maximum size of 50 mm, but 37,5 mm is preferred (after compaction).

In the absence of experience, the following grading envelopes are suggested as a guide.

TABLE 25
Suggested grading of gravel wearing courses

Sieve size (mm)	Percentage passing by mass			
	Maximum size of particle (mm)			
	37,5	26,5	19,0	13,2
37,5	100	100	100	100
26,5	85-100	100	100	100
19,0	70-100	80-100	100	100
13,2	60-85	60-85	75-100	100
4,75	40-60	45-65	50-75	60-100
2 00*	25-45	30-50	35-55	45-70
0,425	15-40	15-40	18-45	25-50
0,075*	7-30	7-30	7-30	7-30

* These sieves are the most important.

5.2 ATTERBERG LIMITS

The optimum value of the Plasticity Index (PI) is predominantly dependent on the climate of the area in which the material is to be used and on the grading of the material. The following range is suggested:

$$PI = 8 - 20$$

In a dry area (N-value from 5 to 10; see Section 6 of Appendix A) a PI in the upper half of the range is desirable, particularly where the gravel has a coarse grading. In this type of climate, a high PI reduces the tendency to ravel and corrugate.

In a very dry area, i.e. where N is greater than 10, the upper limit of PI may be exceeded. In a wet area, i.e. where N is less than 5, a PI in the lower half of the range is desirable particularly when a fine-grained gravel, such as a sand-clay mixture, is used. In such an environment a low PI reduces the tendency of the gravel to pothole or to become slippery in rain.

In the case of pedogenic materials the upper and lower limits of the PI can be extended as the material generally has self-cementing properties, e.g. calcrete or ferricrete.

5.3 FIELD COMPACTION

It is desirable to compact the material while it is damp. The moisture content should not be so high as to cause instability or excessive movement under the compaction plant employed.

The minimum in-situ density should be 93 per cent Mod. AASHTO density.

5.4 CALCRETE WEARING COURSE

Experience has shown that in the case of calcrete gravels the strength of the larger fractions has an important bearing on the performance of the gravel.

The best all-weather calcrete wearing course material will probably have a maximum particle size of about 20 mm; 20 to 50 per cent soil fines (passing 0,425 mm sieve); a linear shrinkage of between 3 and 6; an Aggregate Fingers Value (AFV)* in excess of 60 per cent; an Aggregate Pliers Value (APV)** of 20 to 75 per cent; and a value of linear shrinkage multiplied by percentage soil fines of between 100 and 200. The selection of calcrete for a wearing course should be based on local experience.

* The AFV test is carried out on a number of particles (-19,0 mm + 13,2 mm) and involves an attempt to break these particles with the fingers. The total percentage passing the test is known as the Aggregate Fingers Value. The particles passing this test are then tested for their APV. For further details see Reference 26.

** The APV test is carried out on a number of particles (-19,0 mm + 13,2 mm) and involves an attempt to break these particles with a pair of ordinary 180 mm pliers. The total percentage passing this test is known as the Aggregate Pliers Value.

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APPENDIX A

CLASSIFICATION AND PROPERTIES OF ROAD CONSTRUCTION MATERIALS

1 CLASSIFICATION OF ROCK TYPES USED IN ROAD CONSTRUCTION

Many more types of rock are used in road construction in southern Africa than types of soil. Each rock type is distinguished from others by structural, textural and mineralogical characteristics and each should be treated on its own merits. This is of course impractical and, for the purpose of road construction, these rocks may be grouped on the basis of similar road-construction properties in such a way that there is no need to dispute whether a certain material is, for example, a diabase, dolerite, gabbro or norite. Since the road-construction properties are the basis of this grouping, deviations from the traditional genetic classification of rocks are unavoidable.

This grouping is based on the presence or absence of quartz and the effect this has on the weathering of the rocks. Since quartz is so important for the assessment of the durability of a natural roadbuilding material, every road engineer should be able to recognize this mineral.

In the following description of the groups, many of the rocks are familiar. There are others, however, which are only used locally and which are, therefore, of little importance to southern Africa as a whole although they are important in the area concerned: phonolite, for instance, is not known to many materials engineers but it is a frequently used road-construction material in the central parts of South West Africa, especially in the vicinity of Windhoek; dolerite, on the other hand, which is so widely and frequently used in the Republic of South Africa and a few other territories, is only of minor importance in Zimbabwe.

Since every rock is suitable for use somewhere in a pavement, the rocks can only be rated as "good" or "bad" in terms of the various classes of road and their structural layers.

The groups and their members are listed and briefly described as follows:

Group: *Basic crystalline rocks*

Granular to dense.

Usually dark coloured, but anorthosite and diorite are light coloured.

Very little or no quartz.

Decompose to expansive montmorillonite clay where N is less than 5 (see Section 6 below).

Most frequently used member of group: **dolerite**.

Members of group:

amphibolite
andesite
anorthosite
basalt

diabase
diorite
dolerite
gabbro
greenschist

norite
peridotite
phonolite
serpentine

Group: Acid crystalline rocks

Granular to dense.

Light colours.

Much quartz except in syenite and certain felsites (rhyolites).

Minerals other than quartz decompose to non-expansive kaolinite clay where N is less than 5 (see Section 6 below).

Most frequently used member of group: **granite**.

Members of group:

- felsite
- gneiss
- granite
- pegmatite
- rhyolite
- syenite

Group: High-silica rocks

Light and dark colours.

Very dense.

Very hard, can usually not be scratched with a pocket knife.

Composed almost entirely of quartz or silica.

Only disintegrate to gravel or sand.

Most frequently used member of group: **quartzite**.

Members of group:

- chert
- hornfels
- vein quartz
- quartzite

Group: Arenaceous rocks

Various colours possible.

Sand grains (quartz) of various sizes (up to pebbles) and shapes in cementing matrix which may be siliceous, calcareous, ferruginous or clayey.

Cementing matrix is principal determinant of suitability for road construction, but shape of grains is also important.

Disintegrate to various types of sand.

Most frequently used member of group: **sandstone**.

Members of group:

- arkose
- conglomerate
- gritstone
- mica schist
- sandstone
- siltstone

Group: Argillaceous rocks

Medium (e.g. phyllite) to very fine-grained rocks.

Colour variable.

Composed mainly of clay minerals with varying quantities of sand grains (quartz).

Absorb water easily.

Softer members of group, e.g. shale, stick to wet tongue like blotting paper.

Disintegrate to more or less sandy clay.

Most frequently used member of group: **shale**.

Members of group:

mudstone

phyllite

sericite schist

shale

slate

Certain materials which have been used successfully for some kind of surfacing aggregate, e.g. near Keetmanshoop, have been called "shale". The very great strength of the materials (10 per cent FACT (dry) more than 230 kN, ACV less than 20 per cent), however, classifies them as hornfels (see high-silica rocks in this section).

Only fairly highly indurated "baked" argillaceous rocks are sufficiently strong and durable for use in the base of roads which carry traffic that is not too heavy (<E2 category suggested). The minimum strength should be 180 kN in the 10 per cent FACT on the dry material provided that the crushing strength of the wet material is not less than 125 kN. Such a material is durable in any environment and sufficiently strong for use in the base of the road. Rubbing together of the rock pieces, particularly during construction operations but also under traffic, causes fine rock-powder to develop and cover the aggregate. The powder, especially if it gets wet, acts as a lubricant which makes the pieces of aggregate move under traffic, thus reducing the overall bearing capacity of the base without affecting the strength of the individual rock pieces. Chemical stabilization of such bases is therefore advisable.

Certain indurated argillaceous rocks may absorb lime, particularly those in which montmorillonite is the clay mineral or in which some amorphous silica is present. The determination of the initial consumption of lime is advisable.

Group: Carbonate rocks

Granular to dense.

Light and dark colours.

Can be scratched with a pocket knife.

Disintegrate to rubble.

Effervesce in diluted hydrochloric acid (HCl).

Most frequently used member of group: **dolomite**.

Members of group:

- dolomite
- limestone
- marble

Certain white limestones may look very similar to certain dense hardpan calcretes and it may be difficult to distinguish between them. In an outcrop, the contact between the two types of rock can usually be recognized readily and in many cases a softer layer, which is mostly calcified limestone rubble, underlies the hardpan calcrete. In a hand specimen, hardpan calcrete is mostly distinguishable from limestone by the very rough, almost 'sandy' surface texture of the former and the numerous inclusions (sand grains, pieces of rock) it often contains. A hand specimen of a very dense hardpan calcrete, however, may be so similar to limestone that distinguishing becomes very difficult. In most cases, inspection of a specimen of such calcrete with a hand lens will disclose the presence of tiny cavities and widely spaced inclusions, especially more or less well-rounded sand grains.

Marble is distinguishable from dolomite and limestone by its crystalline texture.

Group: *Diamictites*

Mostly dark colours but lighter colours are also possible.

Grain size variable but mostly fine.

Random composition.

Depending on composition, these rocks may disintegrate, or they may decompose to clay where N is less than 5 (see Section 6 below).

Large textural and compositional variability makes constant expert advice desirable.

Most frequently used member of group: **tillite**.

Members of group (in southern Africa):

- greywacke
- tillite

The 10% dry FACT value of 160 kN should be adjusted to 200 kN for tillite. A value below 200 kN is reported to result in degrading of the material.

Group: *Metalliferous rocks*

Various kinds of mine waste.

Colours variable, although dark colours predominate.

Mostly heavy to very heavy materials, which makes haulage expensive.

Disintegrate to gravelly or sandy materials.

Most frequently used member of group: **ironstone**.

Members of group:

- ironstone
- magnesite
- magnetite

Group: *Pedogenic materials*

Powdery to rock-like materials.

Colours: white, brown, red, occasionally black. Consist of some kind of soil more or less strongly cemented by calcareous, ferruginous, siliceous or phosphatic material.

Nature of host soil, cementing material and degree of cementation (strength) determine suitability for road construction.

Most frequently used members of group: **calcrete** and **ferricrete**.

Members of group:

calcrete*

ferricrete

phoscrete

silcrete

The fluent transition between many of the above rock types, especially between members of a group but also between groups, would allow a much larger number of rock names to be added to those listed above. This would only be confusing, however, and only those rock names which are more or less familiar to road engineers in southern Africa have been given.

Influence of weathering on road aggregates

All these rocks may weather and through the stages of weathering the road construction properties of the rocks change. Although weathering is a complicated process, it is sufficient for the purposes of road engineering to distinguish between the principal forms of weathering, i.e. disintegration and decomposition.

In **disintegration** the rock merely breaks and crumbles down to ever smaller pieces while its minerals remain unaffected. Acid and basic crystalline rocks and certain diamictites disintegrate only where N is greater than 5 (see Section 6 below), but all other types of rock disintegrate in all climatic environments. The effect of this form of weathering on the roadbuilding properties of the rock is a continuous decrease in strength without the durability of the material being affected. This means that any material which satisfies the design requirements can be used without the risk that it will change in service to the extent that it could cause problems.

In **decomposition**, which is of significance only where N is less than 5 (see Section 6 below), secondary minerals consisting mostly of clay are formed and an entirely different material develops. Since this process affects only those minerals which are known as "primary", decomposition is restricted to the acid and basic crystalline rocks and those diamictites which contain large quantities of primary minerals of crystalline rocks. Since acid and basic crystalline rocks are the most frequently used natural roadbuilding materials in more than half of the surface area of southern Africa, these conditions are particularly important and great care is required in the selection of weathered crystalline rocks and diamictites. The stage of decomposition must be considered in relation to the N-value at the site because the rate of decomposition increases within a pavement. This is best done by determining the percentage of secondary minerals. The maximum percentage of secondary minerals permissible to ensure adequate durability

* See remarks on limestone under "carbonate rock" in this section.

ity of a crystalline rock for different climatic regions (N-values) when used in bases or subbases is shown in Figure 1A.

The figure refers only to untreated material. If the material is treated, then a slightly higher degree of decomposition is permissible.

In general, suitable materials can be found everywhere but particular care is required when selecting weathering crystalline rocks or diamictites for use in pavement layers: where N is greater than 5, material on or close to the surface can be used for many more purposes than where N is less than 5. Therefore, materials for a particular purpose can usually be obtained quite easily where N is greater than 5. Where N is less than 5, material satisfying the requirements for the same purpose can only be found at a greater depth.

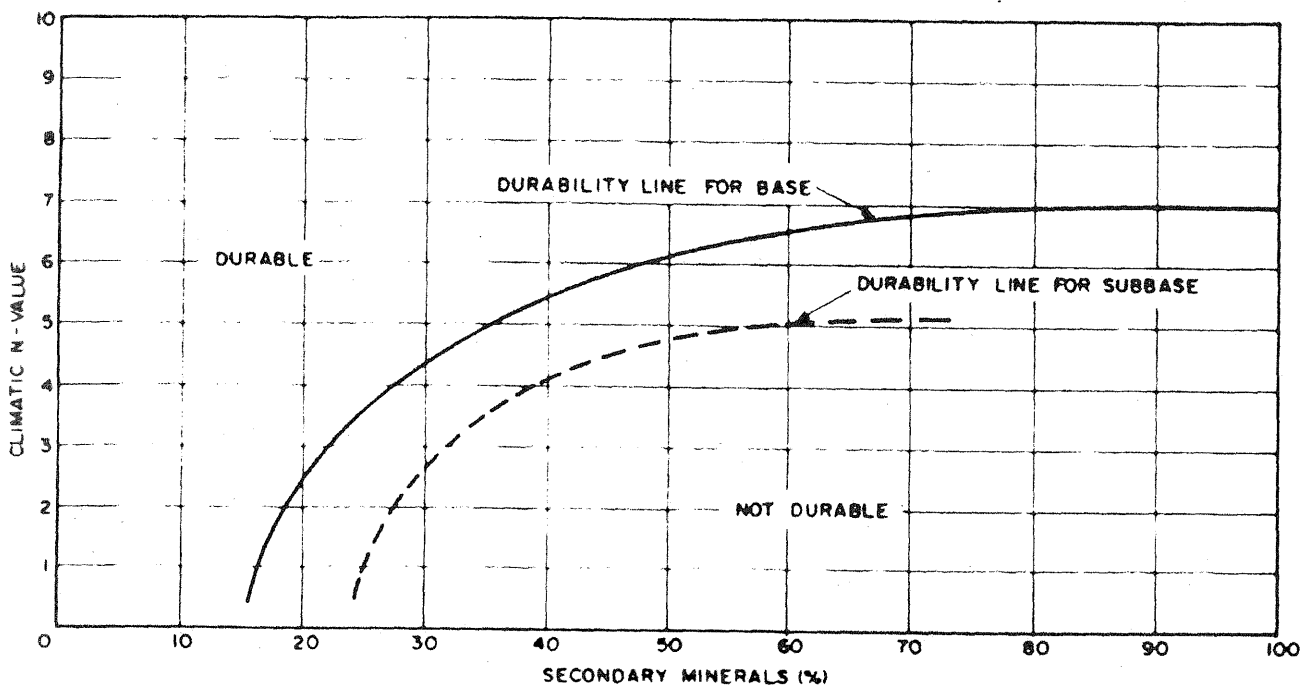


FIGURE 1A

Durability of weathered crystalline rocks in pavements depending on the N-value and the percentage of secondary minerals

2 CRUSHING STRENGTH

Fresh or slightly weathered rock may be crushed for use in surfacings, bases and occasionally also subbases. Strength is one of the most important criteria for the assessment of durability and quality.

Strength, or rather, in the case of road construction, crushing strength, is a natural property of rock which varies with the type of rock and its stage of weathering. Since only fresh or slightly weathered rock can be considered for crushed road aggregate, the crushing strength is not only a measure of the suitability of the rock for the various purposes of road construction, but is also a measure of the degree of disintegration which the rock has attained and thus its durability.

The crushing strength values, either obtained from the 10 per cent Fines Aggregate Crushing Test (10 per cent FACT) on dry and wet material or expressed as the Aggregate Crushing Value (ACV), both given in the SABS standard specification for aggregates from natural sources (SABS 1083)⁹, apply to most types of rock, but these recommendations also draw attention to certain exceptions which follow:

Arenaceous rocks

The specification SABS 1083⁹ applies only to arenaceous rocks whose cementing matrix is siliceous. A siliceous matrix can often be recognized by its glassy appearance which makes the sand (quartz) grains and the matrix look very much alike while other types of matrix are usually very different from the quartz grains in colour and appearance. Since it is not always easy to be certain, the following may serve as a guide:

- (a) Many arenaceous rocks, which are known commercially as "quartzite" and which have been used successfully for **surfacing aggregate**, are in fact quartzitic sandstones, i.e. arenaceous rocks with a siliceous matrix.
- (b) Only arenaceous rocks with a siliceous matrix will be sufficiently strong to satisfy the strength requirements for the various types of **surfacing aggregate** as specified in SABS 1083⁹.

The strength specification for **bases** in SABS 1083⁹ however, is too low for arenaceous rocks with a cementing matrix other than a siliceous one, and there is the risk of selecting materials that would not stand up to the service requirements. Again, the crushing strength for bases as specified in SABS 1083⁹ could be applied to arenaceous rocks with a siliceous cementing matrix. Unless the nature of the cementing matrix can be ascertained, however, it is recommended that slightly stronger arenaceous rocks, as shown in Table A1, be selected.

Argillaceous rocks

These are not suitable for use as **surfacing aggregate**. So-called 'shales' which have been crushed and used successfully in surfacings, e.g. Malmesbury shale, are actually hornfels and as such are members of the group of high-silica rocks.

Argillaceous rocks suitable for use in **bases** must be much stronger than specified in SABS 1083⁹, i.e. they must be indurated to a considerable degree. Such rocks occur mostly in a condition which makes them akin to natural gravel. Such a material is usually not crushed commercially but crushing tests should be done for selection. Recommended values for the crushing strength of these materials are given in Table A1. Attention is drawn to the fixed value given in this table for the wet test instead of a permissible percentage decrease of the strength of the wet aggregate relative to the dry one. This fixed value for the wet crushing strength must not, however, be taken to imply that only the wet strength need be determined, because the decisive value for the selection of durable aggregate is obtained from the dry test.

TABLE A1
Recommended crushing strengths of natural road construction materials

Group of natural road construction materials	Recommended crushing strength for				Remarks
	Surfacing aggregate			Base	
	Rolled-on chips	Surface treatment	Bituminous mixtures		
Basic crystalline rocks	10 % FACT: 210 kN/75 % ACV: 18 %	10 % FACT: 210 kN/75 % ACV: 21 %	10 % FACT: 160 kN/75 % ACV: 25 %	10 % FACT: 110 kN/75 % ACV: 29 %	
Acid crystalline rocks	10 % FACT: 210 kN/75 % ACV: 13 %	10 % FACT: 210 kN/75 % ACV: 21 %	10 % FACT: 160 kN/75 % ACV: 25 %	10 % FACT: 110 kN/75 % ACV: 29 %	
High-silica rocks	10 % FACT: 210 kN/75 % ACV: 18 %	10 % FACT: 210 kN/75 % ACV: 21 %	10 % FACT: 160 kN/75 % ACV: 25 %	10 % FACT: 110 kN/75 % ACV: 29 %	The wet test is of very limited relevance; it could be disregarded*
Arenaceous rocks	10 % FACT: 210 kN/75 % ACV: 18 %	10 % FACT: 210 kN/75 % ACV: 21 %	10 % FACT: 160 kN/75 % ACV: 25 %	10 % FACT: 140 kN/75 % ACV: 27 %	110 kN/75 % or 29 % are only acceptable for base if cementing matrix is siliceous
Argillaceous rocks		Unsuitable		10 % FACT: 180 kN/125 [†] kN ACV: 24 %	Subbase* 10 % FACT: 160 kN/125 kN ACV: 25 %
Carbonate rocks	10 % FACT: 210 kN/75 % ACV: 18 %	10 % FACT: 210 kN/75 % ACV: 21 %	10 % FACT: 160 kN/75 % ACV: 25 %	10 % FACT: 110 kN/75 % ACV: 29 %	
Diamictites*	10 % FACT: 220 kN/70 % ACV: 17 %	10 % FACT: 220 kN/70 % ACV: 21 %	10 % FACT: 170 kN/70 % ACV: 24 %	10 % FACT: 200 kN/70 % ACV: 21 %	140 kN/70 % or 27 % acceptable for bases of lightly trafficked roads
Metalliferous rocks		Not used		10 % FACT: 110 kN/75 % ACV: 29 %	
Pedogenic materials					
Calcrete**	No information available for southern Africa		10 % FACT: 180 kN/65 %	10 % FACT: 110 kN/50 % (medium traffic)	Bases of lightly trafficked roads perhaps 80 kN/50 %
Ferricrete	Not suitable for crushing tests				
Silcrete	10 % FACT: 210 kN/75 % ACV: 18 %	10 % FACT: 210 kN/75 % ACV: 21 %	10 % FACT: 160 kN/75 % ACV: 25 %	10 % FACT: 110 kN/75 % ACV: 29 %	Compare high-silica rocks

* Deviations from SABS 1083⁹ - suggested by H.H. Weinerl

** Deviations from SABS 1083⁹ - suggested by F. Netterberg

[†] Only to be used on lightly trafficked roads

Diamictites

These are extremely variable and it is difficult to describe their properties in general terms. Their road construction properties are largely a combination of those of the arenaceous and argillaceous rocks and they are often influenced by those of the acid and basic crystalline rocks. It would be safer therefore to select slightly stronger material than specified in SABS 1083⁹ for use in all types of **surfacing** and noticeably stronger material for use as crushed rock in **bases**. Recommended values are given in Table A1.

A peculiarity of diamictites is that the wet test result in the 10 per cent FACT need only be 70 per cent of that obtained in the dry test, but the limits set for the dry material should be slightly increased (see Table A1). This stems from the way in which the fine silt and clay size components which consist mainly of quartz, several types of clay mineral and rock powder are intermixed and intergrown.

Tillite is the only diamictite which is certainly known to have been used in southern African road construction, although greywacke from the Cape Supergroup and Karoo Sequence, thought to be sandstone, may also have been used. Other rocks in this group would be volcanic ejecta (tuff, breccia) but owing to their scarcity in southern Africa they have not so far been used in road construction.

Pedogenic materials

Ferricrete has rarely been crushed in South Africa.

Hardpan and honeycomb silcrete can be, and probably have been, used as crushed rock especially in minor roads in certain parts of South Africa. A well-cemented hardpan or honeycomb silcrete is similar to quartzite and the strength of crushed silcrete should be assessed along the same lines as that of the high-silica rocks (see Table A1).

According to the SABS specification 1083⁹, crushed hardpan or honeycomb calcrete, if not too porous, would only be suitable for **bituminous mixtures** since the material, even if it were silicified to some extent, would hardly ever satisfy the strength requirements for **surface treatments** and **rolled-on chips**.

Since calcrete consists mostly of calcite, even if somewhat silicified, its physical break-down easily leads to excessive fines. It is therefore advisable to base selection on a slightly greater crushing strength than specified in SABS 1083⁹. Such a slightly greater crushing strength would also make provision for the acceptable decrease of the wet strength of 65 per cent as suggested by Netterberg¹⁰ because of the self-cementing properties of calcrete.

Crushed calcrete may be used for bases and, since usable calcrete occurs almost only in semi-arid areas (N greater than 5) with a regional factor of 0.4, a dry strength of 110 kN in the 10 per cent FACT, and an ACV of 29 per cent as specified in SABS 1083⁹ are acceptable for a road carrying light to medium traffic. Netterberg¹⁰ has considered the possibility of even lower strengths for bases of lightly trafficked roads and stabilized bases. Since some of the carbonate in calcrete goes into solution when wetted repeatedly during construction, and since it is reprecipitated during drying, a wet strength of only 50 per cent of the dry strength is acceptable.

The above values are given in Table A1. The 10 per cent FACT is given in kN for the required minimum strength of the dry material and the permissible percentage decrease in the strength of the wet sample in relation to that of the dry one. In the case of argillaceous rocks, however, the wet strength is also given in kN. No wet strength is given for the ACV because this relationship, which may not be linear, has never been investigated.

The results obtained from one of these crushing tests on dry material can be converted by approximation to those of the other by using the expressions:

$$\text{ACV (in percentage)} \approx 38 - 0,08 (10 \text{ per cent FACT (in kN)})$$

$$10 \text{ per cent FACT (in kN)} \approx 12,5 (38 - \text{ACV (in percentage)})$$

provided the 10 per cent FACT is between 100 and 300 kN and the ACV between 14 and 30 per cent. If the crushing strength of the aggregate is greater than 350 to 400 kN in the 10 per cent FACT or between 6 and 10 per cent for the ACV, the shape of the aggregate, i.e. its flakiness, becomes increasingly important until it is practically the only property which determines the amount of fines produced in the tests.

3 POLISHING

Aggregate exposed to the surface of a road is subject to abrasion under traffic. As a result of the abrasion, the stone may attain a smooth, polished surface and may eventually become flush with the road surface with a loss of the frictional resistance exerted by such a surface on moving traffic, especially under wet conditions. This frictional resistance is expressed as the Polished Stone Value (PSV), and a stone whose PSV is greater than 80 is considered not to polish.

The resistance to abrasion is a natural property of rocks which depends mainly on their mineral composition, but textural features and weathering may have a modifying effect. The resistance of minerals to abrasion is largely a function of their hardness. The hardness of the rock-forming minerals according to Mohs' scale is given in Table A2.

TABLE A2
Hardness of rock-forming minerals (Mohs' scale)

Mineral	Hardness
Quartz	7,0
Olivine	6,5-7,0
Pyroxenes	6,0-6,5
Feldspars	6,0
Opal	5,5-6,5
Amphibole	5,5-6,0
Dolomite	3,5
Calcite	3,0
Micas	2,5-3,0
Chlorite	2,0-2,5
Clay minerals	1-2

From this table it follows that quartz is the hardest of the important rock-forming minerals and that olivine is almost as hard, while all the others are noticeably softer. It is obvious that a rock that is composed of minerals of different hardness will polish less than a rock whose minerals are all of about the same hardness. The difference in the hardness of the components is more important than their absolute hardness, and therefore a rock which is composed of quartz only will also polish eventually.

The distribution of minerals in a rock gives it a textural property such that the more evenly hard and soft minerals distributed within a rock, the less the rock will polish. Thus the occurrence of olivine in certain basalts (olivine basalt) is of little advantage to the polishing resistance of the rock because olivines occur mostly in clusters. If such a basalt is crushed, many chips will be entirely free of olivine, and where a cluster of olivine occurs in a chip, the olivine will be inclined to break out, leaving an olivine-free chip behind. Very slight weathering, especially decomposition of crystalline rocks, for example if the percentage of secondary minerals is between 10 and 15, may cause the polishing to be less severe, especially in quartz-free rocks. At this stage of weathering, only a limited number of minerals are affected by decomposition, and even if the rock were composed only of minerals of similar hardness, the slightly weathered minerals would abrade faster than unweathered ones, and the rough surface texture of the chips would be retained.

The polishing characteristics of the groups of natural roadbuilding materials that are used for surfacing aggregate are:

Basic crystalline rocks

All of them are liable to polish. Variations of the PSV, which are observed occasionally, are due either to some evenly distributed olivine, e.g. in dolerite or norite, or to slight weathering.

Acid crystalline rocks

Generally, these rocks do not polish much; syenite and many felsites are likely to polish more than the others.

High silica rocks

These polish in time but their great overall resistance makes the binder abrade faster than the aggregate and a rough surface is retained.

Arenaceous rocks

Only quartzitic sandstone, i.e. sandstone with a siliceous cementing matrix (which is opal), can be used for surfacing aggregate. These rocks are often known commercially by the name "quartzite". Such sandstones do not cause problems due to polishing.

Carbonate rocks

These polish more than any other group of natural roadbuilding materials. Southern African carbonate rocks hardly ever contain sufficient sand grains (quartz) to improve the polishing characteristics.

Diamictites

Their very variable composition has of course an influence on the polishing characteristics but most of them polish so little that they may even approach a PSV of 80.

Pedogenic materials

The polishing of **calcrete** depends entirely on the presence or absence of sand (quartz) grains; therefore some calcretes polish like carbonate rocks and others may be compared to the non-polishing diamictites. **Silcrete** has the same polishing characteristics as the high-silica rocks.

4 FLAKINESS

The tendency of a rock to break into cubical or flaky chips during crushing is only partly a natural property. Schistose, laminated, very fine-grained or dense rocks are more inclined to produce flaky chips than are rocks of another texture. The technique of crushing, especially the rate of reduction, however, probably has a greater influence on the production of cubical or flaky chips than the petrological nature of the rock. Cubical chips could thus be produced from any type of rock and would be durable if produced from fine to medium-grained or dense rocks. Cubical chips from schistose or laminated rocks, however, will not last because they will crack and change into little plates during handling and in service. This cracking will occur along the predetermined planes of weakness, i.e. the layers of schistosity or lamination. It may often be difficult to crush coarse-grained rocks satisfactorily because the crushing process may enhance the breakdown of the rock into its individual minerals. Cubical chips obtained from such rock may also be insufficiently durable.

Since the production of more or less flaky chips is mainly due to the crushing process, an assessment of the relationship between the groups of natural roadbuilding materials and the greater or lesser tendency of rocks to break into flaky chips would not be very revealing. In general, it can only be stated that very fine-grained and dense rocks have a greater tendency to break into flaky chips than others, a fact which was known to prehistoric man who selected such rocks for making his stone implements.

5 DELETERIOUS MINERALS

A number of minerals which may occur in rocks from various groups of natural roadbuilding materials may affect the performance of the road adversely. Such minerals are:

Mica

This is a very elastic mineral, especially the light-coloured muscovite, the platelets of which, if bent, tend to return to their original shape. This "spring action" leads to an increase in the voids ratio with undesirable consequences for compacted layers and concrete. Compacted layers lose their density and the increased voids ratio promotes the ingress of water. Muscovite in concrete aggre-

gate increases the water demand and drying shrinkage, decreases the cube strength and has an adverse effect on the workability.

These problems occur particularly if the muscovite platelets have a diameter of more than 0,5 mm and if the quantity is near to, or more than, 10 per cent by volume of the total aggregate. Cement stabilization helps to counteract the "spring action" to some degree.

The above statements apply only to mica which occurs loosely in a soil or weathered rock. Since the determination of the mica content is complex, the following rule of thumb may be used: if mica can be recognized at a glance, i.e. without searching for the platelets, perhaps with a hand lens, caution is required and remedial measures are advisable.

Sulphide minerals

These minerals, especially pyrite (colour: gold), marcasite (colour: silver) and chalcopyrite, also called copper pyrite (colour: iridescent gold, red and green) decompose easily in the presence of air and water. The decomposition is aggravated by the presence of certain bacteria which occur frequently in nature. One result of the decomposition of sulphide minerals is the formation of acids, especially sulphuric acid, with deleterious effects on concrete and lime or cement stabilized road layers. These acids are not stable and change into sulphate salts.

Experience has shown that the total quantity of sulphide minerals as well as their distribution in the rock determines the severity of their deleterious effect on roads. An aggregate containing more than 1 per cent by volume of such minerals, particularly if they are evenly distributed through the rock and not concentrated in clusters here and there, should not be used in the base and subbase if used for waterbound macadam, although up to 2 per cent will do no harm in bituminous mixtures or surface treatments.

Soluble salts

These do not only develop from the decomposition of sulphide minerals but may also be contained in the natural soil, especially in areas where N is greater than 5 (see Section 6 below). If they are only contained in or develop from the imported aggregate, their quantity is limited and the supply may be exhausted in time. If they are contained in the natural soil, however, there is an almost inexhaustible source of such salts - similar conditions prevail where a road passes a mine dump, especially if it contains yellow mine sand.

It follows that limits on the acceptable contents of soluble salts can be applied to imported materials if the natural subgrade is free of salts, but such limits are purely hypothetical if the natural subgrade provides an inexhaustible source of soluble salts. In the latter case, countermeasures can only be taken by appropriate pavement design.

The easiest and fastest method for the determination of potentially harmful quantities of soluble salts in roadbuilding materials is to measure the electrical conductivity of the soil paste. Depending on the grain size of the tested aggregate, the following limits are recommended:

Size -0,425 mm : not more than 2,0 mS/cm at 25 °C

Size -6,7 mm : not more than 1,5 mS/cm at 25 °C

Materials which give higher conductivity values should be rejected.

Notwithstanding the above, if the conductivity is -

more than 0,04 mS/cm on the -0,425 mm material, or

more than 0,02 mS/cm on the -6,7 mm material,

the quantity of sulphate, expressed as SO_3 , should always be determined according to one of the following methods: TMH¹³ Method B17, SABS Method 850³¹ or BS 1377¹⁸. The maximum sulphate content, expressed as SO_3 , should not exceed 0,25 per cent if the PI or the -0,002 mm fraction is greater than 12 per cent, but it may be up to 1 per cent SO_3 if both the PI and the -0,002 mm fraction are less than 12 per cent before stabilization and if the pH of the saturated soils fines paste is not less than 6. The determination of the pH is required because sulphate in the quantities given above always involves the risk of some free sulphuric acid being present.

The above limits are tentative and the reader is therefore advised to keep up to date with developments resulting from experience.

Reactive silica

This occurs in nature in a variety of forms which may have an adverse effect on concrete made with certain cements, especially those known as 'high alkali' cements. Such forms of reactive silica are **amorphous silica**, the mineral **opal** and **siliceous skeletons** of microfossils. These three modifications are often called amorphous or opaline silica. There are also the minerals **cristoballite** and **tridymite**, high-temperature modifications of quartz, which become unstable under surface conditions, and the mineral **chalcedony**, a cryptocrystalline (extremely fine-grained) fibrous form of quartz which develops over geological times from amorphous silica. Even **quartz** itself, when it is strained owing to metamorphism or is intensely fractured, may be unsuitable for use with certain cements if these minerals make up 30 per cent or more of the constituents of the aggregate.

The number of rocks which may contain one or more of these forms of silica is so large - they are represented in all nine groups of natural road construction materials - that they cannot be listed here. So far, in the Republic of South Africa, adverse cement-aggregate reaction due to reactive silica has been shown to affect concrete roads containing Malmesbury hornfels, more commonly known by the misleading name "Malmesbury shale", in the Cape Peninsula. The use of high-alkali cements could lead to similar failures with many other types of aggregate. Concrete aggregate should therefore always be subject to petrological examinations, especially if an as yet unknown rock is used. Even long-established aggregates should be examined from time to time to keep track of gradual changes or to discover sudden compositional changes²⁵.

Amorphous silica consumes very much lime, but at the same time it has a welcome pozzolanic effect if contained in a roadbuilding material which is stabilized

with lime. It is therefore recommended that such a material be stabilized with lime, irrespective of its plasticity, as has already been shown by Netterberg ¹⁰, for calcretes containing diatomaceous skeletons.

Nepheline/analcime

Nepheline is a primary mineral in certain basic crystalline rocks, and analcime, a zeolite-type mineral, is an alteration product of nepheline. The change from nepheline to analcime is associated with a considerable increase in the volume of the new mineral, which in turn is associated with the complete disintegration of the rock. This change occurs rapidly if the rock is exposed to the atmosphere. It is particularly severe if the rock contains cryptocrystalline or amorphous nepheline. The process has become known as "sunburn" and occurs particularly in nepheline-basalt, but it cannot be completely excluded from other basalts and rocks such as phonolite and andesite.

A rock which is liable to sunburn may look entirely fresh when first exposed. The sunburn potential may be determined by boiling pieces of the suspect rock in distilled water for at least 36 hours. If the rock is a sunburner, white spots will be seen on the treated surface. Sunburn has not yet been proved to occur in southern Africa although a few cases of fast breakdown of surfacing aggregate obtained from the Drakensberg Group may have been the result of this process.

Clay minerals

These are part of a large group of rather different minerals. They all form platy crystals based on silica sheets and all have the ability to absorb water in their crystal lattice or to release it if conditions change. The two most important groups of clay minerals, as far as road construction is concerned, are the kaolinite and the smectite groups. The best-known member of the latter group is montmorillonite, whose unfavourable properties are well known. The climatic and geological conditions of southern Africa are such that montmorillonite occurs much more frequently than kaolinite. Therefore, whenever a clay is encountered, montmorillonite should be suspected until the contrary can be proved.

6 ENVIRONMENTAL INFLUENCES

When natural roadbuilding materials other than fresh rock are selected, the environmental influences must not be disregarded, since they determine the mode and stage of weathering of the material and its potential durability. The particular environmental factors that need to be considered in this context are climate and topography.

Climate is the most important single influence on weathering and the durability of natural roadbuilding materials. The relevant climatic conditions are expressed by the N-value:

$$N = \frac{12E_J}{P_a}$$

where E_J is the computed evaporation during January and P_a is the total annual precipitation. There are only three N-values of importance to road construction. They are:

$$\begin{aligned} N &= 2 \\ N &= 5 \\ N &= 10 \end{aligned}$$

$N=5$ is the most important. When N is less than 5, many rocks, especially the crystalline types, decompose; when N is greater than 5, all rocks disintegrate. Decomposition and disintegration are the principal forms of weathering and decomposition is more detrimental to the quality and durability of natural roadbuilding materials than disintegration (see Section 1 of Appendix A).

Figure 2A gives the contours of $N=2$, $N=5$ and $N=10$. Note that most administrative areas of southern Africa fall predominantly within one of the major categories of N : less than 2, 2 to 5, more than 5. For example, nearly all of Natal has an N -value of less than 2 and most of the Cape Province an N -value of more than 5. Each area may therefore use its own standards for the assessment of the suitability and especially the durability of natural roadbuilding materials, particularly if they are derived from acid and basic crystalline rocks and from diamicrites. The selection of weathered dolerite, for instance, is therefore subject to fewer restrictions in the Cape Province than in Natal. The criteria used for selection in the Cape Province must not be transferred to Natal. This document, however, covers the whole of southern Africa and note must therefore be taken of the differences in quality and suitability resulting from the climatic variation of the subcontinent.

The N -value can best be calculated by the expression:

$$N = \frac{372c_J(0.58 + kW_J)}{P_a W_0 S_J}$$

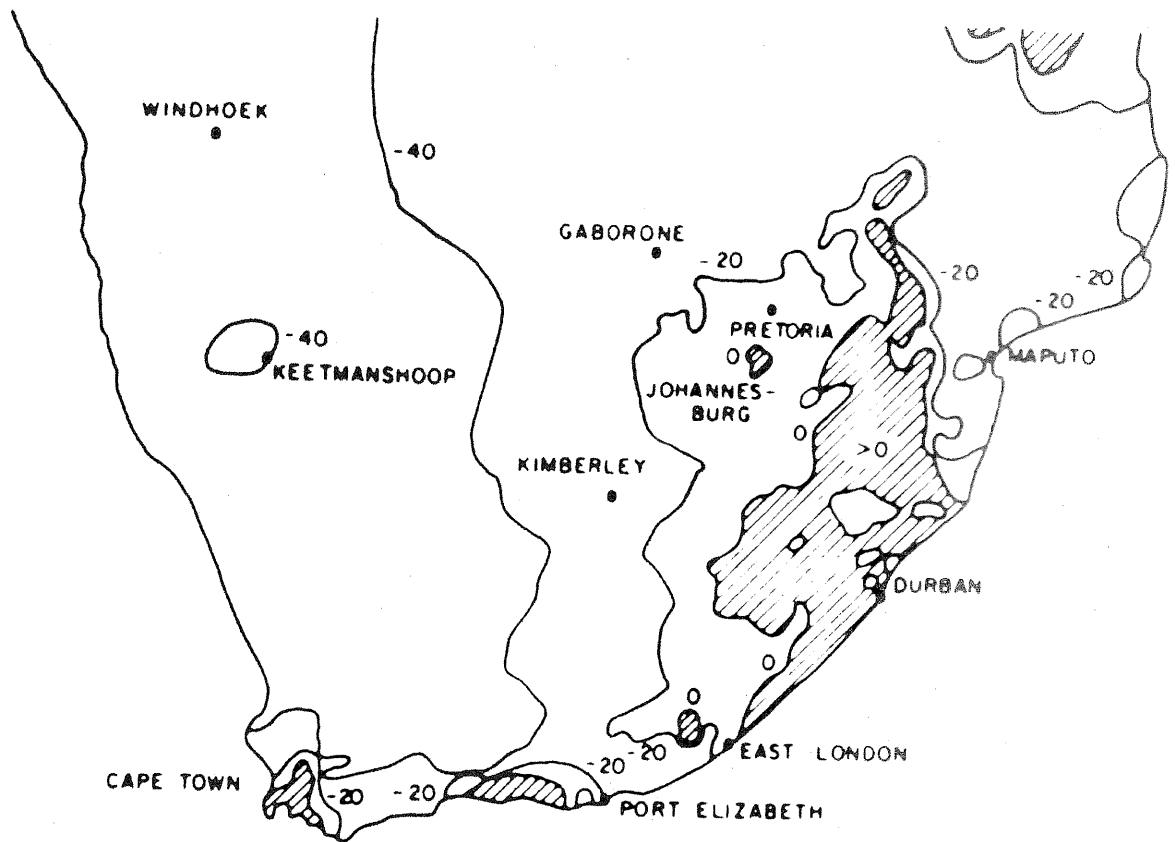
in which Olivier's expression for evaporation is used, where:

c_J is the average decrease of the wet bulb reading for January. This value can be obtained from direct measurements or it may be read off from hypsometric tables (published e.g. in the Handbook for Physics and Chemistry) if the average relative humidity and temperature for January are known.

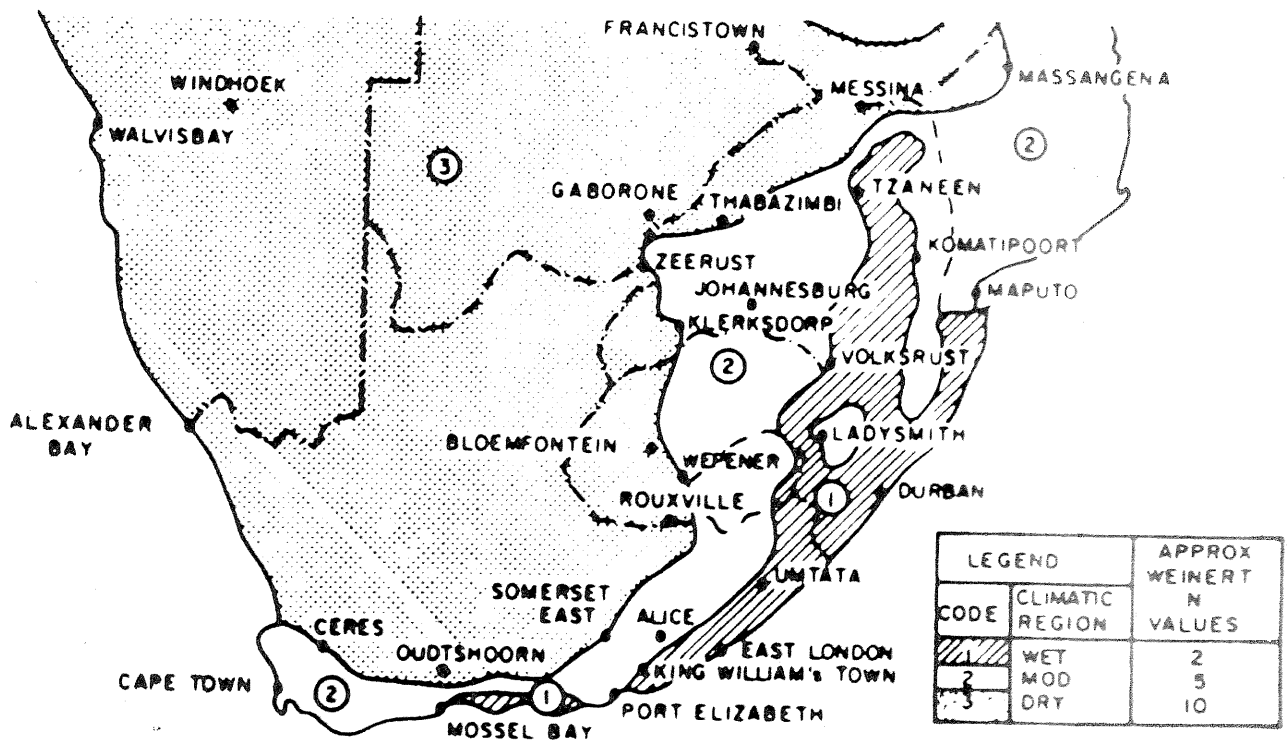
kW_J is part of the wind correction $(0.58 + kW_J)$ where W_J is the average wind speed for January and k is 0.14 if W_J is measured in m/s; 0.0626 if W_J is measured in mph; and 0.0719 if W_J is measured in knots.

P_a is the average annual rainfall in mm.

$W_0 S_J$ is a correction for latitude called the "Monthly Field Water Requirements Characteristic Constant" whose values for January for the southern hemisphere are given in Table A3.



a) MOISTURE INDEX (1m) ACCORDING TO THORNTHWAITE'S CLASSIFICATION



b) MOISTURE REGIONS ACCORDING TO WEINERT N VALUES

FIGURE 2A
Climatic regions of southern Africa

TABLE A3

Characteristic constants ($W\phi S$) for monthly field water requirements for southern Africa in January

Lat. S	10°	15°	20°	25°	30°	35°	40°
$W\phi S$ January	0,70	0,74	0,76	0,79	0,82	0,83	0,85

Climatic conditions may change abruptly in areas with an irregular relief, e.g. mountain ranges or escarpments, and the determination of the local N -value according to a method published elsewhere ²⁶ is recommended. These areas are shown in Figure 3A.

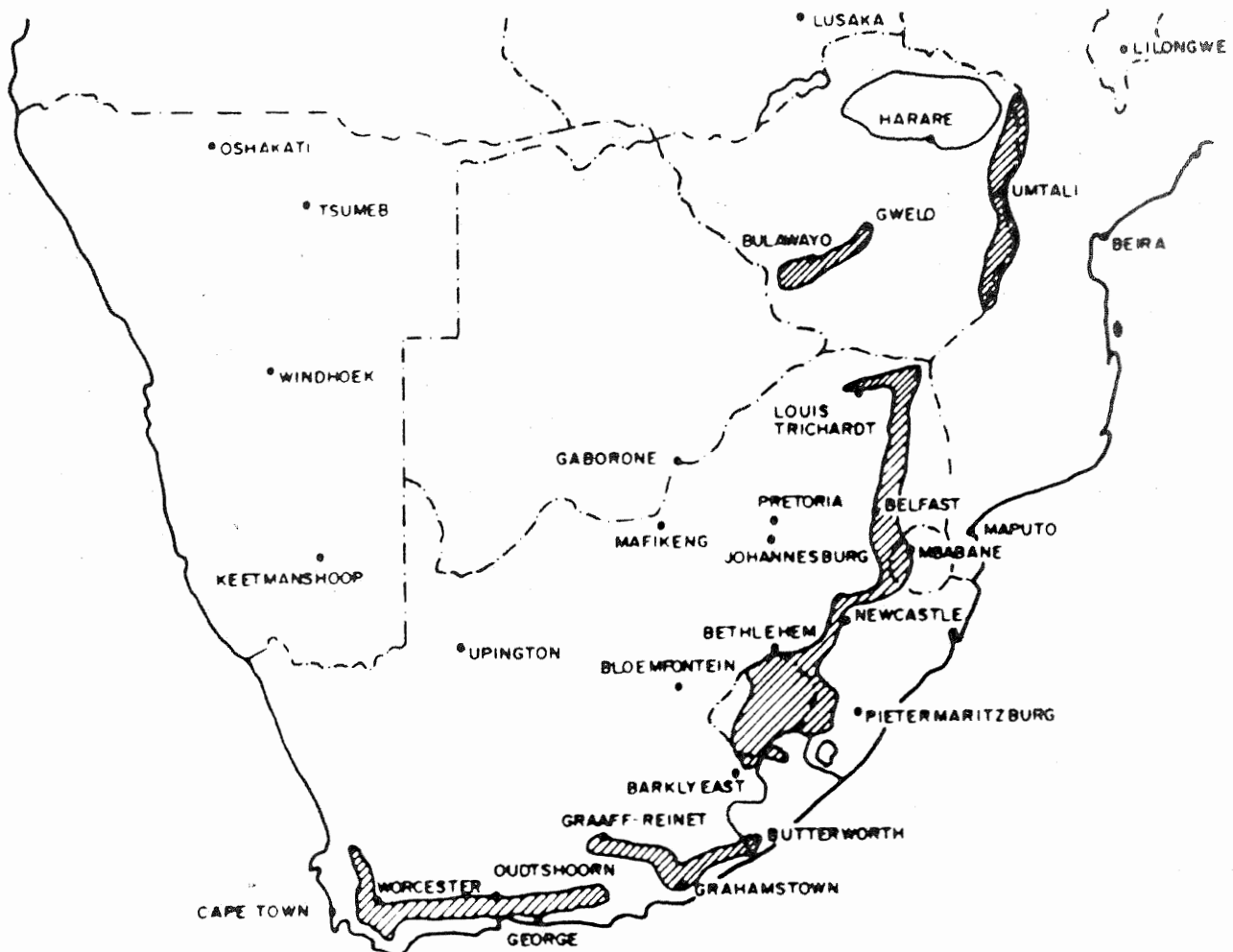


FIGURE 3A

Areas which warrant local determination of N -values (these areas are shaded)

Topography may modify the effect of the N-value. The influence of the N-value on weathering refers mainly to flat country with very little or no run-off. In general, variations of topography are variations of slopes, and since such variations have an effect on run-off they also influence the mode, the intensity and the depth of weathering. Four ranges of slopes can be considered important in this context:

- 0° - 5° : Flat country to which the N-value applies as shown in Figure 2A.
- 5° - 15° : Most suitable sites for borrow pits in weathered rock; material of suitable quality is obtained more easily than in flat country.
- 15° - 30° : Soil cover is very much reduced in thickness. Too steep for borrow pits but suitable for quarries. Profound changes take place in clay minerals: much of the montmorillonite changes into kaolinite.
- >30° : Mostly solid rock, exposed. Very suitable for quarries if access can be gained on less sloping ground.

APPENDIX B

METHOD OF SAMPLING AND TESTING STABILIZED MATERIALS FOR THE DETERMINATION OF THE MAXIMUM DRY DENSITY, OPTIMUM MOISTURE CONTENT AND CBR OR UCS

1 SCOPE

This method describes the sampling of stabilized material just before the commencement of field compaction and the subsequent determination of the OMC, MDD and compaction of the material for the CBR or UCS determinations.

2 SAMPLING

Determine the prescribed number of sampling points for doing the tests on a random basis. To reduce testing it is suggested that these points should coincide with some of the density points (see Subsection 7.1 below).

Take a sample at the pre-determined sampling point as soon as the stabilizer and compaction moisture have been mixed in and compaction is due to start. Sample the material at four to five points across the total width of the layer and to the full depth of the layer to obtain a composite sample of at least 80 kg.

3 PREPARATION

Prepare the sample as described in TMH13, Method A7, and mix it thoroughly.

Add enough water to the material to attain a moisture content of approximately 2 per cent below optimum. If the material is already over optimum, stir it a few times to lower the moisture content by evaporation but do not spread it out or allow it to dry out too much (see Subsection 7.3 below).

Quarter out nine portions of the material: five portions for determining the optimum moisture content and maximum density and three portions for doing the CBR or UCS compaction (see Subsection 7.2 below). Cover the material with wet hessian bags or put it in closed containers.

4 DETERMINATION OF THE OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY

Start compacting the Mod. AASHTO specimens as described in TMH13, Method A7. (Keep the quartered-out material covered by wet hessian bags or in closed containers.)

Plot the percentage (or mP) of water added to the soil against density to get a moisture/density relationship curve. Moisture content samples are also taken and dried to establish the proper relationship for density control.

5 DETERMINATION OF THE CBR OR UCS

As soon as the optimum added moisture content has been determined, add this amount of water to the CBR or UCS samples and compact and determine the CBR or UCS as described in TMH13, Methods A8, A9 and A14.

Note: The whole procedure should be completed in less than three hours.

6 FIELD DENSITY CONTROL

The maximum density determined as above is used for field density control. The field density should be determined as soon as possible after final compaction as hydration of the stabilized material may change the moisture content, and therefore the density, considerably. The field density should be determined at the same spot where the uncompacted laboratory sample was taken.

7 NOTES

- 7.1 Usually more density tests and therefore Mod. AASHTO compaction tests than strength control tests have to be done. Considerable effort is therefore saved if some of these points can coincide.
- 7.2 The CBR or UCS sample can be split out at a later stage to facilitate handling and to keep the three points at more or less the same moisture content.
- 7.3 If the material is too moist it will not be compactable in the field, as the field compaction moisture content is usually 1 to 2 per cent below Mod. AASHTO optimum because of the higher compaction energy used in the field.

ERRATA

(Corrected January 1986)

TRH14 GUIDELINES FOR ROAD CONSTRUCTION MATERIALS

Page 4, seventh-last line:

Delete (ST) and substitute the following:

(S1 and S2)

Page 5, Table 3, second heading:

Delete (ST), and in the next line delete (SC) and substitute the following:

(S1 and S2)

(S4)

Page 6, Table 4, second heading:

Delete ST and SC, and substitute the following:

S1, S2 and S4

Page 14, third paragraph:

Delete the existing third paragraph and substitute the following:

Materia9 G1, G2, G3, G4 should comply with the following grading envelopes, and in the case of G2 and G3 at least 50 per cent by mass of the individual fractions in excess of 4.75 mm shall have at least one fractured face.

Page 43

The second-last sentence should read:

Attention is drawn to the fixed value given in this table for the wet test instead of a minimum permissible percentage strength of the wet aggregate relative to the dry one.

Page 44

Delete the existing page 44 and substitute the attached new page.

Page 45, second-last paragraph:

The last sentence should read:

Such a slightly greater crushing strength would ava9o make provision for the acceptable wet:dry strength ratio of 65 per cent as suggested by Netterberg¹⁰ because of the self-cementing properties of calcrete.

Page 46

The second sentence should read:

The 10 per cent FACT is given in kN for the required minimum strength of the dry material and the minimum permissible percentage strength of the wet sample in relation to that of the dry one.