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DEPARTMENT OF TRANSPORT

Appropriate use of locally available materials in concrete, bituminous surfacings and layerworks for roads in rural areas

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SINOPSIS:

Die gebruik van plaaslike materiale in lae volume paaie kan groot kostebesparings meebring, maar ook 'n hoër risiko van vervroegde verswakking of swigting. 'n Kompromis moet dus gevind word tussen moontlike vervroegde verswakking of swigting (bv verhoogde onderhoudskoste) en die koste om die strukturele vermoë te verbeter. Alhoewel die gebruik van beton in lae volume paaie beperk is, sal die gebruik van lae kwaliteit aggregaat die sterkte en duursaamheid verlaag en moontlik ook krimping veroorsaak. Bitumineuse deklae is tans onderhewig aan streng spesifikasies en vergelykings met ander lande toon dat sekere verslappings moontlik is, veral met betrekking tot die gradering en hardheid van aggregate. Dit word voorsien dat die gebruik van gewysigde bitumens met marginale aggregate 'n laer lewens siklus koste vir deklae sal meebring. 'n Breedvoerige ontleding van die strukturele gedrag van plaveisellae toon dat die huidige tegnieke ernstige tekortkominge het. Dit is gevind dat dun plaveisel strukture, waar maksimaal van in situ materiale gebruik gemaak word, 'n bevredigende werkverrigting gee indien konstruksie goed beheer word en daar voldoende dreinering is.

SYNOPSIS:

The use of local materials in low volume rural roads can result in significant cost savings. However, this is associated with an increased risk of premature distress or failure. A compromise needs to be found between the probability of premature distress or failure (ie increased maintenance costs) and the cost of improving the structural capacity. Although the use of concrete in low volume roads is very limited, the effect of reducing the aggregate quality will be to decrease the strength and durability of the concrete and probably to increase the shrinkage. Bituminous surfacings are currently tightly specified and comparison with other countries indicates that certain areas of relaxation are possible. These are mostly with respect to the grading and aggregate hardness. It is anticipated that the use of modified bitumens with marginal aggregates will result in lower life-cycle costs for surfacings. Detailed analysis of the structural behaviour of pavement layers has shown that current design techniques have some major deficiencies. It has been clearly identified that thin pavement structures which make the maximum use of the in situ and local materials can provide satisfactory performance provided the drainage and construction quality is well controlled.

KEYWORDS Low volume roads, specifications, marginal materials.
TREFWOORDE Lae volume paaie, spesifikasies, marginale materiale.

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DISCLAIMER

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REVIEW STATEMENT

This report has been reviewed by:

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LIST OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

The divergence between the escalating cost of road construction according to traditional standards and the need for roads of the appropriate standard in rural areas (particularly developing rural areas) is increasing rapidly with time. It has thus become more important to make maximum use of local materials for road construction. Adjustments to existing standard specifications for the appropriate use of these locally available materials for rural roads need to be investigated and documented and should be more in line with an acceptable level of risk for the facilities. It should also be noted that the traffic distribution on rural roads in developing areas may differ significantly from that occurring on typical Category C rural roads as described in TRH4[1].

A project involving a full study of local and overseas literature concerning the use of local materials, the collection of existing information and an analysis of the findings has been carried out over a period of two years. The information covered appropriate components for concrete and bituminous materials, certain aggregate requirements and materials deeper in the pavement layers. Base materials were not included in this project as these have recently been investigated in another Department of Transport research project (91/201)[2].

Experience has shown[3] that many materials, subject to the appropriate climatic and traffic conditions can perform well, even though their properties differ significantly from traditional requirements[4]. These materials are often available locally, significantly reducing the processing and haulage requirements. However, very little of the experiences with local materials have been documented and minimal laboratory and field testing of roads constructed with local materials has been carried out.

In order to capitalise on the cost advantages of using local materials it is necessary to understand the areas of possible relaxation of specifications and the consequences of this relaxation in terms of the savings which could be realised and the increased risk. This report reviews the possible relaxations and makes recommendations on the use of local materials for surfacings, concrete and pavement layers beneath the base.

The roads which are appropriate to the discussions in this report are all classified as lightly trafficked and should generally carry less than 200 vehicles per day of which less than 10 per cent should be classified as heavy vehicles. These heavy vehicles should not exceed the equivalent of one standard 80 kN axle per vehicle. This would result in a cumulative equivalent axle count of about 35 000 axles/lane over a design life of 10 years and fall into the new E0-3 category[5,6].

2. OBJECTIVES OF THE PROJECT

The objectives of the project as defined in the proposal were as follows:

(i) Carry out a literature study and interviews to establish current practice for:
- concrete in pavements, drainage and other structures;
- bituminous materials and aggregates;
- layer work.

(ii) Carry out laboratory and field testing to evaluate potential areas for reducing standards and the effects on the performance of materials and relate to actual field performance.

(iii) Propose new guidelines and appropriate material specifications for local materials with specific reference to appropriateness in terms of climate, traffic and purpose of the road and cost effectiveness.

This research will culminate in guidelines for the specification of locally available materials for use in construction of rural roads.

The project was scheduled to run over a two year period, the first comprising mostly background work and identification of possible areas of material relaxation. The second half of the project involved theoretical, laboratory and field verification of the possible relaxations.

3. CONCRETE

3.1 BACKGROUND

The use of concrete in lightly trafficked rural roads typically consists of applications associated with drainage structures (bridges, drifts and causeways, drains, culverts, culvert inverts and wingwalls, erosion protection measures) and short lengths of concrete road (currently associated mainly with steep grades). Recently the use of roller-compacted concrete has been investigated as well and may be particularly cost-effective on steep grades and in areas where erodibility of soils is a problem. One other area of concrete usage is for the manufacture of blocks for the construction of block paved roads, particularly useful where labour-enhanced activity is viable. The use of cement in soil stabilisation is not addressed in this project.

Concrete is an expensive construction material and is typically limited as far as possible in rural roads but it should be noted that it is also a strong and durable material providing a long life if designed and prepared according to existing specifications. Concrete is a construction material the engineer can readily formulate, within limits, to meet specific individual job requirements of strength and durability. The rising cost of concrete aggregate and cement, together with the dwindling supplies of high quality aggregate has forced a reappraisal for roads in rural areas. Means of using lower quality materials such that they are as capable of withstanding traffic and weather influences as the current high quality aggregates, need to be found. In many cases the quality currently specified for concrete may not be appropriate for the use to which the concrete will be put. Lower quality materials may produce a concrete far more appropriate for the particular use with the concomitant cost advantages.

Many concrete applications make use of pre-cast sections with thin walls, eg culverts, and it can be assumed that this type of structure is unlikely to be made from sub-standard concrete.

Marginal quality aggregates (aggregates that do not meet existing specifications and which are often locally derived) have been used in the manufacture of dry lean concrete (DLC), roller-compacted concrete (RCC) and *Econocrete*. RCC and DLC have been used mostly for mass construction, viz. concrete gravity dams but have also been used for the sub-base and base (RCC) of pavement structures. DLC has, however, been defined as a concrete of extremely low workability and has only a modest strength[7].

Econocrete pavements are Portland cement concrete structures that may be made with low-cost, locally available aggregates or recycled materials that do not meet conventional specifications, and a relatively low cement content[8]. In areas where high-quality aggregates are in short supply, substantial quantities of substandard local aggregates are often available and can be used[9].

Some sources of marginal aggregates are bank-run-gravel, crusher run, shale, sandstone, conglomerate, quartzite, limestone, breccia and scoria. It must be noted that concrete made with low grade aggregates will inherently be of inferior quality[7].

Implementation of the Reconstruction and Development Programme (RDP) will inevitably involve the construction of significant concrete structures associated with the provision of the transportation infrastructure in many remote parts of South Africa. Many of the minor structures will be constructed far away from commercial rock crushers which would mean that the required aggregate would need to be imported at high cost. In effect this reduces the development which can be carried out with the available investment funds. Apart from the hauling in of aggregate, two other alternatives exist for the sourcing of concrete aggregate:

(i) Installation of a mobile crushing plant in the vicinity. This is generally not economically viable for limited volumes of aggregate for small projects;
(ii) The use of local materials such as natural deposits or river gravels;
(iii) Hand crushing.

3.2 EXISTING SPECIFICATIONS AND REQUIREMENTS

The standard requirements for concrete are tight with strict specifications pertaining to the cement, the aggregate and sand as well as the hydration water. The current specifications for concrete in roads and structures in South Africa mostly follow those issued by the South African Bureau of Standards[10]. The CSRA[11] requirements which are apparently used by most of the Provincial Road Authorities refer to the SABS criteria and these are used as the basis for this project as they broadly represent the recommendations of all the major road authorities in South Africa. The TRH14[4] specifications are also discussed. In addition to the standard specifications described below and which are typically applied, numerous others exist. A good example is the discussion in Fultons' Concrete Technology[12] on all the aspects described below, each aspect comprising a full chapter.

3.2.1 Aggregates

The CSRA[11] specification for structural concrete requires that both coarse aggregate (stone) and fine aggregate (sand) shall comply with the requirements of SABS 1083[10], subject to the following:

(i) The shrinkage of both the fine and coarse aggregate when tested in accordance with SABS Method 836 shall not exceed the following limits:
   (a) For use in prestressed concrete, concrete bridge decks and slender columns the shrinkage of both fine and coarse aggregate shall not exceed 130 per cent of that of the reference aggregate.

(b) For use in other reinforced concrete members the shrinkage of the fine aggregate shall not exceed 175 per cent and of the coarse aggregate 150 per cent of that of the reference aggregate.

(c) For use in mass concrete substructures and unreinforced concrete head walls and wing walls, the shrinkage of both the fine and coarse aggregate shall not exceed 200 per cent of that of the reference aggregate.

Where there is any doubt about the shrinkage characteristics of aggregates, the contractor shall submit a certificate by an approved laboratory which gives the shrinkage characteristics of the aggregate.

(ii) The flakiness index of the stone as determined by TMH1 Method B3[13] shall not exceed 35.

(iii) Aggregates shall not contain any deleterious amounts of organic materials such as grass, timber or similar materials.

(iv) Aggregates which may be potentially alkali reactive shall be assessed in accordance with subsection C-15 of SABS 1083, and if there is any danger of alkali-aggregate reaction, its use shall be subject to approval by the engineer.

Aggregates for concrete pavements[11] shall comply with the requirement of SABS 1083, but subject to the following:

(i) The drying shrinkage of concrete samples made from each of the required three concrete mixtures for preparing the compressive-strength and flexural-strength samples in accordance with clause 7104(b) shall not exceed 0,040 per cent. Drying-shrinkage tests shall be conducted in accordance with SABS Method 1085.

Where the drying shrinkage exceeds the specified maximum value, either alternative aggregate shall be used, or further investigations shall be made, or evidence shall be produced with a view to confirming the suitability of the aggregates proposed for use. The historical behaviour of the aggregate in concrete may serve as a recommendation in such cases.

(ii) Coarse aggregate shall comply with the 10% FACT values specified for aggregate used in concrete subject to abrasion.

In addition, the aggregates shall comply with the following requirements:

(i) The fine aggregate shall be either a natural or crusher-produced sand or a blend of natural and crusher sands where a mixture is used, and the quantity passing through each of the 0,150 mm and 0,075 mm sieves shall not exceed the value obtained for the mix by reference to the permissible limits for each sieve size.

(ii) The acid insolubility of the sand or the mixes of sand types as determined by the test described in subclause 8105(b) shall exceed 40 per cent.

(iii) The fineness modulus of the fine aggregate (or mixtures thereof) shall not deviate from the approved fine aggregate (or mixtures thereof) by more than 0,40.

(iv) Unless otherwise permitted by the engineer, the water demand for a concrete mix containing the sand or sand mix proposed for the concrete mix shall not exceed 220 ℓ/m^3 at a slump of 75 mm. The testing procedure shall be in accordance with the provisions of subclause 8105(e).

(v) Aggregates shall not contain any deleterious quantities of organic material, particularly pieces of timber, which will cause unacceptable surface defects when they float to the top of fresh concrete during vibration.

(vi) The nominal sizes of coarse aggregate shall be as follows:

Slab thickness of 175 mm and over:

37,5 mm, plus one or more of the following:

19,0 mm, 13,2 mm and 9,5 mm.

Slab thickness of 150 mm and over but less than 175 mm:

26,5 mm *

Slab thickness of 100 mm and over but less than 150 mm:

19,0 mm *

* Where recommended by an approved laboratory, smaller sizes of stone (nominal 26,5 mm and 19,0 mm sizes) shall be provided.

(vii) The flakiness index of the coarse aggregate, as determined by the TMH1 Method B3T, shall not exceed 35.

(viii) Where there is any danger of a particular combination of aggregate and cement giving rise to a harmful alkali-aggregate reaction, the particular combination shall be tested in accordance with the testing method as described in subclause 8105(f), and, where the result points to such reaction, either the aggregate or the cement or both shall be replaced so that an acceptable combination may be obtained.

According to TRH14[4] single-sized coarse aggregate to be used in concrete pavements should comply with the following:

Grading

The recommended grading analyses are shown in Table 3.1.

Table 3.1 : Grading of coarse aggregate (concrete pavements)[4]							
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.

Crushing strength

The minimum 10% FACT value should be 110 kN or the maximum Aggregate Crushing Value should not exceed 29 per cent.

In certain cases it may also be desirable to do the 10% 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.

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.

Other coarse aggregate characteristics

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

Shape - As indicated by voids content, SABS Method 845.

Water absorbtion - SABS Method 843.

Content of material of low density - SABS Method 837.

Abrasion resistance - SABS Method 846. 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

The main requirements recommended by TRH14[4] for sand to be used in concrete pavements are as follows:

Grading

The recommended particle size distribution is shown in Table 3.2.

Table 3.2 : Grading of fine aggregate (concrete pavements)[4]

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

Note: a. 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.

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.

Fineness Modulus

The Fineness Modulus of the fine aggregate (determined by Method B13 of TMH1[13]) should lie between 1,6 and 3,5.

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 830.

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.

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.

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 837. 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 812 - sedimentation method.

Content of material of low density - SABS Method 837.

Shell content - SABS Method 840.

Cement-aggregate reaction - Aggregates containing certain minerals, especially certain types of silica, react adversely with high-alkali cements. Aggregates and cements suspected of adverse reaction should be laboratory checked before use.

3.2.2 Cement

Cement used for structural concrete shall be any of the following[11]:

(i) Portland cement or rapid-hardening Portland cement complying with the requirements of SABS 471.

(ii) Portland blast-furnace cement complying with the requirements of SABS 626.

(iii) Portland cement 15 or rapid-hardening Portland cement 15 which complies with the requirements of SABS 831.

(iv) A 50/50 mixture of Portland cement and milled granulated blast-furnace slag which complies with all the requirements of SABS 626, except for composition. Granulated blast-furnace slag shall be milled to a specific area of not less than 3 500 cm^2/g. The Portland cement and milled granulated blast-furnace slag may be mixed in the concrete mixer together with the other ingredients of the concrete.

In prestressed concrete members or units the use of Portland blast-furnace cement will not be permitted and the use of a 50/50 mixture of Portland cement and milled granulated blast-furnace slag may be used only if authorised in the project specifications or by the engineer, in writing.

Cement used for concrete for pavements shall be any one of the following[11]:

- Ordinary Portland cement in accordance with SABS 471.
- Ordinary Portland cement 15 SL in accordance with SABS 831.
- Ordinary Portland cement 15 FA in accordance with SABS 831.
- A 50/50 mix of Portland cement and milled granulated blast-furnace slag which complies with all the requirements of SABS 626, except for its composition, may be used if approved in writing by the engineer. Granulated blast-furnace slag shall be milled to a specific area of at least 3 500 cm^2/g. The Portland cement and the milled granulated blast-furnace slag may be mixed in the concrete mixer with the other concrete components.

TRH14[4] recommends that ordinary Portland cement (OPC) (complying with SABS Specification 471) 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.

3.2.3 Water

Water for both structural and pavement concrete shall be clean and free from detrimental concentrations of acids, alkalis, salts, sugar and other organic or chemical substances[11]. If the water used is not obtained from a public drinking water main, the engineer may require the contractor to have the suitability of the water proved by way of tests conducted by an approved laboratory as often as may be deemed expedient by him.

The water used for concrete[4] 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. TRH14 goes somewhat further by recommending that if required, the suitability of the water must be tested by an approved laboratory with respect of the following properties:

- pH value
- conductivity
- total dissolved solids
- 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.

3.2.4 Admixtures to concrete

Admixtures shall not be used in structural concrete without the approval of the engineer, who may require that tests be conducted before the admixtures are used to prove their suitability[11].

Admixtures, if their use is allowed, shall comply with the following:

(i) Admixtures shall be used only in liquid form and shall be batched in solution in the mixing water by mechanical batcher capable of dispensing the admixture in quantities accurate to within 5 per cent of the required quantity.

(ii) All admixtures shall comply with the requirements of ASTM C 494 or AASHTO M-194 and shall be of an approved brand and type.

(iii) Air entraining agents shall comply with the requirements of ASTM C 260 or AASHTO M-154.

(iv) Admixtures shall not contain any chlorides.

Any admixtures used for concrete pavements[11] shall comply with the requirements of ASTM C 494 (or AASHTO M-194) for chemical admixtures, or ASTM C 260 (or AASHTO M-154) for air-entraining admixtures, and shall be of an approved type and brand. In addition, admixtures shall be subject to such tests as may be prescribed by the engineer, carried out at an approved testing laboratory at the contractor's expense to determine the effect of the admixture on the concrete mix. Admixtures containing calcium chloride shall not be used.

TRH14[4] highly recommends the use of an air-entraining admixture in concrete for pavements 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 admixtures 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.

TRH14[4] specifies that admixtures should be used as in 3.2.4 (i) with the addendum that they should be mixed 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 C 260.

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 C 494, but should not be used without the express approval of the engineer.

3.2.5 Strength requirements for concrete

The minimum 28 day flexural strength should be 3,8 MPa[4]. The required corresponding 28 day compressive strength, meeting the specified flexural strength requirement, should then be determined as follows:

fc = Corresponding compressive strength for 28 day flexural strength of 3,8 MPa

fcw = Working (required) compressive strength for 28 day flexural strength of 3,8 MPa
= 0,85 fc

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 regulations[14] and all cube compression strength tests should be carried out in accordance with TMH1[13].

3.2.6 Econocrete

Specifications have been proposed specifically for *Econocrete*[8] and these are summarised below:

Aggregate

The aggregate may be locally available stone or gravel, crushed or uncrushed. The aggregate shall not contain an excess of flat, elongated pieces or other objectionable matter. The aggregate may contain crushed portland cement concrete, crushed bituminous concrete (termed asphalt in South Africa), or base course materials.

The aggregate shall not have an abrasion loss of more than 55 per cent at 500 revolutions as determined by AASHTO T-96 or ASTM C 131.

Any of the following gradations may be used (Table 3.3):

Table 3.3 : Aggregate gradations for concrete stone

Sieve designation (mm)	Percentage by weight passing sieves (%)		
	A	B	C
50,0	100		
37,5		100	
25,0	55-85	70-95	100
19,0	50-80	55-85	70-100
4,75	30-60	30-60	35-65
0,425	10-30	10-30	15-30
0,075	0-15	0-15	0-15

The gradations may be modified to meet local aggregate gradations if suitable mixtures can be produced.

Portland Cement

Portland cement shall comply with the latest specifications for the type specified (ASTM C 150, CSA Standard A5-M, or AASHTO M-85), or for blended hydraulic cements (ASTM C 595 or AASHTO M-240) excluding slag cement, Types S and SA.

Water

Water used in mixing shall be as clean and as free as possible of oil, salt, acid, alkali, sugar, vegetable, or other substances injurious to the finished product. Water will be tested in accordance with and shall meet the suggested requirement of AASHTO T-26. Water known to be of potable quality may be used without testing.

Admixtures

The contractor shall submit certificates indicating that the material meets all of the requirements indicated below.

Air-entraining admixtures shall meet the requirements of AASHTO M-154, ASTM C 260, or CSA A266.1.

Water-reducing, set-controlling admixtures shall meet the requirements of ASTM C 494; AASHTO Type D, water-reducing and retarding; or CSA A266.2. Water-reducing admixtures shall be added at the mixer separately from air-entraining admixtures in accordance with the manufacturer's printed instructions. Water-reducing and set-retarding admixtures shall be compatible with the admixture used for air-entraining.

3.3 CURRENT APPLICATION OF LOCAL MATERIALS

The quality necessary for concrete will depend on the specific application of that concrete. Structural and reinforced concrete should be mixed and controlled to the high standards typically specified for this material. Mass concrete gravity retaining structures and wing walls can afford some relaxation of specification and allowance for marginal materials but aspects such as excessive shrinkage should be controlled.

Pre-cast thin concrete culverts should be constructed with high quality concrete but it is often possible to use local labour to cast in situ concrete for the construction of drainage structures, mostly culverts. These structures will typically require significantly more concrete but by using local materials and labour, significant savings could possibly be made.

One of the largest potential uses for concrete containing marginal materials is for pavement structures and the bulk of the discussion in this report involves this aspect. However, much of the discussion is relevant to concrete for alternative uses, mostly associated with low cost drainage structures. It is still important to evaluate the life-cycle cost of any proposed concrete pavement and compare it with the alternative designs appropriate for that road.

3.3.1 Econocrete

Econocrete is a name that has been given to a portland cement concrete that may be made with a relatively low cement content and with low-cost aggregates or recycled materials that do not necessarily meet standards for normal concrete aggregates. In areas where high-quality aggregates are in short supply, substantial quantities of substandard local aggregates are often available. The bulk of the published literature on the use of marginal materials in concrete involves its use in econocrete pavements.

According to Meininger[15] the idea of econocrete is to design a concrete pavement system which will minimise costs by using local material for those layers of the pavement construction where it will perform satisfactorily, bringing in imported aggregates only for that part of the work which requires properties not attainable using local materials.

The general definition of econocrete[15] is that it is a portland cement concrete designed for a special application and environment and, in general, making use of local commercially produced aggregate. Often these aggregates would not meet conventional quality standards for concrete aggregate. Econocrete is not necessarily lean (low cement factor) concrete. The definition is a broad one which is meant to encourage innovation and flexibility in the design process to consider various alternative approaches with the ultimate objectives of lowering per kilometre paving costs and to allow a greater amount of work to be done with the available funds. However, performance of new systems must always be the primary criteria. *If there is not reasonable assurance of long service for a particular pavement system, choice of materials or new mix-design, it is certainly not worth effecting a small saving now in trade for perpetual maintenance headaches later.*

Econocrete is a durable concrete with a well-distributed air void system[9]. Water reducing admixtures are often included in the mix to improve workability at the low slump ranges that result in higher strengths.

Econocrete mixes are based on laboratory tests and are designed for specific strength and durability levels in accordance with their intended use.

The biggest use of econocrete is in subbases where recycled materials are used as aggregate in the econocrete. One of the reasons for selecting an econocrete subbase is to provide an erosion-resistant subbase surface.

Econocrete has been used as a lean concrete base or subbase under a conventional pavement or base courses under asphalt surface. In these cases a cement content of 120 to 210 kg/m^3 has been used to provide a compressive strength of 500 to 2000 psi (3,5 to 13,8 MPa). The layer thickness is 100 to 150 mm. These layers were made with a typical base course type aggregate with fine and coarse sizes mixed in one long gradation and were considered superior in performance to a cement treated base.

Econocrete is often used in composite concrete pavements where the econocrete is overlaid with a thin (75 to 100 mm) traditional concrete wearing course.

Econocrete shoulders have been constructed with new concrete pavements or rebuilt shoulders adjacent to existing concrete pavements. The requirements for the shoulder concrete are not as demanding as for the main roadway pavement. Normally, shoulders carry very little traffic, and the abrasion resistance, strength requirements, and aggregate quality requirements are lower[16].

Low traffic street paving and parking areas with low cement and non-standard aggregate gradations have been constructed in one-layer construction, ie without a wearing course. Because of the low abrasion resistance, however, this is one application for which econocrete is not particularly suitable[15].

Other econocrete uses involve the use of low strength mixtures as fill or back fill, bedding, embankments, etc. where a stable non-settling material is needed in a remote area and where the use of a concrete type mixture may be more practical than the placing of soil or aggregate mixtures and compacting them to a high percentage of maximum density.

Aggregate requirements and mix design

Some of the restrictive specification requirements for concrete aggregates relate to the performance characteristics of the exposed pavement surface, where substandard aggregates may cause undesirable surface conditions, such as lack of abrasion resistance, slippery pavements or pop-outs[16]. Many substandard or marginal aggregates that do not meet normal specifications may be acceptable when used in econocrete as a lower course in the pavement structure subject to economic viability.

Aggregate gradation requirements for econocrete are also not as strict as those for normal concrete. In many cases the regular gradation of aggregate from the crushing plant, (ie crusher-run) is satisfactory without the addition of sand.

Some specifications designate only the maximum size of aggregate and the amount that passes a 75 μm sieve. It is noted that gradation specifications should be modified to meet local aggregate gradations if suitable econocrete mixtures can be produced. A specific gradation requirement for the project can be written to control the variability of the aggregate.

Data obtained from laboratory test programmes and econocrete construction projects indicate that a wide range of aggregates may be used. Some of these aggregates are materials not processed to the degree that normal aggregates are. Most have more fine material passing the 150 and 75 μm sieves than is acceptable for normal concrete, but this is not necessarily objectionable for econocrete as the extra fines can improve workability for mixes that have low cement contents.

On several recycling projects in the United States, old concrete and asphalt pavements have been crushed and used as aggregates for econocrete.

Meininger[15] gives a number of gradations used for econocrete. All of the materials had a maximum size of 37,5 mm with between 1 and 15 per cent passing the 75 μm sieve. Unlike the bimodal aggregate content (coarse and fine) specified in TRH14[4] the materials were well graded with coefficients of uniformity mostly in excess of 60. The long base course type gradings are being used in many instances. In general, these are good quality aggregates but are not processed to the same degree as normal aggregates. They are frequently used in a long grading, with the permissible amount passing the 75 μm sieve increased. In a lean concrete the extra fines actually help in supplying needed workability. Typical cement contents and compressive strengths obtained with these mixtures varied between 154 and 251 kg/m^3 and 4,3 and 16,6 MPa respectively.

The normal procedures and tests for concrete are followed for the proportioning of econocrete mixtures with the following exceptions:

- a single aggregate is sometimes used rather than a combination of coarse aggregate and fine aggregate; and
- the cement content is usually (but not necessarily) less than that for normal concrete.

A primary requirement is that the econocrete be workable, ie it is easy to mix and place, capable of adequate consolidation by vibration, and cohesive enough to resist excessive edge slumping when placed with a slip-form paver. The second requirement is that the hardened concrete has the level of strength and durability appropriate for the exposure conditions.

Workability of concrete depends primarily on the aggregate characteristics, air content, the cement content and the cement:water ratio. Since the cement content is low in lean concrete (which could cause poor workability for normal aggregates), the workability may be enhanced by:

- the existence of extra fines in the aggregate;
- higher than normal amounts of entrained air;
- addition of fly-ash, water-reducing admixtures, or workability agents; or
- a combination of these.

Ruth and Larsen[17] have developed procedures and models to predict the composition of econocretes and to give good estimates of the strength and modulus parameters for econocrete mixtures prepared with a variety of low to high quality Florida aggregates. The procedure requires only aggregate test values and cement content to predict the mix components and strength values.

An alternative procedure[17] uses the properties of the fresh concrete trial mix for estimating econocrete strength. Predicted strength values and estimated costs can be used to eliminate all but the most cost-effective aggregate sources.

According to Meininger[15] the Federal Highway Administration has classified econocrete into non-experimental and experimental categories. The non-experimental uses include:

(i) Lean concrete bases (constructed with a slipform paver)

- replacement for Cement Treated Bases (CTB) using air-entrainment and higher cement contents than CTB;
- the use of lower quality aggregates is encouraged;
- the use of recycled aggregates is encouraged;
- road surface abrasion resistance;
- no joints in lean concrete base;
- no bond between pavement and base.

(ii) Two-course monolithic pavement

- only with the use of high strength-high quality concrete in both courses;
- good quality but lower cost aggregates in lower course;
- aggregates should not be ones that will develop problems with freeze-thaw, D-cracking, alkali-aggregate expansion, etc.

The only practical reason for constructing a two-course pavement with two layers of high strength concrete is for skid resistance.

(iii) Concrete shoulders

- the concrete should have good freeze-thaw resistance.

In the Federal Highways experimental category, two-course composite monolithic pavements (lower strength in lower course), econocrete surfaces with asphaltic mixtures and fly-ash use are included.

Meininger[15] does not see the econocrete concept causing any great increase in the use of aggregates from roadside pits. This is a result of the potential problems from both an environmental and public relations aspect (in the United States). (Different considerations will affect the decisions in developing and rural areas in South Africa.) Commercial sources will have to be used in most instances. Deposits of marginal quality materials should be evaluated for possible use in specially designed econocrete mixtures for lean concrete bases or the lower course of a two-course pavement.

Another potential opportunity is the use of gradation of aggregate designed to reduce waste from an operation. For example, if the sand to gravel ratio is high, concrete might be made with higher percentages of sand. If pea gravel or course sand sizes are abundant, mixes might be used with a grading using extra quantities in these sizes. On the other hand, if there is another market for certain size fractions, econocrete might be proposed with a gap graded material. (This could be particularly useful if surfacing chippings are produced from a nearby quarry).

Weaker concrete means thicker pavements. The widespread use of lean, low cement-factor concrete for the top pavement surface was not considered to be feasible or economical[15]. Lean concretes can be used in underlying layers very satisfactorily if the mixture and the pavement are properly designed.

In summary the use of econocrete for lightly paved roads in rural areas in South Africa is unlikely to be cost-effective except on steep slopes, areas particularly prone to erosion and which consequently have high maintenance costs or specific localised applications. In sandy areas, it may be possible to make an econocrete from the local material for very light traffic.

Many of the references evaluate econocrete as a substitute for cement-treated bases but this is considered not to be an economically viable alternative for most lightly trafficked roads.

3.3.2 Rollcrete

Rollcrete consists of a zero-slump cement concrete mix which is spread with an asphalt-type paver, compacted with vibratory steel-wheeled rollers and finished with rubber-tyred rollers[16]. The concrete mix consists of an ordinary portland cement/pulverised fuel ash (PFA) mixture and crushed aggregate.

The earliest rollcrete pavements were constructed in Spain in the early 1970's[19] for roads carrying light traffic. These were very successful and since the 1980's its use has spread through Europe, the USA and Australasia[18]. Its use in South Africa was confined to dam construction until 1989 when research was initiated and trial sections of rollcrete were constructed. This investigation of rollcrete was specifically oriented towards low volume roads in developing areas[20].

It is generally recommended that crushed aggregate is more suitable for rollcrete than natural aggregate[18]. The Spanish rollcrete specifications stipulate that at least 66 per cent of the aggregate should be crushed stone. However, an investigation of rollcrete roads with marginal aggregates consisting of weathered shale, metamorphosed shale, greywacke, dune sand and fine sand containing a large percentage of silt[21] showed that these can be used for rollcrete of adequate strength for certain pavement structures.

Fairly tight specifications are given for the aggregate particle size distributions for both Spanish and USA (Corps of Engineers) rollcretes[18]. It is important that the quantity of fines is carefully controlled as an excess will cause surface depressions during rolling[19]. Opinions differ regarding the permissible plasticity: one author states that the materials should be non-plastic[22] whilst another suggests that a significant amount of plastic fines can improve the surface finish[23].

As rollcrete is, by definition, a zero-slump concrete[18], its most important parameter is the moisture content. This typically varies between four and seven per cent and depends to a large degree on the aggregate quality. The optimum moisture content is determined on the basis of the maximum density which involves laboratory testing of mixes with varying moisture contents. The moisture at which the maximum density is obtained is taken as the construction moisture content. It is extremely important that this is very closely controlled as the strength is very sensitive to the moisture content (ie cement:water ratio). A variation of one per cent in moisture can result in a 20 per cent decrease in the concrete strength[24].

Total cementitious binder contents vary between 10 and 14 per cent depending on the application. If a lower course is used, the cement content in this layer can be reduced to 7 or 8 per cent. However, between 20 and 40 per cent of the cement can be replaced with pulverised fuel ash (PFA). The compressive strength of rollcrete typically varies between 20 and 40 MPa, although the flexural strength has been found to be greater than that of conventional concrete, mostly as a result of the good compaction.

Marginal materials which would not normally be used for conventional concrete can be used for rollcrete because of the compaction process. Should they be used for rollcrete, the resultant product may have significantly different properties which should be thoroughly tested before the design stages of the project[25].

From a review of the literature it is clear that rollcrete has certain potential for limited applications but the economics (ie the cost of 10 to 14 per cent cement) will probably make its use difficult to justify in most rural situations.

3.3.3 Dry lean concrete

Dry lean concrete[26], as its name implies, is a concrete mix with two main differences from normal structural concrete:

- It has an earth-moist consistency because it is made with a low moisture content - only about 6 per cent of the weight of the aggregate plus cement. The only way to compact concrete of this consistency is by rolling;
- It contains a small amount of cement - the aggregate:cement ratio is usually between 15:1 and 22:1 by weight (normal structural concrete is about 6:1 or richer). The cement content of lean concrete usually lies between 70 and 140 kg/m^3.

Dry lean concrete differs from rollcrete mostly in the significantly lower cement content and is used extensively as a roadbase under bituminous surfacings from motorways to housing estate roads and other paved areas. It has the advantage that it can be used by site traffic before the final surfacing is applied. It can also be used to advantage as a sub-base beneath concrete pavement slabs and concrete block paving.

The object of all specifications for lean concrete is to ensure that the complete roadbase will have the required in situ strength. One way of checking that the in situ strength is satisfactory would be to cut cores and test them in compression, but this would be expensive. Because it is known from experience that if the mix is right to begin with and if fully compacted in place, then the in situ strength will be satisfactory, all specifications rely on ensuring that the mix itself is right and that it has been properly compacted.

Cubes are made and tested to check the strength and ensure that the mix is satisfactory (a check on the mix proportions). Full compaction is ensured by careful rolling and control, checking if necessary by measuring the in situ density.

Most specifications for lean concrete are based on the requirements of the British Department of Transport Specification for Road and Bridge Works[27]. These requirements, which are primarily intended for major works, include a high rate of testing to check both that the mix is right and that full compaction has been obtained.

Ordinary washed concreting aggregates should be used with the fine and coarse aggregates stored and batched separately: this allows better control over the proportion of sand and also permits the sand content to be adjusted if required. The maximum size of aggregate may be either 40 mm or 20 mm, the latter being easier to handle on small jobs and less prone to segregation.

The proportion of sand will need to be between 30 and 40 per cent for 40 mm maximum aggregate size and between 35 and 45 per cent for 20 mm maximum aggregate size. Too little sand will tend to aggravate segregation during discharge and spreading, whereas too much sand will lead to corrugations in the compacted surface.

Specifications for housing estate roads and other minor paving jobs often required the aggregate:cement ratio to be between 15:1 and 22:1, the proportion of aggregate to cement being based on the saturated surface dry weight of the aggregate.

Ordinary Portland cement is invariably used. With normal concreting aggregates, the minimum cube strength requirements will usually be satisfied with a mix of 20:1 (a cement content of about 105 kg/m^3 of compacted concrete).

The right amount of water for lean concrete is that which will give full compaction under rolling and is best assessed at the time of rolling: too much water will cause the lean concrete to be picked up on the wheels of the roller, and too little will lead to inadequate compaction, a low in situ strength and an open-textured surface.

The water content will usually be about 6 per cent of the combined weight of the cement and the aggregate[26]. At this level, the mix will be "earth moist": tests for workability are neither applicable nor used for lean concrete.

Like ordinary concrete, and in fact more important because of the small amount of water in the mix, loss of moisture has to be prevented in order to stop the top of the lean concrete surface from becoming weak and friable. This is best and most conveniently done by the spray application of a bituminous or cut-back emulsion.

Alternatively, and only for small areas, curing can be done by covering the lean concrete with plastic sheeting, but it is important to ensure that it is properly held down along the edges to prevent it from being blown off: any joints in the sheeting should be lapped at least 300 mm. The sheeting should be kept in place for 7 days.

No traffic must be allowed on the lean concrete until at least 3 days after laying: between 3 and 7 days after construction only light vehicles such as cars and vans should be allowed. Although this is often a major inconvenience, it must be strictly observed.

Site trafficking of a lean concrete surface, even after 7 days, may cause some surface damage and wear which may well require making good before the surfacing is applied. In any case, the surface should be cleaned before the wearing course is put down.

Haque[28] made dry lean concrete with marginal aggregates such as shale, greywacke and dune sand. These were all mechanically weak and poorly graded materials and were investigated for use as sub-base and/or base of a pavement and mass concrete for a gravity dam. The rate of strength development in relation to the cement added was considerably less than that for dry lean concretes using conventional aggregate and a higher water content was necessary. The large amount of silt in the materials did not have any adverse effect but appeared to improve the workability and compactability of the DLC mixes[28].

Williams[29] states that lean concrete mixes involve the use of washed aggregate of concreting quality, a low cement content and a low water content. The material is laid without joints other than construction joints, compacted by vibrating rollers, cured, and in flexible pavements, surfaced with bituminous material. One of its advantages is that it provides an all-weather working platform for site traffic.

Lean dry concrete pavements have been laid successfully in many parts of the United Kingdom using local sources of processed aggregate and have been described as essentially a British method of construction[29]. One of the major disadvantages of DLC base courses is the reflection cracking, even through thick layers of asphalt.

Williams[29] attempted to achieve an optimum strength which retained favourable load spreading but eased the cracking problem. Initially, the criterion was a minimum 28 day strength of 6,9 MPa but this was reduced to a minimum 28 day strength of 5,2 MPa and a maximum of 13,8 MPa, the upper limit subsequently being increased to 20,7 MPa.

Although dry lean concrete has been used in schemes ranging from minor housing estates to motorways in Britain, the economics, like those of rollcrete, appear to make this type of material inappropriate for construction in rural or developing areas except for limited specific applications. It requires an equipment intensive construction process further reducing its usefulness in rural areas.

A number of experimental "zero-slump" concrete road sections using marginal materials were constructed and monitored by the US Army Corps of Engineers[30]. The materials involved were non-plastic poorly graded sands, gravelly sands and poorly graded gravels and a gravelly clayey sand with a plasticity index of 24. The rationale behind these experiments was that the construction of roads for military use according to conventional standards was becoming less cost-effective as the cost of labour, materials and equipment increased. It was concluded from these experiments that:

- The use of marginal materials in zero-slump Portland cement concrete is appropriate for pavements that are to be used as secondary roads, streets, parking lots, storage areas, or for those with a relatively short service-life;
- Highly plastic aggregate materials that are to be used in zero-slump concrete mix should be thoroughly processed prior to incorporating portland cement to ensure a uniform mixture;
- The sand:aggregate ratio of a marginal material to be used in a zero-slump concrete mixture should be 25 or more;
- Satisfactory placement of zero-slump concrete can be accomplished using conventional base course spreading and mixing or an asphalt paver; and
- Zero-slump concrete can be adequately compacted with heavy vibratory rollers (an 11 tonne tandem was used in the experiments).

A laboratory investigation into the use of marginal materials in dry lean concrete used a natural weathered shale gravel, a mixture of fresh metamorphosed shale (rippable in a borrow pit) and single-sized dune sand and a mixture of crushed greywacke and dune sand[28]. The cement to aggregate ratio was varied between 1:8 and 1:19 whilst the water cement ratio was varied between 0,5 and 1,5. This resulted in effective cement contents of between 5 and 12 per cent of the total mix and compressive strengths of between 4 and 18,5 MPa were developed.

Since the water content of DLC mixes is dictated by practical considerations, its strength can be studied purely in relation to the cement content rather than becoming involved with the water:cement ratio.

It was concluded that, from the laboratory examination of concrete made with all-in weathered shale, metamorphosed shale, and greywacke with dune sand as the fine aggregate that these mechanically weak and poorly-graded materials can produce a DLC with an adequate strength for applications such as concrete fill, gravity dams and a subbase and/or base of a rigid pavement.

The optimum water contents of the DLC mixes which used marginal aggregates ranged between 5,6 and 7,9 per cent of the total weight of the mix. These optima are higher than those reported for DLC made from conventional good quality aggregates.

The strength of water-cured DLC was about 46 per cent higher than that of air-cured DLC (exposed to an ambient humidity of between 40 and 70 per cent) for the mixes with high silt contents which were tested. Although there is a significant difference in the strength between the two different curing regimes, the difference is much higher for conventional concrete. It was thus concluded that water curing is not as critical for DLC as it is for conventional concrete.

3.3.4 Hyson cells

A new concept in low volume road building has been introduced by Hyson-cells[31]. The Hyson-cells product is manufactured from polyethylene sheeting to form a 3-dimensional honeycomb mat comprising connected, square, hollow, thin-walled cells. The cells are available in a variety of sizes and are from 50 to 2 000 mm deep. The mats may be trimmed or joined together in both directions, and may be laid on top of each other to form layered structures. Hyson-cell geogrids have wide application in civil engineering, one being an alternative to conventional concrete for roads.

One innovative application uses Hyson-cells with grouted stone. The single-sized 53 mm stone (with no fines) is placed dry in the cells and then compacted to with 2 or 3 passes of a vibrating drum roller. The voids in the compacted stone are then filled with a cement slurry using a grout pump or vibrated in with a vibrating roller. This pre-compaction technique has proved to yield significantly improved strengths compared to conventional use of similar aggregate:cement mix proportions. The cement grout only serves to lock the load-bearing aggregate in position with the stone taking a larger portion of the load than is normally the case[31]. It is critical for the grouted stone method that complete penetration of the stone layer contained in the cells by the grout is achieved. The addition of PFA ("fly ash") to the grout improves its flow characteristics significantly[32].

Hyson cells have been used with ashcrete[32] (a concrete made with waste ash instead of fine and coarse aggregate) in a similar process to that described above. The confinement of the ashcrete in the cells apparently facilitates its compaction allowing appropriate standards of densification to be achieved with materials that would otherwise be unsuitable.

3.3.5 Karoo basalts

The Lesotho Highlands Water Project will be requiring large volumes of concrete for the tunnels and dams. The only available materials in most of the area are dolerites and basalt, the latter predominating. These materials have in the past performed poorly in concrete and have thus been largely avoided. Core samples of amygdaloidal basalt, cores cut from existing concrete structures and cubes cast during construction of a weir in Lesotho were tested by the Division of Building Technology[33]. These indicated that although the basalt had high smectite clay contents and performed poorly in terms of certain recognised tests, they perform well in terms of the Brazilian wetting and drying test which is used to assess durability. High drying shrinkages were measured in the laboratory and yet only 54 to 73 per cent of this potential shrinkage had actually been recorded in the field.

3.4 MECHANISM AND ACTIONS

An understanding of the mechanism of strength development in concrete is basic to the successful use of the material. The hydration of a mixture of cement and coarse and fine aggregate results in an exothermic reaction. The constituents of the cement (mostly calcium silicates and aluminates) combine with the water to form various hydrated calcium silicates and aluminates and generating calcium hydroxide. The hydrated calcium aluminates normally harden rapidly but this reaction is slowed down and controlled by the addition of an appropriate quantity of gypsum to the cement during manufacture. The hydrated calcium silicates initially form a gel which hardens with time (at a rate considerably slower than that of the calcium aluminates)[12].

As the cement cures with time, the crystallisation of the cementing products around the fine and coarse aggregate results in a strong conglomerate of sand, stone and cementing agent. The strength of the concrete is primarily a function of the quantity of cement and the water:cement ratio, ie the cement paste. It has been shown that there is no relationship between the strength of the aggregate and the strength of concrete made from it[12]. For very low strength concrete (less than 15 MPa) the crushing value of the stone is of little importance[12].

The compressive strength of mature cement/water pastes typically varies between 80 and 170 MPa depending on the water:cement ratio, considerably higher than any concrete strengths specified in practice. The compressive strength of cubical or cylindrical specimens of typical good concrete aggregate varies between 70 and 400 MPa. It is thus clear that it is the strength of bonding of the cement paste to the aggregate and the porosity of the concrete (a function of compaction) which is the prime determinant of the strength of concrete cubes. In addition to this, aspects such as cracking through excessive thermal variations or shrinkage reduces the strength.

Addis[12] notes that recent research has shown that the aggregate needs to be studied in both its relation to the hardened cement paste and also in relation to the environment of the concrete during its service life. Factors other than the aggregate strength discussed above which should be considered are the elasticity, porosity, thermal expansion, volumetric change associated with moisture variation, particle shape and surface texture, relative density, thermal properties, resistance to abrasion, and soundness.

The grading of both the fine (<4,75 mm) and the coarse aggregates (>4,75 mm) is critical with respect to the workability and performance of the concrete[12]. The grading of the sand affects mostly the workability, cohesiveness and bleeding whilst the larger the maximum size of the aggregate is, the lower the water content required and hence the lower the cost of cement for a given strength.

It is clear that the specifications for concrete, irrespective of their origin, are all very similar. The main parameters specified which appear to be of prime importance are grading- and deleterious component-related.

There is very little specification with respect to the durability of concretes and it is suggested that should marginal materials be utilised to a greater extent, this aspect will require additional attention.

The thickness design of concrete pavements, like flexible pavements, is a function of the strength of the support layers, the traffic and the quality of the concrete[8,34]. Any relaxation in concrete quality would need to be made up in concrete thickness, although for low volume roads lower thicknesses are possible. It should, however, be noted that rigid pavements are more susceptible to overloading and care should be taken not to make the pavements too thin. Many examples have been noted in the urban situation where commercially constructed domestic driveways have failed prematurely. Observation of the concrete used has invariably shown an almost total lack of coarse aggregate and thicknesses of between 75 and 100 mm.

3.5 AREAS OF RELAXATION AND POSSIBLE CONSEQUENCES

This project does not deal with the effects of cement specification or variability and does not go into detail about admixtures or their effects on the concrete. The discussion is restricted to the quality of the coarse and fine aggregate and the implications of using local materials. A brief discussion on the use of local water is also included.

3.5.1 Coarse aggregate

The main influence which marginal materials will have on concrete properties relate to the strength, stiffness, shrinkage, durability and the water:cement ratio. It is these aspects which need to be evaluated during the second part of the project. Although aspects such as skid resistance, in terms of the polished stone value, are important (safety considerations should not be underestimated or neglected), it is considered that relaxations in certain areas are justifiable. This should be on the basis of the low traffic volumes, but allowing for poorer geometrics and increased hazard warnings.

The influence of marginal aggregates on the shrinkage of concrete is well publicised but some disagreement exists as to the actual mechanism. Aggregate stiffness is considered to be directly related to shrinkage with stiffer aggregates giving higher shrinkage[12]. However, other workers have concluded that for high quality dense aggregates the shrinkage is independent of aggregate stiffness. Therefore the effect of aggregate stiffness is only important when low-stiffness aggregates are involved. However, compressible low-stiffness aggregates are also those with a high surface area and these would be expected to give high shrinkages. It would thus appear that aggregate porosity as well as stiffness were factors in the investigations described.

The fact that aggregate for concrete roads which is distinctly angular, even somewhat flaky, will lead to higher flexural strengths[12] indicates that some relaxation of the properties is possible.

3.5.2 Fine aggregate

If local sources of fine aggregate for concrete pavement fail to meet new skid-resistance requirements but produce satisfactory concrete strength and durability, why import material from a hundred kilometres or more away to be incorporated into the full slab depth?[12] Only the pavement surface requires aggregate providing maximum skid resistance.

The fineness modulus recommended by SABS 1083 may be relaxed provided that suitable adjustments are made to the concrete mix proportions[12].

3.5.3 Water

Although this report is specific to the use of local materials, mixing water for concrete should not be neglected. It is generally required that the water used for the hydration of cement in concrete should be potable. Tests have shown that a sodium chloride concentration of 3 000 parts per million is about the limit of potability although it should be noted that rural residents may adapt to higher salinities over time. However, much plain concrete has apparently been successfully made even with seawater. It is recommended that any potential local water for use in concrete should be used in the manufacture of test cubes and the results compared with cubes made from potable water to identify deleterious effects.

3.5.4 General

It should be noted that relaxation of certain of the specified parameters will usually result in a change in the water: cement ratio which will typically result in the need for more cement to achieve the same strength. However, if the required strength is also reduced economies can be made. The economic implications of relaxation of standards will be addressed in detail.

SABS 1083[10] is commendable in that it is a practical document in which an attempt has been made to avoid unnecessary or restrictive requirements[12]. The specification permits the use of materials falling outside the suggested limits provided sufficient testing has been undertaken to show the suitability of a particular material for a specific purpose. To this end the standard provides additional tests which may be necessary for further evaluation. Addis[12] notes however, that with the ever increasing demand on, and depletion of, high-quality aggregates, some revision of the present standard may become necessary in the interests of economy to broaden some of the existing limits. Technological advances over the past few decades coupled with the experiences of satisfactory case histories can assist in providing guidelines to extending the present limits.

3.5.5 Laboratory testing

As there is little practical experience in designing concrete mixes using natural gravels, it was decided to use the Compactability software package[35] o determine the optimum moisture content (OMC) of natural gravels for optimal compaction (ie maximum dry density (MDD))(see examples of output in Appendix A). The OMC is dependent on the grading (ie particle size distribution), Atterberg Limits and Linear Shrinkage of the aggregate source. Because the strength of Ordinary Portland Cement (OPC) concrete is basically dependent on the cement:water ratio, it is very important to keep the water requirements of the concrete as low as practically possible to limit the cost of the added cement. However, the Optimum Moisture Content (OMC) for compaction purposes is normally a very dry mix because the material is normally compacted by heavy vibratory and pneumatic-tyred rollers. The starting point for the minimum moisture content was therefore taken as the Zero Air Voids Moisture Content (ZAVMC)(for MDD) which is slightly higher than OMC. With this predicted ZAVMC known from the Compactability Software for the properties of the natural gravel, the amount of cement required was determined according to the strength requirement of the concrete. As most of these structures will be relatively small (ie minor) and therefore only require normal concrete for reinforced structures, the general strength requirement of the concrete was taken as 20 MPa. A limited amount of work was done on one natural gravel looking at higher strengths namely 30 MPa and 40 MPa (see Table 3.6 for cement:water ratios for different strengths).

Starting with the dry natural gravel, the ZAVMC and the amount of cement required for the particular strength, mixes were manufactured. Concrete used in structures should normally be able to flow freely into place to ensure that it takes the required shape of the structural element. As expected, it was found that the ZAVMC (for the tightest packing ie maximum dry density (MDD)) was still too dry for this to happen. The moisture content of the samples was therefore increased in increments of 1 per cent at a time so as to determine the point where the material would flow relatively easily but would not be excessively wet. It would seem that this point is approximately 4 to 5 per cent above ZAVMC as determined by the Compactability software package.

The following materials were evaluated:

- Bultfontein HVS site subgrade material (ferricrete)
- PPC calcrete obtained from the PPC cement factory
- TPA2 quartzitic gravel used in the compactability study
- TPA1 norite gravel used in the compactability study
- NPAB decomposed dolerite gravel used in the compactability study

Additional information on these materials is provided in Appendix A.

To rapidly assess the quality of the mixes it was decided to make use of a rapid curing method whereby the samples were cured in a moist environment for 24 hours at 80°C. This rapid curing test was originally developed by final year students of the Department of Civil Engineering at the University of Pretoria under the mentorship of Dr C.J. Semmelink. The correlation between the 24 hour strength and the 28 day strength is very high ($r^2 = 0,98$) and the strength of the concrete after 24 hours is about 80 per cent of its 28 day strength.

For this rapid curing phase use was made of circular split moulds as these samples could be sealed in plastic oven bags after initial set and then be placed inside a billy can with free water available below the sample (see Figure 3.1). The billy can was sealed with a lid before the sample was placed in an soils oven for 24 hours at a temperature of 80°C. The sample was removed after 24 hours and cooled down in a water bath for 1 to 2 hours before the concrete cylinder was crushed. The height of the sample was normally about 100 mm and the diameter was 152,4 mm. Because the height/diameter ratio was less than 2, a strength reduction factor was initially used according to ASTM C 39-86. However, the equivalent cube crushing strength of concrete is approximately 1,2 times the core crushing strength of a concrete cylinder with height/diameter ratio of 2. Therefore the reduction factors were multiplied by 1,2. To reflect the 28 day crushing strength the crushing strength values obtained in this manner were further divided by 0,80, because the 24 hour rapid curing strength is approximately equal to 80 per cent of the 28 day strength (see Table 3.4).

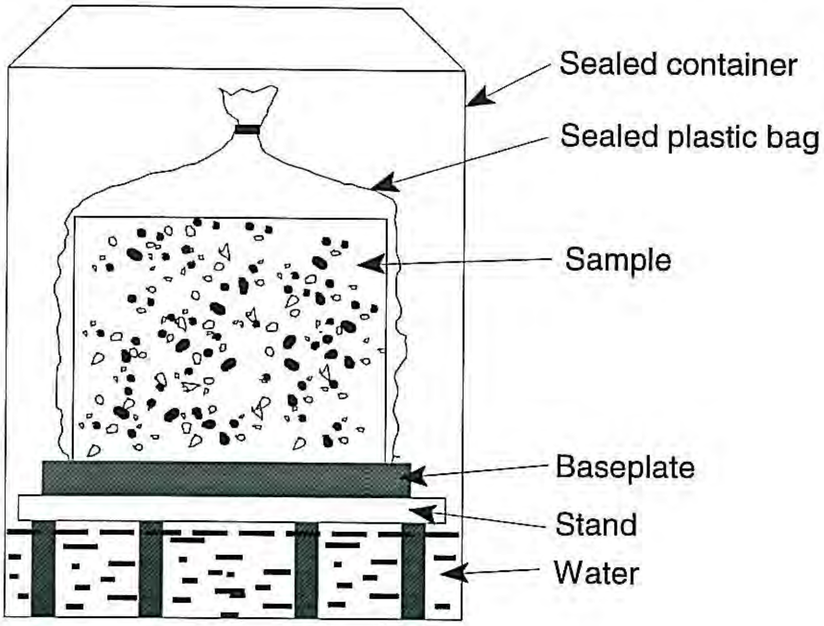

Figure 3.1 : Schematic view of samples in the rapid curing process

Subsequently it was noticed in the literature that there is no good correlation between the cylinder (or core) crushing strength and the cube crushing strength. For this reason a switch was made to cubes for the latter part of the work. Because the general strength requirement of concrete is the average strength of 3 cubes, sets of 3 cubes each of a number of natural gravel sources were evaluated (see Table 3.5). However, a comparison was made between the cylinder crushing strength (without the reduction factor) and the average 28 day curing cube crushing strength (see Table 3.5).

The initial aim had been to also evaluate some river gravels obtained from the previous homeland QwaQwa. Unfortunately the best quality river gravel deposit was still extremely fine and on the clayey side. In the light of our experience with the other natural gravel samples it was decided that this material was not suitable and therefore no work was done on this sample.

Table 3.4 : Equivalent 28 day crushing strengths of rapid cured cylinders

Design	%MC	h(mm)	RF[a]	Break (kN)	MPa[b]	c/w ratio	added mc(%)	theor c/w ratio[c]	Mix[d]
20	10	95,5	0,929	170	10,8	0,96	10	1,38	1
20	14	110,6	0,964	182	12,0	1,01	16	1,21	1
20	15	111,8	0,966	172	11,4	0,99	17	1,22	1
20	18	106,9	0,955	185	12,1	1,02	18	1,38	1
20	17	108,5	0,959	185	12,2	1,02	18	1,30	1
30	15	112,3	0,968	150	9,9	0,92	17	1,62	2
30	14	110,0	0,962	160	10,6	0,95	16	1,60	2
30	10	100,0	0,939	230	14,8	1,14	10	1,83	2
30	18	108,7	0,960	295	19,4	1,35	20	1,65	
30	17	108,3	0,959	240	15,8	1,18	20	1,56	
40	18	109,0	0,960	235	15,5	1,17	21	1,96	
40	17	108,5	0,959	250	16,4	1,21	21	1,85	
20	16	104,9	0,951	235	15,3	1,16	16	1,38	TPA2
20	15	103,9	0,949	220	14,3	1,12	15	1,38	TPA2
20	14	104,5	0,950	210	13,7	1,09	14	1,38	TPA2
20	16	113,5	0,970	70	4,7	0,68	16	1,38	TPA1
20	15	106,4	0,954	95	6,2	0,75	15	1,38	TPA1
20	14	102,8	0,946	80	5,2	0,70	14	1,38	TPA1
20	17	122,3	0,989	100	6,8	0,78	19	1,23	NPAB
20	16	116,3	0,976	105	7,0	0,79	19	1,16	NPAB
20	15	111,0	0,965	105	6,9	0,78	19	1,09	NPAB
20	20	100,0	0,940	205	13,2	1,07	20	1,38	PPC
20	19	101,3	0,943	195	12,6	1,04	19	1,38	PPC
20	18	100,4	0,941	250	16,1	1,20	18	1,38	PPC

Note:
a. Reduction factor of cylinder multiplied by 1,20 to get equivalent cube strength.
b. Equivalent 28 day crushing strength.
c. The theoretical c/w ratio was calculated and modified to account for the variation between %mc and added %mc.
d. Mixes 1 and 2 were obtained from the Bultfontein sample but differed to produce the various design strengths.

Table 3.5 : Crushing strengths of normal 28 day cured cubes and corrected accelerated cured cylinders for the different natural gravels.

Sample	Concrete cubes (28 days) 28 day		Corrected 24 hour (MPa)
	(kN)	(MPa)	
BULT	260	11,6	
BULT	280	12,4	
BULT	215	9,6	
	mean	11,1	11,7
TPA1	240	10,7	
TPA1	235	10,4	
TPA1	205	9,1	
	mean	10,1	5,4
TPA2	340	15,1	
TPA2	355	15,8	
TPA2	325	14,4	
	mean	15,1	14,4
NPAB	150	6,7	
NPAB	155	6,9	
NPAB	150	6,6	
	mean	6,7	6,9
PPC	305	13,6	
PPC	315	14,0	
PPC	310	13,8	
	mean	13,8	14,0

Table 3.6 : Cement:water (c/w) ratio for different crushing strengths (MPa)

MPa	c/w	MPa	c/w	MPa	c/w	MPa	c/w
1	0,51	11	0,97	21	1,42	31	1,88
2	0,56	12	1,01	22	1,47	32	1,92
3	0,60	13	1,06	23	1,51	33	1,97
4	0,65	14	1,10	24	1,56	34	2,01
5	0,69	15	1,15	25	1,61	35	2,06
6	0,74	16	1,20	26	1,65	36	2,11
7	0,79	17	1,24	27	1,70	37	2,15
8	0,83	18	1,29	28	1,74	38	2,20
9	0,88	19	1,33	29	1,79	39	2,24
10	0,92	20	1,38	30	1,83	40	2,29

3.6 CONCLUSIONS AND RECOMMENDATIONS

From the results of both the concrete cube and rapidly cured cylinders testing, it was concluded that there is no clear cut answer to the use of local materials for the manufacture of concrete. None of the natural gravels met the required crushing strength. However, a few came reasonably close, namely the quartzite gravel (TPA2) and the calcrete gravel (PPC), and one of the other materials gave a reasonable strength, namely the Bultfontein gravel, if the strength factor of 3 for normal concrete is taken into consideration. The reason for the low crushing strength of the rapidly cured TPA1 is not clear, but in general there was a very good comparison between the 24 hour estimates of the concrete cube crushing strength and the 28 day cube crushing strength.

On the other hand the strength of the decomposed dolerite (NPAB) was just about equal to the design strength (ie 20 divided by 3). All these mixes were designed in exactly the same manner. This shows that the addition of the correct amount of cement and controlling the water content will not necessarily ensure that concrete of a reasonable quality is manufactured each time. The cause of the problem is probably the amount of clay contained by the material as well as the amount of fine material in the mix.

When a material becomes too fine, a point will be reached where the percentage of cement required for the correct strength will become excessive because of the high moisture requirement. It would therefore make sense to lay down certain minimum requirements for the quality of the natural gravel to be used in making concrete from local gravel sources. These could include the percentage of the material which is allowed to pass the 4,75 mm sieve as well as a possible limit to the maximum values of the Atterberg limits of the fines. The bulk of the natural gravel should preferably consist of sound aggregate particles with a reasonable crushing strength. To produce these aggregates the use of small portable crushers (manually or power driven), such as used with Hyson cell projects, may be considered.

For the above-mentioned reasons it is not possible to make specific recommendations on the use of marginal material for concrete other than to note that their use may be possible. However, each potential source should be investigated using appropriate laboratory tests to indicate whether its use is economically viable. In many cases the cement content at a suitable cement:water ratio required to produce an acceptable strength will be uneconomic. Should a laboratory test programme show that it is possible to obtain the required strength using the material with an economically viable cement content, aspects such as workability and durability will need to be further investigated. Durability is enhanced by constructing a dense, impermeable concrete but excessive deleterious materials will have an adverse effect on the durability.

4. BITUMINOUS SURFACINGS

4.1 BACKGROUND

Bituminous surfacings are applied to roads in order to bind the surface particles together and to resist abrasion by traffic as well as to waterproof and protect the pavement as far as possible. One of the major advantages of bituminous surfacing is that all-weather passability of the road is maintained. In addition, the routine maintenance requirement associated with unpaved roads is significantly reduced. The bituminous surfacings mostly applied to rural roads are thin sand or chip seals although recent research has shown that, depending on the prevailing milieu, asphalt surfacing may be more cost effective[36]. Other surfacing types may also be used.

The performance of bituminous surfaced low volume roads can not be isolated from the subgrade or other layers in the pavement. Netterberg and Paige-Green[3] state that "the whole pavement and its environment should be considered and not only the materials". Relevant factors affecting the performance which need to be considered include likely roadbed and subgrade problems, moisture conditions, drainage, water table, material type, climate, plant available to excavate, mix, compact and level materials, variability of materials, quality of workmanship, testing, supervision, design method, surfacing type, width, thickness, pavement cross section, traffic, likelihood of adequate and timely maintenance, time, frequency and consequences of failure. Toole and Newill[37] show that satisfactory performance of marginal quality materials is likely to be very sensitive to the local climatic environment as well as pavement drainage, construction method, design traffic level and maintenance input. It is therefore necessary to also consider specifications for aggregates where these are relevant in the context of the use of bituminous binders.

4.2 BITUMINOUS BINDERS: SPECIFICATIONS AND REQUIREMENTS

In order to investigate the possible relaxation of certain specifications, it is first necessary to look at the current specifications for various materials and applications. These are usually well documented but are summarised here for expedience.

4.2.1 Bituminous binder types

Various types of bituminous binders have been, and are still, used in road construction for various purposes. Although the term "bituminous binders" includes tars and bitumens the use of tar in surfacings has generally decreased during the past few years, especially on routes with high traffic levels and urban roads and streets. Tar is mostly used for primes, tack coats and in surfacings for parking areas. Currently bitumens are the predominant binder in surfacing layers.

The following grades and types of bitumens are available in South Africa:[38]

Road grade bitumens:

B24 (previously 40/50 pen)
B12 (previously 60/70 pen)
B8 (previously 80/100 pen)
B4 (previously 150/200 pen)

Cutback bitumens:

MC-30 (MC = medium curing)
MC-70
MC-800
MC-3000
RC-250 (RC = rapid curing)

Bitumen-rubber:

Blends of bitumen, crumbed rubber, additives

Bitumen/tar blends:

A blend of 70 to 80 per cent bitumen and 30 to 20 per cent tar

Bitumen emulsions:

Anionic	Spray grade
	Premix grade
	Stable mix grade
Cationic	Spray grade
	Premix grade

Modified bitumen emulsions:

Polymer, rubber or latex-modified.

The following grades and types of tars are available in South Africa:[38]

Tar primes:

RTH1/4 P Special quick drying
RTH 3/12 P

Spray grade tars:

RTH 15/20 S
RTH 45/50 S
RTH 50/55 S

Hotmix grade tars:

RTH 50/55 HM

RTH 55/60 HM

RTH 60/65 HM

PVC tars:

1,5 per cent by mass of PVC (polyvinyl chloride) (no longer generally available)

Tar-rubber:

Blends of tar, crumbed rubber, additives

4.2.2 Binder specifications

Specifications of bituminous binders for use in road construction in South Africa are published by the South African Bureau of Standards (SABS)[38].

SABS 307	- Penetration grade bitumens
SABS 308	- Cutback bitumens
SABS 309	- Anionic bitumen emulsions
SABS 548	- Cationic bitumen emulsions
SABS 748	- Tar from coke oven crudes
SABS 749	- Road tar from SASOL synthetic fuel manufacturing process

These specifications are reviewed from time to time.

4.2.3 Adhesion between binder and aggregate

The purpose of a bituminous binder in road construction is to act as an agent which binds aggregates together in order to improve the load-bearing capacity of a material (in the case of asphalt) and protect it against loss of particles due to abrasion (in the case of asphalt and surface dressings). Adhesion between the binder and the aggregate is therefore of primary importance. Adhesion results from a combination of a number of factors, the most important being electrical and mechanical forces as well as adsorption and diffusion. No chemical reaction takes place. The role of stone in its interaction with a bituminous binder can be conceived of in three ways: chemical, mechanical and the interface energy concepts[39].

Chemical concept

Aggregate is either hydrophilic (water-loving) or hydrophobic (water-hating) and either acid or basic. Rocks are acid if the "acidic components" (SiO_2 and CO_2) comprise more than 50 per cent of their total composition. The 50 per cent could refer to either the percentage of mass as determined in a chemical analysis or the percentage of the number of relevant molecules. The state of hydrophyllicity and acidity affects their chemical interaction with various bitumens.

Mechanical concept

This concept considers only the degree of adhesion between a bituminous binder and stone. Only geometrical shape of the stone surface (flat/smooth, irregularities, protrusions, indentations, cavities) is considered in such a way that the more irregular the surface, the better the adhesion.

Interface energy

This together with the mechanical concept explains part of the role played by stone in obtaining the desired bond with bitumen. Each stone surface contains free energy which results in an attractive force towards the stone surface. Rough surfaces contain more free energy than smooth ones. The adhesion depends on the surface texture of stone more than on the "acidity". Most rocks are weakly negatively charged and there is no noticeable difference in regard to the total charge between the different types of rock. The chemical composition strongly influences the texture of crushed rock.

A binder can generally be expected to adhere to untreated aggregate provided:

- the aggregate is clean, dry and free from dust;
- the weather is fine and dry;
- large temperature variations do not occur soon after sealing;
- cutter oil is just adequate to give the binder a viscosity appropriate to the road temperature.

A number of factors affect adhesion:

- Electrical polarity - better adhesion is obtained when the binder and aggregate surfaces have opposite polarities. Bitumen is substantially non-polar.
- Water - most aggregates have a greater affinity for water than for bitumen, thus damp aggregate needs to be treated with a precoat or adhesion agent. When damp aggregate is used, rollers and traffic should be kept away from the road until moisture has evaporated.
- Binder viscosity - low viscosity binders wet the stone more easily and thus adhere better than high viscosity binders.
- Dust - prevents good coating of aggregate with binder. Binder viscosity needs to be adjusted accordingly, or precoating may be used.
- Aggregate shape and surface texture - angular stone adheres more readily to binder than rounded ones. Smooth surfaces wet easier, but also strip more easily.
- Weather - low temperatures can usually be counteracted by precoating the stone and lowering the binder viscosity.

Adhesion can be promoted by:

- Precoating
- Adhesion agents
- Rubber addition - latex, powder or in solution with kerosene incorporated in the binder.

4.3 AGGREGATES IN BITUMINOUS SURFACINGS

4.3.1 Aggregates used in surfacings

Weinert[39] classified the South African natural construction materials into nine groups and then differentiated between those which decompose and those which disintegrate. The materials in each of the groups were then evaluated in terms of their suitability and problems for surfacing stone.

Class: Decomposing rocks

- Basic crystalline rocks: Diorite, gabbro, norite, serpentinite, diabase, dolerite, andesite, basalt, phonolite, amphibolite, green schist are used in surfacings in southern Africa. Most of them are acceptable in regard to their adhesion to bituminous binders. Only occasionally will they have unacceptably smooth surfaces and thus adhesion problems. Most tend to exhibit polishing under traffic because of similar hardnesses of their constituent minerals.
- Acid crystalline rocks: Felsite, pegmatite, rhyolite and syenite are used in surfacings in southern Africa. It is possible that a number of cases recorded the use of granite whilst the stone may actually have been orthogneiss. Materials of this group generally make good surfacing stone if not too coarse grained, porous or glassy. Adhesion of medium grained rock to bituminous binders is acceptable but stripping may occur on the faces of large minerals in coarse grained rocks. Most of these rocks do not polish excessively under traffic, felsite and syenite being the exceptions.

Class: Disintegrating rocks

These rocks only disintegrate, meaning that decomposition (weathering through chemical alteration of minerals) does not need to be considered for the assessment of the road building quality, although some exceptions are possible.

- High-silica rocks: Hornfels, quartzite and vein quartz have been used in surfacings in southern Africa. Chert is the only one in this group not used in surfacings. Crushed high-silica rocks are always sufficiently strong for use as a surfacing aggregate. Crushing is likely to produce smooth surfaces and stripping is likely. Polishing occurs very slowly.

- Arenaceous rocks: Quartzitic sandstone is the only member of this category of materials which has been used in surfacings in southern Africa. It may sometimes be mistaken for quartzite. Others include arkose, conglomerate, gritstone and mica schist. Quartzitic sandstone which complies with crushing strength requirements, makes a better surfacing aggregate than quartzite because it does not develop smooth surfaces when crushed. Polishing is less than that of quartzite.
- Argillaceous rocks: (phyllite, sericite schist, slate, shale, mudstone, "baked" shale) These are never used in surfacings.
- Carbonate rocks: Dolomite and marble have been used in surfacings in southern Africa. Limestone is the other member of this group. Every carbonate rock which is suitable for crushing is suitable for use as a surfacing material. These aggregates are mostly only used under light traffic. They exhibit the best adhesion to bitumen of all rock types but are the least suitable materials with respect to polishing.

Class: Special groups of rocks

- Diamictites: (tillite, volcanic breccia, volcanic tuff) Only tillite has been used in road surfacings in southern Africa. Crushed tillite is very good if it is not too water-absorbent. Tillite adheres well to bitumen, but can absorb water when weathering has initiated. It is among the least liable materials to polish.
- Metalliferous rocks: (ironstone, magnetite, megnesite) These are not used in surfacings.

Class: Soils

- Pedogenic materials: Calcrete and, sometimes, silcrete has been used as surfacing aggregate in southern Africa. Calcrete is used on a limited scale for lightly trafficked roads. According to Netterberg[40], calcretes have been used successfully in asphalt in New Mexico and surface treatments in South Australia. In view of its generally satisfactory adhesion and resistance to polishing, it can be used in surfacings, provided it meets usual strength requirements and is not too porous. Some calcretes contain deleterious amounts of soluble salts which can cause disintegration of the upper base and blistering of blacktops.

It is very seldom that non-standard materials are used for bituminous surfacings in South Africa. In remote areas materials such as hardpan calcretes have occasionally been crushed as surfacing aggregate but very little use has been made of natural gravels or lightly processed materials of marginal quality. The use of natural river sands for slurry seals in the Karoo during the late 1970's resulted in significant problems and premature failures. These were investigated[41] resulting in a proposal that the extended sand-equivalent test be used to identify potentially unsuitable materials. Adhesion of the bitumen emulsion to the smooth surfaces of the river sand was also identified as a possible contributory factor.

The National Association of Australian State Road Authorities[42] provides the following recommendations regarding aggregate for possible use in surfacings:

- Crushed and screened rock - this should be either clean spalls of sound rock, free from bedding planes or lines of weakness, or clean hard gravel. It should be free from any deleterious matter, weathered, disintegrated stone or pyrite and should be uniform in quality;
- Wholly crushed and screened gravel;
- Washed and screened gravel;
- Screened gravel - this should be derived from hard wearing stone and should be free from vegetable matter, clay balls and soil;
- Partly crushed and screened gravel - this should be derived from hard wearing stone and should be free from vegetable matter, clay balls and soil;
- Bank run or pit sand - should be derived from hard wearing stone and should be free from vegetable matter, clay balls and soil;
- Scoria (volcanic ash, tuff) - should be reasonably hard, free from overburden, decomposed scoria, an excess of fines, or any vegetable matter;
- Crushed slag - should be crushed from air-cooled, blast furnace slag of uniform quality, reasonably free from flat, elongated, vesicular, glassy, or brittle pieces;
- Synthetic aggregate.

It is significant that material obtained solely by screening can be used for surfacing aggregate.

The following properties need to be considered prior to the use of the aggregate:

- Uniformity - all material should be of the same grading and shape. Segregation of fine and course particles must be avoided;
- Cleanliness - particle surfaces should be as free as possible of dust;
- Resistance to wear - Maximum limits for the Los Angeles Abrasion loss of 20, 27 and 35 per cent are recommended for traffic volumes exceeding 1000 vpd, 300 to 1000 vpd and less than 300 vpd respectively.
- Soundness - the aggregate should be sound and able to resist decomposition;
- Shape - the best shape is approximately cubical with a flakiness of not more than 35;
- Grading - an appropriate grading must be achieved;
- Affinity for bitumen - good adhesion properties are required;
- Resistance to polishing - the material should resist polishing but for low volume roads this factor is considered to be of lesser importance than aspects such as durability and hardness.

4.3.2 Asphalt

Specifications for the aggregate components of asphalt mixtures in South Africa are documented in TRH14[4]. It is stated 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". Specifications are given for the following mixture types: gap-graded, continuous graded, semi-gap graded and open graded.

Aggregates may be obtained from several sources. The coarse aggregate (>4,75 mm) is typically crushed rock. Fine aggregate (>0,075 mm) may be crusher sand, clean natural sand, mine sand, selected river gravel or a mixture of these. The filler (<0,075 mm) may be cyclone dust, rock flour, fly-ash (pulverized fuel ash), portland cement, milled granulated blast-furnace slag, hydrated lime or a combination of these. Small quantities of hydrated lime (1 to 1,5 per cent) may improve the anti-stripping properties of the asphalt.

Grading

A number of fairly tight gradings are specified for asphalt[4]. These are summarised in Table 4.1.

Table 4.1 : Grading of aggregate used in asphalt surfacings

Sieve size (mm)	Percentage passing sieve by mass								
	Gap graded Stone content			Continuously graded			Semi-gap graded	Open-graded	
	Low	Med	High	Coarse	Med	Fine		Coarse	Fine
19,0	100	100	100	100			100		
13,2	75-100	75-100	75-100	84-100	100		80-100	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	53-55	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	12-32	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: a. The maximum size of aggregate should not exceed one half the compacted thickness of the asphalt.

Crushing strength[1]

The aggregate should have a minimum 10% FACT value of the dry material of 160 kN or a maximum Aggregate Crushing Value of 25 per cent. The 10% FACT should be a little higher for tillite (170 kN) and calcrete (180 kN). Additional detail is given in Table 4.2.

Table 4.2 : Recommended crushing strengths of materials for surfacings[39]				
Group of natural road construction materials	Recommended crushing strength			Remarks
	Rolled-on chips	Surface treatment	Bituminous mixtures	
Basic crystalline rocks	210/75% 18	210/75% 21	160/75% 25	
Acid crystalline rocks	210/75% 13	210/75% 21	160/75% 25	
High-silica rocks	210/75% 18	210/75% 21	160/75% 25	Wet test of little relevance, could be disregarded
Arenaceous rocks	210/75% 18	210/75% 21	160/75% 25	
Argillaceous rocks	Unsuitable			
Carbonate rocks	210/75% 18	210/75% 21	160/75% 25	
Diamictites	220/70% 17	220/70% 21	170/70% 24	
Metalliferous rocks	Not used			
Pedogenic materials				
Calcrete	No information available		180/65%	
Ferricrete	Not suitable for crushing test			
Silcrete	210/75% 18	210/75% 21	160/75% 25	Compare high-silica rocks

Open-graded asphalt requires a slightly stronger material as there is less particle interlock. The recommended values are:

minimum 10% FACT (dry)	210 kN
or maximum ACV	21 per cent

Argillaceous rocks (phyllite, sericite schist, slate, shale, mudstone, "baked" shale) and ferricrete are unsuitable for asphalt. Pedogenic materials such as ferricrete and silcrete do not perform satisfactorily in asphalt surfacings (although they have been used with variable results).

Polished stone value (PSV) (SABS Method 848[43])

For difficult sites, eg approaches to intersections, roundabouts, steep grades and tight curves with insufficient superelevation, a minimum PSV of 55 is recommended for coarse aggregate. For other sites a value of 50 is adequate. PSV requirements are very important under heavy traffic for open-graded asphalts.

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.

Water absorption

This should be limited to less than 1,5 per cent by mass for fine aggregates and not more than one per cent for the coarse aggregate.

Properties of asphalt mixture

Asphalt should meet the criteria of Table 4.3 at 100 per cent Marshall density.

Table 4.3 : Design criteria for asphalt surfacing mixtures (all traffic categories)

Type of mixture	Continuous grading		Gap or semi-gap	
	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 (%)	Not specified	-	1,10	
Film thickness (μm)	Not specified	5,0	-	
Filler/bitumen ratio	Not specified	1,0	1,5	
Voids in mineral aggregate (min)	Max size aggr.	VMA (%)	Not specified	
	19,0 13,2 9,5	14,0 15,0 16,0		

4.3.3 Double stone seals[44]

These are apparently the most common seal type used in South Africa. Various combinations of individual seals make up typical double seals eg 13 mm and sand; 13 mm plus slurry; 13 plus 6 mm and fog spray.

The criteria for surfacing stone according to Weinert[39] are:

- it must be sufficiently strong not to break or crush during rolling or under traffic;
- it must not abrade excessively under traffic (min polished stone value of 45);
- the stone must adhere well to the binder.

Dickinson[45] suggests the following specifications for aggregates used in seals in Australia (Table 4.4):

Table 4.4 : Specifications for aggregates used in seals		
	AADT	
	300-6000	<300
Max. flakiness index (%)	35	35
Max. Los Angeles abrasion (%) Coarse grained Fine grained	 30 20	 40 30
10% FACT - min. wet strength (kN)	80	70
Max. decrease in wet strength as % of dry	40	45
Min. dry strength (calculated from above) (kN)	200	156

No specifications for polishing are specified because of the "poor interlaboratory precision of the polishing test"[45].

According to Weinert, 10% FACT results for surfacing materials generally fall between 160 and 210 kN[39]. Changes in binder application rates can make limited allowance for slightly weaker and more porous stone. Actual strength requirements vary among different road authorities. In Zimbabwe a 10% FACT result of 160 kN (ACV of 26 per cent) is specified for surfacings and 80 kN (ACV 32 per cent) for coarse aggregate in asphalts with continuous gradings.

TRH14[4] recommends that aggregate for surface treatments should consist 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.

Crushing strength

Either 10% FACT or ACV can be used on the 13,2 to 9,5 mm fraction. Full limits are provided in Table 4.2. The values in each cell are the 10% FACT (kN), the ratio of soaked to dry 10% FACT (%) and the ACV (%).

Polished stone value

The same limits as for asphalt are recommended. PSV requirements are very important for single and multiple seals, and under heavy traffic for Cape seals.

Grading

The particle size distributions of various nominal sized stone for use in double stone seals are summarised in Table 4.5.

Table 4.5 : Gradings of single-sized stone					
Sieve size (mm)	Percentage passing by mass				
	Nominal size (mm)				
	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					

Stone-bitumen adhesion

Bitumen does not generally adhere well to "acid" rocks. However, when tar or cationic bitumen emulsions are used, or stones are precoated, adhesion problems can be reduced or avoided.

Medium to fine-grained rocks have rough-textured crushed faces which are not likely to result in stripping of the binder. Most of the carbonate rocks and calcretes, many acid or basic crystalline rocks, tillites (diamictites) and any arenaceous rocks which possess a strong siliceous cementing matrix could fall into this category. Crushed coarse-grained rocks possess numerous flat and smooth faces of large minerals (particularly feldspars in certain acid crystalline rocks). Very dense to glassy rocks develop a smooth-textured crushed face from which bitumen strips readily (all high-silica rocks and certain volcanic members of the acid and basic crystalline rocks fall into this category). Precautions should be taken with natural transported materials which are unprocessed as transportation results in rounding and smoothing of the particles with a detrimental effect on bitumen adhesion.

The Riedel and Weber test[13] is carried out to evaluate bitumen adhesion and the following classification of adhesion has been developed:[4]

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

Flakiness index

The permissible flakiness indices recommended in TRH14[4] are summarised in Table 4.6.

Table 4.6 : 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

Fines

The presence of dust should be limited to that recommended in Table 4.7.

Table 4.7 : Permissible fines and dust contents

Nominal size (mm)	Stone			
	Grade N		Grade S	
	Max Fines content (%<0,425 mm)	Max Dust content (%<0,075 mm)	Max Fines content (%<0,425 mm)	Max Dust content (%<0,075 mm)
19,0 to 9,5	0,5	-	2,0	1,5
6,7	0,5	-	3,0	1,5

4.3.4 Cape seal

The requirements for aggregate grading, flakiness, crushing strength and polished stone values (PSV) are the same as those for double seals discussed in the previous section.

Grading of natural sand

The grading of all sands for sand seals and multiple seals should comply with the following recommendations (Table 4.8):

Table 4.8 : Grading of natural sands	
Sieve size (mm)	Percentage passing by mass
6,7	100
0,3	0-15
0,15	0-2

Sand equivalent value (natural sand)

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

4.3.5 Single stone seal

The requirements for aggregate grading, flakiness, crushing strength and polished stone value (PSV) are the same as those for double seals discussed in an earlier section.

4.3.6 Sand seal

This type of seal has been used very successfully in South Africa and Zimbabwe[46] normally where difficulty is experienced in obtaining suitable aggregate and where river sand is available. The usual practice is to prime the base with a tar primer (about 1 ℓ/m^2, depending on porosity of the base) which results in a penetration of about 5 mm. One application of a B4 road grade (150/200 pen) bitumen follows the prime (typically 1,5 ℓ/m^2). In Zimbabwe a sand of between 10 and 1 mm is used, and in Natal one of between 4,75 and 0,075 mm is usual. Typical application is 200 m^2/m^3. Natal prefers a double seal, without a primer and with an MC 3000 cutback bitumen.

The disadvantages of sand seals are:

- poorer abrasive resistance than surface treatments;
- more susceptible to ravelling;
- blotchy appearance;
- low skid resistance;
- can be used only on gradients less than 7 per cent.

The specification of the grading for natural sands for sand seals is summarised in Table 4.8.

4.3.7 Sand asphalt

This type of surfacing is useful on rough bases and is applied up to 10 mm thick using a B4 road grade (150/200 pen) bitumen at a binder content of between 6 and 7 per cent.

The specification of the natural sands recommended for sand-asphalt is that shown in Table 4.8.

4.3.8 Slurries

According to Wolff et al[47] the binder used mostly for slurries is an anionic bitumen emulsion, although internationally the use of cationic bitumen emulsions seems to be favoured. Anionic emulsions are generally compatible with a wider range of aggregate types than are cationic emulsions[48]. However, cationic emulsions give less stripping and consequently greater durability with hydrophilic aggregates.

The use of natural sand is not recommended. Where it is to be used, the sand content must be limited to 25 per cent and a lower durability of the slurry seal must be accepted. The sand equivalent of the sand to be used should not be less than 45.

Properties of slurry aggregate[4]

Slurry aggregate is generally crusher sand or a blend of this and natural sand.

The grading should comply with the limits summarised in Table 4.9. Natural sand should not make up 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 4.9 : Grading of slurry aggregate

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

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

4.3.9 Gravel (Otta) seals

These seals have not been used regularly in South Africa, although they are described in the Botswana Road Design Manual[49] and have given extremely good service in urban and remote rural areas in Botswana. They consist of a graded gravel seal with a particle size range from 0,075 mm (1 to 10 per cent passing depending on nominal size) to 20 mm (40 to 100 per cent passing 9,5 mm sieve)[50]. They are extremely cost-effective when material is in short supply as none of the grading is wasted compared to typical nominal sized surfacing stone.

4.3.10 Primer seals

These are used in Australia when the application of a bitumen surface treatment is delayed. Primer seals consist of a prime and a layer of aggregate (6 mm maximum size)[51]. The binder which is used most often is a cutback bitumen.

4.3.11 Marginal aggregates

A number of cases where locally available marginal materials have been used successfully in road construction have been documented[3]. In none of these cases was a sub-standard bituminous surfacing constructed with a marginal bituminous binder. The manufacture of bituminous binders is controlled by SABS specifications (see section 4.2.1). The different grades of binder are made up by blending hard and soft bitumens and by sometimes using additives. The quality of bitumens is carefully controlled during manufacturing. It is therefore clear that only good quality binders should be delivered for road construction. Substandard binders are usually the result of wrong treatment or processing prior to application on the road, eg overheating, prolonged storage at elevated temperatures or contamination with other substances, eg diesel, oil, other binders, etc. This section thus concentrates on the use of standard quality bituminous binders together with aggregate of poor quality.

If the quality of pavement layers is suspect, it is desirable or even essential to have an adequate surfacing. *It may be better to tolerate some bleeding in the interest of flexibility, waterproofing, salt resistance and a generally longer life than to have a dry, brittle, permeable surfacing*[3]. As the marginal pavement materials often used in low volume roads are typically highly moisture susceptible, good maintenance in the form of timely resealing and regular crack-sealing is probably more important than the quality of the base course in many cases. The possible cost of this must obviously be taken into account when considering the use of substandard materials.

The presence of saline materials in the pavement layers can often have a detrimental effect on the performance of bituminous surfacings. An interesting case study indicates that a marginal surfacing aggregate can be used together with highly saline soils (as well as sea water for compaction) with good results provided that the appropriate construction procedures are followed[53]. In fact, later damage which occurred was the result of the variable and dusty nature of the chippings and not their hardness nor the saline conditions.

A gravel road along the north-west coast of the Cape, carrying less than 50 vpd (30 per cent heavy), in poor and dangerous condition was surfaced using low quality materials and sea water for compaction of the layers. Rainfall in the area averages 100 mm/year. The road consisted of a sand subgrade and calcareous base. Maintenance of the gravel road had been performed using sea water which added to the natural salt content of the base material (0,5 to over 2 per cent).

Fresh water for construction was not available due to drought conditions in the area and suitable aggregates for surfacing could not be found.

The existing crushers at Kleinzee and Koingnaas (for concrete aggregate) were not capable of producing stone complying with SABS 1083 for even Grade S stone. The stone which was available consisted of crushed quartzites and granites. The available crusher dust had about 20 per cent passing the 0,075 mm sieve and the fines were plastic, which thus rendered the material unsuitable for use in a sand seal. It would have required blending with good quality river sand before it could be used in a slurry. The river sand was in short supply and excessively dusty for use in a sand seal. Washed crushed quartzitic tailings from a mine dump varied in diameter from about 1 mm to about 12 mm.

Conductivities of all the materials were far in excess of the recommended safe maximum of 0,2 S/m on the <0,425 mm fractions for use in base courses. This recommendation avoids damage to the prime and surfacing due to upward migration of salts and crystallisation at the base surface regardless of construction procedures. Experience on road experiments in Namibia[52] indicates that omitting priming and surfacing within two weeks of finishing the base, could be a solution where sea water is used for compaction and up to 2 per cent of salt is contained in the base. Very severe damage to any prime could be anticipated. To contain the salts, an impervious seal is needed. The prime was omitted since it cannot by itself provide an impervious seal and would all have lifted off very rapidly as a results of salt crystallisation. Additional benefits of omitting prime are:

- cost savings, some of which can be used to increase amount of binder in the seal;
- time savings, ie the seal could be applied earlier;
- accommodation of traffic would be facilitated sooner.

A stable grade anionic bitumen emulsion was used and gave good adhesion when used on both the calcretes and the mine tailings. A double seal consisting of poorly graded 25 to 6 mm flaky crushed quartzitic chips followed by a layer of either sand or crusher dust remained in a good condition for over a year with no evidence of salt damage. No grading of the chips is given. The first layer consisted of 19,0 to 9,5 mm followed by a 9,5 to 4,75 mm nominal size with high dust content. The binder application rate was between 1,8 and 2,2 ℓ/m^2. In certain areas the base was allowed to dry out, which should have been avoided, and subsequent sea water applications eventually increased the salt contents to as high as 3,5 per cent. In these areas noticeable ravelling of the seal at the shoulders and where a good double seal was not obtained (sites of stone loss) occurred and potholing eventually affected nearly all of the road.

Traffic increased to 100 vpd after completion of the surfacing. Regular maintenance of the seal is necessary to ensure that its impermeability is maintained. This case study indicates that the specification of the grading and shape of stone chips for the surfacing of low volume roads may be relaxed, but not where the salinity of the base is high or soluble salt damage is likely.

4.3.12 Factors influencing surfacing performance

The level of service provided by a road to the road user is affected by the functional requirements which primarily governs riding comfort and safety. The surfacing is only a part of the road, but a very important one as far as riding quality is concerned. The extent to which a pavement meets the requirements of the road user is the serviceability[54]. Performance is the variation in serviceability with time during the design life, until it reaches some predefined terminal level, at which point it is said to have failed.

It is therefore clear that the performance is related to certain predefined terminal conditions. These terminal conditions depend on the importance of a road in terms of the traffic volumes using the road, the type of road and the area in which the road is. Roads of the same construction may therefore have different terminal serviceability levels depending on their environment.

Before the effect of marginal aggregates on the performance of surfacings is investigated, it needs to be put into perspective as far as the other factors influencing surfacing performance are concerned. Because of the interrelationships between these factors, it is sometimes impossible to isolate any one of the factors from the influence of the others. The factors which influence the performance of a surfacing seal are listed in TRH3[44]:

- the characteristics of the stone and bituminous binder;
- the rate of application of the stone and binder;
- the initial development of good adhesion, which must be retained throughout the life of the seal;
- the initial compaction at the construction stage to obtain a dense interlocking mosaic of stone (applicable to asphalt mixtures);
- the construction technique and the control of traffic during construction and the early life of the seal;
- the strength and flexural properties of the pavement layers, especially those of the upper base;
- the type and condition of the existing road surface;
- the volume and type of traffic;
- the climate and the road surface drainage conditions;
- the geometry of the road, for example steep upgrades and sharp curves;
- the impermeability of the seal;
- the embedment of the stone into the underlying surface, and
- the presence of salts in the base.

The more important factors which influence performance of single and double seals have been summarised as follows[55]:

Non-controllable:

Traffic	heavy/light
Environment	micro/macro climate
	road surface condition

Controllable:

Stone type	strength
	precoating
	wearing properties
Stone size	grading
	average least dimension
	flakiness
Binder type	bitumen
	tar
	modified binders
Stone application rate	
Binder application rate	

From the literature it is clear that there are many factors which govern the performance of pavements. In the case of low-volume roads in areas where funding is usually limited and high quality materials are scarce and expensive, these factors are exacerbated further. It is also clear that most of these factors are interrelated. For example, in a dry area a certain substandard stone may be used in a surfacing with great success while it may fail when used in a wet area because of the effect of moisture on its strength and polishing resistance and possibly adhesion with the binder. The interrelated nature of certain factors may on the other hand be used beneficially. Using a lower grade surfacing stone may perhaps be adjusted for in a way by using a modified binder. All factors should thus be taken into account at the same time and on every site.

4.4 RECOMMENDATIONS AND POSSIBLE RELAXATION

It has been shown that aggregate costs can make up about 20 per cent of total surfacing cost[56]. It is estimated that by using marginal materials this cost could decrease by about a third (ie 7 per cent of the total surfacing cost). In portions of countries such as Namibia and Botswana where no suitable material for stone chipping is available, potential savings may be greater. It is also clear that the performance of low volume road surfacings is dominated by factors other than marginal surfacing materials, as indicated above. The cost of the surfacing as a percentage of the total cost of the road is highly variable, depending on the pavement structure, quantity and haulage of imported material, location, etc. but for a typical low volume road could be in excess of 50 percent of the total cost of the road. The advantage of lower cost must be weighed against the increased risk of failure. In addition, factors such as long surfacing life are likely to be more important in reducing the life-cycle cost of bituminous surfacings.

"Great strength is not required of chippings and the usual minimum 10% FACT requirement of about 210 kN (or maximum ACV of 21 per cent) can be relaxed to about 160 kN as used in Zimbabwe[57] or even to 80 kN as used in Australia [45] and elsewhere with the proviso that the soaked value is at least 75 per cent of the dry value. The equivalent ACV requirement of 30 per cent should probably not be relaxed further. Screened laterite and calcrete nodules and crushed hardpan even somewhat inferior to this have been used overseas in surface dressings and asphalt surfacings. Only a pneumatic roller should be used on the weaker chippings and the possibility of binder absorption must be considered."[3]

"The maximum chip size for light traffic should be 13 mm[58]. While dusty chips are likely to cause problems in conventional seal design, consideration can be given to using an Otta graded gravel or chip seal[50]. One road in Botswana with such a seal has used aggregate with an ACV of 40 per cent under 100 vpd successfully since 1978[59]."[3]

"Loss of chippings due to absorption of the prime and seal binder into the base course (often calcrete) has been encountered in Australia and probably in Zimbabwe and Botswana. Binder application rates must be increased in such cases."[3]

"The dry BS 812 Aggregate Impact Value Test (AIV), which gives results about numerically equal to the dry Aggregate Crushing Value (ACV) on most materials, should be used as an inexpensive site test. This test is probably superior to the Treton Test used in the Cape although recent research has shown that the results from the Treton test as an indicator of aggregate hardness correlate better with performance than the AIV for wearing course materials for unpaved roads[60]."[3]

"Care should be taken that a significant proportion of weak aggregate is not used, since even 0,1 per cent of soft or weatherable aggregate might mean a hole in the surfacing every square metre or so. This could lead to serious consequences in wet areas or where the base has a high salt content."[3]

"Good sand for seals and slurries is advisable, but single-sized fine-medium sand such as Kalahari sand can be used where there is no significant hoof traffic. Kalahari sand-calcrete mixtures have been used for surfacing and base in Namibia[61,62]."[3]

"Sealing of lightly trafficked roads in Australia is only carried out during the hot summer months (January to March). The roads which are constructed at other times of the year are thus sealed with a primer seal until the proper seal is added the next summer. The primer seals usually consist of a cutback bitumen in the AMC2-4 range followed by a layer of graded or single sized chippings, sand, fine natural gravel, crusher waste, sea-shells or any other suitable "aggregate" rolled in. These generally last for at least a year but may be left for two or three years before addition of a single chip seal."[3]

"The width of a seal only appears to be important if traffic can be kept away from the edge, as the base material requirements for the single lane roads with seal widths of about 3,5 m used in Zimbabwe, Zambia, Malawi and Australia appear to be the same as those for double lane roads. Shoulder sealing in wet areas is probably only advantageous if traffic is discouraged from using them."[3]

The Botswana Road Design Manual[49] suggests relaxation of several material specifications when used in constructing low volume roads. Certain precautions need to be taken as some calcrete base courses tend to have a fairly high bitumen demand and should therefore be primed. Bitumen is recommended in preference to tar.

Many calcretes have high soluble salt contents. In these cases two applications of prime should be applied, the second spray as soon as the construction vehicles can drive on the first spray without lifting the prime[49]. Calcrete bases should be surfaced as soon as construction vehicles can drive on the primed basecourse without lifting the prime. Where primed base appears dry it may be necessary to increase the binder content in the surfacing by 10 to 15 per cent to compensate for the high bitumen demand of calcrete bases.

For materials other than calcrete, no prime may be necessary for double sand and double gravel seals[49].

The Botswana requirements for a double sand seal are:

First seal: RTH or RTL 30/35 tar or MC 800 at 0,7 ℓ/m^2 to 0,8 ℓ/m^2 (without a prime: 0,8 ℓ/m^2 to 0,9 ℓ/m^2); sand application rate 0,006 m^3/m^2

Second seal: MC 3000 at 0,8 ℓ/m^2 (without a prime: 0,85 ℓ/m^2); sand at 0,006 m^3/m^2.

The sand shall consist of crusher fines or clean sand free of organic matter with the following grading:

Screen size (mm)	per cent by mass passing
9,5	100
4,75	70-90
2,36	45-70
0,6	16-34

Jordaan[63] has shown that large aggregates can be successfully used in surface treatments with significant benefits in terms of performance and economy.

The theory behind the use of large aggregates is that, because of their size, shear forces on the surface are transmitted to the base through punching of the stone into the top of the base. The surface of the base is also strengthened and base material of lower standard can thus be used. Coarse aggregate surfacings are noisy to drive on and difficult to walk on, and are thus not recommended for use in the urban environment.

A coarse aggregate seal consists of:

- A prime which is used only if it is needed as a curing membrane. The use of a tar is preferred if the base is not stabilized, otherwise a cutback bitumen should be used.
- B4 road grade (150/200 pen) bitumen or MC 3000 at 1 ℓ/m² nett bitumen.
- 26,5 mm concrete aggregate at 72 m²/m³ (over- or under application is corrected by back chipping and hand brooming).
- MC 3000 cutback at 1,2 ℓ/m².
- Clean sand or washed crusher sand at 150 to 200 m²/m³

The cutback bitumen is preferred to a penetration grade as it has better wetting properties. Because of the inherent stability of the seal and the particle interlock, the soft binder does not affect the stability detrimentally.

The advantages of using this type of construction are:

- Construction tolerances can be relaxed as bleeding and ravelling is virtually impossible.
- It is insensitive to aspects such as variable punching of aggregate into the base.
- Except for rolling, construction can be done by hand.
- As the aggregate is pressed into the base during rolling, the riding quality is exceptionally good.
- Because the top binder spray is blinded with sand, the road can be opened to traffic almost immediately after sealing.
- Coarse aggregate seals look and ride much like more expensive seals or asphalt.

A comparison of the cost of large aggregate seals with other types of surfacings is summarised in Table 4.10.

Table 4.10 : Comparison of cost of coarse aggregate seal with other seals[63]

Type of seal	Material cost (R/m^2)	Construction cost (R/m^2)	Total cost (R/m^2)
26,5 mm + sand	2,32	0,85	3,17
19,0 mm + sand	2,22	0,87	3,09
19,0 / 9,5 double seal	2,81	0,98	3,79
13,2 / 6,7 double seal	2,11	1,02	3,13
19,0 / slurry (Cape seal)	2,66	0,95	3,61
13,2 mm single seal	1,37	0,57	1,94
9,5 mm single seal	1,21	0,59	1,80
6,7 mm single seal	0,94	0,61	1,55
Sand seal (5 mm)	1,05	0,45	1,50

Note: a. Material cost does not include handling, transport, VAT.
b. Construction cost does not include profit, mass rebates, etc.
c. Values in 1992 Rands

It is important that the appropriate surfacing type is used in the correct location. Table 4.11 summarises the possible uses of the various seals.

4.4.1 Grading

One of the main problem areas with surfacing aggregate is producing materials with a suitable grading. Only minor deviations from the specified grading of natural sand for sand seals or stone/sand double seals can usually be accepted without a significant increase in risk for lightly trafficked roads.

4.4.2 Crushing strength

The requirements of TRH14 can be relaxed to those summarised previously in this section provided that rolling with steel wheeled rollers is minimised and rubber tyred rollers are used and taking cognisance of the types of traffic which will use the road. Conditions specific to each site need to be considered, eg traffic and climate.

Table 4.11 : Recommended application of various surfacing types[36]

Factor		Asphalt	Cape seal	Thick slurry	Double seal	Single seal	Sand seal
Environment	1st world - high pavement standard	Y	Y	Y	Y	Y	Y
	1st world - low pavement standard	Y	Y	Y	Y	Y	Y
Maintenance	High	Y	Y	Y	Y	Y	Y
	Medium	Y	Y	Y[a]	Y	Y[b]	
	Low	Y	Y	Y	Y[c]		
	None	Y					
Gradient	<6%	Y	Y	Y	Y	Y	Y
	6-8%	Y	Y[g]	Y[eg]	Y[fg]	Y[efg]	Y[ef]
	8-12%	Y	Y[gh]		Y[fgh]	Y[defgh]	Y[def]
	12-16%	Y	Y[dg]		Y[dfg]		
	>16%	Concrete blocks					

Notes:
a. Thin slurries can lead to construction problems.
b. Rural only
c. Sensitive to construction problems, only used where there is a longer maintenance period.
d. Not on stabilised basecourse.
e. Not if channelling of water flow expected due to soil wash.
f. Not if urban drainage.
g. Not if communal water systems present - leads to detergents on the road, erosion of bitumen.
h. Not gradients >10% if channelling of flow expected - soil wash.

4.4.3 Polished stone value

For traffic of less than 500 vpd, PSV requirements can probably be reduced (by 5 PSV points) taking cognisance of the traffic type, climate and road geometry. In areas where high speeds, a large proportion of high occupancy vehicles, sharp bends, steep grades or frequent rainfall are typical, caution should be exercised before relaxations are permitted.

4.4.4 Fines and sand equivalent

No reduction from TRH14 is considered possible. Sand equivalents should not be less than indicated in Table 4.12. Should the sand equivalent values be marginally low (by less than 5 points), precoating of the material may allow its use provided caking of the fines does not occur.

Table 4.12 : Recommended sand equivalent values for sands for surfacings[5]

Application	Sand equivalent (min)
Sand seal	30
Otta seal	25
Slurry seal	30
Dust palliative	30

4.4.5 Stone-bitumen adhesion

No relaxation of TRH14 is considered permissible. Problems with bitumen adhesion may be overcome with the use of modified bitumens.

4.5 ECONOMICS AND CHOICE OF SURFACING

4.5.1 General

The cost effectiveness of a project is a very important aspect to consider. Several factors need to be taken into consideration during calculations. There are several available methods which can be used to do this. It is not the purpose of this report to go into great detail of any of these methods here, although certain basic terms are explained in order to facilitate understanding of the methodologies followed. Following is a list of certain of the more general terms as they are used in later sections:

construction cost

the cost of upgrading an existing gravel road to a surfaced condition and does not include the cost of realignment. It does, however, include the cost of preparing the existing gravel surface by means of either sweeping, ripping and recompaction or adding a new base layer 150 mm thick.

maintenance cost

includes the cost of routine maintenance on the road surface (patching, crack sealing, grading, etc.) as well as, in the case of gravel roads, the cost of regravelling. Non-pavement related costs (cutting grass, etc.) are excluded.

financial analysis

including only agency costs.

economic analysis

including agency costs (construction and maintenance) as well as road-user costs and non-road user benefits.

partial economic

the same as for economic analysis, but excluding time costs.

road-user costs

the cost to the road-user when using a road, including the value of travel time, collision costs and vehicle operating costs.

non-road user benefit

the benefit to the community brought about by the provision or upgrading of a road. This is not often used because of the difficulty in properly defining it and quantifying the parameters.

nett present value

the present worth of a series of costs expected to occur in the future, discounted using an acceptable interest rate.

benefit/cost ratio

the ratio of the present worth of benefits to the present worth of investment (eg construction cost).

agency costs

the cost to the road authority or agency to provide and maintain a road.

The point at which it becomes economically justified (in terms only of agency costs) to upgrade an unpaved road to surfaced standard is a decision based on the total cost of the various alternatives. The life-cycle cost of the unpaved road must be compared with that of a paved equivalent for a finite period. The costs of the paved roads used in the analysis will depend on, among other factors the selected surfacing option.

A number of guidelines as to when it is viable to surface an unpaved road have been proposed. Richards[64] suggested the following:

- Reconstruction or new alignment - the Cape Provincial Administration surfaces roads when traffic reaches between 120 and 150 vehicles per day (no economic analysis is done);
- Upgrade gravel to surfaced - an economic appraisal is the best method. However, a consensus estimate of 150 to 200 vehicles per day (vpd) in dry areas and 200 to 300 vpd in wet areas was obtained from a panel.

Netterberg and Paige-Green[3] stated that general practice in South Africa for low-volume roads is to upgrade to bitumen standard if traffic exceeds 300 vehicles per day. Current practice varies from province to province. The TPA only surface roads with traffic volumes in excess of 500 vehicles per day. In Natal, the previous policy of providing surfacings only where traffic exceeds 300 vpd was raised to 600 vpd because of funding limitations. In the Orange Free State roads are upgraded when traffic reaches 800 vpd[65]. It must be noted that if no funding is available, no upgrading can be performed, regardless of traffic volume.

As noted above, Richards[64] observed that "experience differs from region to region". This is clearly the case. An example was found where a road carrying traffic of less than 50 vpd was surfaced with a bituminous surfacing[53] while others carrying larger volumes of traffic were left unsurfaced. This was, however, a private road and other factors not normally associated with a public road may have affected the decision.

The economic justification of upgrading unpaved roads has been extensively studied and fully described in a number of documents[5,6,66]. It has been calculated that the break-even traffic at which sealing of an unpaved road becomes economically beneficial can be as low as 20 vehicles per day (taking full economic costs) and financially between 100 and 200 vpd (using direct costs ie agency costs only)[66]. In addition to the economic or financial justification for upgrading, the decision as to what the most appropriate surfacing for the particular situation is, needs to be made. It is recommended that this is made taking into account the environment, maintenance capability, gradient and location with respect to intersections[36].

Several surfacing types are well documented. The choice of surfacing type is limited for low-volume roads because of the high cost of certain types, eg asphalts. However, the life-cycle costs and environment may result in asphalt becoming the primary choice. Where the maintenance capability is low, for instance, those surfacings which have a high durability such as asphalt, Cape seal and double seal should be considered[36].

Jordaan[63] states that in order "to keep the cost of low-cost roads within acceptable limits, the seal, which may contribute up to 50 per cent of the cost of the pavement, is usually kept as cheap as possible. The main problem with less expensive seals is that they have very low tensile strengths and should be placed on solid bases. On high speed straights they usually perform well, but not under traffic turning, stopping and accelerating". There are thus certain factors which must be taken into account in the selection of surfacing type, of which, in the case of low-volume roads, construction and maintenance costs as well as the available materials must certainly be the most important.

4.5.2 Results of cost/risk analyses

An investigation into the possible effects on long-term costs of using marginal aggregates must include cost analyses. This section discusses the results of analyses which were performed in order to determine the possible economic advantages and/or risks associated with the use of lower quality local aggregates in bituminous surfacings on low volume rural roads. In the calculation of the costs of different surfacing types, certain assumptions have had to be made in order to reduce the magnitude of possibilities. The recommendations made in the Appropriate Standards for Bituminous Surfacings-projects (Sabita)[36] were used as basis for these assumptions. More detail is given in Appendix B. In this section, only the results of the analyses are discussed.

The analyses included financial analysis (agency costs only) and partial economic analysis (excluding time costs). The results showed that in all cases, the *financial* break-even point for an analysis period of ten years (in terms of break-even traffic), was always in excess of 1 000 vpd. A road carrying such a volume of traffic may not be classified as a low volume road anymore. Only the results of the partial economic analysis are therefore discussed below.

Reduced aggregate cost (Run 2 and Run 3)

The effect of savings in the cost of aggregate and in the haulage costs on life-cycle costs was determined first. The expected life of all the surfacings was kept unchanged from that of the standard, high cost alternative (Run 1). Two levels of saving have been used in the analysis. For Run 2, a 10 per cent saving on both aggregate and haulage was used and 20 per cent for Run 3.

From the results it is clear that the break-even traffic is reduced for all the surfacing types when these savings are calculated. It seems, however, that the advantage gained for the single and slurry seals is generally less than that for the thicker surfacings.

Reduced resealing frequency (Run 4)

As can be expected, a decrease in the life of a surfacing (or the frequency at which it needs to be resealed) has a negative effect on the economic feasibility of surfacing a gravel road. A reduced surfacing life (of two years in this analysis) does not have such a marked influence on the results of an asphalt surfacing. This can be expected because of the relatively longer expected life of an asphalt surfacing when compared to that of any of the thinner surfacings. A reduction of two years from an original expected life of four years for a single seal means a 50 per cent reduction. On the other hand, a reduction of two years from an original twenty years for an asphalt is a reduction of only 10 per cent. It is clear, however, that the thin surfacings with short expected lives are affected more by a reduced life.

Increased routine maintenance (Run 5 and Run 6)

Increases in the routine maintenance of bituminous surfacings may be expected where marginal aggregates are used. This may be caused by the loss of stones from the surfacing due to soft material breaking up under traffic, porous stone absorbing binder and leaving the surfacing dry and brittle or it may be caused by excessive polishing of the stone by traffic where a good skid resistance is of importance.

Once again, the effect of an increase in routine maintenance cost does not have such a marked effect on the life-cycle cost of an asphalt surfacing. A 10 per cent reduction in the annual cost of maintaining a surfaced road generally seems to have virtually no effect on the break-even traffic, except for the single seal and the slurry surfacings. An increase of 20 per cent does seem to have a more marked effect, but once again it is relatively small in the case of the asphalt surfacings.

Length of the analysis period (Run 7)

The TRH4 document[1] gives certain guidelines for analysis periods of different road categories. For low volume roads (category C), analysis periods of between 10 and 30 years are recommended. It is furthermore said that, for a fixed alignment, a period of 30 years is recommended. In the analysis of the effect of the various cost components on life cycle-costing, an analysis period of 10 years was used throughout. To indicate the effect of using a different analysis period, an analysis period of 20 years was selected.

Increasing the analysis period from 10 to 20 years has a drastic influence on the break-even point of all the alternatives and on all the roads analysed. Break-even traffic is reduced by between 25 and 35 per cent. *From an economic point of view, it can therefore not be simply stated that a gravel road in a certain area needs to be surfaced once the traffic reaches a certain level without also taking into account the length of the analysis period.*

4.6 CONCLUSIONS AND RECOMMENDATIONS

From the survey and analyses done it is possible to draw a number of conclusions.

The cost of the surfacing can comprise more than 50 per cent of the cost of the pavement structure on low volume roads consisting of light pavement structures but significantly less on traditional pavement structures. Significant savings can thus be made by relaxing some of the requirements for the material when used on low volume roads. However, relaxation needs to be carried out with great circumspection as the risk of premature failure and the need for increased maintenance can be significantly increased.

Certain possible relaxations are discussed in the text but these need to be implemented taking into account the particular attributes of the pavement being designed:

(i) Traffic - if the pavement is likely to carry very heavy traffic, vehicles with high tyre pressures (long hauls in hot weather is a typical cause of excessive tyre pressures as is the increasing use of "super-single wheels") and/or slow moving, overloaded traffic, the relaxation of parameters such as crushing strength may lead to disintegration of the seal.

(ii) Geometrics - if the road has poor geometrics with sharp bends and frequent areas requiring braking which impart high shearing stresses to the pavement surface, caution should be exercised prior to relaxing standards.

(iii) Climate - in wet areas the aggregate must be durable enough and have an adequately low porosity to avoid decomposition and fracture caused by high pore water pressures under traffic loading.

(iv) Maintenance capacity - Relaxation of the surfacing aggregate requirements is not recommended where there is a low maintenance capability. Any defects which do occur on seals over light pavement structures require immediate repair to avoid extensive failure of the pavement structure.

The following recommendations are thus made:

Once all the implications of relaxation of standards are evaluated, with a good understanding of these implications, the proposed design should make the optimum use of engineering judgement.

Care should be taken when more than one characteristic of the pavement is relaxed ie *savings in only one of the pavement structure, the drainage or the surfacing should be permitted.* The relaxation of all three of these in one pavement should never occur and the relaxation of two of these should only be allowed where failure is unlikely to result in major political repercussions.

5. LAYER WORKS

5.1 BACKGROUND

The design of layer works for roads in Southern Africa normally follows either TRH4[1] or CSRA[11] requirements or local specifications typically based on manuals similar to these.[67,68,69,70] Most of the traditional specifications allow for roads carrying up to 200 000 E80s over their design lives. In most rural areas very few of the roads which require development carry traffic anywhere near this and can be designed to considerably lower standards without an undue increase in risk.

Most of the structural capacity of this type of road is designed into the base course but it is important to have an adequate platform on which to construct the base. This is necessary both to allow the passage of construction equipment as well as to provide a reaction for the compaction effort applied to the base. Typically the minimum requirement for the subgrade is a G7 quality material (CBR>15 at 93% Mod AASHTO), compacted to 93% Mod AASHTO.

The bulk of the cost of road pavements is in the layer works with materials estimated to make up about 70 per cent of the cost of a typical rural road.[71] Most standard pavement designs typically call for a base, a subbase and, depending on the strength of the lower layers, up to two selected layers. It is thus clear that any elimination of pavement layers will result in significant cost savings. As discussed earlier in this report, no attention is paid to the base in the layer works portion of this project, with the emphasis being placed on the underlying support layers.

During a field investigation of over fifty lightly trafficked roads, carried out for the marginal base course materials project,[2] it was frequently noted that the total pavement structure consisted solely of the base on the in situ material. This clearly indicates that it is not always necessary to import layers beneath the base particularly as South African subgrades are often of a high quality.

5.2 REQUIREMENTS FOR LOWER LAYERS

The primary purpose of the pavement layers beneath the base course is to spread the loads applied to the road surface in such a way that the subgrade does not become overstressed. At the same time the layers beneath the base should have a certain degree of balance to prevent excessive high modular ratios between adjacent layers. With typically strong subgrades prevailing over much of southern Africa[2] it is clear that the importation of thick layers of selected materials is unnecessary for many lightly trafficked roads.

The discussion in this report ignores localised problem subgrades such as those with active clays, collapsible soils, dispersive and erodible soils and other potential subgrade problems. These should be identified during the centre line investigation and the appropriate remedial action taken.

The requirements of the pavement layers all revolve around their strength at the in situ density and moisture conditions prevailing at any specific time. This strength is directly related to the stiffness or E-modulus of the material, which is the main factor affecting its performance. The in situ strength is in turn a function of the material properties (Atterberg constants, grading, clay mineralogy, etc.), which can usually be considered to be constant during the life of the road, and the prevailing density and moisture regime which change with time.

5.3 EXISTING SPECIFICATIONS

Subgrade materials are adequately defined by TRH14[4], which classes in situ materials as G7 to G10 materials. However many subgrades fall into higher material categories so all the strength specifications for classes G4 to G10 are listed in Table 5.1. As can be seen from the table all the material strengths are specified in a soaked state since this is the worst possible condition under the pavement. It is known that the water table in South Africa is generally deep[72,73] and the subgrade is unlikely to become soaked, except in localised areas with poor drainage.

Table 5.1 : Standard CBR and swell requirements for layer works[4]

	Compaction (% Mod AASHTO MDD)	Material type						
		G4	G5	G6	G7	G8	G9	G10
Min Soaked CBR (%)	100							
	98	80						
	95		45					
	93			25	15			
	in situ					10	7	3
Max Mod AASHTO[a] Swell (%)	100	0,2	0,5	1,0	1,5	1,5	1,5	1,5

There are several recommended standards for layer works in southern Africa, each following a similar format of specifying the minimum strength and compaction of the material and the minimum thickness of material required. The tables that follow therefore have the following notation: [Thickness in mm] [Material type] ([Minimum compaction as a percentage of Mod

[a] Although the terminology Mod AASHTO is strictly speaking incorrect, it is used in this report to be consistent with TRH14[4] and CSRA[11].

AASHTO MDD]). A minimum compaction of 100% Mod AASHTO is specified for sands in all the standards.

5.3.1 TRH Series[1,4]

The subgrade must be brought up to CBR 15 (G7) standard. The normal process is to rip and re-compact the top 150mm of in situ material, and then add either one or two 150mm thick selected layers as required. Table 5.2 gives the minimum material specification and compaction requirements. If the in situ subgrade CBR is less than 3 then special actions must be taken, which are not covered in this report. The material depth is normally taken as 800 mm for lightly trafficked roads.

Table 5.2 : TRH4 Subgrade Specifications[1]

	Subgrade CBR		
	3-7	7-15	>15
Selected Layers	150 G7 (93%) 150 G9 (93%)	150 G7 (93%) -	- -
Subgrade Treatment	R&R 150 G10 (90%)	R&R 150 G9 (90%)	R&R 150 G7 (90%)
To material depth	G10 (85%)	G9 (85%)	G7 (85%)

5.3.2 Committee of State Road Authorities (CSRA)[11]

No specifications are given for material strengths, other than for fill. The required minimum standards of compaction for various layers are, however, specified.

Fill	: CBR 3 at 90% Mod AASHTO for depth 0 to 1,2 m
	: CBR 3 at 100% Mod AASHTO for depth 1,2 to 9 m
	: 90% or 93% Mod AASHTO as required
Lower Selected Layer	: 90% or 93% Mod AASHTO as required
Upper Selected Layer	: 93% or 95% Mod AASHTO as required

5.3.3 Cape Provincial Administration[67]

The CPA Material Standards give minimum CBR strengths for the subgrade and each layer at the specified density (% Mod AASHTO). The compaction of the subgrade must be 93 per cent if settlement problems are expected and 100 per cent if the subgrade is a sand.

Table 5.3 : CPA Subgrade specifications[67]			
	Subgrade CBR		
	3-5	5-10	10-15
Selected Layers	150 CBR 15 (95%) 150 CBR 10 (93%) 150 CBR 10 (93%)	150 CBR 15 (95%) 150 CBR 10 (93%) -	150 CBR 15 (95%) - -
To material depth	CBR 3 (90%)	CBR 5 (90%)	CBR 10 (90%)

5.3.4 Natal Provincial Administration[68]

The NPA Materials Specification gives minimum CBR values and compaction requirements for different layers.

Top of Subgrade : CBR 10 at 90% Mod AASHTO, 93% Mod AASHTO
Lower Layers : CBR 3 at 90% Mod AASHTO

5.3.5 Transvaal Provincial Administration[69]

The TPA Materials Specifications give minimum CBR values at in situ density.

Subgrade : CBR 3, 90% Mod AASHTO
Lower Selected : CBR 7, 90% Mod AASHTO
Upper Selected : CBR 15, 93% Mod AASHTO

5.3.6 Road Note 31 (British Department of Transport)[70]

Road Note 31 does not give any minimum CBR requirements for the subgrade, but does give a recommended compaction of 95% British Standard Compaction (BSC), which is only about 40 per cent of the Mod AASHTO compactive effort. The subbase thickness is dependent on the CBR of the subgrade (at 95% BSC) but the in situ moisture content can be taken into account to get the subgrade design CBR. The subbase must have a CBR of 25 at 100% BSC.

The revised Road Note 31[74] has followed the example of TRH4[1] and defined subgrade strength classes for use in the Catalogue of designs as follows:

Class	CBR Range (%)
S1	2
S2	3-4
S3	5-7
S4	8-14
S5	15-29
S6	30

5.4 THEORETICAL ANALYSIS OF SUBGRADE

5.4.1 The Effect of Subgrade E-modulus

Since the stiffness of the lower layers of the pavement plays a major role in determining the structural capacity of a granular pavement it was decided to perform a theoretical study of the effect of subgrade elastic modulus on the predicted structural capacity of a pavement. The pavement was analysed using the standard South African Mechanistic Design Method[75] (MDM). The MDM only has one transfer function to predict the rate of subgrade rut formation, which does not take into account the moisture condition of the subgrade or the in situ density, both of which will have a major effect on the predicted structural capacity of the pavement.

The standard TRH4[1] design for a Category C, Traffic Class E0 road was used as the starting point for evaluating the sensitivity of the design and the design method to changes in different constants. The pavement design consists of a 100 mm G4 base on a 125 mm G5 subbase for both wet and dry conditions. A subgrade of G7 quality is assumed. The analyses were performed using the minor and major stresses as generated by ELSYM and a standard dual wheel load of 2x20 kN wheels at 350 mm spacing with a tyre pressure of 520 kPa.

The subgrade and subbase material properties were then varied to see how different factors affected the structural capacity of the pavement. Five different test runs were performed, in each case the subgrade E modulus was varied from 10 MPa to 200 MPa based on the standard ranges of Elastic Moduli summarised in Table 5.4 for typical subgrade and subbase materials. The test runs varied subbase thickness, subgrade Poisson's Ratio (υ), subbase E-modulus (E), subbase cohesion (c) and subbase angle of internal friction (ϕ) respectively.

Table 5.4 : Standard Elastic Moduli[75]

Material	E (MPa)			
	Dry		Wet	
	Good Support	Poor Support	Good Support	Poor Support
G4	300 (100-600)	75-350	50-150	30-200
G5	250 (50-400)	40-300	30-200	20-150
G6	150 (50-200)	30-200	20-150	20-150
G7	30-200		20-120	
G8	30-180		20-90	
G9	30-140		20-70	
G10	20-90		10-45	

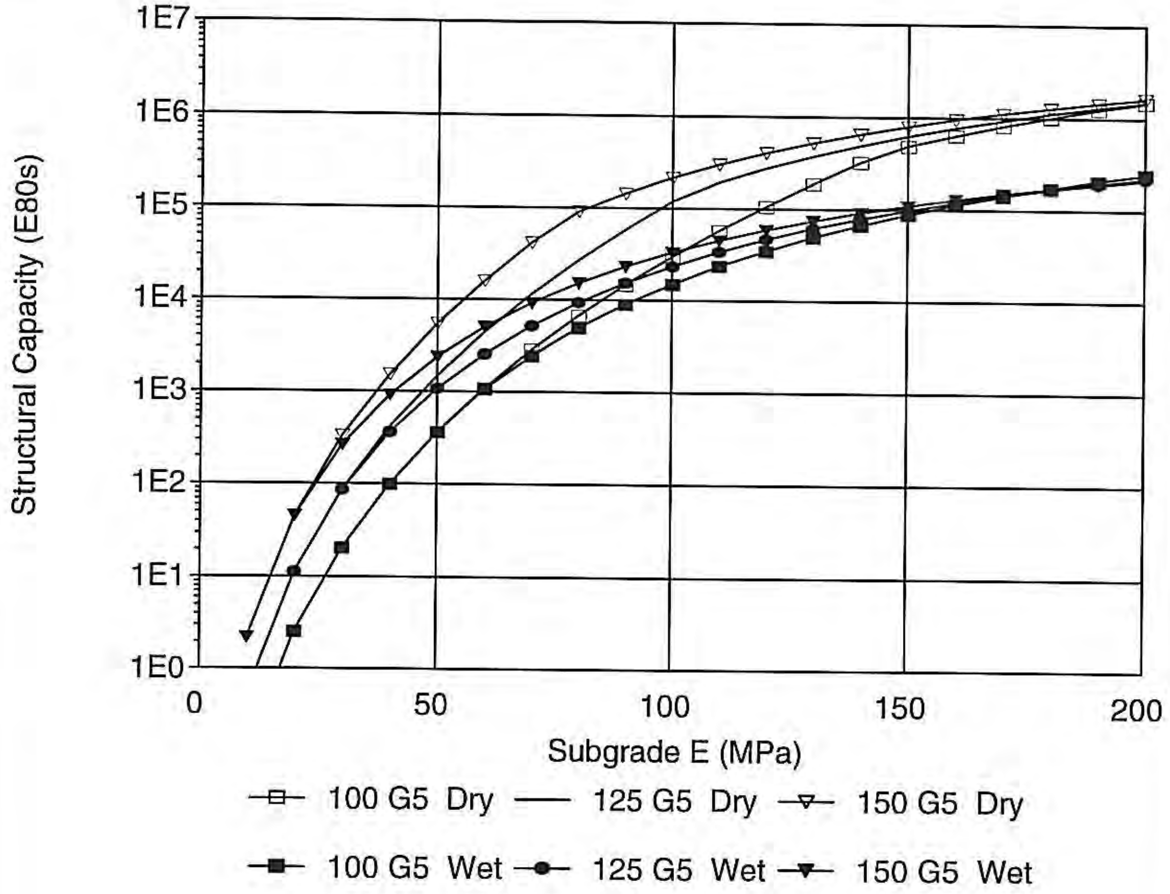

Figure 5.1 : Effect of Subbase thickness

The subbase thickness has a marked effect on the structural capacity of the pavement but the effect decreases as the subgrade E approaches that of the subbase (200 MPa) (see Figure 5.1). This is to be expected as the stresses will be transferred into the subgrade as its modulus increases, and so high shear forces (which lead to low factors of safety) will not develop in the subbase. On each line a sudden change of gradient can be noticed (at the point where the wet line separates from the dry line and higher up along the dry line); this is the point where the structural capacity of the subbase falls below that of the subgrade. There is a large change in structural capacity across the different subgrade moduli, from one of less than 1 E80 to 1 640 000 E80s (dry) and 250 000 E80s (wet).

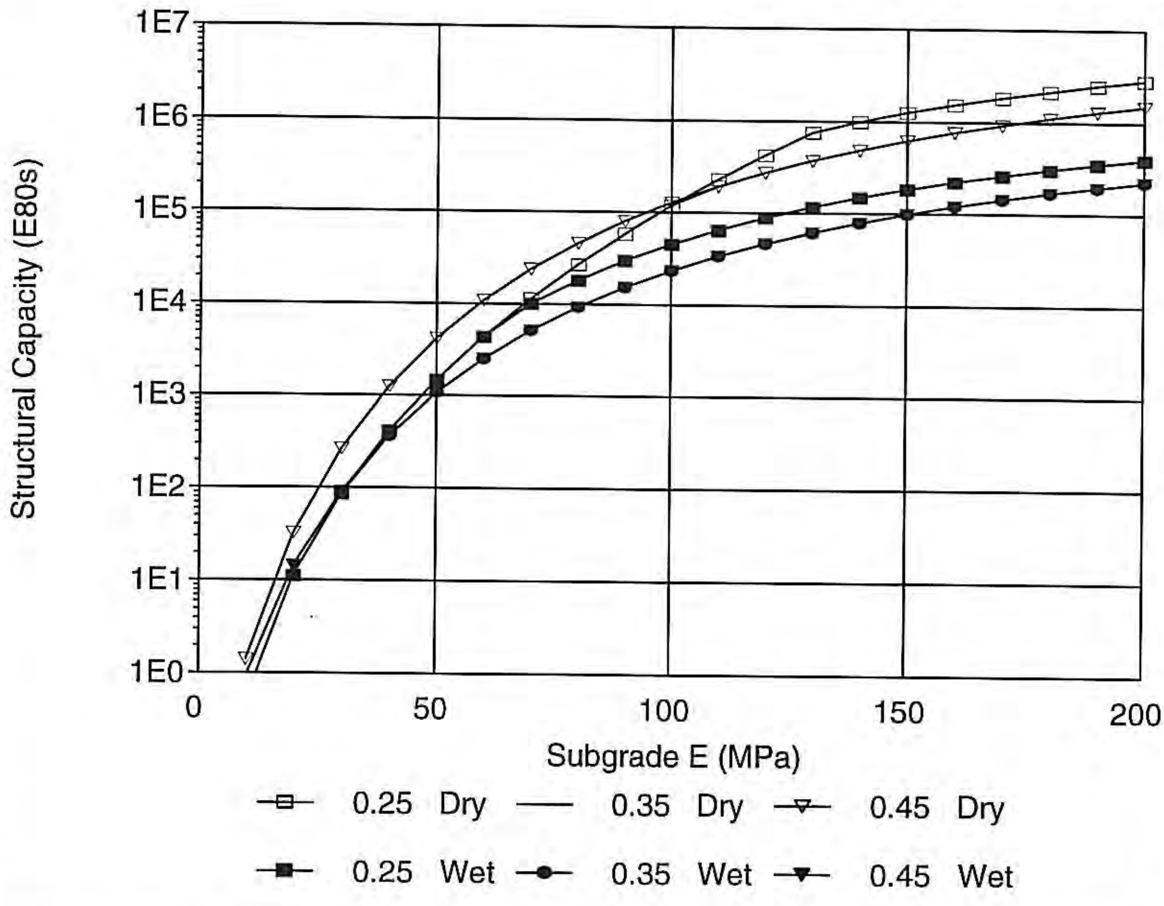

Figure 5.2 : Effect of Subgrade u

The effects of varying the subgrade Poisson's ratio (u), shown in Figure 5.2, are fairly interesting. If u is below the standard (0,35) then the structural capacity of the subbase increases but not that of the subgrade, this is because the subgrade is more rigid and therefore absorbs more of the stress - however the vertical subgrade strain remains constant. If u is higher than the standard the subgrade is more flexible and so absorbs less stress - however the stress in the subbase stays the same, only the subgrade strain is decreased. This behaviour is due to the assumed Poisson's ratio of 0,35 for the subbase. If the subgrade has a higher Poisson's ratio than the subbase it will have the same stresses but lower strains and if u is lower than the subbase will have the same strains but higher stresses.

This implies that the use of an assumed standard value of u can have important ramifications in the analysis of a pavement and does not have as low a sensitivity as usually assumed.

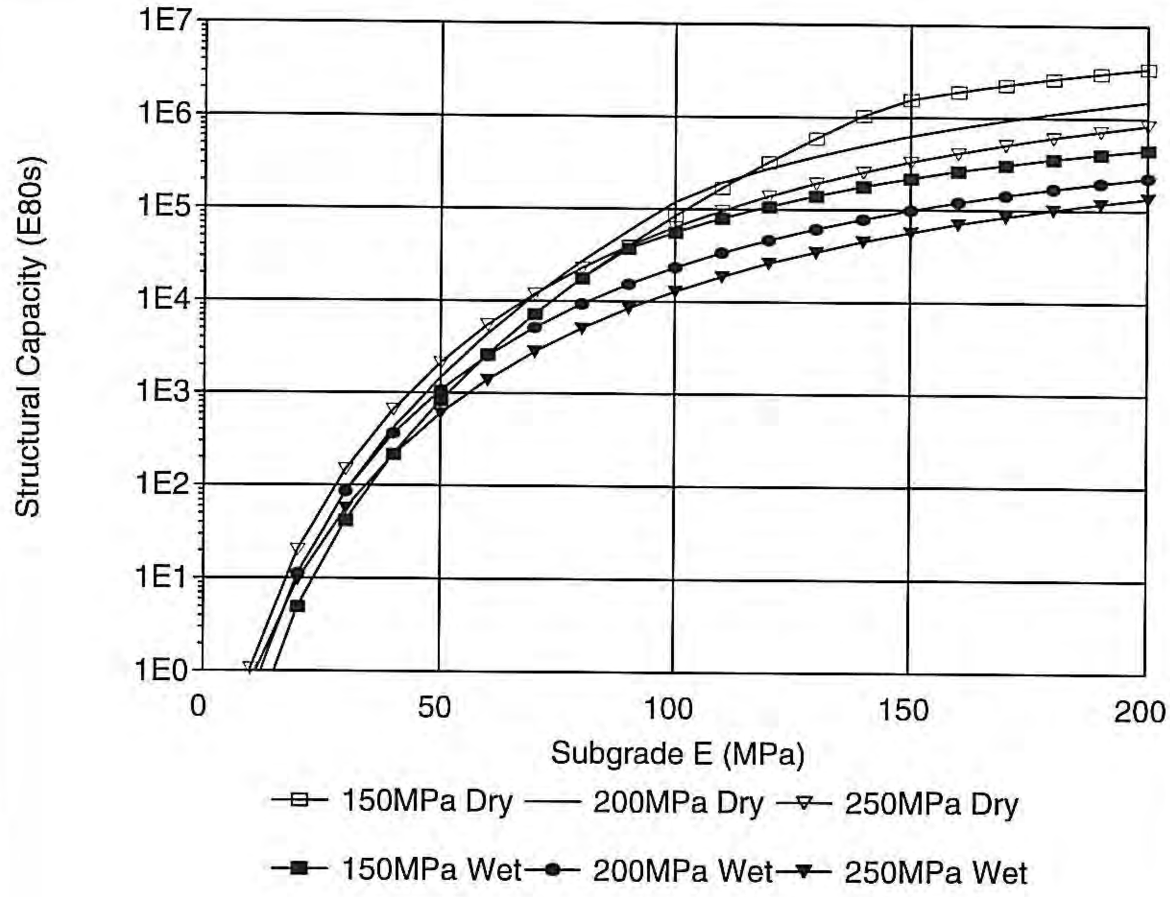

Figure 5.3 : Effect of Subbase Elastic modulus

The effects of varying the subgrade E modulus show some of the shortcomings in the design method (Figure 5.3). If the subbase E modulus is below the standard (200 MPa) then the layer is weak and so absorbs less of the stresses; this leads to higher subgrade strains and a lower predicted structural capacity initially, but the lower shear stress in the subbase gives it a higher structural capacity. If the subbase modulus is increased it absorbs more of the stress and so its predicted structural capacity is lower, but it protects the subgrade and so subgrade structural capacity is higher. In reality the structural capacity of the subbase would decrease as it got weaker because the weaker layer would undergo plastic deformation under the loading.

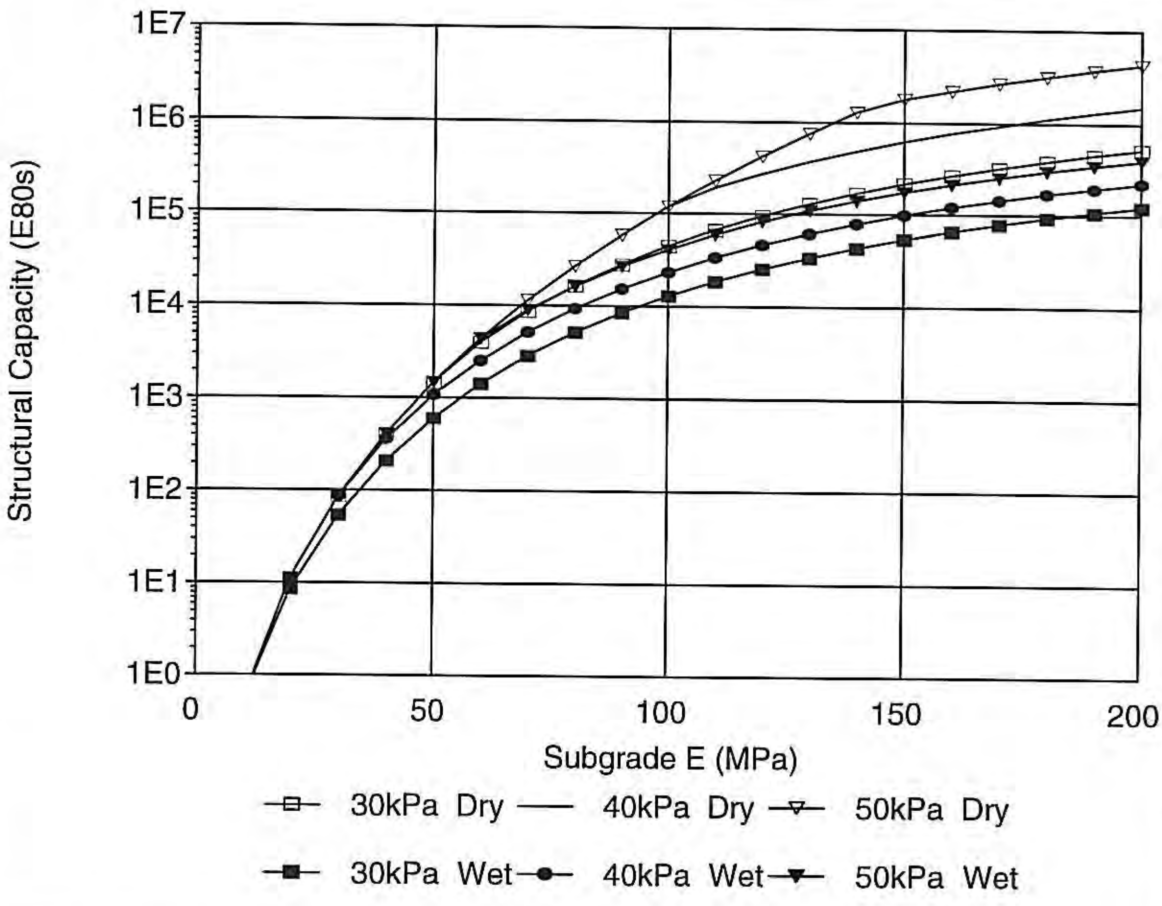

Figure 5.4 : Effect of Subbase cohesion

The effects of varying subbase cohesion are to increase the predicted structural capacity as the cohesion is increased. The predicted structural capacity of the subgrade is not changed (Figure 5.4).

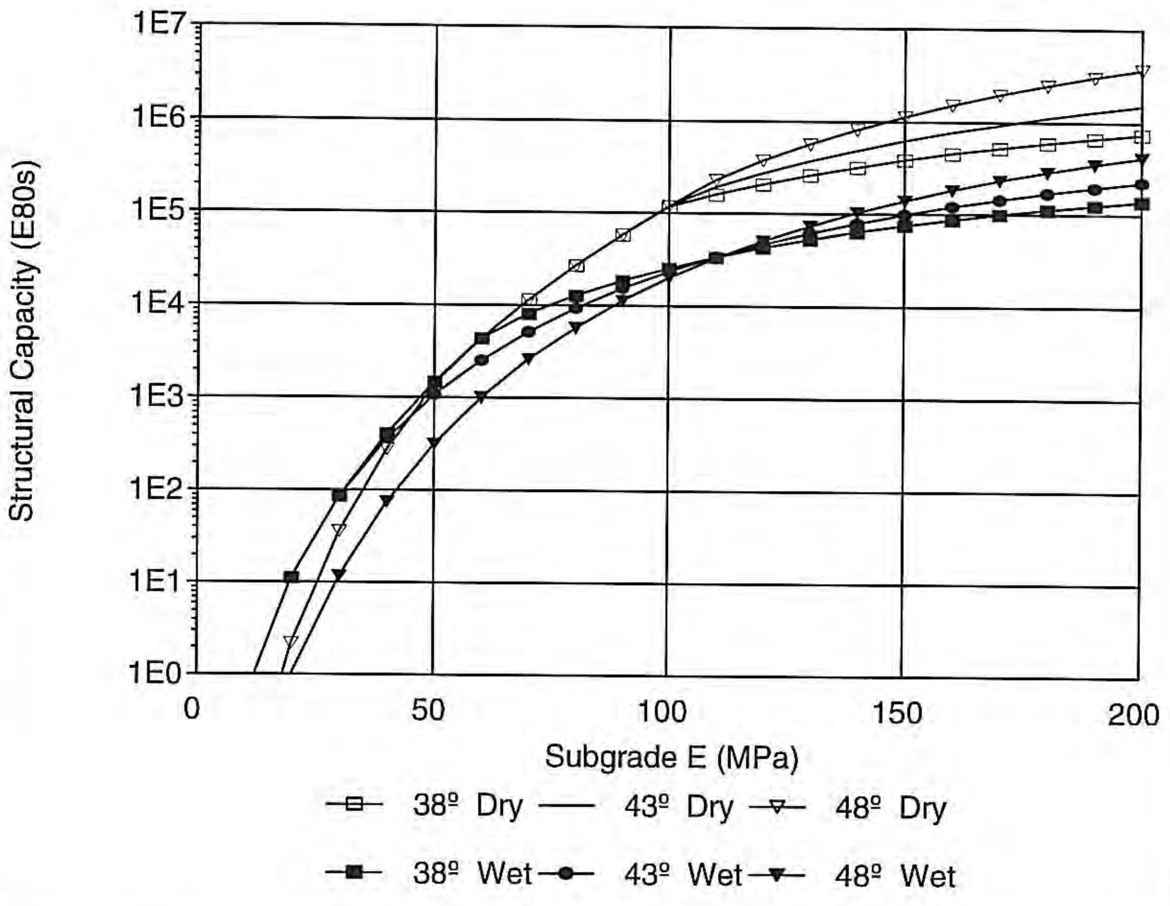

Figure 5.5 : Effect of Subbase ɸ

As ɸ is increased so the structural capacity of the subbase is increased (Figure 5.5). The structural capacity of the subgrade is not affected. At very low subgrade E values the structural capacity of the subbase falls below the structural capacity of the subgrade - this is due to predicted tensile stresses in the subbase that reduce the factors of safety. This demonstrates another problem with the Factor of Safety method: Mohr-Coulomb failure plane theory is not valid for tensile stresses, the only criteria for failure under tension are that the tensile stresses do not exceed the cohesion (ie the failure envelope on the diagram extends horizontally backwards in the tensile region). It is possible to generate tensile forces in a granular layer (because of cohesion) but those forces should not be relied on in the design since the material may become soaked and loose its cohesion.

5.4.2 Effect of Moisture on Elastic Moduli

The elastic moduli of the granular materials are assumed from standard tables, as given in Table 5.4. These values do not take recent findings[73] about the in-situ moisture content of pavement layers into account. These show that the equilibrium moisture content (EMC) of granular materials under a pavement layer is usually lower than the optimum moisture content (OMC), which can result in significant increases in the elastic modulus of the material. However there is still no model for predicting the elastic modulus at different densities. The Proctor compaction results can be used to give an indication of how CBR, and thus elastic modulus, vary with lower compaction but there is no set method to get an accurate value for use as an input parameter. K-mould testing[76] has shown that E values tend to decrease when the material is first loaded and then remain constant or decrease as the material is loaded further.

There have been several attempts to model the relationship between material strength, normally given by the CBR, and the moisture and density of the material. The most recent of these is that given by Emery[73], which relates soaked CBR to in situ CBR at different moisture contents. The materials were compacted using Mod AASHTO compaction for G1 to G5 materials and Proctor compaction for G6 to G10 materials, although the OMC listed is Mod AASHTO OMC. This is the reason that the G6 to G10 materials can be more than 110% OMC, since at the lower compaction they can absorb more moisture. Relationships are given for CBR_u and for resilient modulus.

$$E=187(e^{-0.59\frac{EMC}{OMC}})(CBR_s)^{0.20} \qquad \text{.....Eq. 1}$$

$$CBR_u=59.13(e^{-1.33\frac{EMC}{OMC}})(CBR_s)^{0.46} \qquad \text{.....Eq. 2}$$

where:

CBR_u = Unsoaked CBR

E = Resilient Modulus (MPa)

CBR_s = Soaked CBR

OMC = Optimum Moisture Content at Mod AASHTO compaction

EMC = Equilibrium Moisture Content of Layer

Table 5.5 : Maximum Moisture Contents[73]	
Material	Soaked Moisture Content (as a % of OMC)
G1-G4	110%
G5	110-150%
G6	150%
G7	150-170%
G8	170-200%
G9	200-220%
G10	>220%

Research around the DCP test[77] has also resulted in relationships to predict resilient modulus and in situ CBR from the DN value. Combining the formulae given above and these relationships the following relationships are obtained.

$$E=9.778(CBR_u)^{0.836} \qquad \text{.....Eq. 3}$$

$$E=296(e^{-1.11\frac{EMC}{OMC}})(CBR_s)^{0.385} \qquad \text{.....Eq. 4}$$

with the same notation as above.

5.4.3 Influence of Compaction on Pavement Performance

The compaction of pavement layers is important in that as the density of a material increases its bearing capacity and elastic modulus also increases. Also the compaction density of the material determines the amount of permanent deformation that can take place in a layer due to traffic compaction and therefore the amount of rutting that can take place in the pavement, as well as affecting the permeability and therefore the water absorption capacity of the material.

The standard compaction specifications for different layers in the pavement, as shown in Table 5.6, are given by TRH4[1] and TRH14.[4] Only granular materials are covered since these encompass all of the locally available materials.

Table 5.6 : Minimum compaction requirements[4]	
Layer	Compacted density (% Mod AASHTO)
Base (G4)	98%
Subbase	95%
Selected Layer	93%
Subgrade	
Top 200 mm	90%
within material depth	85%
Fill	90%

When Low Volume Roads with light structures are constructed, or when the in situ material has a high strength and is used to replace some of the lower structural layers, these densities are often used without any consideration for possible rut formation. It is often found that a base course, compacted to 98% Mod AASHTO is constructed directly on a high quality subgrade that is only compacted to 90% Mod AASHTO, since the specification only requires that the subgrade be compacted to this level.

A quick comparison of the possible rut formation can be obtained by assuming that the layers will increase in compaction by 1 per cent in a normal pavement, and in the sub-standard pavement will increase to the same as those in the normal pavement. Table 5.7 shows the compaction and permanent deformations in each of the layers.

Table 5.7 : Rut formation due to traffic compaction

Layer	t (mm)	Normal			Sub-standard		
		Compaction (% mod AASHTO)		Rut (mm)	Compaction (% mod AASHTO)		Rut (mm)
		Before	After		Before	After	
Base	150	98%	99%	1,5	98%	99%	1,5
Subbase	150	95%	96%	1,5	90%	96%	9,0
Select Layer	150	93%	94%	1,5	90%	94%	6,0
Subgrade	200	90%	91%	2,0	85%	91%	12,0
Material Depth	150	85%	86%	1,5	85%	86%	1,5
Total				8,0			30.0

As can be seen from this it is vitally important that the compaction profile specified is achieved, along with the strength profile, or else the pavement will undergo a rapid bedding in phase and premature failure. The compaction profile is also important since the upper layers can only be compacted sufficiently if the lower layers provide a reaction to the compaction force.

5.5 OVERVIEW OF LOW VOLUME ROAD TEST RESULTS

In a major study of low volume roads[2,78,79] to determine specifications for the use of marginal base course materials a large amount of data on in situ conditions in these roads, along with material properties in the Inner Wheel Track (IWT), Outer Wheel Track (OWT) and the Centre Line (CL) was collected. Rut depths and traffic counts were also obtained. This data was then used to analyse the pavements using the various design methods and compare the predicted structural capacities with the design structural capacities. The results from this study are summarised here to give an indication of how the moisture condition and in situ densities compare to those normally specified.

Table 5.8 : Average results for in-situ compaction, moisture content and bearing capacity

Position	Moisture content (% mod OMC)			Moisture content (% predicted EMC)			Saturation ratio (S_r) (%)		
	OWT	IWT	CL	OWT	IWT	CL	OWT	IWT	CL
Base	87,24	83,00	79,53	129,92	121,26	118,54	49,36	44,16	44,38
Subbase	99,40	90,67		137,60	127,88		49,92	47,31	
Subgrade	103,71	92,39		141,28	121,85		46,76	42,73	
Average	96,92	88,51		136,45	122,88		48,33	44,22	
Position	OMC CBR			Sat CBR			DCP CBR		
	OWT	IWT	CL	OWT	IWT	CL	OWT	IWT	CL
Base	77,04	83,78	94,96	86,48	76,33	87,85	88,00	131,73	116,20
Subbase	80,03	97,60		61,53	68,20		68,33	100,58	
Subgrade	69,55	67,27		54,85	51,20		73,91	59,24	
Average	74,40	79,78		67,57	64,17		77,43	90,45	
Position	Compaction (% mod MDD)								
	OWT			IWT			CL		
Base	98,46			96,52			98,25		
Subbase	96,31			96,28					
Subgrade	93,55			92,88					
Average	95,87			94,95					

The average compaction increased just under 1 per cent from the Inner Wheel Track to the Outer Wheel Track. The normal standard densities were achieved in all layers except the base course in the Inner Wheel Track. The moisture contents were all over the predicted Equilibrium Moisture Content (PEMC), and were on average very close to OMC. The material was on average 48 per cent saturated in the OWT.

The CBR results show the poor repeatability of CBR testing. The base course CBR was on average lower at OMC than under soaked conditions, which is not likely[b]. The average CBR of the subbase was also higher than that of the bases in the IWT, but this is because not all the roads had a subbase. The CBRs vary widely for the OWT, IWT and CL samples, which should agree fairly closely although the materials were marginal with respect to durability. The possible differential degradation of the materials in the different areas of the road could contribute to variations in CBR as recorded. Significant variability in materials from different wheel paths and adjacent areas of the road was noted frequently during sampling, particularly as the layers often consisted of differing, poorly mixed materials which showed segregation/separation.

Together these results show that the material being used in low volume roads, especially roads constructed with local materials is very variable in its properties, which has resulted in many of these roads failing prematurely since the material was either not strong enough or was not compacted properly. The moisture condition in low volume roads is wetter than in stronger pavements, but designing on OMC CBR values rather than soaked CBR values is still far closer to the actual conditions. The material is not being sufficiently compacted because, although the specified densities are being reached, the required density profile is not. The layer thickness in the roads also varies widely, which shows that construction standards are also low.

Many of the roads in the study consisted of very light pavement structures that have performed very well. The lightest structure analysed was a 150 mm G6 base on a G7 subgrade, which had taken 35 000 E80s at the time of testing and developed a 20 mm rut. This is despite the fact that the compaction in the base had increased from 97 per cent in the IWT to 102 per cent in the OWT, and in the subgrade from 86 per cent to 98 per cent, although the climate was very dry, and the pavement was the only pavement whose moisture condition was close to EMC. The standard mechanistic design method predicted a structural capacity of 2 200 E80s and a mechanistic design using OMC strengths predicted a structural capacity of 86 000 E80s, which may have been achieved if the compaction had been better.

5.6 CONCLUSIONS AND RECOMMENDATIONS

The environmental and traffic factors are the primary influences on the performance of any road, however in many cases these cannot be quantified accurately enough to develop an appropriate pavement design with confidence. More work needs to be done in these areas to improve the design of low volume roads. There is also an inadequate understanding of the behaviour of low volume roads and the materials used in them, and more work needs to be done to find

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[b] It should be noted that the CBR at OMC was extrapolated from CBR tests carried out on the moulds used to determine the OMC and MDD as often there were not actual tests done exactly at OMC. This could account for the discrepancies in the test results.

relationships between the moisture and density of a material and its bearing capacity, and to its deformation under repeated loading.

The standards set for the design of low volume roads are too high, and the pavement structures can be reduced. The strengths of the materials can also be determined at OMC conditions and in situ density, or at in situ density and saturation ratio of 50 per cent.

The two most important factors in determining the structural capacity of the pavement are drainage and compaction. If the drainage of the pavement is not adequately designed, constructed or maintained than the road will fail. Also cracks in the surface of the road must be sealed at regular intervals because once the road begins to deteriorate it is very difficult to fix the problems without major rehabilitation work. During construction the compaction standards and layer thickness must be carefully controlled. The compaction profile specified must be maintained; if a subbase layer is omitted because the subgrade is of adequate strength then the subgrade compaction must be that of a subbase and not a selected layer or else the pavement may rut quickly under traffic compaction. If possible the layers should be compacted to refusal density, since the extra rolling does not significantly increase the cost and can be counterbalanced against the cost of the material that would have had to be included to increase the pavement structural capacity by the same amount. However for higher densities to be achieved the material must be compacted at OMC, which again requires good quality control on site.

A secondary problem associated with reducing the pavement structure is that it will lead to higher surface deflections under loading, which will cause problems with cracking in the surface seal (particularly stiff and aged seals) and also generate high shear forces between the base and the seal which could cause the seal to separate from the base. The use of modified binders for surface treatments will go a long way towards reducing the risk of premature failure of the surfacing due to high deflections.

It is clear from this study that there are still a number of aspects regarding the optimum design of low volume roads which need to be investigated. These include: a satisfactory design method - current mechanistic design methods are essentially inappropriate, and better predictions of E and υ at various stress levels and moisture regimes.

In the interim the following design method is proposed, based on the DCP design method.[77] The first stage is to fix the compaction of the layers not by their thickness or type (base, subbase, etc.), but to fix the minimum compaction according to depth, as shown in Figure 5.6. The in-situ compaction must be better than this through the entire layer. These values are however minimum values and not target values, if a higher compaction can be achieved economically, which is normally the case, then it must become the target value. In cases where the top of the layer has a lower density than the bottom (which can occur with vibratory compaction) then the surface must be compacted using static compaction equipment.

CBR curves are given for three balance numbers (B) (20, 30 and 40) since a lower balance number can be used if the subgrade is consistently strong. The DCP design formulae were however only developed for balance numbers in the region of 40, but research indicates that pavements with lower balance numbers seem to perform adequately. The in situ material CBR required at the centre of the layer is then determined from Figures 5.7 to 5.9 (depending on the desired balance number). The strength profile is determined by the traffic and moisture condition as given in literature on the DCP design method.

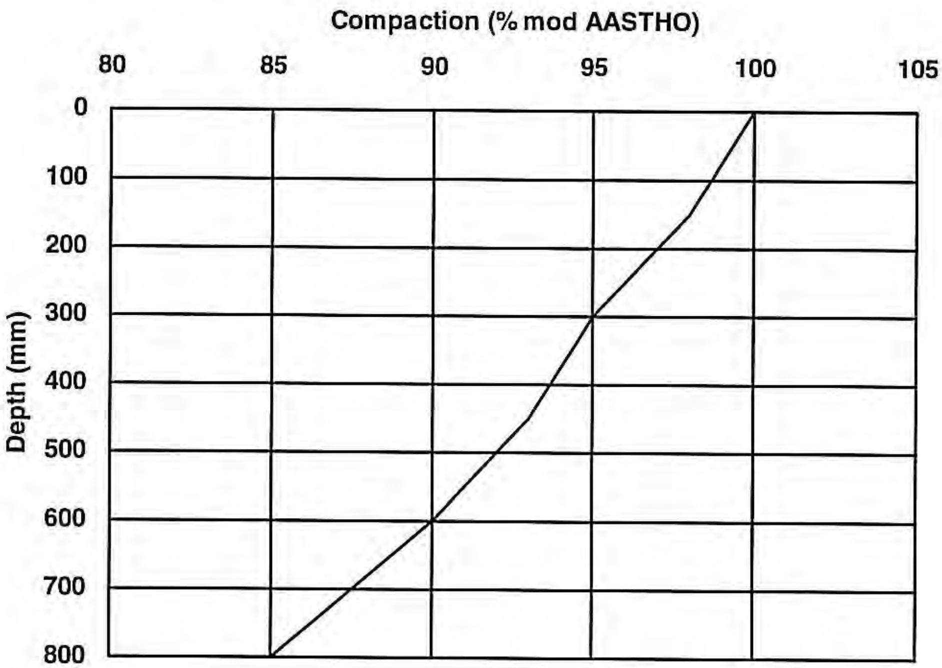

Figure 5.6 : Minimum Compaction with Depth

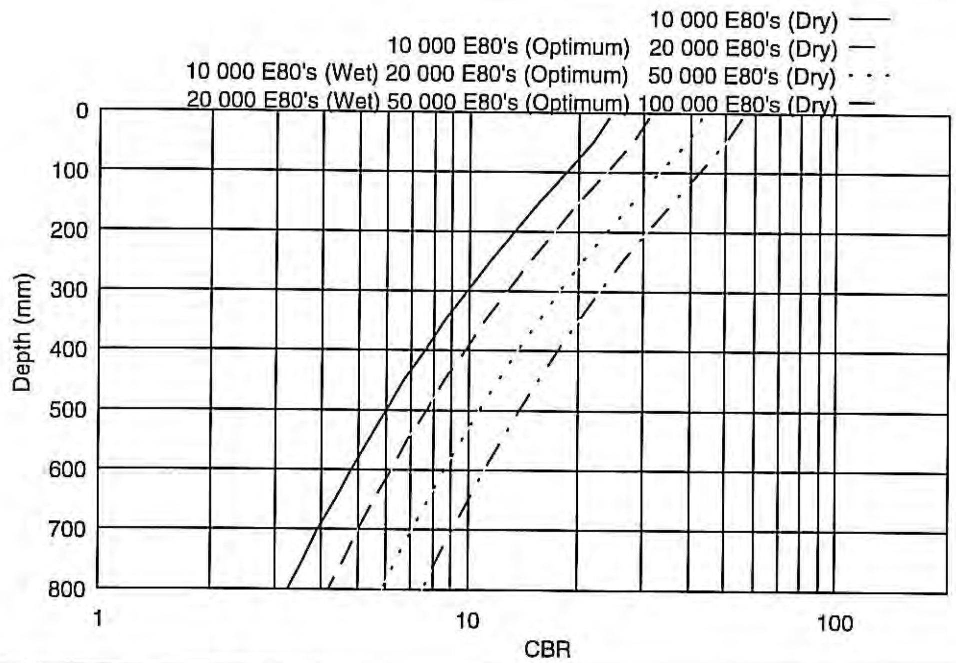

Figure 5.7 : CBR curves for B=20

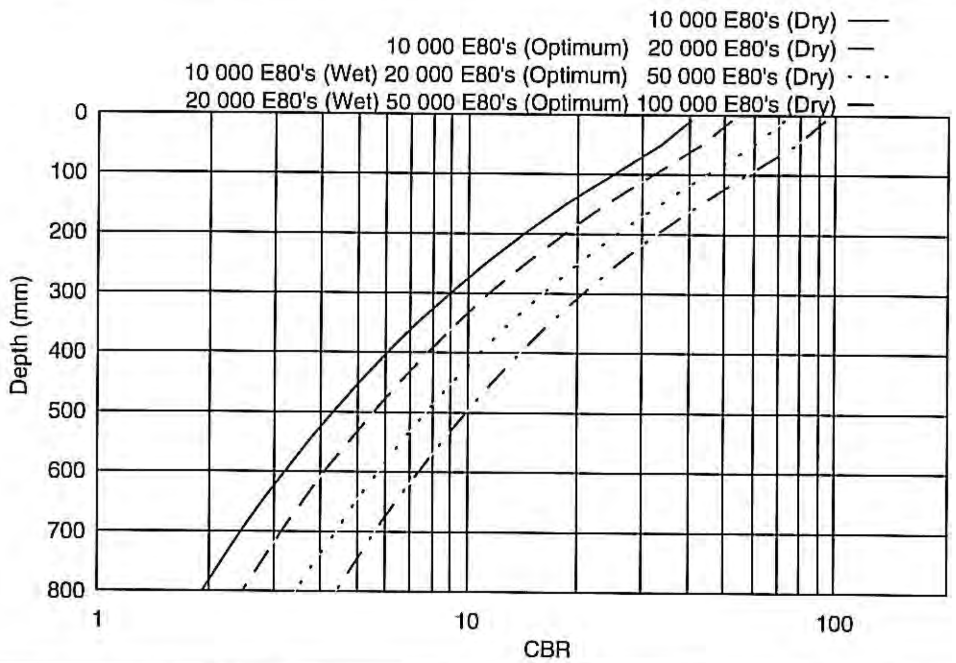

Figure 5.8 : CBR curves for B=30

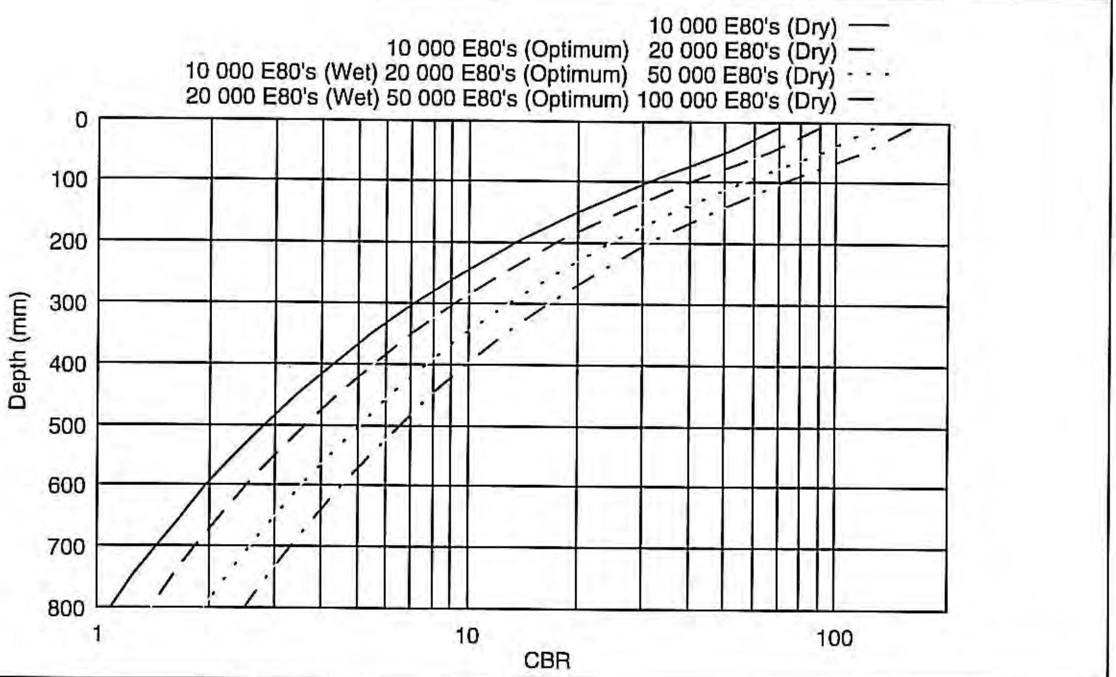

Figure 5.9 : CBR curves for B=40

6. GENERAL

6.1 CONCRETE

In view of the need for job creation and the employment of local residents in rural areas, labour intensive techniques should be utilised as far as possible. The use of concrete for structures other than causeways and minor drainage structures requires a high capital equipment component and relatively concentrated quality and construction control. In most cases unprocessed natural materials would appear to be unsuitable and the cost of processing the local materials combined with the high cost of cement are likely to make alternative construction techniques more appropriate.

The use of innovative techniques such as the Hyson cell concept[31] with a lean concrete or cement grout could have application in certain cases. The economics of this process in terms of life-cycle costs would need to be evaluated against alternative treatments.

6.2 BITUMINOUS SURFACINGS

It has been clearly shown that labour-enhanced construction of bituminous surfacings is viable, practical and cost-effective[80]. Road surfacings can in fact generate a high level of employment and skills transfer. However, a number of organisation, labour and training issues need to be addressed before labour-enhanced construction of surfacings can be implemented successfully[80].

6.3 LAYER WORKS

Layer works can make full use of local materials but are not particularly cost-effective in terms of labour-enhanced construction methods[80]. It has been clearly shown that high densities, low material variability and good construction control are necessary for successful performance of low volume roads. These are all difficult (but not always insurmountable) problems to overcome using labour enhanced techniques.

6.4 ECONOMIC INFLUENCES

Haulage is one of the major components of the cost of construction materials. By using local materials as far as possible this component can be considerably reduced. However, the balance between the economics of hauling better quality material and using the local material needs to be carefully considered.

A detailed life-cycle cost analysis of all the possible options needs to be carried out for each project. This needs to be viewed in relation to the maintenance capability and environmental factors prevailing at the site.

6.5 RISK FACTORS

Departure from established material quality specifications will indubitably carry an increased level of risk of failure. This risk should be calculated and not a gamble[3]. There are, however, a number of ways of managing this risk.

- It is imperative that the staff resident during construction are aware of the assumptions that went into the design process and are thus able to identify any deviations timeously during construction. Aspects such as moisture- and density-sensitivity of the materials should be clearly identified and their implications understood.
- The control of construction quality should be increased in comparison with traditional construction procedures and not decreased which is what typically happens.
- Particular attention should be paid to the design, construction and maintenance of good drainage to ensure that the pavement structure does not become excessively wet. *This should extend to the maintenance of the shoulders (a useful labour intensive activity) where many moisture problems originate.*
- During the design process sensitivity analyses should be carried out to determine what the implications of a change in the material properties or construction quality will be.
- It is suggested that end-result specifications are used as far as possible. In this way, densities achieved, in situ strengths and moisture contents and the finish of the pavement are used as acceptance criteria. Only those sections of road where the design objectives have been achieved are thus accepted.
- The probability of a change in certain parameters related to the in situ moisture content can be evaluated to actually quantify what is probably the major contributor to premature failure[73].

The political and engineering acceptance of a philosophy of increased risk of failure due to an infrequent combination of adverse circumstances has enabled sealed roads to be provided over larger distances at traffic counts as low as 50 to 100 vpd in for example Australia which would normally have been prohibitively expensive and impossible to provide had normal standards been applied[3].

Because of differences and uncertainties of construction and traffic and weights between industrialised and developing countries, one either has to adopt a larger factor of safety or accept a greater risk of failure[3]. The risk is further compounded by differences in types of geological materials used, uncertainties regarding their performance and the best methods for their characterisation.

7. CONCLUSIONS

As the cost of providing roads to traditional standards increases a smaller length of road can be constructed with the available funding. The use of local materials in preference to the importation of traditional high quality materials which need to be transported can result in significant cost savings. However, this is associated with an increased risk of premature distress or failure. A compromise needs to be found between the probability of premature distress or failure (ie increased maintenance costs) and the cost of improving the structural capacity.

The use of concrete in low volume roads is very limited and is probably only cost-effective for low water crossings, mass concrete wing-walls and erosion protection measures and possibly thick culverts and culvert inverts. Innovative applications using techniques such as Hyson cells may prove useful but will need economic evaluation. The effect of reducing the aggregate quality will be to decrease the strength and durability of the concrete and probably to increase the shrinkage. In addition, the water:cement ratio may be effected which could result in an unacceptably high increase in the cement content to achieve the same strength. This may raise the cost of the concrete substantially. The use of econocrete and rollcrete is probably only economically viable in limited areas and as it is equipment intensive should be avoided in rural areas where job-creation is a priority. If the use of local materials is proposed each material needs to be investigated in the laboratory to ensure that the design strengths can be obtained with an economical cement content. Only if this is confirmed should the workability and durability aspects be investigated and then the material can be used.

Bituminous surfacings are currently tightly specified and comparison with other countries indicates that certain areas of relaxation are possible. These are mostly with respect to the grading and aggregate strength. The effect of relaxing these requirements needs to be quantified and the economic benefits evaluated in terms of the increased risk. It is anticipated that the use of modified bitumens with marginal aggregates will result in lower life-cycle costs for surfacings. Methods of treating marginal materials to improve their quality should be investigated.

Detailed analysis of the structural behaviour of pavement layers has shown that current techniques have some major deficiencies. It has been clearly identified that thin pavement structures which make the maximum use of the in situ and local materials can provide satisfactory performance provided the drainage and construction quality is well controlled. Analyses have shown that it is important to ensure some degree of pavement balance in order to obtain good performance and the modulus ratio between adjacent pavement layers should be controlled.

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APPENDIX A: CONCRETE NATURAL GRAVEL LABORATORY RESULTS

APPENDIX A: CONCRETE NATURAL GRAVEL LABORATORY RESULTS

A.1 CORRECTION OF CYLINDER CRUSHING STRENGTH VALUES FOR SPECIMEN HEIGHT DIFFERENCES

The height of the cylindrical samples prepared using the split moulds was not fixed. The *maximum* height of the specimens in the split moulds is 125 mm. As the height was less than twice the diameter, the measured crushing strength values had to be corrected for the difference in height, because height/diameter ratio of the sample has an effect on the measured strength.

For the determination of the compressive strength of cylindrical concrete specimens, ASTM C 39-86 normally requires samples with height/diameter ratio of 2. However, if the sample height is less the measured strength is reduced by the following reduction factor:

Height/Diameter	**Factor**
2,00	1,00
1,75	0,98
1,50	0,96
1,25	0,93
1,00	0,87

These values were extrapolated to cover samples down to 90 mm in height and corrected for equivalent cube strength by multiplying by 1,2 (see Figure A.1 and values in Table A.1).

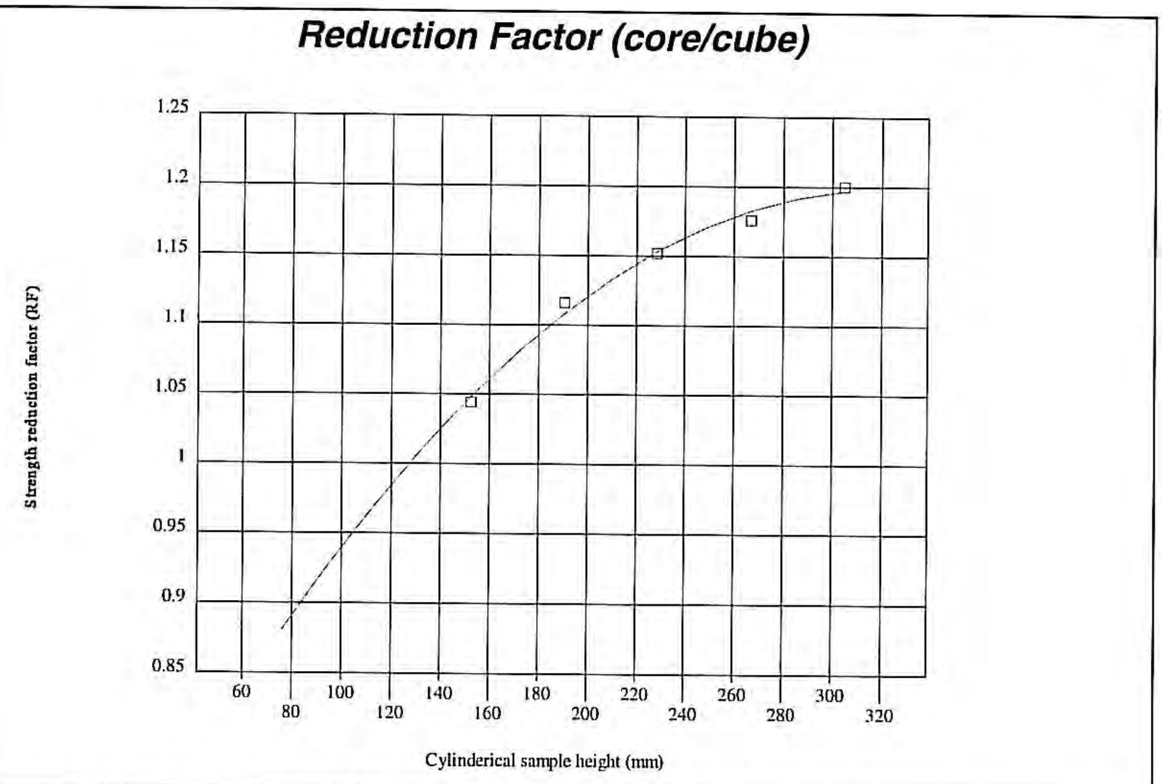

Figure A.1 : **Equivalent cube strength reduction factor for cylindrical concrete specimens against the height/diameter ratio of the specimen**

A.2 MIX DESIGNS WITH COMPACTABILITY SOFTWARE PACKAGE

Bultfontein gravel

MMD(mod)	2010	20 MPa	P<4.75=	79.1			
EffMC(%)	MC*	c/w	cement*	fines*	total*	ARD	cem(%)
8	160.8	1.377	221.4	1589.9	1811.3	2.707	11.016
9	180.9	1.377	249.1	1589.9	1839.0	2.712	12.393
10	201.0	1.377	276.8	1589.9	1866.7	2.716	13.770
14	281.4	1.377	387.5	1589.9	1977.4	2.732	19.278
15	301.5	1.377	415.2	1589.9	2005.1	2.735	20.655
16	321.6	1.377	442.8	1589.9	2032.8	2.739	22.032
17	341.7	1.377	470.5	1589.9	2060.4	2.742	23.409

	EffMC(%)	8	9	10	11	12	13	14
Sieve(mm)	cem(%)	11.016	12.393	13.77	19.278	20.655	22.032	23.409
75.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
63.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
53.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
37.500	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
26.500	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
19.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
13.200	99.50	99.55	99.56	99.56	99.58	99.59	99.59	99.59
4.750	79.10	81.17	81.40	81.63	82.48	82.68	82.87	83.06
2.000	56.60	60.91	61.39	61.85	63.61	64.03	64.44	64.83
0.425	37.70	43.88	44.57	45.24	47.77	48.37	48.95	49.52
0.075	21.80	29.56	30.42	31.26	34.44	35.19	35.92	36.63

Estimated quantities for 100 mm high cylinders (Diameter 152,4 mm)(20 MPa)

MDD(mod)	MC(%)	EffMC(%)	cem(%)	grav(g)	cem(g)	water(g)	Total	c/w
2010	9	8	11.016	3302.7	363.8	297.2	3963.8	1.377
2010	10	9	12.393	3262.2	404.3	326.2	3992.8	1.377
2010	11	10	13.770	3222.8	443.8	354.5	4021.0	1.377
2010	15	14	19.278	3073.9	592.6	461.1	4127.6	1.377
2010	16	15	20.655	3038.9	627.7	486.2	4152.8	1.377
2010	17	16	22.032	3004.6	662.0	510.8	4177.3	1.377
2010	18	17	23.409	2971.0	695.5	534.8	4201.3	1.377

Estimated quantities for 153 mm cubes (20 MPa)

MDD(mod)	MC(%)	EffMC(%)	cem(%)	grav(g)	cem(g)	water(g)	Total	c/w
2010	9	8	11.016	6484.6	714.3	583.6	7782.6	1.377
2010	10	9	12.393	6405.2	793.8	640.5	7839.5	1.377
2010	11	10	13.770	6327.7	871.3	696.0	7895.0	1.377
2010	15	14	19.278	6035.5	1163.5	905.3	8104.3	1.377
2010	16	15	20.655	5966.6	1232.4	954.7	8153.6	1.377
2010	17	16	22.032	5899.2	1299.7	1002.9	8201.8	1.377
2010	18	17	23.409	5833.4	1365.5	1050.0	8249.0	1.377

* = Kg/m^3

Bultfontein gravel

MMD(mod)	2010	30 MPa	P<4.75=	79.1			
EffMC(%)	MC*	c/w	cement*	fines*	total*	ARD	cem(%)
8	160.8	1.832	294.6	1589.9	1884.5	2.719	14.656
9	180.9	1.832	331.4	1589.9	1921.3	2.724	16.488
10	201.0	1.832	368.2	1589.9	1958.1	2.729	18.320
14	281.4	1.832	515.5	1589.9	2105.4	2.748	25.648
15	301.5	1.832	552.3	1589.9	2142.3	2.752	27.480
16	321.6	1.832	589.2	1589.9	2179.1	2.756	29.312
17	341.7	1.832	626.0	1589.9	2215.9	2.760	31.144

	EffMC(%)	8	9	10	11	12	13	14
Sieve(mm)	cem(%)	14.656	16.488	18.320	25.648	27.480	29.312	31.144
75.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
63.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
53.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
37.500	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
26.500	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
19.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
13.200	99.50	99.56	99.57	99.58	99.60	99.61	99.61	99.62
4.750	79.10	81.77	82.06	82.34	83.37	83.61	83.84	84.06
2.000	56.60	62.15	62.74	63.32	65.46	65.96	66.44	66.91
0.425	37.70	45.66	46.52	47.35	50.42	51.13	51.82	52.49
0.075	21.80	31.80	32.87	33.91	37.76	38.66	39.53	40.37

Estimated quantities for 100 mm high cylinders (Diameter 152,4 mm)(30 MPa)

MDD(mod)	MC(%)	EffMC(%)	cem(%)	grav(g)	cem(g)	water(g)	Total	c/w
2010	9	8	14.656	3197.9	468.7	287.8	3954.3	1.832
2010	10	9	16.488	3147.6	519.0	314.8	3981.3	1.832
2010	11	10	18.320	3098.8	567.7	340.9	4007.4	1.832
2010	15	14	25.648	2918.1	748.4	437.7	4104.3	1.832
2010	16	15	27.480	2876.2	790.4	460.2	4126.7	1.832
2010	17	16	29.312	2835.4	831.1	482.0	4148.6	1.832
2010	18	17	31.144	2795.8	870.7	503.2	4169.8	1.832

Bultfontein gravel

MMD(mod)	2010	40 MPa	P<4.75=	79.1			
EffMC(%)	MC*	c/w	cement*	fines*	total*	ARD	cem(%)
8	160.8	2.288	367.9	1589.9	1957.8	2.729	18.304
9	180.9	2.288	413.9	1589.9	2003.8	2.735	20.592
10	201.0	2.288	459.9	1589.9	2049.8	2.741	22.880
14	281.4	2.288	643.8	1589.9	2233.8	2.762	32.032
15	301.5	2.288	689.8	1589.9	2279.7	2.767	34.320
16	321.6	2.288	735.8	1589.9	2325.7	2.771	36.608
17	341.7	2.288	781.8	1589.9	2371.7	2.776	38.896

	EffMC(%)	8	9	10	11	12	13	14
Sieve(mm)	cem(%)	18.304	20.592	22.880	32.032	34.320	36.608	38.896
75.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
63.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
53.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
37.500	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
26.500	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
19.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
13.200	99.50	99.58	99.59	99.59	99.62	99.63	99.63	99.64
4.750	79.10	82.33	82.67	82.99	84.17	84.44	84.70	84.95
2.000	56.60	63.31	64.01	64.68	67.13	67.69	68.23	68.75
0.425	37.70	47.34	48.34	49.30	52.81	53.62	54.40	55.15
0.075	21.80	33.90	35.15	36.36	40.77	41.78	42.76	43.70

Estimated quantities for 100 mm high cylinders (Diameter 152,4 mm)(40 MPa)

MDD(mod)	MC(%)	EffMC(%)	cem(%)	grav(g)	cem(g)	water(g)	Total	c/w
2010	9	8	18.304	3099.2	567.3	278.9	3945.5	2.288
2010	10	9	20.592	3040.4	626.1	304.0	3970.6	2.288
2010	11	10	22.880	2983.8	682.7	328.2	3994.8	2.288
2010	15	14	32.032	2777.0	889.5	416.6	4083.1	2.288
2010	16	15	34.320	2729.7	936.8	436.8	4103.3	2.288
2010	17	16	36.608	2684.0	982.6	456.3	4122.8	2.288
2010	18	17	38.896	2639.8	1026.8	475.2	4141.7	2.288

Quartzite gravel (TPA2)

MMD(mod) EffMC(%)	2062 MC*	20 MPa c/w	P<4.75= cement*	62 fines*	total*	ARD	cem(%)
10	206.2	1.377	283.9	1278.4	1562.4	2.730	13.770
11	226.8	1.377	312.3	1278.4	1590.8	2.735	15.147
12	247.4	1.377	340.7	1278.4	1619.2	2.740	16.524
13	268.1	1.377	369.1	1278.4	1647.6	2.744	17.901
14	288.7	1.377	397.5	1278.4	1676.0	2.748	19.278
15	309.3	1.377	425.9	1278.4	1704.3	2.752	20.655
16	329.9	1.377	454.3	1278.4	1732.7	2.756	22.032

Sieve(mm)	EffMC(%) cem(%)	10 13.770	11 15.147	12 16.524	13 17.901	14 19.278	15 20.655	16 22.032
75.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
63.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
53.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
37.500	96.00	96.40	96.44	96.48	96.65	96.68	96.72	96.76
26.500	92.00	92.79	92.88	92.97	93.29	93.37	93.44	93.52
19.000	88.00	89.19	89.32	89.45	89.94	90.05	90.17	90.28
13.200	84.00	85.59	85.76	85.94	86.59	86.74	86.89	87.03
4.750	62.00	65.77	66.19	66.60	68.14	68.51	68.86	69.21
2.000	40.00	45.95	46.62	47.26	49.70	50.27	50.83	51.38
0.425	14.00	22.53	23.48	24.41	27.90	28.72	29.53	30.31
0.075	4.00	13.53	14.59	15.62	19.52	20.43	21.33	22.21

Estimated quantities for 100 mm high cylinders (Diameter 152,4 mm)(20 MPa)

MDD(mod)	MC(%)	EffMC(%)	cem(%)	grav(g)	cem(g)	water(g)	Total	c/w
2062	10	10	13.770	3306.1	455.3	330.6	4092.0	1.377
2062	11	11	15.147	3266.6	494.8	359.3	4120.7	1.377
2062	12	12	16.524	3228.0	533.4	387.4	4148.8	1.377
2062	13	13	17.901	3190.3	571.1	414.7	4176.1	1.377
2062	14	14	19.278	3153.5	607.9	441.5	4202.9	1.377
2062	15	15	20.655	3117.5	643.9	467.6	4229.0	1.377
2062	16	16	22.032	3082.3	679.1	493.2	4254.6	1.377

Estimated quantities for 153mm cubes (20 MPa)

MDD(mod)	MC(%)	EffMC(%)	cem(%)	grav(g)	cem(g)	water(g)	Total	c/w
2062	10	10	13.770	6415.3	883.4	641.5	7940.2	1.377
2062	11	11	15.147	6338.6	960.1	697.2	7995.9	1.377
2062	12	12	16.524	6263.7	1035.0	751.6	8050.3	1.377
2062	13	13	17.901	6190.5	1108.2	804.8	8103.4	1.377
2062	14	14	19.278	6119.0	1179.6	856.7	8155.3	1.377
2062	15	15	20.655	6049.2	1249.5	907.4	8206.0	1.377
2062	16	16	22.032	5980.9	1317.7	957.0	8255.6	1.377

Norite gravel (TPA1)

MMD(mod)	2234	20 MPa	P<4.75=	69		2.961	
EffMC(%)	MC*	c/w	cement*	fines*	total*	ARD	cem(%)
10	201.5	1.377	277.5	1422.8	1700.3	2.967	13.457
11	221.7	1.377	305.2	1422.8	1728.0	2.967	14.803
12	241.8	1.377	333.0	1422.8	1755.8	2.968	16.149
13	262.0	1.377	360.7	1422.8	1783.5	2.968	17.495
14	282.1	1.377	388.5	1422.8	1811.3	2.969	18.840
15	302.3	1.377	416.2	1422.8	1839.0	2.969	20.186
16	322.4	1.377	444.0	1422.8	1866.8	2.970	21.532

	EffMC(%)	10	11	12	13	14	15	16
Sieve(mm)	cem(%)	13.457	14.803	16.149	17.495	18.840	20.186	21.532
75.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
63.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
53.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
37.500	91.00	91.89	91.99	92.09	92.45	92.54	92.62	92.71
26.500	85.00	86.49	86.65	86.82	87.42	87.57	87.71	87.85
19.000	79.00	81.08	81.32	81.54	82.39	82.60	82.79	82.98
13.200	78.00	80.18	80.43	80.66	81.56	81.77	81.97	82.17
4.750	69.00	72.08	72.42	72.75	74.01	74.31	74.60	74.88
2.000	56.00	60.37	60.85	61.33	63.11	63.53	63.94	64.35
0.425	21.00	28.84	29.71	30.56	33.77	34.52	35.26	35.99
0.075	7.00	16.23	17.25	18.26	22.03	22.92	23.79	24.64

Estimated quantities for 100 mm high cylinders (Diameter 152,4 mm)(20 MPa)

MDD(mod)	MC(%)	EffMC(%)	cem(%)	grav(g)	cem(g)	water(g)	Total	c/w
2234	10	10	13.457	3591.8	483.4	351.0	4426.2	1.377
2234	11	11	14.803	3549.7	525.5	381.6	4456.7	1.377
2234	12	12	16.149	3508.5	566.6	411.5	4486.6	1.377
2234	13	13	17.495	3468.4	606.8	440.7	4515.8	1.377
2234	14	14	18.840	3429.1	646.1	469.2	4544.3	1.377
2234	15	15	20.186	3390.7	684.5	497.1	4572.2	1.377
2234	16	16	21.532	3353.1	722.0	524.3	4599.5	1.377

Estimated quantities for 153 mm cubes(20 MPa)

MDD(mod)	MC(%)	EffMC(%)	cem(%)	grav(g)	cem(g)	water(g)	Total	c/w
2234	10	10	13.457	6969.6	937.9	681.1	8588.6	1.377
2234	11	11	14.803	6887.9	1019.6	740.5	8647.9	1.377
2234	12	12	16.149	6808.0	1099.4	798.4	8705.9	1.377
2234	13	13	17.495	6730.1	1177.4	855.1	8762.5	1.377
2234	14	14	18.840	6653.9	1253.6	910.4	8817.9	1.377
2234	15	15	20.186	6579.4	1328.1	964.5	8872.0	1.377
2234	16	16	21.532	6506.5	1401.0	1017.4	8924.9	1.377

Decomposed dolerite gravel (NPAB)

MMD(mod)	2150	20 MPa	P<4.75=	52		2.827	
EffMC(%)	MC*	c/w	cement*	fines*	total*	ARD	cem(%)
10	203.1	1.377	279.7	1072.2	1352.0	2.862	13.010
11	223.4	1.377	307.7	1072.2	1379.9	2.865	14.311
12	243.8	1.377	335.7	1072.2	1407.9	2.868	15.612
13	264.1	1.377	363.6	1072.2	1435.9	2.870	16.913
14	284.4	1.377	391.6	1072.2	1463.8	2.872	18.214
15	304.7	1.377	419.6	1072.2	1491.8	2.875	19.515
16	325.0	1.377	447.5	1072.2	1519.8	2.877	20.816

	EffMC(%)	10	11	12	13	14	15	16
Sieve(mm)	cem(%)	13.010	14.311	15.612	16.913	18.214	19.515	20.816
75.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
63.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
53.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
37.500	93.00	93.69	93.77	93.85	94.13	94.20	94.26	94.33
26.500	87.00	88.29	88.43	88.57	89.10	89.23	89.35	89.47
19.000	79.00	81.08	81.32	81.54	82.39	82.60	82.79	82.98
13.200	73.00	75.68	75.98	76.27	77.36	77.62	77.87	78.12
4.750	52.00	56.76	57.29	57.81	59.76	60.22	60.67	61.10
2.000	46.00	51.36	51.95	52.54	54.73	55.24	55.75	56.24
0.425	30.00	36.95	37.72	38.47	41.31	41.98	42.64	43.28
0.075	20.00	27.94	28.82	29.68	32.93	33.70	34.44	35.17

Estimated quantities for 100 mm high cylinders (Diameter 152,4 mm)(20 MPa)

MDD(mod)	MC(%)	EffMC(%)	cem(%)	grav(g)	cem(g)	water(g)	Total	c/w
2150	11	10	13.010	3470.4	451.5	360.7	4282.6	1.377
2150	12	11	14.311	3430.9	491.0	389.0	4310.9	1.377
2150	13	12	15.612	3392.3	529.6	416.7	4338.6	1.377
2150	14	13	16.913	3354.6	567.4	443.7	4365.6	1.377
2150	15	14	18.214	3317.6	604.3	470.2	4392.1	1.377
2150	16	15	19.515	3281.5	640.4	496.1	4418.0	1.377
2150	17	16	20.816	3246.2	675.7	521.4	4443.3	1.377

Estimated quantities for 153 mm cubes(20 MPa)

MDD(mod)	MC(%)	EffMC(%)	cem(%)	grav(g)	cem(g)	water(g)	Total	c/w
2150	14	13	16.913	6509.2	1100.9	861.0	8471.2	1.377
2150	15	14	18.214	6437.6	1172.6	912.4	8522.5	1.377
2150	16	15	19.515	6367.5	1242.6	962.6	8572.7	1.377
2150	17	16	20.816	6298.9	1311.2	1011.7	8621.9	1.377
2150	18	17	22.117	6231.8	1378.3	1059.8	8670.0	1.377
2150	19	18	23.418	6166.1	1444.0	1106.9	8717.1	1.377
2150	20	19	24.719	6101.8	1508.3	1153.0	8763.2	1.377

Calcrete gravel (PPC)

MMD(mod) EffMC(%)	1849 MC*	20 MPa c/w	P<4.75= cement*	38.15 fines*	total*	2.551 ARD	cem(%)
14	271.0	1.377	373.2	705.4	1078.6	2.705	18.100
15	316.6	1.377	436.0	705.4	1141.4	2.721	21.143
16	337.7	1.377	465.0	705.4	1170.4	2.728	22.553
17	358.8	1.377	494.1	705.4	1199.5	2.735	23.962
18	379.9	1.377	523.2	705.4	1228.6	2.741	25.372
19	401.0	1.377	552.2	705.4	1257.6	2.747	26.781
20	422.1	1.377	581.3	705.4	1286.7	2.752	28.191

Sieve(mm)	EffMC(%) cem(%)	10 18.100	11 21.143	12 22.553	13 23.962	14 25.372	15 26.781	16 28.191
75.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
63.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
53.000	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
37.500	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
26.500	92.75	93.47	93.55	93.63	93.92	93.99	94.06	94.13
19.000	72.68	75.39	75.69	75.99	77.10	77.36	77.61	77.86
13.200	55.81	60.19	60.68	61.16	62.95	63.37	63.79	64.19
4.750	38.15	44.29	44.97	45.64	48.15	48.74	49.32	49.88
2.000	31.43	38.23	38.99	39.73	42.51	43.17	43.81	44.44
0.425	26.61	33.89	34.70	35.49	38.47	39.17	39.86	40.53
0.075	14.77	23.23	24.17	25.09	28.55	29.36	30.16	30.94

Estimated quantities for 100 mm high cylinders (Diameter 152,4 mm)(20 MPa)

MDD(mod)	MC(%)	EffMC(%)	cem(%)	grav(g)	cem(g)	water(g)	Total	c/w
1849	15.817	14	18.100	2855.9	516.9	424.1	3797.0	1.377
1849	16.817	15	21.143	2784.2	588.7	479.3	3852.1	1.377
1849	17.817	16	22.553	2752.2	620.7	501.9	3874.8	1.377
1849	18.817	17	23.962	2720.9	652.0	524.1	3896.9	1.377
1849	19.817	18	25.372	2690.3	682.6	545.7	3918.6	1.377
1849	20.817	19	26.781	2660.4	712.5	566.9	3939.7	1.377
1849	21.817	20	28.191	2631.1	741.7	587.6	3960.4	1.377

Estimated quantities for 153 mm cubes(20 MPa)

MDD(mod)	MC(%)	EffMC(%)	cem(%)	grav(g)	cem(g)	water(g)	Total	c/w
1849	15.817	14	18.100	5541.7	1003.0	823.0	7367.7	1.377
1849	16.817	15	21.143	5402.5	1142.2	930.0	7474.7	1.377
1849	17.817	16	22.553	5340.3	1204.4	974.0	7518.7	1.377
1849	18.817	17	23.962	5279.6	1265.1	1016.9	7561.7	1.377
1849	19.817	18	25.372	5220.3	1324.5	1058.9	7603.7	1.377
1849	20.817	19	26.781	5162.2	1382.5	1100.0	7644.7	1.377
1849	21.817	20	28.191	5105.5	1439.3	1140.2	7684.9	1.377

SAMPLE No. : TPA1 DATE : 1994/10/04

DESCRIPTION : Norite gravel

REMARK :

INPUT INFORMATION MATERIAL TYPE 2

GRADING Metric Units		ATTERBERG LIMITS (Casagrande Apparatus)		
SIEVE (mm)	% PASSING	L.L.	P.I.	L.S.
75	100,00	27,00	10,00	4,50
63	100,00			
53	100,00	DENSITY AND SHAPE INFORMATION		
37,5	91,00			
26,5	85,00	ARD (+4,75)	ARD (-4,75)	S B D
19,0	79,00			
13,2	78,00	2,961	2,961	0,00
4,75	69,00			
2,00	56,00	BRD (+4,75)	BRD (-4,75)	W F D
0,425	21,00			
0,075	7,00	2,961	2,961	0,00

OUTPUT PREDICTIONS MDD in kg/cub. metre

M D D (VIB)	O M C (VIB)	ZAVMC (VIB)	WA	C M C
2359,59	8,85	9,23 - 8,61	0,000	6,384
MDD Mod AASHTO	OMC Mod AASHTO	ZAVMC Mod.	WA	R S D
2276,94	8,79	11,20 - 10,15	0,000	2,961

C B R NOT PREDICTED WITHOUT "SBD" AND "WFD"

SAMPLE No. : TPA1/20/14 DATE : 1994/10/04

DESCRIPTION : Norite gravel concrete

REMARK :

INPUT INFORMATION MATERIAL TYPE 2

GRADING Metric Units		ATTERBERG LIMITS (Casagrande Apparatus)		
SIEVE (mm)	% PASSING	L.L.	P.I.	L.S.
75	100,00	27,00	10,00	4,50
63	100,00			
53	100,00	DENSITY AND SHAPE INFORMATION		
37,5	92,54			
26,5	87,57	ARD (+4,75)	ARD (-4,75)	S B D
19,0	82,59			
13,2	81,77	2,961	2,969	0,00
4,75	74,31			
2,00	63,53	BRD (+4,75)	BRD (-4,75)	W F D
0,425	34,52			
0,075	22,92	2,961	2,969	0,00

OUTPUT PREDICTIONS MDD in kg/cub. metre

M D D (VIB)	O M C (VIB)	ZAVMC (VIB)	WA	C M C
2308,09	9,83	9,92 - 9,62	0,000	7,284
MDD Mod AASHTO	OMC Mod AASHTO	ZAVMC Mod.	WA	R S D
2258,36	9,50	11,72 - 10,58	0,000	2,967

C B R NOT PREDICTED WITHOUT "SBD" AND "WFD"

SAMPLE No. : TPA1/20/16 DATE : 1994/10/04

DESCRIPTION : Norite gravel concrete

REMARK :

INPUT INFORMATION MATERIAL TYPE 2

GRADING Metric Units	
SIEVE (mm)	% PASSING
75	100,00
63	100,00
53	100,00
37,5	92,71
26,5	87,85
19,0	82,98
13,2	82,17
4,75	74,88
2,00	64,35
0,425	35,99
0,075	24,64

ATTERBERG LIMITS (Casagrande Apparatus)		
L.L.	P.I.	L.S.
27,00	10,00	4,50
DENSITY AND SHAPE INFORMATION		
ARD (+4,75)	ARD (-4,75)	S B D
2,961	2,970	0,00
BRD (+4,75)	BRD (-4,75)	W F D
2,961	2,970	0,00

OUTPUT PREDICTIONS MDD in kg/cub. metre

M D D (VIB)	O M C (VIB)	ZAVMC (VIB)	WA	C M C
2303,52	9,91	10,00 - 9,72	0,000	7,365
MDD Mod AASHTO	OMC Mod AASHTO	ZAVMC Mod.	WA	R S D
2256,49	9,57	11,78 - 10,62	0,000	2,968

C B R NOT PREDICTED WITHOUT "SBD" AND "WFD"

SAMPLE No. : NPAB DATE : 1994/10/04

DESCRIPTION : Decomposed dolerite gravel

REMARK :

INPUT INFORMATION MATERIAL TYPE 2

GRADING Metric Units	
SIEVE (mm)	% PASSING
75	100,00
63	100,00
53	100,00
37,5	93,00
26,5	87,00
19,0	79,00
13,2	73,00
4,75	52,00
2,00	46,00
0,425	30,00
0,075	20,00

ATTERBERG LIMITS (Casagrande Apparatus)		
L.L.	P.I.	L.S.
44,00	13,00	8,50
DENSITY AND SHAPE INFORMATION		
ARD (+4,75)	ARD (-4,75)	S B D
2,827	2,827	0,00
BRD (+4,75)	BRD (-4,75)	W F D
2,827	2,827	0,00

OUTPUT PREDICTIONS MDD in kg/cub. metre

M D D (VIB)	O M C (VIB)	ZAVMC (VIB)	WA	C M C
2240,03	9,49	8,98 - 9,27	0,000	6,784
MDD Mod AASHTO	OMC Mod AASHTO	ZAVMC Mod.	WA	R S D
2196,54	8,85	10,74 - 10,15	0,000	2,827

C B R NOT PREDICTED WITHOUT "SBD" AND "WFD"

SAMPLE No. : NPAB/20/14 DATE : 1994/10/04

DESCRIPTION : Decomposed dolerite gravel concrete

REMARK :

INPUT INFORMATION MATERIAL TYPE 2

GRADING Metric Units	
SIEVE (mm)	% PASSING
75	100,00
63	100,00
53	100,00
37,5	94,20
26,5	89,23
19,0	82,60
13,2	77,62
4,75	60,22
2,00	55,24
0,425	41,98
0,075	33,70

ATTERBERG LIMITS (Casagrande Apparatus)		
L.L.	P.I.	L.S.
44,00	13,00	8,50

DENSITY AND SHAPE INFORMATION		
ARD (+4,75)	ARD (-4,75)	S B D
2,827	2,874	0,00
BRD (+4,75)	BRD (-4,75)	W F D
2,827	2,874	0,00

OUTPUT PREDICTIONS MDD in kg/cub. metre

M D D (VIB)	O M C (VIB)	ZAVMC (VIB)	WA	C M C
2166,84	11,52	10,69 - 11,13	0,000	8,067
MDD Mod AASHTO	OMC Mod AASHTO	ZAVMC Mod.	WA	R S D
2159,41	10,31	12,21 - 11,29	0,000	2,855

C B R NOT PREDICTED WITHOUT "SBD" AND "WFD"

SAMPLE No. : NPAB/20/16 DATE : 1994/10/04

DESCRIPTION : Decomposed dolerite gravel concrete

REMARK :

INPUT INFORMATION MATERIAL TYPE 2

GRADING Metric Units	
SIEVE (mm)	% PASSING
75	100,00
63	100,00
53	100,00
37,5	94,33
26,5	89,47
19,0	82,98
13,2	78,12
4,75	61,10
2,00	56,24
0,425	43,28
0,075	35,17

ATTERBERG LIMITS (Casagrande Apparatus)		
L.L.	P.I.	L.S.
44,00	13,00	8,50

DENSITY AND SHAPE INFORMATION		
ARD (+4,75)	ARD (-4,75)	S B D
2,827	2,878	0,00
BRD (+4,75)	BRD (-4,75)	W F D
2,827	2,878	0,00

OUTPUT PREDICTIONS MDD in kg/cub. metre

M D D (VIB)	O M C (VIB)	ZAVMC (VIB)	WA	C M C
2159,71	11,72	10,87 - 11,32	0,000	8,186
MDD Mod AASHTO	OMC Mod AASHTO	ZAVMC Mod.	WA	R S D
2155,24	10,45	12,37 - 11,41	0,000	2,858

APPENDIX B: COST CALCULATIONS - INPUT DATA AND METHOD OF ANALYSIS

APPENDIX B: COST CALCULATIONS - INPUT DATA AND METHOD OF ANALYSIS

B.1 GENERAL

A study of the effects of using marginal aggregate in bituminous surfacings for rural roads carrying low traffic volumes must at some stage include cost analyses in order to determine the effects of various factors on the life-cycle costs of various options. In this study, a number of imaginary roads in different regions within South Africa were analysed. A description of the analysis including the data used to calculate costs follows with the assumptions made and the results of the analysis.

It should be noted that the costs used in the analysis may not always reflect realistic costs, although all values are based on recommended values. The analysis is, however, aimed at determining the risk incorporated in using marginal materials in bituminous surfacings and the effect of certain factors on life-cycle costs, rather than at the actual rand values.

B.2 METHOD OF ANALYSIS

The SURF+ software package[B1] was used as an aid in the cost analyses. The program is a rapid means of comparing the costs of maintaining an existing gravel road with that of upgrading the road to a surfaced condition and maintaining it for a certain period. The user is allowed to use standard values built into the program or to input values from experience. The cost comparisons are made using the methods of nett present value, benefit/cost ratios and internal rate of return. These methods require that an alternative be compared to a base option, which is the existing gravel road.

In the analysis a traffic volume was determined by means of iterative calculations at which it becomes economically viable to pave the gravel road with a bituminous surfacing. This traffic, the break-even traffic, is the number of vehicles per day using the road at the initial stages and which will lead to a zero nett present value when all costs are discounted over the analysis period to present values. Another method may be to use the internal rate of return (IRR) method and to determine the traffic volume at which a rate of return equal to the selected discount rate will result. Other methods can be used to give the same result.

B.3 DATA USED IN THE ANALYSIS

In order to do the analysis, certain assumptions have had to be made. The most important of these assumptions concern the cost of maintaining the surfaced road. In the Sabita recommendations it is assumed that maintenance on all the surfacing types will be the same for roads that are in the same area. When a surfacing containing marginal aggregates is compared with a standard surfacing, it is clear that this may not always be true, because of many factors that may influence the effective life of a surfacing. Certain assumptions had therefore been made regarding maintenance costs of different surfacings.

Maintenance costs of bitumen-surfaced roads are calculated in SURF+ using the CB-Roads[82] methodology. Distinctions are made between wet and dry climatic areas and certain traffic volumes. It was decided not to use these costs for all the surfacing types, but rather to make certain assumptions, using the SURF+ maintenance calculations for an asphalt in good condition as basis. Adjustments were subsequently made to maintenance figures for the other surfacing types.

An analysis period of 10 years was used for the bulk of the analysis. It was assumed that no major rehabilitation will take place during this period for any of the surfaced alternatives. In other words, it is assumed that the routine maintenance will be of such quality that it would be sufficient to keep the pavement on an acceptable level of serviceability throughout the analysis period. It is also assumed that the construction of the supporting pavement layers will be such that they will also not fail under traffic or because of other factors during the analysis period. An analysis period of 20 years was used in one analysis to indicate the effect on the life-cycle calculations.

The surfacing types were selected according to the guidelines given in the Sabita manual.[36] Different terrain types and environmental conditions were selected. Construction costs from the Sabita project are used because of the limited number of cost categories in SURF+. The data used in the calculations are given in Table B.1. The assumptions for each of the cost categories (low, medium, high) are given in reference 36. These costs were taken as the standard costs for a properly constructed surfacing. The use of marginal materials will result in a reduction in the construction costs of a surfacing. The costs in Table B.1 were broken into the different factors of material cost, haulage cost, binder cost, compaction, etc. Suggested surfacing lives of various surfacings are given in Table B.2[36].

Table B.1 : Surfacing construction cost.[80]			
Surfacing	Cost (R/m^2) (1990/91)		
	Low	Medium	High
Slurry (10 mm thick)	2,71	3,42	4,13
Asphalt (25 mm thick)	7,88	8,70	9,52
Asphalt (30 mm thick)	9,41	10,36	11,31
Single seal (10 mm stone)	1,41	1,92	2,43
Double seal (13 mm / 6 mm)	3,28	4,17	5,06
Cape seal (19 mm & slurry)	4,13	5,15	6,17

Table B.2 : Suggested surfacing lives for cost calculations.[36]				
Surfacing	Urban environment (years)		Rural environment (years)	
	Poor conditions	Good conditions	Poor conditions	Good conditions
Asphalt	10-14	15-20	10-14	15-20
Double seal, Cape seal	5-7	8-11	6-8	9-13
Single seal	3-5	5-8	4-6	5-9
Slurry				
thin	2-4	4-6	2-4	4-6
thick	4-7	7-9	4-7	8-10

Note:
a. Poor conditions means third world environment or problems such as weak pavement structure, poor quality control, inadequate provision, etc.
b. Good conditions means first world environment with no problems.
c. surfacing life is highly variable and this table can only be a guide to life.
d. this assumes that the surfacing is being used in an appropriate context.

A number of surfacing types were selected and costs calculated for constructing and maintaining them on imaginary roads in various regions throughout South Africa. The regions were selected in order to represent different climatic conditions within South Africa. The selected roads are in the following regions:

- Road 1: Bloemfontein;
- Road 2: Magoebaskloof;
- Road 3: Messina;
- Road 4: Mosselbay;
- Road 5: Richardsbay;
- Road 6: Rustenburg;
- Road 7: Umtata, and
- Road 8: Upington.

The details of the selected gravel road are given in Table B.3. The same road was used as the base option for all the analyses. An assumption had to be made regarding the frequency of regravelling the gravel road in order to enable the calculation of the zero nett present value. It was decided that an 8 year period would be used throughout.

Table B.3 : Gravel road details (base option)

Item	Description
Road length	1 km
Road width	9 m
Thickness of wearing course	150 mm
Riding quality	fair (QI=80)

The break-even traffic volumes were calculated for each of these options, both in financial and partial economic terms. It was found that the break-even traffic for the financial analyses in all cases above 1 000 vpd. It was subsequently decided to only do the partial economic analyses. A number of additional analyses were then performed in order to determine the effects of various factors related to the use of marginal aggregate on the break-even traffic of the standard options.

These are:

- Run 1: standard costs used, as explained above, using a 10 year analysis period;
- Run 2: 10 per cent savings on the costs of aggregate and of haulage;
- Run 3: 20 per cent savings on the costs of aggregate and of haulage;
- Run 4: as for Run 2, with a 2 year reduction in resealing frequency;
- Run 5: as for Run 2, with a 20 per cent increase in routine maintenance costs;
- Run 6: as for Run 2, with a 10 per cent increase in routine maintenance costs, and
- Run 7: standard costs as for Run 1, but with a 20 year analysis period.

The results of these analyses are shown in Tables B.4 to B.9 and summarised in Figures B.1 to B.6. The results are discussed in the main text (Sec 4.5).

Table B.4 : Break-even traffic (vehicles per day) for asphalt surfacings (partial economic analyses)

Analysis	Road 1	Road 2	Road 3	Road 4	Road 5	Road 6	Road 7	Road 8
Run 1	225	204	211	218	214	249	197	262
Run 2	220	197	203	207	203	241	189	253
Run 3	215	189	195	197	193	233	181	245
Run 4	220	197	203	227	223	241	204	253
Run 5	230	198	213	218	214	252	198	263
Run 6	225	192	208	213	208	247	193	258
Run 7	140	139	139	156	148	166	140	168

Table B.5 : Break-even traffic (vehicles per day) for Cape seal (partial economic analyses)

Analysis	Road 1	Road 2	Road 3	Road 4	Road 5	Road 6	Road 7	Road 8
Run 1	121	126	109	152	148	146	145	182
Run 2	119	124	108	149	145	145	143	180
Run 3	118	122	106	146	142	143	141	178
Run 4	119	146	120	171	167	161	160	180
Run 5	130	135	118	162	155	156	152	190
Run 6	125	130	113	154	150	150	147	185
Run 7	76	85	78	105	97	108	100	122

Table B.6 : Break-even traffic (vehicles per day) for double seal (partial economic analyses)

Analysis	Road 1	Road 2	Road 3	Road 4	Road 5	Road 6	Road 7	Road 8
Run 1	103	107	90	125	126	126	128	164
Run 2	101	103	86	125	121	123	124	160
Run 3	98	100	83	120	116	120	120	157
Run 4	101	126	99	147	143	139	142	161
Run 5	111	117	97	136	132	134	133	170
Run 6	106	109	91	131	12	129	129	165
Run 7	64	73	65	96	83	94	89	110

Table B.7 : Break-even traffic (vehicles per day) for single seal (9,5 mm) (partial economic analyses)

Analysis	Road 1	Road 2	Road 3	Road 4	Road 5	Road 6	Road 7	Road 8
Run 1	85	95	72	109	105	114	112	142
Run 2	85	94	71	107	103	113	110	141
Run 3	84	91	70	105	101	111	108	131
Run 4	91	110	94	189	185	131	175	148
Run 5	97	106	83	119	115	126	120	152
Run 6	91	100	77	113	109	119	115	146
Run 7	52	65	52	87	79	82	86	94

Table B.8 : Break-even traffic (vehicles per day) for single seal (9,5 mm) with a modified bitumen (partial economic analyses)

Analysis	Road 1	Road 2	Road 3	Road 4	Road 5	Road 6	Road 7	Road 8
Run 1	82	83	63	100	96	117	105	140
Run 2	81	82	63	99	95	115	103	139
Run 3	81	80	62	97	93	114	102	138
Run 4	93	104	78	121	116	124	121	152
Run 5	93	95	74	111	107	128	114	150
Run 6	87	88	68	105	101	122	108	145
Run 7	55	59	53	73	65	81	75	98

Table B.9 : Break-even traffic (vehicles per day) for slurry seal (10 mm) (partial economic analyses)

Analysis	Road 1	Road 2	Road 3	Road 4	Road 5	Road 6	Road 7	Road 8
Run 1	107	121	93	140	136	130	136	154
Run 2	106	119	91	137	132	128	134	153
Run 3	104	117	90	134	129	127	128	152
Run 4	111	136	98	219	214	135	199	169
Run 5	117	131	102	149	144	140	143	163
Run 6	111	125	97	143	138	137	139	158
Run 7	65	81	63	107	98	89	100	111

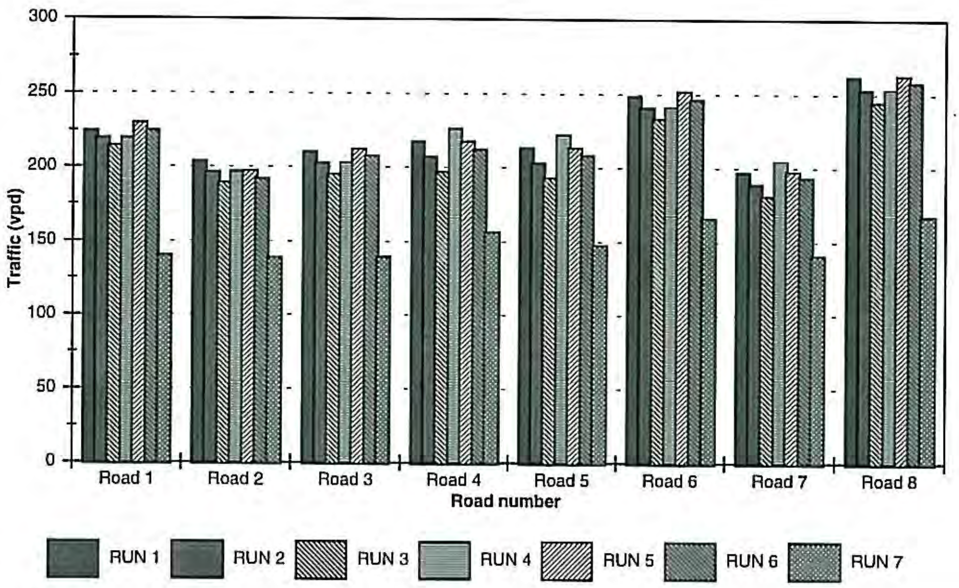

Figure B.1 : Break-even traffic - Asphalt

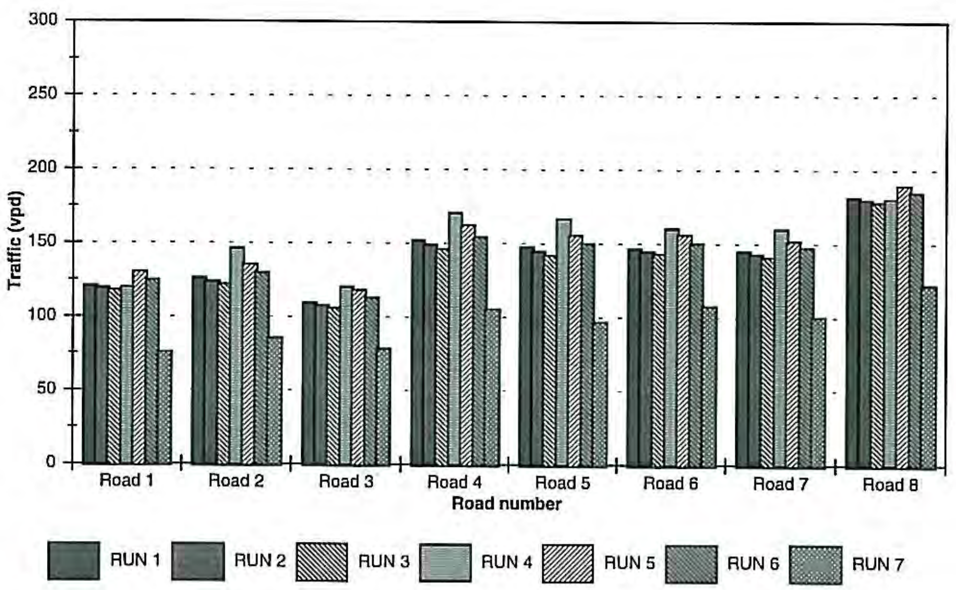

Figure B.2 : Break-even traffic - Cape Seal

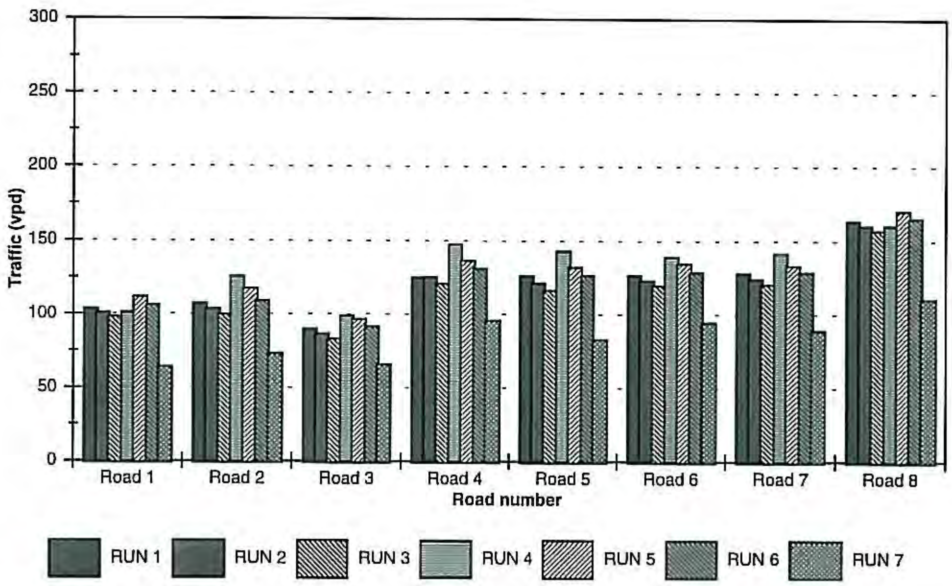

Figure B.3 : Break-even traffic - Double seal

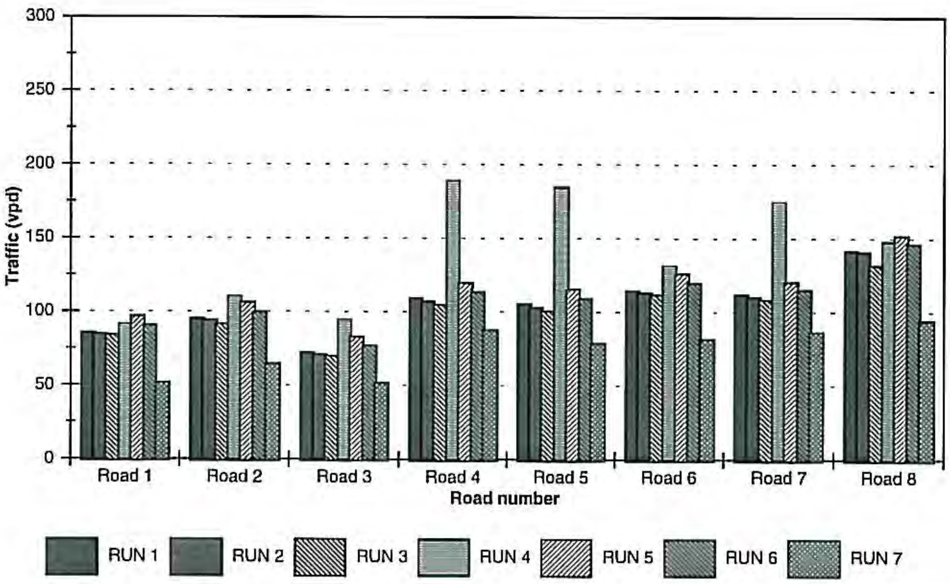

Figure B.4 : Break-even traffic - Single seal

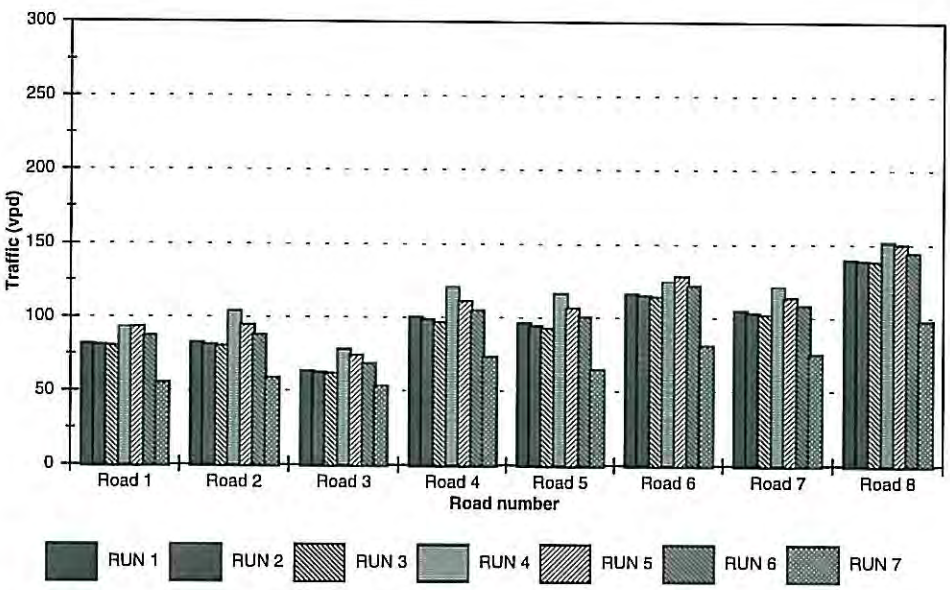

Figure B.5 : Break-even traffic - Single seal (modified bitumen)

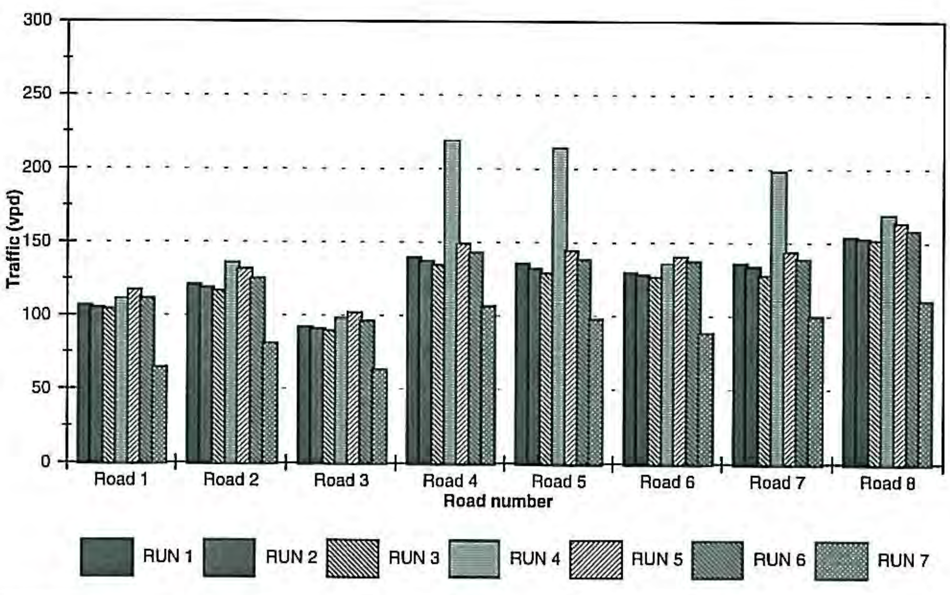

Figure B.6 : Break-even traffic - Slurry seal