THE USE OF NATURALLY OCCURRING MATERIALS FOR PAVEMENTS IN WESTERN AUSTRALIA

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ABSTRACT

Over many years, it has been demonstrated that the application of engineering judgment has been a significant aspect of the continuing use of naturally occurring materials in roads and regional airports in Western Australia. The information contained in this document has been assembled so that, where possible, engineering judgment can be supported by relevant tests and an understanding of the significance of the test results. Materials such as laterite and calcrete (pedocretes), clayey-sands and scree gravels, are discussed against a background of "conventional" selection criteria, strength tests and critical service applications.

This document has not been prepared to provide specifications for general use in the application of natural materials in roads. Rather, one of its aims is to guide personnel involved in the preparation of specifications, towards practical criteria, which do not exclude useable resources. Wherever possible, Main Roads Western Australia practices have been incorporated into this guideline.

This document has been produced by a working group from the Western Australian Pavements Group (a subcommittee of Australian Geomechanics Society comprising Consultants, Main Roads WA, Local Government, Material Suppliers and Researchers). It is an update of the 2003 document published by Main Roads WA. New data has been included and the scope expanded to cover unsealed roads and minor airports.

"Although laboratory experiments may be thought indispensable for a complete knowledge of metalling materials, they are, however not sufficient to define the coefficient of quality of such material" Bredtschneider *et al*. (1911)

1 INTRODUCTION

Naturally occurring granular materials are an important source for basecourse or as subbase courses in the construction of flexible pavements in Western Australia (WA). They include fine-grained materials such as well graded silty and clayey sands (sand-clay), coarse and medium-grained materials such as natural gravels and materials produced by ripping and rolling rock which breaks down.

These materials are often used, but are not limited to, roads with low to medium traffic (<5000 vpd) and surfaced with sprayed seals. However, when correctly applied their use on much more heavily trafficked roads has been successful. In recent years there have been a number of minor airports constructed for new mines in the arid Pilbara Region of Western Australia. Natural materials have been used for runway construction on some of these airports where aircraft movements are less than about 10 per day and maximum aircraft size is about 100 seats.

The term "natural material" is used here to mean a gravelly material occurring in nature as such, or which can be produced with only minimal crushing. Some processing to remove or breakdown oversize may still be necessary. However a distinction is made between these "natural materials" and material produced by crushing hard rock and referred to as "crushed rock base".

The performance of a material as a basecourse or subbase is largely dependent upon its strength and stiffness. For conventional materials, strength comes mainly from mechanical interlock and may be reasonably inferred from simple tests such as particle size distribution and plasticity index. Conventional criteria based on classification tests are generally adequate to exclude almost all unsatisfactory materials. However they have the disadvantage of also excluding some materials capable of giving satisfactory performance.

The WA road network includes more than 18,000 km of highways and main roads and about 170,000 km of secondary and local roads. With such a vast road network and small population, a strong commitment to low cost road construction is necessary.

WA experience has demonstrated that there are considerable cost savings associated with using locally occurring natural materials for pavement construction wherever possible. Many materials successfully used in the past have not met conventional selection criteria but have still given satisfactory performance (Pedersen, 1978).

As these materials are a scarce and valuable resource, it is important that knowledge and experience is applied to their use and management to ensure that they are not wasted or misused. This document provides a guide to the principles and practices used to select appropriate natural materials for construction of roads and regional airports.

The origins of this document can be traced to a visit to WA by Dr Frank Netterberg in the 1980s. The visit by Dr Netterberg, lead to the preparation of a WA guide to selection of natural gravels. The 1989 guide was updated in 2003 by a joint working group from Main Roads WA and Australian Geomechanics Society. The current document was prepared by a working group from the WA Pavements Group, a subcommittee of Australian Geomechanics Society.

2 FUNCTION AND GENERAL REQUIREMENTS OF A BASECOURSE MATERIAL

The principal function of a basecourse and subbase in a flexible pavement is to provide sufficient cover over the subgrade, to limit the stresses and strains induced by wheel loading such that subgrade shear failures do not occur and the subgrade does not densify significantly, thus contributing to a loss of shape of the pavement surface.

To perform satisfactorily, a basecourse must have a number of attributes:

- stability.
- durability including resistance to wear.
- impermeability.
- workability.

Stability of the basecourse is an important component to ensure adequate strength and stiffness under the applied traffic loads, whereas durability ensures that this strength is maintained during its designed service life.

Resistance to wear and erosion is also important, particularly as many rural roads in WA have unsealed shoulders.

Normal practice in WA is for the road shoulder to be constructed of the same material as the basecourse (full width construction). If the unsealed shoulder (and basecourse) has a high permeability, then moisture from rain and runoff may infiltrate leading to increased moisture content and loss of strength. The extent of lateral wetting, under the seal, is likely to be more extensive if the subgrade (or subbase) permeability is less than that of the basecourse. Impermeability of' the base is generally desirable to prevent ingress of water. An exception can occur when very high moisture contents cannot be prevented. Under these circumstances an open graded permeable material may be advantageous to reduce the development of excess pore water pressures. This paper does not deal with this special case.

A material must be capable of being excavated, spread, shaped and compacted with reasonable economy. Segregation of material within the basecourse can be of concern, particularly if large particles are permitted or if extensive water binding is used during compaction. Construction techniques may need to be modified to produce a homogeneous material. Workability may be assessed directly during the initial stages of construction or by trial sections. It can also be inferred from classification tests.

3 GEOLOGICAL PROCESSES WHICH PRODUCE NATURAL GRAVELS

Rocks and soils may be formed by many processes including crystallisation, metamorphism, lithification, weathering, erosion, transportation, deposition and pedogenesis. The material of particular interest for this report is pedogenic material in the form of natural gravels.

The process of pedogenesis consists of the formation of pedogenic soil or rock by precipitation from soil or groundwater of minerals, which have been leached from *in situ* soils under tropical climatic conditions of high rainfall and/or high evaporation. This process can produce pedocretes, which are massive or granular materials weakly to strongly cemented by minerals such as iron oxides, aluminium oxides, silica and carbonates. Examples of these pedocretes are laterite, silcrete and calcrete. Gravels formed by pedogenesis may remain *in situ* or be transported where secondary pedogenic processes may continue. Alluvial river gravels, sands and colluvial scree gravels are occasionally originally formed by pedogenesis.

The best naturally occurring gravel for basecourse and subbase in WA is generally considered to be lateritic gravel. Many lateritic gravels are well graded and have similar strength characteristics and durability as a basecourse manufactured from crushed rock. Lateritic gravels are very common in WA, however quality varies widely and deposits are often unsuitable.

Although laterites are the most sought after gravels, natural gravels of almost every common rock type have been successfully used as basecourse in WA. These include limestone (calcrete) gravels, a variety of scree gravels, decomposed granite, hardpan, sand-clay and river shingle. These materials are described in more detail by Butkus (2001).

4 CONVENTIONAL SPECIFICATIONS FOR BASECOURSE

Basecourse and subbase properties described in Section 2 of this report (stability, durability, resistance to wear, impermeability, workability) are difficult to measure directly. Classification tests and specifications based upon these tests including particle size distribution, particle durability and fines plasticity were developed in Europe and North America (e.g. British and American Standards) from which desirable and undesirable properties could be inferred. These European and North American tests, specifications and the materials they covered are described as 'conventional' as they were initially accepted as the basis for selection of road pavement materials in various parts of the world including Australia. A discussion of these properties recognised in practice as being "conventional" follows.

4.1 PARTICLE SIZE DISTRIBUTION

Experience indicates that for maximum density and stability the grading of a granular material should lie within an envelope described by the equation:

$$\frac{P_1}{P_2} = \left(\frac{d_1}{d_2}\right)^n \qquad (1)$$

Where P_1 = % of particles smaller than sieve size d_1
P_2 = % of particles smaller than sieve size d_2
n = an exponent between 0.3 and 0.5

The gradings represented by the curves in Figure 1 are referred to as maximum density gradings. The distribution of particle sizes is such that each particle (in theory) fits into the voids created by interparticle contact of the larger sizes. Particle size distributions of this type are also commonly known as Fuller gradings after the author who first proposed their use. The percentage of particles is normally assessed by mass. As packing is by volume, Equation 1 is only true for particles where density is independent of size. For some natural materials, particle density varies with size and grading curves for good packing may not fit those described by Equation 1.

An indication of the probable suitability of a material for use as a basecourse can be inferred from its grading curve:

- Material containing an excess of fines ($n < 0.3$) may lack stability particularly when wet.
- Material having deficiency of fines ($n > 0.5$) is likely to be permeable, difficult to obtain a surface suitable for sealing and in extreme cases, lack stability.
- Material with a high proportion greater than about 37.5 mm may be difficult to work and shape particularly if the coarse material is larger than about 50 mm.
- Materials which vary from one side of the grading envelope to the other (gap graded) may be porous, lack stability when wet or be susceptible to segregation depending on where the deficiencies and excesses occur.
- Maximum particle size has a significant influence on material stiffness. Lekarp (1999) carried out repeated load triaxial tests in a specially constructed cell, which could test gravel and sand mixtures up to 90mm maximum size. They reported resilient moduli, which were about 25% higher for well-graded material with a maximum size of 90 mm compared with material with a maximum size of 16 mm.
- The use of material with sizes larger than 37.5 mm is usually restricted to subbase layers where surface finish requirements are not as critical. The use of subbase material with large particles, say up to 75 mm or 100 mm maximum size can be considered particularly where the stone is sufficiently friable for some break down during construction. A maximum size of 75mm is commonly specified for crushed limestone subbase.

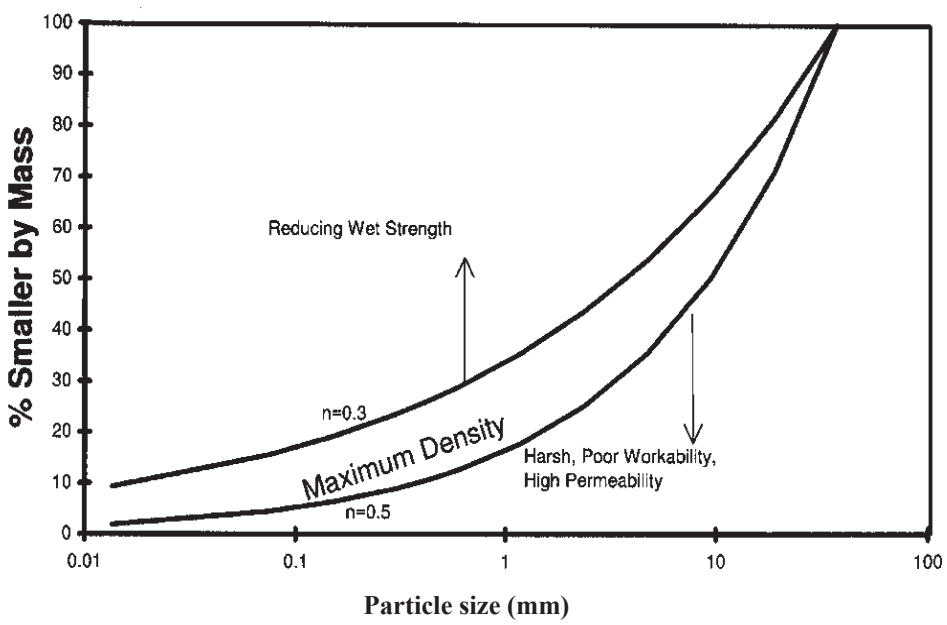

Figure 1: Maximum density grading envelope (37.5 mm max size).

4.2 CONSISTENCY LIMITS

Consistency limits (plastic limit, liquid limit, plasticity index, linear shrinkage) are related to the type and amount of clay in a material. In terms of engineering classification of soils particles finer than 2 microns are regarded as clay. These particles may not all be clay minerals. A fraction of particles finer than 2 µm may be quartz. The most common clay minerals (kaolinite, illite and montmorillonite) have a flat platy shape, though other shapes (needles, etc.) do exist. These plates may be stacked together in a parallel manner (dispersed clays) or in a card house structure (flocculated).

The shape, size, charge imbalances and large specific surface area of clay particles impart properties of cohesion, shrinkage, swell, sensitivity to moisture and low permeability to soils. Fine-grained material at its liquid limit has shear strength of about 1.7 kPa. At its plastic limit, the shear strength is about 170 kPa (Atkinson, 2007).

It should be noted that clay structure (dispersed or flocculated) is influenced by salt content and mechanical energy. The standard laboratory tests use distilled water and a relatively large amount of energy in sample preparation compared to field mixing. These differences can influence the relationship between field performance and index test results.

The following points are a guide to the interpretation of index tests on road building materials.

- A high plastic limit (W_P) may indicate an excessive amount or undesirable type of clay.
- A high liquid limit (W_L) may indicate the possibility of poor packing, the presence of mica or high porosity due to undesirable soil components. However a high liquid limit may just be the result of porous particles.
- A high plasticity index (PI) is generally associated with moisture sensitivity (loss of strength with increased moisture content). However non-plastic materials may also give poor performance due to lack of cohesion.
- Linear shrinkage: this test gives an indication of volume changes resulting from variation in moisture content. It is also useful as a check on the plasticity index, particularly for low PI materials. The PI is typically 2 to 3 times the linear shrinkage depending on the type of clay minerals.

Specification limits on W_L, PI and linear shrinkage can generally be relaxed in well drained arid areas where loss of strength due to moisture is less likely. The tests to determine W_L, PI and linear shrinkage are conducted on material passing 0.425 mm. The proportion of 0.425 mm fines in the material should also be considered. Clearly a highly plastic material with a large proportion of fines will be less stable when wet than a material, which has a low proportion of fines. This is best taken into account by combining the properties of PI (or linear shrinkage) and percentage finer than 0.425 mm as a product.

An example of a “conventional” specification based on grading and index tests is the AASHTO (1995) specification detailed in Table 1.

Table 1: AASHTO specification for base and subbase materials

Particle Size Distribution						
Sieve size mm	Grading A % Passing	Grading B % Passing	Grading C % Passing	Grading D % Passing	Grading E % Passing	Grading F % Passing
50.0	100	100				
25.0	-	75-95	100	100	100	100
9.5	30-65	40-75	50-85	60-100		
4.75	22-55	30-66	35-65	50-85	55-100	70-100
2.00	15-40	20-45	25-50	40-70	40-100	55-100
0.425	8-20	15-30	15-30	25-45	20-50	30-70
0.075	2-8	5-20	5-15	5-20	6-20	8-25
Other limits						
Dust Ratio $P_{0.075}/P_{0.425}$	≤ 0.67					
Liquid Limit (%)	≤ 25					
Plasticity Index (%)	≤ 6					
Los Angeles Abrasion (%) (coarse aggregate)	≤ 50					

When adopting basecourse specifications from elsewhere allowance must be made for differences in the liquid limit test. Subtle differences in method and equipment exist between authorities and countries, which may lead to differences in W_L and subsequently to PI. The PI could vary from 4% to 8% depending on which test method was used. The properties of some non standard materials can be affected by sample preparation (air drying versus oven drying and time of mixing and curing).

Caution is also required when rejecting natural gravel on the basis of W_L. Porous particles are common with natural gravels. A W_L greater than 25% may not be an indicator of poor performance. There is little if any published evidence demonstrating that moderate liquid limits (up to say 35%) in natural gravel base course gravels have led to failures on low volume roads.

5 DIFFERENCES BETWEEN CONVENTIONAL AND PEDOGENIC MATERIALS

Table 2 shows comparative properties for these two types of materials. Some of the reasons why pedogenic materials behave differently to materials commonly used as basecourse in North America and Europe are apparent from Table 2.

Caution is needed in adopting basecourse specifications from elsewhere because of the variability of the liquid limit test. Subtle differences in method and equipment exist between authorities and countries, which may lead to differences in W_L and subsequently to PI. The PI could vary from 4% to 8% depending on which test method was used.

Table 2: Differences between conventional and pedogenic materials (adapted from Netterberg, 1985)

Property	*Conventional (crushed rock base, river gravels, glacial outwash)*	*Pedogenic (Laterite, Calcrete, Silcrete)*
Composition	Natural or crushed aggregate with fines	Varies from clay to rock
Aggregate	Solid, strong rock	Sometimes porous, weakly cemented fines
Clay minerals	Mostly illite or montmorillonite	Wide variety, e.g. halloysite, attapulgite
Cement	None (usually)	Iron oxides, aluminium hydroxide, calcium carbonate, etc
Hydration	None	Variable
Chemical Reactivity	Inert	Reactive
Solubility	Insoluble	May be soluble
Weathering	Weathering or stable	Forming or weathering
Consistency Limits	Stable	Sensitive to drying and mixing
Grading	Stable	Sensitive to drying and working
Salinity	Non-saline	May be saline
Self-stabilisation	Non self-stabilising	May be self-stabilising
Stabilisation (cement)	Increases strength and stiffness	Usually increases strength and stiffness
Stabilisation (lime)	Decreases plasticity	Usually decreases plasticity and/or increases strength and stiffness
Variability	Homogeneous	Extremely variable

5.1 PLASTICITY LIMITS FOR BASECOURSE MATERIALS IN TROPICAL COUNTRIES

Table 3 shows typical limits on plasticity index from some tropical countries. The differences from values in Table 1 may be a result of the materials, economic constraints, service climatic conditions, test procedures and the traffic conditions.

It must be noted that the limits in Table 3 may have been derived using different test methods and sample preparation techniques and may not be directly comparable.

5.2 PEDOCRETES IN WESTERN AUSTRALIA

In WA, natural materials used for basecourse and subbase in road and airport pavements are most commonly pedocretes, which were formed in previous geological times when the climate was tropical (Bardossy, 1979, Sueko, 1988). Some colluvial scree gravels are also used but suitable alluvial (river) gravels are uncommon. Butkus (2001) contains a description of the most common materials used in WA for road construction.

The use of conventional limits (e.g. Table 1) with pedogenic materials results in selection of materials that give highly satisfactory performance but excludes many materials that will perform satisfactorily. Conventional limits, including expanded limits published by NAASRA (1980) for arid areas, may be unnecessarily restrictive when applied to soils such as pedocretes.

Table 3: Typical limits on plasticity index of basecourse material for various tropical countries

Country	*Material*	*Limits on PI (%)*
Botswana	Natural Gravel	≤ 10 (low traffic) ≤ 6 (med traffic)
	Calcrete	≤ 15
Brazil	Lateritic Gravel	< 15
France (overseas provinces)	Lateritic Gravel	≤ 15
Gambia	Lateritic Gravel	≤ 13-22
Ghana	Lateritic Gravel	< 10
Kenya	Natural Gravel	≤ 20 (low traffic) ≤ 15 (med traffic)
Malawi	Lateritic Gravel	≤ 6
Mali	Lateritic Gravel	≤ 6-16
Nigeria	Lateritic Gravel	≤ 12
Portugal (former overseas provinces including Angola and Mozambique)	Natural Gravel	≤ 15
Uganda	Lateritic Gravel	16-25
USA (Theatre of Operations Airfields)	Lateritic Gravel	< 10
Zambia	Lateritic Gravel	≤ 6

Notes:

(1) Based on information collated by Charman (1988), Gidigasu (1988), Hight *et al.* (1988), Netterberg (1986), Netterberg & Paige-Green (1988) and ARRB (1998).

5.2.1 Self Stabilisation of Pedocretes in Western Australia

Some of the gravels of pedogenic origin in common use as basecourses in WA are:

- laterite
- ferricrete
- bauxite
- silcrete
- calcrete

These materials are not necessarily chemically inert and may be capable of self-stabilisation under the influence of wetting and drying cycles. Where strength is inferred on the basis of particle size distribution and plasticity tests only, this potential for self-stabilisation is not taken into account. Similarly tests such as the California Bearing Ratio (CBR) if applied immediately after compaction do not measure the potential for strength gain with time. The magnitude of possible strength gain is evident from the data presented in Figure 2 for a lateritic gravel basecourse on the Meenaar Deviation of the Great Eastern Highway. Clegg impact tests were carried out across the seal together with sampling for moisture content. Impact values corresponding to moisture ratios of 70% and 100% were obtained by interpolation and/or extrapolation. Data was presented in Hamory (1989). Similar evidence of self-bonding with time is evident from monitoring of calcrete and lateritic gravel base course sections in the Kimberley region (Kilvington and Hamory, 1986) and for the red clayey-sand used as basecourse on the Hamelin Denham Road (Cocks and Hamory, 1988). The process of self-stabilisation is complex and probably involves many factors, both physical and chemical. Grant (1974) suggests that initial bonding in lateritic gravels is due to solution and deposition of the aluminium hydroxide.

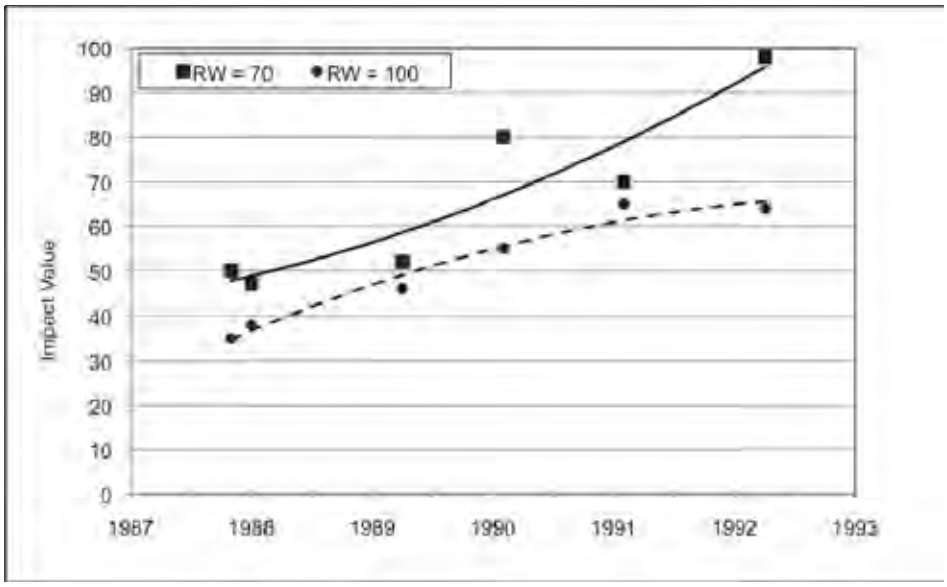

Figure 2: Change of stiffness/strength of lateritic gravel base, Great Eastern Highway Meenaar Deviation. (RW refers to moisture ratio as a percentage of OMC_{mod})

The need for special tests to evaluate and identify natural gravels with self-bonding potential was identified by Grant and Aitchison (1970). However, little Australian or international research has been directed to this problem.

Whilst evidence of self-stabilisation of pedogenic materials exists (Netterberg, 1975) it is difficult at this time to utilise this information in a formal way in material selection and evaluation. Until such time as the mechanism of self-stabilisation and the factors which influence its development are better understood, it can only be considered on the basis of local observation and experience.

A note of caution must be sounded about approving the use of a natural gravel on the basis of its potential to gain strength. The base must still have adequate strength in its early life. This may entail some form of temporary moisture protection by priming or slurrying the shoulders.

5.3 BASIS FOR CHANGING SELECTION CRITERIA FOR PEDOCRETES AND OTHER NATURAL GRAVELS

The need to adapt specifications for base and subbase materials to suit WA conditions has largely been driven by cost. WA has a vast road network and a low population density. The successful use of materials available at low cost, located close to the proposed road works has underpinned the development of the sealed road network. The finding that many natural gravel materials, which do not meet the specification limits adopted in North America and Northern Europe, could still be used successfully (Jewell, 1968), has been largely based on experience.

In addition to the special factors relating to pedocretes, which were set out in Table 2, the following factors are relevant:

- Much of Western Australia has an arid or semi arid climate.
- Regulation axle loads in WA are lower than those adopted in much of Europe.
- Many sealed roads in remote areas of WA have relatively low traffic volumes.
- Appropriate skills in construction with “non standard” materials and high standards of workmanship have been developed.

The typical selection criteria set out in this report, are therefore largely experience based.

6 PROPERTIES THAT AFFECT GRAVEL PERFORMANCE

Properties that are known to exert a major control on the performance of gravel in road pavements, are particle size distribution, plasticity and moisture content and density which effect the strength and stiffness. Strength and stiffness at any moisture content and density can be affected by wetting and drying history or the compaction process to which the material has been exposed in coming to the density involved.

The most important basecourse material parameters are strength and stiffness and there are five aspects which must be satisfied with regard to the selection of basecourse materials:

- Adequate strength under any individual applied load.
- Adequate strength to resist progressive failure under repeated individual loads.
- Ability to retain that strength capacity with time (durability).
- Ability to retain strength under various environmental influences (which relate to material moisture content and in turn to climate, drainage and moisture regime).

Adequate stiffness to avoid creating excessive tensile strains in the bituminous surfacing when load is applied. This is of particular importance where asphaltic concrete is to be applied.

6.1 MOISTURE CONTENT IN WESTERN AUSTRALIAN BASECOURSE MATERIALS

The performance of many naturally occurring pavement materials is strongly dependent on moisture content. For the purpose of assessing moisture content, WA pavements can be subdivided into four drainage units:

- Unit 1 (Well Drained)
- Unit 2 (Permeability Inversion)
- Unit 3 (High Water Table)
- Unit 4 (Pavement Saturation)
  - pavement subject to inundation or
  - very high water table or
  - poor external drainage or
  - moisture surplus climate and
  - permeability inversion

An analysis by Mackenzie (1988), of test results on samples taken from the outer wheel path from 31 locations satisfying the criteria for Unit 1 yielded the following relationship with a correlation coefficient of 0.86:

$$MC = 0.70 \times OMC + [0.29 \times 10^{-3} \ (AR\text{-}PE)] + \left[0.58 \frac{P_{0.425}}{P_{0.075}} \right] - [0.02 \times P_{2.36}] \qquad (2)$$

Where MC = basecourse moisture content (%)
OMC = basecourse optimum moisture content (%) (modified compaction)
$P_{2.36}$ = percentage by mass passing a 2.36 mm sieve
$P_{0.425}$ = percentage by mass passing a 0.425 mm sieve
$P_{0.075}$ = percentage by mass passing a 0.075 mm sieve
AR = annual rainfall (mm)
PE = potential evaporation (mm)

Where the permeability of the layer underlying the basecourse is lower than that of the basecourse (Unit 2), then a "permeability inversion" is said to occur. This generally occurs when the subgrade material has more fines. Under these circumstances water infiltrating into an unsealed shoulder can move laterally under the seal and reduce soil strength under the outer wheel path.

A pilot study by Skender and Leach (1988) has shown that significant basecourse failure can occur where a permeability inversion occurs in combination with a basecourse with marginal strength characteristics.

Desirably, such permeability inversions should be avoided through appropriate material selection and/or construction technique. Typically most basecourse materials are reasonably well graded and include some plastic fines and if properly placed and compacted (water bound) have a low permeability so that failures due to a permeability inversion are rare. When the basecourse is more permeable than about 5×10^{-7} m/s, then the road shoulder should be sealed so that the lateral wetting front does not extend under the trafficked area of pavement.

The extent of lateral wetting can be calculated in a theoretically rigorous manner using the procedures defined by Wallace and Leonardi (1976). However the calculations are somewhat complex and the computer program THEWET developed by Wallace and Leonardi is not particularly user friendly.

For a more detailed discussion of permeability and selection of design moisture content, the reader is referred to the paper by and Skender and Leach (1988) and Main Roads (2003) Engineering Road Note 5. Comparison of measured moisture content in a basecourse with the expected moisture content given by Equation 2 can provide useful guidance as to whether there is some underlying drainage problem at an investigation site.

Notwithstanding the findings regarding moisture content of basecourses, it is more normal in WA to evaluate CBR of prospective pavement material and subgrade on a soaked basis. In contrast, repeated load triaxial tests are commonly conducted at 70% or 85% of OMC.

Except for cement modified materials, normal practice in WA is to dry back the natural basecourse prior to application of bituminous surfacing. Dry back is based on achieving a moisture content less than or equal to 85% of the optimum moisture content (OMC) as derived using modified compaction for roads to receive a sprayed seal surface. For roads that are to have an asphalt surface, then dry back to 70% of OMC is more typical. This facilitates the development of additional strength and stiffness until the natural "setting up" or self stabilisation occurs.

6.2 TESTS FOR STRENGTH AND STIFFNESS

The distinction between strength and stiffness is not always as clear in pavement engineering as it is in other engineering disciplines. Some of the empirical tests used in pavement engineering are indirect indicators of both strength and stiffness. Methods of strength or stiffness measurement used in WA include:

Repeated Load Triaxial Test (RLTT)	laboratory
WA Confined Compression Test (WACCT)	laboratory
California Bearing Ratio (CBR)	laboratory or field
Clegg Impact Test	laboratory or field
Falling Weight Deflectometer	field
Dynamic Cone Penetrometer	field

6.2.1 Repeated Load Triaxial Test

The current repeated load triaxial (RLT) test protocol is published in "AG-PT/T053 - Determination of Pavement Deformation and Resilient Modulus Characteristics of Unbound Granular Materials under Drained Conditions" (Austroads, 2007). The RLT is a stiffness test and in the normal course of events the maximum load is well below that required to induce a shear failure or crushing in the test specimen.

The RLT test allows the measurement of both the permanent deformation and resilient modulus of unbound granular materials, using repeated vertical load and static confining pressures, under drained conditions. Permanent deformation and resilient modulus are measured on specimens (100 mm diameter and 200 mm high) prepared to required density and moisture conditions. Dynamic compaction is used to compact the specimens.

Figure 3 shows the computer controlled testing equipment. Both vertical load and confining pressure are applied pneumatically. Two internal Linear Variable Differential Transducers (LVDT) are used to measure the transient change in the total vertical length of the specimen resulting from a pulsed load. An internal load cell is used to measure axial load, and a pressure transducer is used to measure confining pressure.

The permanent deformation test characterises the vertical permanent strain at three stress stages of repeated deviator stress and a static confining stress. For each stress stage, 10,000 repetitions of deviator stress are applied. Table 4 shows the three stress stages for permanent deformation test on unbound granular materials of base, upper subbase and lower subbase quality. Figure 4 shows an example of permanent strain results.

The resilient modulus test characterises the vertical resilient modulus over 70 stress conditions using combinations of applied repeated deviator stress and static confining stress. Figure 5 shows the stress conditions applied for resilient modulus tests on unbound granular base materials in Main Roads WA. Figure 6 shows an example of resilient modulus test results for base quality ferricrete gravel.

Figure 3: Repeated load triaxial test equipment.

Table 4: Stress levels for permanent deformation

Permanent Deformation Stress Levels						
Stress Stage Number	*Base*		*Upper subbase*		*Lower subbase*	
	$\sigma_3^{(1)}$ (kPa)	$\sigma_d^{(2)}$ (kPa)	σ_3 (kPa)	σ_d (kPa)	σ_3 (kPa)	σ_d (kPa)
1	50	350	50	250	50	150
2	50	450	50	350	50	250
3	50	550	50	450	50	350

Notes:

(1) σ_3 = static confining stress

(2) σ_d = repeated deviator stress = σ_1-σ_3 where σ_1 = vertical stress

Figure 4 shows an example of permanent strain test results for base quality ferricrete gravel.

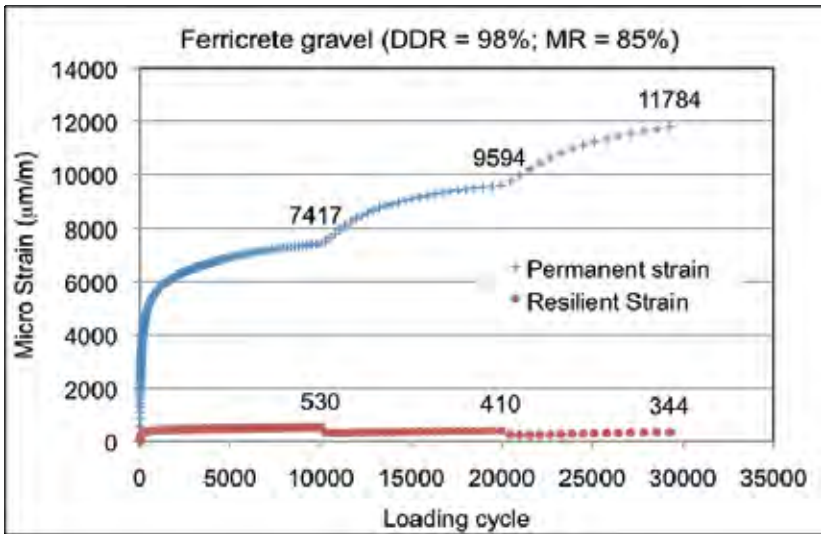

Figure 4: Example of permanent strain test results.

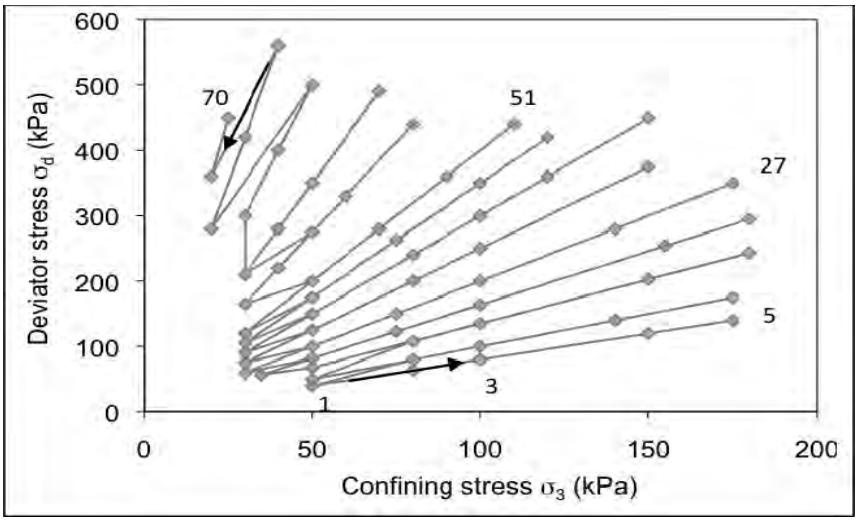

Figure 5: Stress path for resilient modulus test on unbound granular base materials.

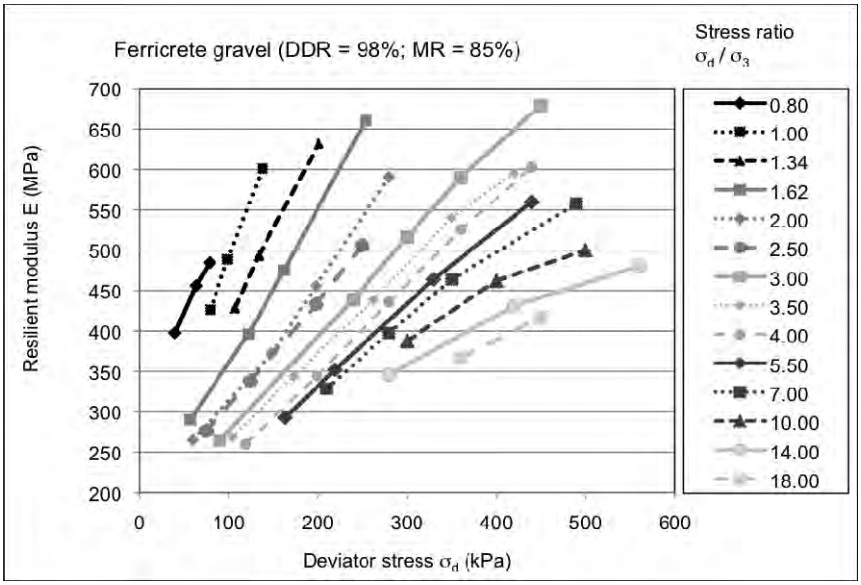

Figure 6: Example of resilient modulus test results.

Resilient moduli of unbound granular material are stress-dependent. This can be described by the following equation as in Austroads Guide to Pavement Technology Part 2: Pavement Structural Design (Austroads, 2012).

$$E = K_1 \times \left(\frac{\sigma_m}{\sigma_{ref}}\right)^{K_2} \times \left(\frac{\tau}{\sigma_{ref}}\right)^{K_3} \quad (3)$$

Where: in a repeated load triaxial test:

E = resilient modulus (MPa)
σ_m = mean normal stress, $(\sigma_1 + 2\sigma_3)/3$ (kPa)
σ_1 = vertical stress applied to the testing specimen (kPa)
σ_3 = confining stress applied to the testing specimen (kPa)
τ = octahedral shear stress, $(\sqrt{2}\,(\sigma_1-\sigma_3)/3)$, (kPa)
σ_{ref} = reference stress (atmospheric pressure = 100 kPa)
K_1, K_2, K_3= experimental test constants.

After a RLT resilient modulus test, a total of 70 resilient moduli E corresponding to the 70 stress steps as shown in Figure 5, can be obtained if the specimen does not fail during testing. The experimental test constants K_1, K_2 and K_3 of the tested specimen can be derived by applying multivariable regression to the test results (i.e., 70 sets of E, σ_m, τ). Therefore, resilient moduli of the tested specimen at given values of σ_m and τ can be calculated using Equation (3). For example, in Main Roads WA, for unbound granular base materials, the resilient modulus used in mechanistic pavement design is calculated using the combination of σ_m of 240 kPa and τ of 120 kPa.

The resilient modulus of an unbound granular base material can be used as an input into mechanistic pavement design processes.

Main Roads WA generally only uses RLT testing when the intended road surfacing is asphaltic concrete. While an RLT test can be carried out at almost any conditions of moisture and density, Main Roads WA, current practice is to carry out the test on specimens compacted to target density (usually 95% to 98% of MMDD at 100% of OMC and then dried back to 70% or 85% of OMC. The intent is to match the stiffness likely to be achieved in a well-drained pavement. This is a very different moisture condition from the soaked CBR test and ranking of materials on the basis of RLT testing may be different to that ranked on the basis of soaked CBR.

Caution is required in applying results of RLT testing on cement treated material. The test loading is compressive and any deterioration in cemented bonds due to tensile stress is not reproduced in the test.

RLT testing is not commonly used as a selection tool for natural gravels as lower bounds of acceptable stiffness and permanent deformation have not been developed.

6.2.2 Accelerated Triaxial Test (Test Method WA 142.1)

The Western Australian Confined Compression Test (WACCT) is similar to the Texas Triaxial Test and may be used to assess the shear strength of a pavement material in what is considered its critical state of moisture and density for its intended use in the field, or over a range of moisture and density conditions if required. A compacted cylindrical specimen of the material is placed in a triaxial cell in which the lateral stress is kept constant, and the vertical stress increased until shear failure occurs. The WACCT was extensively used for evaluation of natural gravels and sand-clay in the 1970s and 1980s but has since fallen out of favour largely due to the unavailability of equipment in the private sector and the cost of the test.

6.2.3 California Bearing Ratio Test (Test Method WA 141.1, AS 1289.6.1.1)

In the laboratory CBR test a cylindrical plunger is penetrated at a standard rate into a compacted, confined sample. The CBR is calculated by expressing the load required to cause a penetration of 2.5 mm as a percentage of 13.2 kN or a penetration of 5 mm as a percentage of 19.8 kN, whichever is the larger. The CBR test was first developed in 1929 and an early description of the test is given in Porter (1938). The test is largely unchanged from that time except that samples were originally prepared with static compaction with a load of 2000 pounds per square inch (13.79 MPa). Porter (1938) reported that “good crusher run bases” had a CBR of 80% to 120% and “good gravel bases – uncrushed” had a CBR of 40% to 80%. The higher CBRs now recorded for such materials are probably the result of adopting dynamic (modified) compaction for sample preparation.

The CBR is an empirical value, and does not accurately relate to any of the fundamental properties governing material strength or stiffness. The material in the test is predominantly subject to shear deformation but stops short of complete shear failure. CBR is commonly referred to as a strength parameter but is also an indicator of stiffness. The test is normally carried out on material passing the 19 mm sieve. There are significant repeatability and reproducibility issues with the CBR test (Rallings, 2014). For increased confidence, the results of several test specimens tested at the same

density and moisture content should be used. Notwithstanding this problem, the test has been successfully used over a long period of time and for a wide range of conditions.

In carrying out the test there are a number of considerations as follows:

- Treatment of particles larger than 19 mm.
- Sensitivity to particle breakdown.
- Selection of target dry density ratio
- Selection of compaction moisture content.
- Soaked or unsoaked test conditions.
- Curing of compacted test specimens prior soaking.

The test is normally carried out on material passing the 19 mm sieve. For coarse (say 75 mm maximum size), naturally occurring basecourse materials, the exclusion of a large portion of stone coarser than 19 mm may result in the exclusion from use of material capable of giving satisfactory performance. Both WA and AS Test Methods acknowledge that there is at present no generally accepted method of testing, or calculation, for dealing with this difficulty. AS 1289.6.1.1 allows the percentage of oversize to be replaced in the test sample with an equal portion by mass of the material passing 19 mm and retained on a 4.75 mm sieve. Other non standard methods used in the past have included cracking the oversize particles to pass 19 mm or the CBR test portion was screened to pass 26.5 mm maximum size, instead of 19 mm.

Since January 2012 the Main Roads WA test method for CBR has prohibited the replacement of material larger than 19 mm. The basis of this decision was to achieve better reproducibility in test results. The attempt at improving consistency of practice between laboratories comes at a cost as material that might be acceptable with replacement of oversize may be excluded from use. International practice regarding treatment of oversize varies (Pinard & Netterberg, 2012). AASHTO T193-81 and South Africa (TMH1 Method 8) include compensation for oversize. The British Standard (BS 1377-4) excludes oversize without compensation.

Where the designer's intent is to include the influence of oversize material on CBR then the Australian Standard should be used with replacement of oversize in the sample.

Some natural gravel material has relatively low strength gravel particles that will breakdown during the placement process in a road. Sensitivity of the CBR can be assessed by testing samples pretreated by one or more cycle of pretreatment by compaction in a CBR mould. Where the CBR is satisfactory without pretreatment but unsatisfactory with two cycles of pretreatment then rework of the material in the field should not be permitted.

Main Roads WA typically requires subbase materials to be assessed on specimens compacted to 94% of MMDD and basecourse materials at 96%. These numbers are not universally adopted and some practitioners adopt higher target densities. The sensitivity of CBR to density can be assessed by compacting and testing specimens for a range of densities and plotting the results on a scale (e.g. Log linear) that gives a straight line to aid interpolation. Where such testing indicates that very high dry density ratios are required (say more than 98%) to achieve a satisfactory CBR result then the risk of material breakdown must be considered.

For the more commonly used natural base course materials such as lateritic gravel, calcrete and scree gravel, evaluation of the CBR is normally carried out on samples compacted at OMC (from the MMDD test) and then soaked for 4 days prior to penetration. On low traffic roads under favourable conditions (arid climate, well drained, few heavy vehicles and sealed shoulders) then the use of unsoaked CBR may be considered. Where unsoaked CBR testing is undertaken, consideration should be given to wrapping the compacted sample in plastic and allowing moisture to equilibrate for 4 days prior to penetration. Evaluation of CBR at moisture contents of less than OMC (without soaking) must take into account the risk of less favourable conditions occurring in the life of the pavement.

For some materials, curing the compacted test specimen for several days prior to soaking may result in higher CBR test results and permit use of material that would otherwise be rejected.

NAASRA (1980) suggests a minimum CBR value of 80% for base course. In WA common limits are minimums of 80% (soaked) for basecourse and (30%) for subbase. Limits do vary depending on material types, climate and loading conditions. A soaked CBR of 60% is sometimes accepted for base course, for example with Tamala Limestone below a 60 mm thick asphalt surface on local streets.

The CBR test can also be carried out in the field. However, this is extremely rare in WA. When applying this test in the field it is important that the weight of the vehicle used to provide the jacking force is sufficient.

6.2.4 Clegg Impact Soil Tester (AS 1289.6.9.1)

The Clegg impact soil tester measures the deceleration of a 4.5 kg hammer (the laboratory soil compaction hammer used for density tests) dropped from a height of 460 mm. One impact value is equivalent to 10g. As with the CBR test, there is some debate as to whether the Clegg impact test is better described as a strength or stiffness test. The manufacturer's website refers to it as a stiffness/strength test. The relationship between strength/stiffness and field impact value for compacted basecourse materials is shown in Table 5.

Table 5: Clegg impact value, basecourse strength/stiffness

Clegg Impact Value	*Basecourse strength/stiffness*
> 75	Very High
60-75	High
45-59	Medium-High
30-44	Low-Medium
< 30	Low

The effect of traffic on a base with very low impact test results can be gauged from the following observations in 1984, on the Meenaar Deviation of Great Eastern Highway.

- On a site with a basecourse impact value of 6, a single pass of a loaded single axle truck resulted in a rut 70 - 80 mm deep.
- On a site with an impact value of 20, a single pass of the same truck resulted in a rut 5 mm deep.

An estimate of the CBR corresponding to impact values can be made using a correlation published by Clegg (1986):

$$CBR=0.06\,(IV)^2 + 0.52\,IV + 1 \qquad (4)$$

Where IV = Impact Value

The impact tester may also be used to assess whether a section of compacted basecourse has dried back sufficiently to be sealed. A characteristic impact value (mean - 0.59 std. dev. for 9 tests) of at least 45 is desirable with an absolute minimum of 40.

6.2.5 Deflection Tests

Falling Weight Deflectometer (FWD) testing is most commonly used in WA as an aid in the structural assessment of road pavements. Provided that the thickness of the various pavement layers is known, estimates of layer moduli can be obtained by back analyses of the shapes of deflection bowls. Numerous methods including the ELMOD, EFROMD and RUBICON programs may be used to analyse bowl shapes. Anisotropy can be included with EFROMD. Some measured values of moduli for WA natural materials, derived from deflection tests, have been reported by McInnes (1993). Deformation under FWD testing is generally small (say 1 mm) and it is a tool to measure stiffness rather than strength.

FWD tests are not commonly used as a selection tool for natural gravels but may be used after construction of a pavement to measure the stiffness of those materials. The curvature function (peak deflection D_0 minus deflection at 200mm D_{200}) is sometimes used as an indicator of basecourse stiffness with high curvature an indicator of low stiffness of the pavement layers.

Horak and Emery (2006) have outlined a method using FWD data whereby the pavement structure is characterised through a Base Layer Index (BLI, D_0 –D_{300}), Middle layer Index (MLI, D_{300}- D_{600}) and Lower Layer Index (LLI, D_{600}-D_{900}). BLI can be used as an indicator of load carrying capability of a base course on roads with highway loading. MLI is an indicator of sub base stiffness and LLI is an indicator of the subgrade stiffness.

6.2.6 Unconfined Compressive Strength

The Unconfined Compressive Strength (UCS) test is used in WA as the prime method of assessing design cement content for cement stabilisation of basecourse materials. The test is carried out on a material passing 19 mm and compacted in a 1 Litre compaction mould. Main Roads WA has adopted a procedure that differs from Austroads. In WA, the normal procedure is to use GP cement for the laboratory testing even when LH cement is to be used for the field stabilisation. The cement is mixed with the proposed basecourse material in a moist condition and allowed to hydrate for about two hours. The sample is then compacted in the mould and extruded, moist cured for 7 days and then soaked for 4 hours before crushing. Test specimens are typically compacted to between 95% or 96% of MMDD. UCS criteria are discussed in Section 8.3 of this paper.

6.2.7 Dynamic Cone Penetrometer Test (South African Test Method ST6, ASTM 6951)

The Australian Standard Dynamic Cone Penetrometer (DCP) uses a 9 kg mass falling 510 mm with a 30° cone. This cone is likely to be damaged if used on gravel and the Australian device is not suitable for testing basecourse materials. Within the last 5 years it has become more common in WA to use the South African DCP when testing basecourse and subbase materials in roads. The South African/ASTM DCP uses an 8 kg mass falling 575 mm and a 60° cone. The energy imparted is about the same as the Australian DCP but the 60° cone is more robust and less likely to be damaged when used on gravel. Where an 8 kg penetrometer is not available then a 9 kg mass dropped 510 mm may be used in conjunction with the 60° cone.

Correlations between CBR and DCP Index (mm/blow) for the South African cone have been developed by Kleyn (1984), Paige-Green & Pinard (2012) and Pinard & Paige-Green (2013). Kleyn (1984) also developed a correlation with UCS for cemented material. Typical values are set out in Table 6.

Table 6: Correlations between DCP Index, CBR and UCS.

DCP index (mm/blow) 60° Cone DN	*Equivalent CBR (%)*	*Equivalent UCS for cement treated material (MPa)*
2	170	1.3
3	100	0.85
4	70	0.6
5	50	Not applicable

For DCP Index (DN) that is greater than 2 mm/blow:

$$CBR = 410\ DN^{-1.27} \quad (5)$$

The above DCP/CBR correlation is based on the South African procedure for CBR, which has some differences to ASTM, BS, AS and Main Roads WA test methods.

6.2.8 Maximum Dry Compressive Strength (Test Method WA 140.1)

The Maximum Dry Compressive Strength (MDCS) test is carried out on material passing 19 mm. Specimens are compacted in a 70mm cubic mould and then dried in an oven at 105°C before crushing. At least three test cubes with moisture contents spanning OMC (from the MMDD test) must be tested. Sometimes a fourth sample must be tested to define a curve to pick the maximum strength.

Due to the extremely dry condition of the test specimens MDCS should be regarded as an "indicator test" rather than a measure of field strength. For non plastic and low plasticity basecourse materials it is an indicator of the ability of the material to develop a tight surface suitable for sealing. For roads with unsealed shoulders and gravel roads a low value of MDCS may indicate a propensity for the material to ravel.

MDCS selection criterion may be waived or reduced under the following conditions:

- Road shoulders are to be sealed.
- There is good stone interlock.
- PI exceeds 6.

It is very rare for a failure in a road pavement to be attributed to low MDCS. Where a nearby source of gravel is available that satisfies other criteria but fails the MDCS requirement, then its use should not be ruled out. Options such as extra working of the base with water and rollers to develop a good surface and sealing road shoulders may be viable alternatives to hauling conforming material long distances.

Reproducibility of the MDCS test can be an issue. It should be noted that the moisture content that gives the MDCS may not be the same as that giving MMDD.

6.3 PREDICTION ON THE BASIS OF PAST PERFORMANCE

Where a material of the same geological classification, physical and chemical properties has been previously used in road construction, then its past performance can be a valuable aid to decision making about future use. Where a material has given unsatisfactory performance in the past then it should not be used unless the cause of the unsatisfactory performance can be identified and rectified.

Caution is still needed when justifying the use of a non standard material based on past performance. Conditions may not be the same as when the material was first used. Some of the changes which need to be considered include:

- Reduced road closures in wet conditions (due to bridges replacing flood crossings).

- Increased traffic volume axle loads and tyre pressures.
- Changes in construction technique and equipment.
- Environmental changes.

6.3.1 Reduced Road Closure

In WA, particularly in the north of the state, much of the road system has, in the past, been subject to closure during flooding. This protected a road from loading when the pavement was in its wettest and weakest condition. With the upgrading of the National Highway and the construction of bridges over rivers, this built in protection has largely disappeared.

6.3.2 Changes in Traffic Loading

Over the years there has been a gradual increase in regulation loads on a single axle in WA, as shown in Figure 7. There has not been a major increase in regulation axle loads since 1988. Based on past trends an increase in regulation axle loads is overdue. If the past trend continues, pavements designed now may have to carry vehicles in the future with a legal single axle (with dual tyres) load of 11 to 13 tonne. This is a potential risk to the continued use of low cost natural materials for road construction. There are opportunities for more freight efficient vehicles that do not result in increased pavement wear (Austroads, 2011).

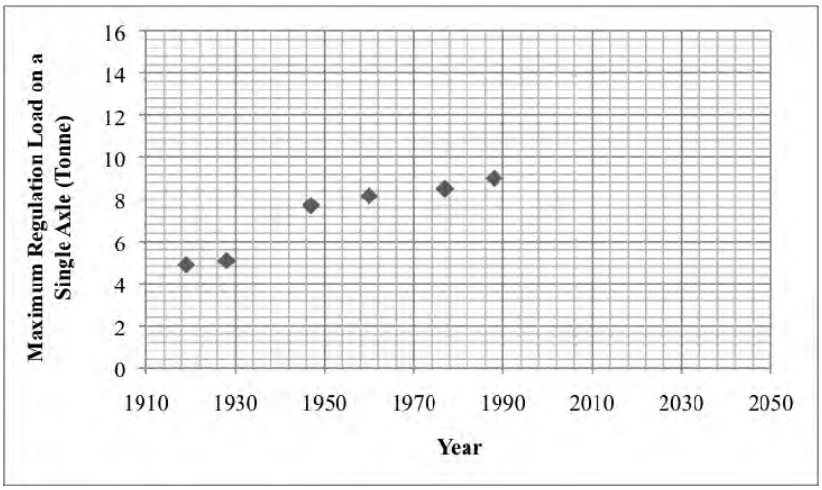

Figure 7: Regulation axle load in Western Australia.

The population has grown rapidly in WA in the last 20 years with a corresponding increase in traffic volume. The consideration of past performance of a natural paving material proposed for future use must take into consideration differences in traffic volume and likely growth rate.

The conventional approach to assessing axle load/pavement damage relationships has its 'empirical origins' in the AASHO road test carried out in Illinois over the period 1958 – 1960. The AASHO test pavements were straight and approximately level and vehicles were driven at a relatively constant speed. The largest vehicle was a five axle semi trailer with a tandem axle drive. As a consequence, the ratio of horizontal contact stress to vertical stress under the drive axles was negligible.

In WA a road train with a tandem drive may have up to 20 axles. Road trains with triple drives may have more than 20 axles. The horizontal contact stresses under the drive axles are large, particularly when the road train is turning, on a steep grade and braking.

Studies by Prem & Potter (1999), Ramsay *et al.* (1999) and Prem *et al*. (2000) suggest that the damaging effect to bituminous surfacing caused by road trains is significant. The damaged surface is more likely to permit the entry of water leading to a reduction of basecourse strength. The risk of pavement damage from turning road trains at intersections and other high stress locations can be reduced by cement modification of natural gravel basecourse materials and the use of high stress multi coat seals or dense graded asphalt over a sprayed seal as a surface.

6.3.3 Changes in Construction Equipment and Methods

There have been significant advances in construction equipment with gear generally becoming larger and heavier. There is a greater requirement today to construct pavements rapidly and pavement materials generally get less of the traditional 'blade' mixing and are constructed in single relatively thick lift using heavier compaction equipment. Often

recycling machines are used to mix and condition the pavement material. The impacts of these changes on material selection generally relate to material uniformity, aggregate breakdown and moisture conditioning.

Material Uniformity: The use of larger dozers in stockpiling operations generally results in less mixing and less control of quality, with greater risk of material segregation in stockpiles. Supervision by experienced materials earthworks personnel is required to optimise mixing of materials during stockpile and load out. Mobile screening or crushing plants are sometimes used to improve material uniformity. The use of recycling machines and compaction of layers in a single lift also reduces the mixing effort and due consideration needs to be given to the uniformity of the material when it is placed on the road.

Aggregate Breakdown: The use of larger dozers for stockpiling, recycling machines for mixing and heavy compaction equipment can result in excessive break down of naturally occurring materials. This may be beneficial with materials with excessive oversize particles, however with the majority of materials this can result in an excessively fine grading in the completed pavement.

Moisture Conditioning: The use of larger compaction equipment generally requires lower compaction moisture contents in the materials being compacted than was required in the past. Where the field compaction effort is effectively greater than that in the laboratory maximum dry density test, the moisture content requirements for compaction are likely to be dry of the laboratory optimum moisture content. As strength and stiffness are a function of moulding moisture content, and not just density, appropriate adjustments need to be made during construction to introduce enough water to get basecourse material to develop its full strength potential caused by suction stresses during drying back.

6.3.4 Environmental Changes in Western Australia

Possibly the most significant environmental change affecting road pavements in WA has been the rising water table and increase in groundwater salinity in the wheat belt region (Wordsworth, 1988; McRobert & Foley, 1999). There is some evidence of increased rainfall in the East Kimberley region leading to wetter pavement conditions.

6.4 AGGREGATE DURABILITY

Where strength tests form the basis of material selection, a check is needed that the material is not going to degrade excessively in service or during construction. Rocks can alter chemically (decompose), or more commonly in the dry climate of WA, existing alteration products (formed by natural weathering processes through geological time) can be mobilised and freed (disintegration). The end result is that the quality of the material during the life of the road is reduced, and the life of the road is also reduced. This can occur within as short a time as a couple of years. In the basecourse, the mode of failure is shear or plastic deformation in the base leading to rutting, crocodile cracking and potholing. This can be due to:

- disintegration of the top 10-15mm in the base, forming excessive fines and loss of adhesion between the basecourse and the surface seal. The most common disintegrating rocks are the high silica (quartzites, quartz gravels and sandstones), arenaceous (mainly sandstones), argillaceous (mudrocks, shales) rocks and carbonates.
- breakdown of the basecourse aggregate in service from the repeated action of traffic in the presence of excess moisture (wetter climate, surface water ingress or poor drainage) with the resultant generation or release of plastic fines (generally expansive smectite clays) which change the nature of the materials and significantly reduce the bearing capacity. Basic igneous rocks (dolerite, basalt) are considered the most likely to release plastic fines. Soaking in ethylene glycol can assist in assessing whether this is likely.

In WA, natural materials that have been observed to develop plastic fines with working and/or traffic are tuff and greenstone basalt from the West Pilbara.

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

6.4.1 Disintegration Tests

- Durability Mill test (modified Texas Ball Mill Test to better measure the effect of water and trafficking (Sampson and Netterberg, 1988).
- Los Angeles Abrasion Test (Main Roads WA 220.1).
- Wet/dry crushing tests, such as Wet/Dry Strength Variation (AS 1141.22).
- Sulphate soundness (little used in modern practice).

- Soaking in ethylene glycol and petrographic analyses.

6.4.2 Decomposition Tests

- Secondary Mineral Count.
- Pick and click (Weinert, 1980).

No single test has proved adequate for defining the durability of all different material types. The Los Angeles Abrasion Test is a harsh test when applied to material used as basecourse or subbase on low volume roads, and many of the materials used successfully as basecourse in WA would be rejected on the basis of conventional Los Angeles test criteria. The presence and effects of deleterious minerals are not always shown by the crushing or compression tests. The Durability Mill test provides more suitable assessment for decomposition. Durability testing should be performed on new material sources for which there is no history of performance and on suspect materials, particularly basic igneous rocks.

One simple test is to soak for 24 hours a sample of the coarse aggregate of a material proposed for use. If a significant proportion of the "gravel" size particles has softened to such an extent that they can be crushed by squeezing between the forefinger and thumb, then the material is unlikely to be suitable for use as a basecourse.

Another simple test is to crush the dry gravel particles with a set of hand pliers. For low volume roads, gravel particles that simply crack into two or three pieces probably have adequate particle strength. However if the gravel particles "shatter" into multiple sand particles then further assessment is required.

Sufficient research has not yet been carried out to develop limits for disintegration and decomposition tests for all WA pavement materials.

7 CONSTRUCTION TECHNIQUE AND WORKABILITY

Adequate strength is in itself an insufficient basis for deciding the suitability of a basecourse material (pedogenic or otherwise) for use in road construction. As the material must also be placed and compacted, consideration must also be given to construction techniques and workability.

Construction technique has as much influence on performance as material properties. While techniques must be varied to suit the material being used and the environmental conditions, there are elements which are common to most materials:

- Winning and blending of material.
- Curing.
- Mixing.
- Spreading.
- Dealing with oversize.
- Compaction.
- Material breakdown.
- Surface preparation.
- Drying back.
- Bituminous surfacing.

7.1 WINNING AND BLENDING

Pedocretes and other natural gravels are more variable than crushed rock products. In particular there is often a change in properties with depth in the borrow pit. For this reason winning by scraper is rarely suitable. The material must be pushed up into windrows by dozer to ensure blending, then loaded and carted onto the roadbed. Properly executed stockpiling has been found to reduce material variability. Procedures for pushing up of natural gravel are discussed in more detail in Butkus (2001).

7.2 CURING

Most Main Roads WA specifications require the compaction of natural gravel basecourse materials in the pavement to a characteristic dry density ratio (modified compaction) of not less than some value between 95% and 98%. This density level is easier to achieve, if the materials are cured at moisture content close to the OMC prior to compaction. Curing will assist the establishment of a uniform moisture regime in the fine fraction of the material. This is particularly important with sand-clay base course and materials with a significant sesquioxide content or coating on the particles. Such a regime will then result in greater uniformity of the density of the compacted pavement and easier compaction.

The water may be added to the material on the road, after which it is placed in windrows for curing, or water may be added to the borrow pit via a trickle irrigation system. The curing time varies from about one day for low plasticity lateritic gravels to 4 or 5 days for red sand–clay such as used in the Hamelin–Denham Road.

7.3 MIXING

Mixing on the road bed is required to further reduce material variability. Mixing may be achieved using motor graders or a recycling machine or a combination of both. Typically, material, which has been hauled and tipped onto the road by truck, is then windrowed using a motor grader to achieve a uniform longitudinal spread of material. Mixing may then be achieved using a motor grader by moving the windrow back and forth across the road. Alternatively the material may be blended and spread by the motor grader and then mixed using a recycling machine.

Water commonly needs to be added to achieve or keep the moisture content of the material close to OMC. Material, which is too dry or too wet during mixing, is prone to segregation. Segregation can also be reduced by tilting the grader blade forward.

The use of recycling machines can reduce the water required for compaction in hot arid areas as evaporation loss is much less than with water carts and grader mixing.

7.4 SPREADING AND DEALING WITH OVERSIZE

The presence of very coarse gravel (>37.5 mm), cobbles and of boulders (referred to generally as 'oversize') makes achievement of design level more difficult. Tearing of the surface during final trimming is also a problem when oversize is present, particularly if the oversize particles are hard and strong. Oversize material also introduces uncertainty into the measurement of density.

Almost all WA natural gravels, including lateritic gravel, calcrete and colluvium, contain some oversize materials. The presence of oversize does not preclude the use of a material, but it does mean that the works process must include some pre-treatment by grid rolling, crushing or screening.

Skilled grader operators can sometimes remove some of the oversize in the windrowing and mixing process so that large particles are gradually moved to the outside of the pavement edge. The moisture content needs to be maintained close to the OMC to avoid gravel segregation during this process. For removing oversize, the grader blade should be tilted back.

Graders are more commonly used with natural gravel basecourse than paving machines. Where a paving machine has been used it is common to have to work the top of the material with a grader to close up the transverse cracks created by the vibrating screed on the paving machine.

7.5 COMPACTION

If a tightly bound surface and the full strength potential are to be achieved, compaction moisture content close to OMC must be used. While adequate density can be achieved using very heavy rollers and compacting dry of OMC, the resulting surface is less likely to be satisfactory and the potential strength may not be achieved as no suction strength from drying back is mobilised. Compacting dry of OMC also increases the risk of the unsealed portion of the shoulder being less tightly bound, more permeable and admitting water.

For fine-grained materials (red sand-clay) laminations can be a problem and the basecourse may have to be placed in a single layer and then trimmed to design level. Compaction of sand clay material requires particular skill and excessive rolling may induce shear planes in the material.

The shape of the laboratory compaction curve is an indicator of likely compaction difficulties. It is possible to define a variable, referred to as 'moisture range,' which is an objective measure of the steepness of the moisture density curve on the dry side of OMC. Moisture range is the difference in moisture content between the point on the curve corresponding to a dry density ratio of 100% (i.e. OMC) and the point on the dry side of the curve corresponding to a dry density ratio of 95%. Descriptive terms for various moisture ranges are set out in Table 7.

Table 7: Moisture density curve shape

Curve Shape	*Moisture Range*
Very Flat	5% or negative intercept for moisture at 95% MDD
Flat	2.5% to 5% (dry of OMC)
Medium	1.5% to 2.5% (dry of OMC)
Steep	0.5% to 1.5% (dry of OMC)
Very Steep	Less than 0.5% (dry of OMC)

Materials with medium, steep and very steep curves are likely to require closer moisture control and hence be more difficult to compact than those with flat curves.

Specifications for compaction are typically in terms of characteristic dry density ratio (a statistical measure) or minimum dry density ratio. Experience in WA has shown that the limits shown in Table 8 are applicable to a wide range of natural gravels and caution should be exercised against "specification creep" whereby compaction requirements are progressively raised as this increases the risk of excessive material breakdown. Either a minimum or characteristic value should be specified but not both.

Table 8: Recommended compaction requirements, WA pavements.

Pavement layer, material	*Minimum Dry Density Ratio (modified compaction)*	*Minimum Characteristic Dry Density Ratio (modified compaction)*
Basecourse, natural gravel	95%	97%
Basecourse, bitumen stabilised limestone, crushed ferricrete.	94%	96%
Basecourse, cement or lime modified natural gravel	94%	95%
Subbase, natural gravel, Tamala Limestone	92%	94%

7.6 MATERIAL BREAKDOWN

Some natural gravel exhibits relatively low particle strength and may be vulnerable to breakdown during stockpiling, mixing and compaction. Recycling machines are particularly harsh on materials and can result in excessive breakdown. Fines generated from breakdown may also adversely affect Atterberg limits and linear shrinkage. The degree to which breakdown occurs can be evaluated by performing particle size distribution and Atterberg limits tests before and after laboratory compaction tests and also before and after mixing and compacting the material in the field. Where breakdown is excessive consideration can be given to lowering the compaction limit provided the performance requirements are not compromised.

7.7 SURFACE PREPARATION

For some materials it is necessary to roll the finished compacted surface with light rubber tyre rollers at moisture contents greater than OMC to mobilise the fines (slurrying) so that a tight, impermeable, well-bound surface can be achieved. The amount and details of the water required will depend on the material type and on the proposed bituminous surfacing.

The potential of a material to produce a tight well-bound surface suitable to receive a primerseal can be assessed by consideration of the dust ratio and/or the Maximum Dry Compressive Strength (MDCS, Test Method WA 140.1).

$$\text{Dust Ratio} = \frac{P_{0.075}}{P_{0.425}} \tag{6}$$

Where $P_{0.075}$ is the percentage by mass passing a 0.075 mm sieve
$P_{0.425}$ is the percentage by mass passing a 0.425 mm sieve

For gravels, a dust ratio of less than 0.3 indicates a deficiency of fines, which may lead to high permeability and difficulties in achieving a tight finish. Additional watering, working and rolling of the material (a process variously known as 'water binding, slurrying or slushing') may be necessary to generate sufficient fines to produce a good surface.

For non plastic materials, the potential for a material to bind sufficiently for a well bound surface to be achieved can be assessed using the MDCS test. A MDCS of at least 1700 kPa is desirable.

Prior to application of bituminous surfacing, the surface needs to be swept to remove loose material. The unsealed portion of the shoulder should only be swept very lightly so that the slurry remains on the surface as a waterproofing layer. The binder application rate on the sealed portion of shoulders is typically higher than on the trafficked lanes.

7.8 DRYING BACK

For many materials, the strength at a moisture content of 100% OMC is inadequate. This is not of serious concern provided adequate strength at the generally lower design moisture content is available. However the basecourse material is required to have sufficient strength from the first day of opening to traffic. It is therefore necessary to dry back the compacted basecourse prior to application of the primerseal. Main Roads WA specifications typically require

dry back to less than 85% of OMC where a spray seal is to be applied or 70% of OMC where the final surface is to be asphalt.

Care should be taken that the subbase and subgrade have also been dried back and do not provide a source of moisture to move into the basecourse.

7.9 BITUMINOUS SURFACING IN WESTERN AUSTRALIA

A full explanation of the selection and design of bituminous surfacing for natural gravel basecourse materials is beyond the scope of this report. However there are some general principles relevant to selection of natural gravel for use as basecourse.

Common practice in WA on low and medium traffic volume rural roads is the following:

- Application of a cutback bitumen prime followed by a one or two coat aggregate seal.
- Application of a cutback primerseal (often using sand, crusher dust or 10 mm aggregate) with a final aggregate seal 6 months to 18 months after the initial treatment.

To be suitable for use as basecourse material, gravel must be capable of developing an adequate bond between the bituminous surfacing and the basecourse, which is enhanced by a degree of penetration of the first bituminous application into the basecourse surface.

For some materials, such as Tamala Limestone, modification by stabilisation with bitumen emulsion is necessary for good adhesion of the seal to be developed.

For materials such as crushed ferricrete, which can produce a very 'tight' surface a low viscosity prime (e.g. 40:60 bitumen: MC cutting oil) is required to achieve penetration. Alternatively a "stone mosaic" finish is required so that a primerseal can adhere to the basecourse. Good practice requires that the binder application rate, blend and aggregate spread rate is adjusted during construction to accommodate the base course and aggregate properties so that the design intent is met.

Generally, calcareous materials (calcrete) are the most troublesome and trials may be required to develop an effective bituminous surfacing practice.

The prime or primerseal blend should not be seen as independent of the basecourse or how it has been constructed. Some flexibility in design and application is required to facilitate the effective and economic use of natural gravel. It will generally be cheaper to modify the bituminous surfacing design (e.g. adding more cutter to the prime) than hauling an alternative basecourse gravel for large distances.

Thin layers of dense graded asphalt can have a relatively high permeability (0.1m/day to 1m/day) particularly in areas near the start of a paving run where segregation occurs or at joints between adjoining paving runs. In most cases it is recommended to first prime and seal the basecourse to create a waterproof membrane before application of the dense graded asphalt surface. This is particularly true for gravels with plastic fines but less necessary on local streets with light traffic and crushed Tamala Limestone (which is itself relatively permeable and insensitive to moisture) directly below the asphalt.

7.10 SALINITY

The damage caused to thin bituminous surfaced pavements by soluble salts has been the subject of numerous studies. The most common type of damage is the blistering of thin bituminous surfacings, which is sometimes referred to as salt heave. The blisters take the form of conical surface disruptions and vary in diameter and height. They may be as large as 100 mm in diameter and 20 mm height and may occur across the full width of a surfacing, or as is fairly common, be confined to the edge of the treatment.

The fluffing or powdering of basecourse is another common form of distress. This presents itself as loss of all cohesion immediately beneath the surfacing or in a blister. The result is a lack of bond between the surface treatment and the base and an unstable layer subject to failure under traffic.

Salt damage is most common in arid, semi arid and warm coastal regions, although it is not limited to these regions. It appears that the thinner and more permeable the surfacing layer is, the more likely the damage, primes being most susceptible and heavy multi coat seals least susceptible. Figure 8 provides guidance on assessing the risk of salt damage.

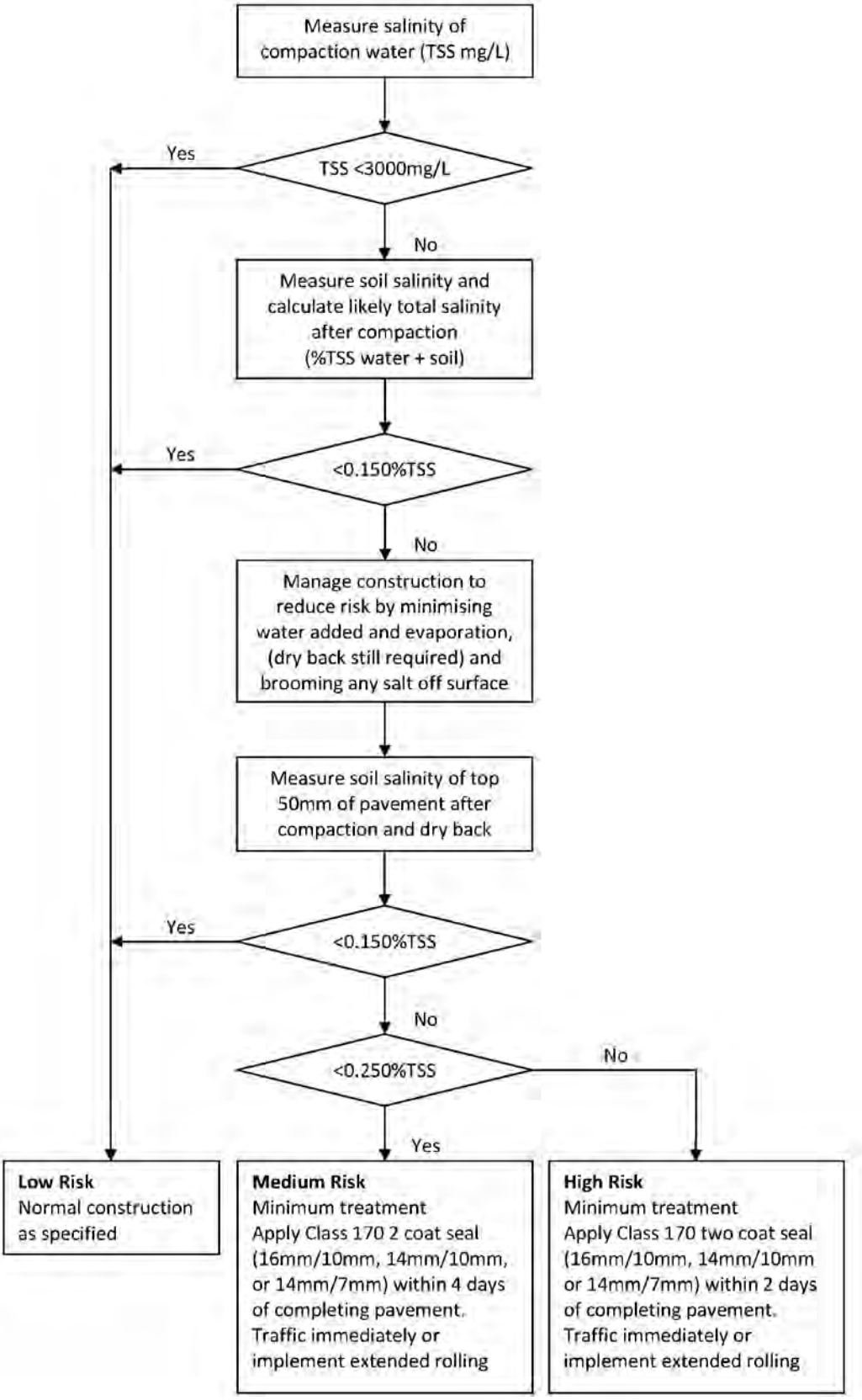

Figure 8: Salinity risk management flowchart.

8 IMPROVING SUBSTANDARD MATERIALS

8.1 MECHANICAL STABILISATION

Not all natural gravels are suitable for use in road construction and some form of improvement may be required to achieve adequate strength and to limit permeability. In some cases, natural gravels can be improved by blending with sand, crusher dust or other inert material, a process referred to as mechanical stabilisation. A method for assessing the likely benefits of mechanical stabilisation is presented below.

The strength and workability of gravel is related to its particle size distribution. As mentioned earlier, acceptable gradings can be defined by equations:

$$\frac{p_1}{p_2} = \left(\frac{d_1}{d_2}\right)^n \tag{7}$$

Where p_1 is the percentage of material finer that d_1
p_2 is the percentage of material finer that d_2
n is the exponent between 0.3 to 0.5

Lloyd (1964) developed a method for estimating CBR based on particle size distribution and linear shrinkage. The method was based on data for gravel and sands in the Moora region of WA.

Lloyd's (1964) method is based on the extent to which the value of "n" in Equation 7 varies from the ideal value of 0.42. The root mean square of n for each pair of successive sieve sizes is calculated. Lloyd (1964) proposed that the soaked CBR of a granular material at a given density condition was proportional to $1/N^3$ where N is a function of n. Lloyd's (1964) work is all the more remarkable as it was completed without the benefit of spreadsheets or a computer. A hand cranked mechanical calculator was used. Lloyd (2014) has recently reanalysed his data. In the revised analyses soaked CBR is approximately proportional to $1/N^{2.5}$.

$$N = \sqrt{\frac{\sum_l^j (n_j - 0.42)^2 + 2.5(n_{0.0135} - 0.42)^2}{j + 2.5}} + 0.42 \tag{8}$$

Where: j is the number of pairs of successive sieves
n_j is the n value corresponding to the jth pair of successive sieves
$n_{0.0135}$ is the n value corresponding to the percent passing the 0.075 mm sieve and the percent finer than 0.0135 mm determined by decantation.

For an 'ideal' material N would equal 0.42. Increasing values of N indicates increasing departure from the 'ideal' grading. The value of Lloyd's approach is not in the absolute determination of basecourse CBR, but rather in a relative mode. The effect of combining two materials can be estimated and optimum blending proportions selected on the basis of particle size distribution tests. The need for expensive and time consuming soaked CBR tests can then be reduced, with only the blend approximating the ideal being fully tested. Two examples of mechanical stabilisation are described in the following sections of this paper.

8.1.1 Mechanical Stabilisation on Newman Tabba Tabba Road (Pilbara Region)

The effects of mechanical stabilisation on basecourse material strength, both in the laboratory (soaked CBR) and in the field (Clegg Impact Test) have been documented by Hardiman (1989) for a section of the Newman Tabba Tabba Road. The pit run gravel was chert/quartzite scree (colluvium) with a marked deficiency of sand. Alluvial sand from the Tabba Tabba Creek was used to improve the grading. The results are summarised in Table 9.

The benefits of improving the grading through mechanical stabilisation are apparent from Table 9. The laboratory strength (soaked CBR) showed a marked improvement and this was confirmed by field measurements using a Clegg Impact Testing device. Subsequent pavement performance has lent further support to the benefits of mechanical stabilisation. The stabilised sections have performed satisfactorily. Short sections left unstabilised for comparison purposes are understood to have developed ruts.

The satisfactory performance of the stabilised material (plasticity index 18) also lends support to the argument that conventional selection criteria can be relaxed for materials in arid areas provided strength is adequate.

The increase in soaked CBR, attributable to the improvement in grading, was reasonably well predicted by Lloyd's (1964) method. The ratio of the $1/N^3$ values was 1.55. The ratio of the soaked CBR values was 1.61.

Table 9: Mechanical stabilisation of natural gravel Newman Tabba Tabba Road (6) (Pit107 000 m).

Material	*Pit run gravel*[1]	*Gravel sand*[1] *blend 3:1*
Particle size distribution		
Sieve Size (mm)	% passing	% passing
37.5	100	100
19.0	83	87
9.5	55	66
4.75	39	53
2.36	29	44
1.18	26	35
0.60	25	25
0.425	24	21
0.300	24	19
0.150	21	16
0.075	17	13
0.0135	9	8
Classification limits		
Liquid Limit (%)	32	29
Plastic Limit (%)	13	11
Plasticity Index (%)	19	18
Linear Shrinkage (%)	9	8
$P_{0.425}$ x PI	471	382
$P_{0.425}$ x LS	216	152
Other limits		
Grading Modulus [5]	2.3	2.2
Dust Ratio $P_{0.075}/P_{0.425}$	0.69	0.62
Chemical analysis on $P_{0.425}$ fraction[2] SiO_2 non quartz (%)	16.9	7.4
Al_2O_3 (%)	12.1	4.8
Fe_2O_3 (%)	7.6	2.9
Silica Sesquioxide ratio	1.7	1.8
Lloyd's N (equation 8)	0.67	0.58
$1/N^3$	3.32	5.13
Soaked CBR (%)	46 (28-54)[3]	74 (61-98)
Clegg Impact Value [4]	41	85

Notes:

(1) Gradings are the average of 6 tests.
(2) Chemical analysis by wet chemical attack and inductively coupled plasma spectrometry. SiO_2 is non-quartz.
(3) Figures in brackets are the range.
(4) Figures quoted are averages. Tests carried out in March 1989 0.4 m from the primerseal edge towards the centreline. The testing followed a period of heavy rain (350 mm).
(5) Grading modulus = $\frac{300-(P_{2.36}+P_{0.425}+P_{0.075})}{100}$
(6) Average annual rainfall at Tabba Tabba is about 300 mm. Average annual pan evaporation is about 3800 mm.

The use of Lloyd's method to assess potential benefits, as described above, assumes that the particle densities of the two materials being blended are similar. If the densities are dissimilar, then the blending and calculation of n values should be based on the percentage passing each sieve by volume, rather than by mass (using particle bulk density to the mass values to apply a correction).

It is essential that during the field stabilisation process, appropriate methods of blending be employed to ensure thorough mixing of the material constituents.

8.1.2 Mechanical Stabilisation on Ord River Irrigation Project (Kimberley Region)

The local naturally occurring laterite gravel was gap graded with weak stone resulting in a fine material after compaction that did not consistently meet basecourse minimum soaked CBR requirement of 80%. To improve grading post compaction, the gravel was blended with crushed basalt aggregate passing 37.5mm and retained on 9.5 mm at a ratio by mass of 3 parts gravel to 1part basalt aggregate. The results are summarised in Table 10.

Table 10: Mechanical stabilisation Ord River Irrigation Project.

Particle Size Distribution Sieve Size (mm)	*Gravel Pre Compaction* % Passing	*Gravel Post Compaction* % Passing	*Gravel aggregate blend 3:1* % Passing
37.5	100	100	100
19.0	97	100	92
9.5	77	95	72
4.75	48	66	51
2.36	38	51	40
1.18	36	46	35
0.425	35	43	31
0.075	8	12	8
0.0135	3	5	3
Plasticity Index	NP		NP
Linear Shrinkage	0.4		0.5
MDCS (kPa)	3400		3100
Soaked CBR(@ 96% MMDD)	60 %(1)		90%(2)
Soaked CBR(near 100% MMDD)			200 % (3)

Notes:
(1) Mean soaked CBR of pit investigation testing (18 test specimens).
(2) Mean soaked CBR of basecourse mixing trial (8 test specimens).
(3) Mean soaked CBR of pit stockpiling blend, high density trials (5 test specimens).

The gravel pre and post compaction gradings together with the blend grading post compaction are shown graphically in Figure 9.

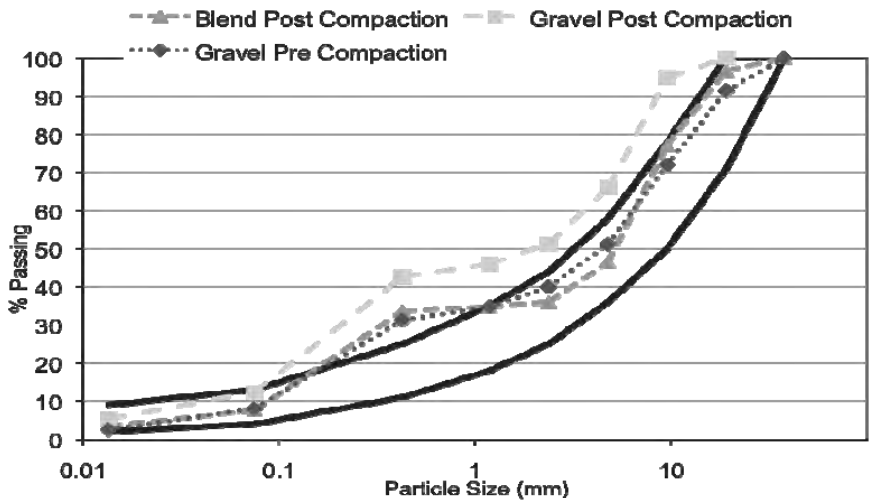

Figure 9: Mechanical stabilisation for Ord River irrigation roads.

The benefits of improving the grading through mechanical stabilisation with aggregate are apparent from Table 10 and the graph in Figure 9 The gravel aggregate blend post compaction is significantly coarser than the gravel post compaction with a marked improvement in soaked CBR.

The average characteristic dry density ratio (modified compaction) obtained during pavement construction was 101%.

8.2 STABILISATION OF LATERITE GRAVELS WITH TAMALA LIMESTONE

In the South West region of WA, the blending of laterite gravels with Tamala Limestone has been undertaken:

- To reduce the quantity and thus haulage cost of lateritic gravel hauled from the Darling Scarp for coastal road construction.
- As a construction expedient to allow road construction during the winter by reducing moisture sensitivity of the lateritic gravel basecourse gravel.
- To reduce the PI of the fines in the lateritic gravel.

The ratio of the limestone/gravel blends varies widely from between 1:1 to 1:3 but may be as low as 1: 10. The blend is typically selected by testing various ratios and selecting the mix which produces the required reduction in plasticity.

Because of the difference in particle density between laterite and limestone (typically 3.2 t/m^3 and 2.4 t/m^3 respectively) the particle size distribution should be determined on the basis of volume rather than mass, however this does not always occur. Where high percentages of limestone are used in a blend, there is a real risk that unless the difference of density is taken into account, a poorly graded material can result, even though the mass based grading is within specification.

Table 11 shows laterite from South Western Highway, south of the township of North Dandalup, prior to blending and after blending with 10% to15% limestone. Particle size distribution is on the basis of mass.

Table 11: Laterite gravel blended with limestone on South West Highway

Parameter	*Laterite gravel prior to blending with limestone*		*Blended product*	*Southwest laterite gravel specification*	*Southwest limestone specification*
Particle Size Distribution					
	Sample 1	Sample 2			
Sieve Size (mm)	% Passing	% Passing	% Passing	% Passing	% Passing
37.5	100	100	100	100	100
19.0	89	85	84	72-100	60-100
9.50	69	56	62	50-78	
4.75	47	36	39	36-58	
2.36	33	25	26	25-44	30-70
1.18	25	19	19	18-35	
0.600	20	16	15		
0.425	18	15	14	11-26	10-50
0.300	16	13	12		
0.150	12	9	8		
0.075	8	6	6	2-13	0-20
0.0135	4	3	4	0-9	
Classification tests					
Parameter	% by mass	% by mass	% by mass	% by mass	
Liquid Limit (% by mass)	26.0	24.3	23.8	≤25.0	
Plastic Limit (% by mass)	19.0	17.3	NP		
Plastic Index (% by mass)	7.0	7.0	NP	≤5.0	
Linear Shrinkage (% by mass)	3.9	3.5	0.8	≤2.0	
Maximum dry compressive strength tests (kPa)					
MDCS (kPa)	General			≥2300	-
	For $CaCO_3$ 30-45%			-	≥1500
	For $CaCO_3$ 45-60%			-	≥1100
	For $CaCO_3$ >60%			-	≥700

8.3 CEMENT STABILISATION

This section of the document deals with cement stabilisation in the following applications:

- Improvement of marginal quality natural gravel that would not otherwise be suitable for use as a basecourse.
- Modification of natural gravel for use in floodways.
- Roads and highways with sprayed seal surfacing.
- Roads and highways with a design traffic of 10^7 ESA or less.

The following applications are specifically excluded from the discussion:

- Cement stabilisation of crushed igneous rock.
- Addition of cement for the purpose of stiffening a granular basecourse under asphalt surfacing.

- Urban freeways.
- Cement treatment of subbase layers.
- Cement treatment of sand-clay materials.
- The process in which the blend of gravel and cement is allowed to hydrate for several days or weeks before placement and compaction.

The addition of small quantities of cement to natural gravel can reduce plasticity and provide some cementitious bonds that contribute to improved performance by increasing shear and bearing strength, especially in wet conditions. In WA only small amounts of cement are added resulting in modified rather than bound pavement layers. Austroads (2013) proposed a revised definition of "modified" as opposed to "bound" materials based on an upper limit on UCS of 1 MPa with assessment at compaction to 100% of Standard Maximum Dry Density and omitting the 4 hour soaking. WA has elected to stay with compaction to a target density, typically 95% of MMDD, and 4 hours of soaking. Current WA practice is to target a UCS (7 day curing, 4 hour soaking, GP cement) of 0.8 MPa. This commonly results in a cement content of less than 2% with most natural gravel types. Consideration should be given to long-term strength gain as there have been examples of delayed hydration and effects from iron oxides in the natural gravel leading to later cracking.

Terminology regarding stabilisation varies between countries and, in Australia's case, between States. In WA the term "stabilisation" is used in a general sense for almost any improvement in soil properties. In the case of cement, stabilised materials are further subdivided into "modified" when the 7 day UCS is less than or equal to 1.0 MPa and "bound" when the 7 day UCS is greater than 1.0 MPa.

In the north of WA type LH cements are often used to provide a longer field working time. The designer needs to consider early strength requirements of the basecourse in selecting the cement type used. The rate of strength gain of type LH is approximately 20% to 30% that of type GP cement with the long-term strength achieved being similar for both cement types. With this in mind the laboratory testing to determine the percent additive is usually conducted with type GP cement. A procedure for assessing working time with cement has been described by Vorobieff (2006).

Cement modification (7 day UCS 0.6 MPa to 1.0 MPa) of the basecourse gravel is standard practice for sealed floodways (low water crossings) on highways and main roads in WA except where flow velocities are so high that concrete pavements are required.

The role of carbonation in the degradation of cement-modified basecourse is a matter worthy of some discussion. Some residual lime is required to retain the high pH required for stability of cement reaction products. The general chemical reaction for carbonation is as follows:

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O$$

The Ca_3Si_2 hydrates from the cement can also absorb CO_2 and swell and lose their structure.

Carbonation is relatively easy to test using Phenolphthalein solution. The role of carbonation in degradation of lime and cement stabilised road construction materials in southern Africa has been described by Paige-Green *et al*. (1990) and Paige-Green (2009, 2012). There have been several dozen cases of road failure attributed to carbonation in southern Africa. Despite the similarities in climate in southern Africa and WA, pavement failure attributed to carbonation is rare with cement modified natural gravel in WA. This is not just a matter of failure to recognise carbonation. Despite the adverse conditions (floodways) in which cement modification is commonly adopted failures of cement modified natural gravel pavements are rare. There are a number of hypotheses as to why carbonation failures are rare in WA:

- WA practice is typically to use cement modified materials that are only marginally outside criteria for use without modification, whereas cement stabilisation of poor quality material is more common in South Africa.
- Low organic content of subgrade soils and hence low CO_2 concentration in the soil.
- The presence of natural cementing agents (Fe_2O_3 and Al_2O_3) in natural gravels in WA that progressively take over the role of cement.
- Primer seals are commonly applied in WA within a few days of cement modification.
- Some (unidentified) aspect of construction practice.

Notwithstanding the rarity of failure due to carbonation in WA, it is prudent to take steps to manage the risk when a cement modified basecourse is used:

- Complete compaction and finishing of cement modified basecourse on the same day that the cement is added.
- Avoid using cement or lime stabilisation as a solution for materials that are highly deficient in gravel size particles.
- Avoid multiple cycles of wetting and drying when working with cement modified basecourse.

- Avoid imposing excessive dry back requirements that result in the unsealed base being exposed to air for long periods of time.
- Ensure any organic rich topsoil and active termite nests are removed from the foundation.
- Apply bituminous surfacing as soon as practicable after cement modification.

The South African Pavement Design Method (SAPDM) (Theyse *et al*., 1996) includes consideration of the crushing performance of cement stabilised layers as well as fatigue performance. Crushing is a mode of failure that occurs in modified basecourse materials and results in compression and shallow rutting of the basecourse under traffic loading. Updated transfer functions for crushing of cement-modified materials are given in Litwinowicz and De Beer (2013). These have not been verified for WA.

8.4 LIME STABILISATION

Lime stabilisation can be used to modify or stabilise a gravel with high plasticity fines. The "conventional" approach would be to conduct range of tests:

- Lime fixation: the percentage of lime that causes the plastic limit to stabilise.
- Lime demand: the percentage of lime to achieve a soil pH of 12.4 or a constant value close to this.
- Strength demand: percentage of lime to achieve an UCS of 1.0 or 1.5 MPa.

In the case where plasticity limits are only marginally exceeded and the deficiency in soaked CBR is minor (say soaked CBR of 60% in lieu of the desired minimum of 80%) then adding 1% or 2% of lime may be adequate. Sufficient lime should be added to raise the soaked CBR to at least 100%. The limited experience of modifying marginal quality lateritic gravels in WA suggests that lower lime contents than are given by the preceding approach have resulted in adequate performance.

Lime stabilisation is rarely used for pavements in WA and standardised design practices for selecting the lime content have not been developed. Three examples of use of lime stabilisation are:

- Stabilisation of Gilgai clay as a lower subbase in a Karratha residential subdivision when there was a shortage of subbase material due to competing demand from iron ore and natural gas projects.
- Modification of marginal quality lateritic gravel on the Dalwallinu bypass on Great Northern Highway.
- Modification of marginal quality lateritic gravel on Great Eastern Highway near Kellerberren.

On the Dalwallinu bypass the lime was added as quicklime (CaO) and then water added in the field to slake the lime and create hydrated lime [$Ca(OH)_2$] before compaction of the gravel.

Results of a laboratory assessment of the effect of lime stabilisation [2% $Ca(OH)_2$] on a lateritic gravel from Cottles pit located near Great Eastern Highway about 212 km east of Perth near Kellerberren are set out in Table 12.

FWD tests on a section of Great Eastern Highway (SLK 201 to 225) with a lime stabilised basecourse 150 mm thick over a 150 mm thick cement-modified sub base showed a trend of curvature ($D_0 - D_{200}$) increasing from 0.06 mm to 0.10 mm between 1999 and 2007 indicating a progressive loss of base stiffness over that time. Pavement performance in terms of roughness has been satisfactory.

For modification of basecourse gravel, cement rather than lime stabilisation is usually preferred in WA.

Table 12: Effect of lime stabilisation on a lateritic gravel from Cottles Pit, Great Eastern Highway.

Material	*Pit run gravel*[(1)]	*Gravel with 2% hydrated lime*[(1)]
Particle size distribution		
Sieve Size (mm)	% passing	% passing
37.5	100	100
19.0	97	98
9.5	77	79
4.75	52	53
2.36	41	42
1.18	36	36
0.425	25	24
0.075	10	8
0.0135	6	4
Classification limits		
Liquid Limit (%)	29	31
Plasticity Index (%)	13	8
Linear Shrinkage (%)	5	3
$P_{0.425}$ x PI	325	192
$P_{0.425}$ x LS	125	72
Other limits		
Grading Modulus [(2)]	2.24	2.26
Dust Ratio $P_{0.075}/P_{0.425}$	0.40	0.33
Maximum Dry Density (t/m^3)	2.171	2.124
Optimum Moisture Content (%)	7.8	8.41
Soaked CBR	58	102

Notes:

(1) Parameters including soaked CBR are the average of not less than 56 tests.

(2) Grading modulus = $\frac{300-(P_{2.36}+P_{0.425}+P_{0.075})}{100}$

8.5 BITUMEN STABILISATION

Bitumen stabilisation has advantages over cement in that it is less brittle, less susceptible to fatigue, and it is less likely to exhibit shrinkage cracking. However, it is relatively expensive.

Bitumen stabilisation can be undertaken using two main methods, either *in situ* or in a pug mill. The pug mill option gives much greater control and uniformity, but is time consuming and expensive. In WA the pug mill option is adopted only for the production of bitumen stabilised Tamala Limestone for new pavements.

For in situ stabilisation, there are again two main methods:

- *In situ* foamed bitumen stabilisation.
- Bitumen emulsion stabilisation.

Both methods can be used either with bitumen only, or combined with active binders such as lime, cement or pozzolanic materials, although these should be limited to a maximum of 2%.

8.5.1 *In Situ* Foamed Bitumen Stabilisation

In situ foamed bitumen stabilisation is an expensive process due to the cost of bitumen and hence optimisation of the bitumen content is required. Often this is assumed to mean adding the amount of bitumen necessary to gain peak material strength properties, but, in reality, the design should consider total bitumen use; it may be preferable to have a lower bitumen content and a thicker pavement that uses less bitumen than an optimised content based on maximizing the material modulus.

There are two potential failure modes with bitumen stabilised pavements:

- Failure by fatigue (cracking).
- Failure by shear (rutting and shoving).

Collings and Jenkins (2011) argue that unlike asphalt, *in situ* foamed bitumen is not homogeneous, nor does the bitumen coat all particles and so the bitumen film is discontinuous. The bitumen is instead distributed as discrete droplets centred on the fines in the mix. Therefore they argue that this material is unlikely to fail by fatigue, but may fail by shear. A fatigue transfer function for *in situ* foamed bitumen stabilised pavements in Perth has been described by Leek *et al.* (2014a).

It is noted that this work was undertaken in South Africa, where different bitumen and active filler contents to those used in WA may have been adopted. In the City of Canning (in WA) where *in situ* foamed bitumen stabilisation has been the mainstay of rehabilitation of heavy trafficked roads, no fatigue failure has been observed, but isolated shear failures have occurred. This does not preclude fatigue at some time in the future when there have been sufficient load repetitions.

By increasing the bitumen content up to a certain point, modulus, fatigue performance and shear performance will increase, but beyond this point, shear performance and modulus will peak, whilst fatigue may plateau. However, all of these properties may not peak at the same binder content and, for materials with rounded particles, shear strength may peak at lower binder contents than those for peak modulus.

Thus if the road is under moderate to heavy traffic, careful characterisation of the material is required. However, the relationship between laboratory characterisation and mixing and actual field performance is not strong and some judgment is required.

8.5.1.1 Investigation for Foamed Bitumen Stabilisation

The investigation of a pavement should consist of a Falling Weight Deflectometer survey to identify changes in pavement structural response, coring to determine the material types and variability found in the pavement and the thickness of the various layers.

In some pavements the pavement makeup may be of sufficient consistency to allow for laboratory characterisation, but some pavements that have been successfully stabilised with bitumen have been so variable that further characterisation was not considered relevant.

Materials should be sampled and laboratory tested for properties such as:

- Optimum active binder for material.
- Variation of modulus or strength with binder content
  - resilient modulus
  - indirect tensile strength.
- Variation of shear strength with binder content (wheel tracking)
  - particularly important with rounded materials under heavy traffic.
- Fatigue performance testing
  - this is not often undertaken as equipment is uncommon
  - a shift factor is applicable between accelerated loading and real time loading, but is not yet determined (Leek *et al*., 2014a).

Sampling should be undertaken using a skid steer profiler, as this gives a balance between replication of the true material breakdown during construction and cost of obtaining the sample.

8.5.1.2 Mix Design for Foamed Bitumen Stabilisation

Bitumen is usually Class 170 complying with AS2008−1997. Whilst most bitumen will foam, bitumen containing silicones may not foam sufficiently and may require a foaming agent. Polymer modified bitumen will not foam.

The foaming properties of the bitumen need to be confirmed by laboratory testing to determine the required water addition ratio. This is based on the following two properties:

- Expansion Ratio: calculated as the ratio of the maximum volume of the foam relative to the original volume of bitumen which should be an absolute minimum of 10.

- Half-life: the time taken in seconds for the foam to collapse to half of its maximum volume which should be greater than 30 seconds.

A particle size distribution (PSD) should be assessed for each material extracted by the skid steer profiler from each representative pavement section. This will indicate the extremes from fine to coarse over the pavement section, and indicate if separate characterisation trials are required.

Where the grading curves are similar and do not vary by more than 10% for any sieve size over 2.36 mm, or by 5% for any sieve size less than 2.36 mm, the mean percentage values of all pavement layers should be used. Materials suitable for foamed bitumen stabilisation have PSD within the envelopes shown in Figure 10.

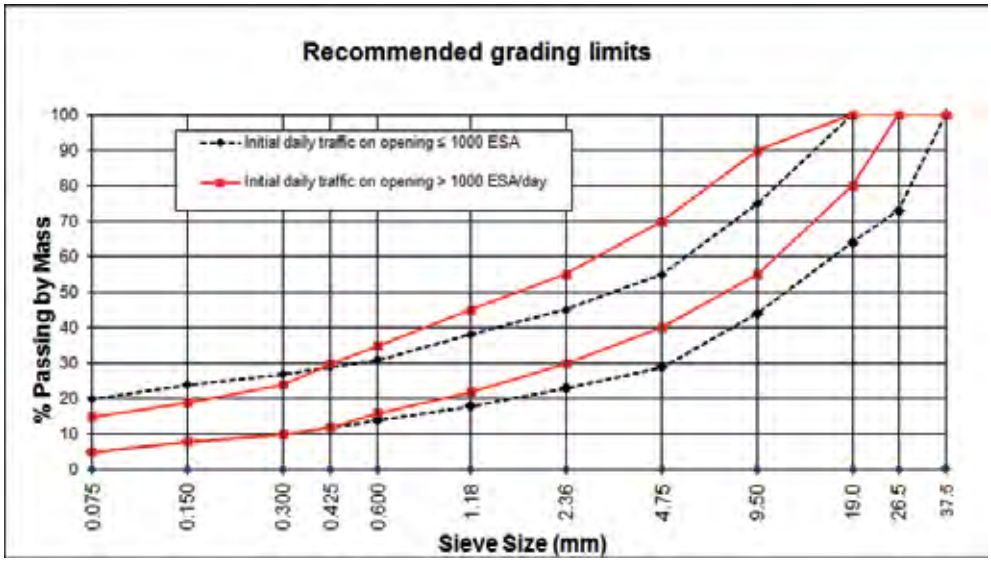

Figure 10: Various grading limits for foamed bitumen stabilisation (Source Austroads 2011b AP-T178/11).

Whilst many references suggest that the 75 micron size material should exceed 5%, experience in WA suggests that material with as little as 2% finer than 75 microns will stabilise satisfactorily.

The Plasticity Index (PI) for materials suitable for foamed bitumen stabilisation is recommended to be a maximum of 10%. However as lime is a material added to the mix, a higher PI may be acceptable with additional lime.

The method currently adopted to assess the optimum combination of bitumen and active filler contents is described in Appendix A of Austroads (2011b) Technical Report AP-T178/11 *"Review of Foamed Bitumen Stabilisation Mix Design Methods"*. Recommended initial target values are shown in Table 13.

Table 13: Recommended target binder contents for foamed bitumen stabilisation.

Percent passing		*Target bitumen content (% mass)*	*Plasticity index (%)*	*Target hydrated lime (% mass)*	*Target moisture content (% OMC)*
4.75 mm sieve	*0.075 mm sieve*				
< 50	5−7.5	3.0	6−10	2.0	75
	7.5−15	3.5			
	15−20	4.0	3−6	1.5	70
> 50	5−7.5	3.5			
	7.5−15	4.0	< 3	1.0	70
	15−20	4.0			

Source: Ramanujan J & Jones J (2008)

Cement may also be used in lieu of hydrated lime, particularly if high early strength is required, but quantities will need to be adjusted. In the construction stage, quicklime is often used instead of hydrated lime and the equivalency of hydrated lime must be calculated.

To assess the optimum bitumen and active filler combination, the characteristic modulus values need to be compared against specification requirement. Table 14 shows values adopted by Department of Transport and Main Roads in Queensland.

Table 14: Foamed bitumen stabilisation: recommended minimum modulus values used by DTMR Queensland.

Average initial daily traffic (ESA)	*Initial 3hr modulus (MPa)*	*3 day cured modulus (MPa)*	*3 day soaked modulus (MPa)*	*Min. retained modulus ratio (soaked/cured)*
< 100	500	2500	1500	0.40
100−1000	700	3000	1800	0.45
> 1000	700	4000	2000	0.50

Source: Ramanujan J & Jones J (2008)

It should be noted that:

- Maximum modulus need not be targeted.
- Binder content should relate to the traffic loads.
- Modulus is not the only criterion used to establish the 'optimum' binder content.
- Wheel tracking testing should be undertaken on the adopted mix when traffic exceeds 500 ESA/day.
- Local experience should be used to establish the most effective bitumen binder content along with the addition of active filler.

Having selected a bitumen and active filler combination from the modulus testing, rut resistance should be determined when the design traffic during the first year of service exceeds 500 ESA/day, or where identified by aggregate angularity requirements. Experience in WA shows that natural lateritic gravels have a high risk of shear failure under heavy traffic, and that aggregate angularity is an important criterion in determining mix design.

The fine aggregate angularity should be investigated for stabilisation of pavements carrying in excess of 300 ESA/day. The material should be tested in accordance with Test method ASTM D 3398: Index of Aggregate Particle Shape and Texture. The particle index for fine material is calculated as:

$$I_a = 1.25V_{10} - 0.25V_{50} - 32 \tag{9}$$

where: I_a = particle index
V_{10} = voids in aggregate compacted to 10 drops per layer
V_{50} = voids in aggregate compacted to 50 drops per layer

If the particle index is less than 10, the material should be tested for rutting resistance. If the particle index is greater than 10, rutting resistance need only be tested for pavements carrying in excess of 500 ESA/day.

The coarse aggregate angularity should be investigated for stabilisation of pavements carrying in excess of 500 ESA/day.

The coarse material should be tested in accordance with Test method ASTM D 3398: Index of Aggregate Particle Shape and Texture. The particle index for coarse aggregate is calculated as:

$$P = 100\left(\frac{F}{F+N}\right) \tag{10}$$

where: P = percentage of particles with the specified number of fractured faces
F = mass or count of particles with at least the specified number of fractured faces = 2
N = mass or count of particles not having the specified number of fractured faces

If P is less than 50%, the material should be tested for rutting resistance in all cases. If P ≥ 80% rutting need only be tested for pavements carrying in excess of 500 ESA/day.

Rutting resistance is assessed by the wheel-tracking test where equipment is available. Samples are tested in accordance with deformation resistance of asphalt mixtures by the wheel-tracking test. The measured rut depth is then compared against the specification requirement. The specification limits adopted by Department of Transport and Main Roads Queensland (DTMRQ) are shown in Table 15.

Research at Curtin University on foamed bitumen stabilisation in WA is summarised in Leek *et al.* (2014a, b and c).

Table 15: DTMR Q limits for wheel track test on foamed bitumen stabilised basecourse.

Average initial daily traffic (ESA)	*Max rut depth at 2000 cycles (mm)*	*Max rate of rut progression (mm/kilocycle)*
< 100	10	0.20
100−1000	7	0.15
> 1000	5	0.10

8.5.1.3 Construction with *In Situ* Foamed Bitumen Stabilisation

In situ foamed bitumen stabilisation relies on mixing active binder and bitumen with water for compaction into the pavement. There are two options available:

- mix lime and water into the pavement, grade and lightly compact to finish level, and add bitumen in a second pass
- a single pass where lime, water for compaction and bitumen are added simultaneously.

When cement is used as the active filler, the single pass operation should be used. However, unless a curing period is required for highly plastic materials, experience has shown that a single pass operation provides excellent results and saves considerably in time and mixing costs. Care must be taken with multiple passes where the material breaks down (becomes finer) with each pass. Works at City of Canning in WA have shown that a third mixing pass results in lower compacted density.

Active filler should be spread with a purpose built spreader capable of spreading accurately at rates as low as 5 kg/m². Active filler may be cement, hydrated lime or quick lime. Lime content is expressed as a rate of hydrated lime and, where quicklime is used, a factor of 0.76 should be applied to convert the design application rate. Where quicklime is used, it must be fully slaked to ensure complete conversion to hydrated lime occurs before mixing. It must be remembered that cement has a higher density than lime thus the volume of cement is less for the same nominal percentage by mass.

If lime is to be blended as a first mixing pass without bitumen, the lime is blended into the pavement using the stabiliser coupled to the water cart. Mixing should be undertaken to 80% of the final mix depth and water added to approximately 80% of OMC. The mix will appear dry, but the addition of bitumen will allow for sufficient fluidity for compaction. It is essential that the pavement be returned to its pre-existing design levels prior to the next bitumen pass, or inaccurate thickness will result.

Figure 11: Foamed bitumen stabilisation, single pass mixing operation.

Following the spreading of lime, or addition of lime and water followed by reshaping as appropriate, the bitumen is added to the pavement via a bitumen tanker coupled to the stabiliser. In a single pass, the bitumen tanker and water cart are linked into a single train as shown in Figure 11. An experienced operator is required to follow the stabiliser to ensure correct bitumen dispersion and moisture content.

Following bitumen stabilisation, compaction is undertaken initially with a heavy pad foot roller. Experience has shown that a 12t vibrating pad foot roller will be sufficient for up to 320 mm thick layers. If vibration cannot be used due to adjacent structures, an 18 tonne pad foot roller in static mode has given the required compacted density. After completion of rolling with the pad foot roller, (taken as being when the roller has "walked out" and the pads are no longer penetrating the material), the area is trimmed and, final rolling being undertaken with a combination of a grader and smooth drum roller, followed by finish rolling with a rubber tyre roller.

8.5.1.4 City of Canning Experience with Foamed Bitumen Stabilisation

The following points have been noted from 14 years of experience in City of Canning:

- Traffic can safely be diverted onto the pavement immediately after finishing.
- The stabilised surface is tight, dust free, free of excessive loose aggregate and is moisture repellent.
- Under heavy traffic, or in intersections, bitumen contents of less than 3.5% to 4% are vulnerable to ravelling if surfacing is delayed beyond 2 days.
- A minimum of 2 days dry back should be allowed prior to surfacing.
- Primer sealing is not required.
- Cement can be used in lieu of lime if high early strength is critical.
- Natural gravels with rounded particles (e.g. lateritic gravel) must be treated with caution, as they are prone to shear failure under very heavy traffic. Consideration to adding mid to coarse size highly angular aggregate should be given prior to stabilisation.

8.5.2 *In Situ* Stabilisation with Bitumen Emulsion

In situ bitumen emulsion stabilisation can be described and characterized in a similar manner to foamed bitumen. Like foamed bitumen stabilisation, bitumen emulsion can be blended with lime or cement, but is often used without active fillers.

Stabilisation with bitumen emulsion is often described as being easier than foamed bitumen, as a tanker containing the emulsion is connected to the recycler and used without the inconvenience of heating and having to conduct a long list of checks.

Bitumen emulsions take longer to cure than foamed bitumen and do not reach the high levels of stiffness that foamed bitumen achieves. Due to the high water content inherent in emulsions, the practical limit for residual bitumen is usually around 2.5% (higher for Tamala Limestone).

8.5.3 Differences between Stabilisation with Foamed Bitumen and Bitumen Emulsion

The following variations are noted between design and construction of bitumen emulsion stabilised pavements compared to foamed bitumen stabilised pavements as noted in the Technical Guideline TG2: Bitumen Stabilised Materials (Asphalt Academy 2009), with additional advice based on WA experience:

- Bitumen emulsion coats the larger aggregate particles to a greater extent than foamed bitumen. A minimum filler content of 2% is sufficient, thus the fines content is less critical.
- Minimization of the Voids in Mineral Aggregate (VMA) is important for emulsion stabilisation, but not as essential as for foam.
- Selection of the bitumen emulsion type for treatment is influenced by the type of aggregate to be treated. Slow setting cationic emulsions are suitable for most aggregates. Tamala Limestone is a notable exception and slow setting anionic emulsion is used with this material.
- pH levels of the water must be checked, as must the compatibility of the bitumen emulsion and the water. These checks are done by performing a dilution test.
- Breaking process with anionic bitumen emulsions is a mechanical process (evaporation), whereas cationic bitumen emulsions produce a chemical break. For dense mixtures, more time is needed to allow for mixing and placement and slower breaking times are required.
- An emulsion mix needs sufficient curing time to acquire sufficient stiffness and cohesion between particles before carrying traffic.

- Rough textured and porous aggregates reduce the breaking and setting time by absorbing water contained in the bitumen emulsion.
- Moisture content of the mix prior to mixing influences breaking time.
- Moisture content of the mix after compaction influences curing rate.
- A minimum of 1 to 2% moisture is required in the aggregate prior to adding the bitumen emulsion.
- Blender-type laboratory mixers and flat-pan mixers with a rotary mixing motion are suited to mix preparation for bitumen stabilised materials (emulsion). Care should be taken to ensure that bitumen does not remain on the mixing drum or paddles, as this will influence the final bitumen content.
- Emulsion usually requires longer curing times due to the higher moisture contents.
- Care must be exercised when adding bitumen emulsion to an *in situ* material that has high moisture content. An over-application on the overlap will result in soft spots in the material along the full length of the longitudinal joint.
- Emulsion can be washed out of the material if additional water is used during compaction before breaking. Water containing 10% to 15% diluted bitumen emulsion should be used when adding moisture and be sprayed across the full recycled width after the initial compaction and immediately ahead of the grader. The effect is to create a binder rich surface layer than can better handle traffic prior to sealing. Again Tamala Limestone is an exception and use of water with a small amount of added detergent is used to aid compaction. The selected detergent must be compatible with the emulsion.

8.6 CHEMICAL STABILISATION

Proprietary chemicals are occasionally proposed to improve materials for basecourse use, but there is limited well documented experience of their successful use. Some of these are:

- Wetting agents (to improve compaction).
- Hygroscopic salts (e.g. calcium, magnesium or sodium chlorides).
- Natural polymers (e.g. ligno sulfonates).
- Synthetic polymer emulsions (e.g. acrylates).
- Modified waxes.
- Sulfonated oils.
- Biological enzymes.

Proprietary chemicals have been successfully used as dust suppressants on unsealed roads, especially in mining operations in WA. There are numerous products available and the suitability of a particular product can initially be assessed in a laboratory by performing a brush test. Often the best way to assess suitability is to trial the product in the field. Chemical dust suppressants have a limited life and periodic application of the product is required. Further information may be found in AustStab (2011).

9 SELECTION CRITERIA FOR WESTERN AUSTRALIAN BASECOURSE MATERIALS

9.1 LATERITIC GRAVEL (FERRICRETE) IN WESTERN AUSTRALIA

One of the difficulties in discussing lateritic gravels is the lack of an internationally agreed definition of the term “laterite”. The word laterite is derived from the Latin word “later” which means “a brick”. The term laterite was originally applied to a soft rock in India, rich in aluminium and iron oxides, which could be cut by hand and when exposed to the air hardened to form a building brick.

There are numerous definitions of the term “laterite” now in use. A common theme to almost all of them is that there has been an enrichment by aluminium and iron oxides (sesquioxides) during the weathering process. This may have occurred by the leaching out of SiO_2 or by the accumulation of the sesquioxides through groundwater fluctuations. The sesquioxide content of most WA “laterites” exceeds 10%.

The results of chemical analysis tests for a range of WA lateritic gravels are presented in Table 16. Results of tests on crushed granite are included for comparative purposes.

Table 16: Chemical composition[1,2] of some WA lateritic gravels and granite road bases

Statistic	Laterite Gravel			Crushed Granite Rock Base		
	$A1_2O_3$	Fe_2O_3	SiO_2[3]	$A1_2O_3$	Fe_2O_3	SiO_2[3]
Average (% by mass)	12.8	6.8	16.8	2.0	1.6	6.8
Standard deviation (% by mass)	7.1	2.6	6.4	0.4	0.2	1.1
Number of samples	13	13	13	3	3	3

Notes:
(1) Expressed as a percentage by mass of the fraction passing a 0.425 mm sieve.
(2) Determined by wet chemistry/inductively coupled plasma spectrometry.
(3) SiO_2 is the non quartz SiO_2.

In some published literature, the use of the term "lateritic" is limited to soils with a silica sesquioxide ratio (S/R) of less than 2. However not all materials regarded as lateritic in WA satisfy this definition.

The silica sesquioxide ratio (S/R) is given by:

$$S/R = \frac{\frac{SiO_2}{60}}{\frac{Al_2O_3}{102} + \frac{Fe_2O_3}{160}} \quad \textbf{(11)}$$

Precise instructions must be given to the analytical laboratory when commissioning tests to measure the Fe_2O_3, Al_2O_3 and non quartz SiO_2 content. If the instructions are vague, then it may be total iron, total aluminium and total silicon that are measured rather than the available oxidised material. The “wet chemistry” method to measure the oxide content is set out in Departmento Nacional De Estradas De Rodagem (1972). If using a method such as ICP, then the Fe_2O_3 and Al_2O_3 must first be extracted by boiling in dilute sulphuric acid (1 part acid to 1 part water). The available silica must be extracted by boiling in 5% Na_2CO_3.

The term "lateritic gravel" as used in this paper includes all natural gravels of lateritic origin including both *in situ* pisolitic and transported material.

9.1.1 Occurrence of Lateritic Soil in Western Australia

Indurated ferricrete and associated pisolitic gravels (lateritic gravels) are the most common surface rocks found in WA (see Figure 12) and have been used extensively for road construction.

* Based on 1: 250 000 Geological Map Series: Geological Survey of WA.

Figure 12: Distribution of lateritic soils and rocks in Western Australia

9.1.2 Selection Criteria for Lateritic Gravel

The properties and behaviour of lateritic gravels used for road construction in WA have been reviewed by Cocks and Hamory (1988). The general approach to the application of these criteria is to representatively sample the material deposit and conduct a testing program based on a large number of relatively simple classification tests to characterise the material and a lesser number of more complex strength tests to enable a correlation between the two. For naturally occurring materials, this usually results in a spread of results, a proportion of which may be outside the criteria limits. In these circumstances the decision on the suitability of the material as basecourse or subbase for road construction is usually a risk based one. Judgement with experience is required to assess the results against the criteria and the predicted in-service design conditions. The likely extent of material breakdown during construction should be assessed and the target horizon in the deposit selected so that the material is likely to fall within the selection limits after breakdown.

The sampling and testing regime should be repeated for the material after it is stockpiled, as the stockpiling process commonly produces material different from that sampled in the deposit. This difference results from variation from the target excavation depth, mixing or segregation of soil horizons and particle breakdown. Experience has shown that stockpiling is a highly critical process. An experienced dozer operator must be used with direct supervision of the stockpiling by experienced technical personnel.

Selection of lateritic gravel for use as basecourse in a sealed road needs to take into consideration many factors including:

- Climate.
- Drainage.
- Traffic Loading.
- Presence of Road Trains.
- Road Geometry.

Gentilli (1972) divided WA into 7 climatic zones as shown in Figure 13.

A matrix of climate and traffic combinations for well-drained conditions is presented in Table 17. For the sake of simplicity, typical selection criteria have been prepared for lateritic gravel as shown in Table 18, Table 19, and Table 20.

Table 17: Required classification numbers for lateritic gravel (well drained conditions).

Climate	*Traffic Loading (ESA)* [(6)]				
	$\leq 5x10^6$	$\leq 10^6$	$\leq 5x10^5$	$\leq 10^5$	$\leq 5x10^4$
Sub humid hot	Lt6[(7)]	Lt6	Lt6	Lt10	Lt10
Semi arid hot	Lt10	Lt10	Lt10	Lt16	Lt16
Arid hot	Lt10	Lt10	Lt16	Lt16	Lt16
Arid warm	Lt10	Lt10	Lt16	Lt16	Lt16
Semi arid warm	Lt10	Lt10	Lt10	Lt10	Lt16
Sub humid warm	Lt6	Lt6	Lt6	Lt10	Lt10
Humid warm	Lt6	Lt6	Lt6	Lt6	Lt6

Notes:

(1) The matrix is for well drained (drainage unit 1) conditions only.
(2) The effect of a permeability inversion (drainage unit 2) can be accommodated by assigning a more adverse climatic zone.
(3) The effect of a high water table (drainage unit 3) can be accommodated by assigning a more adverse climatic zone.
(4) The effect of additional applied horizontal stresses due to road trains, braking at intersections and steep grades should be allowed for by assigning a higher traffic loading.
(5) The use of lateritic gravel for roads carrying more than $5x10^6$ ESA is not precluded. However specialist advice should be obtained, particularly in relation to the selection of bituminous surfacing.
(6) Traffic loading refers to the first 20 years of life.
(7) The designation of Lt6, Lt10 and Lt 16 relate to plasticity index limits of 6, 10 and 16 respectively.

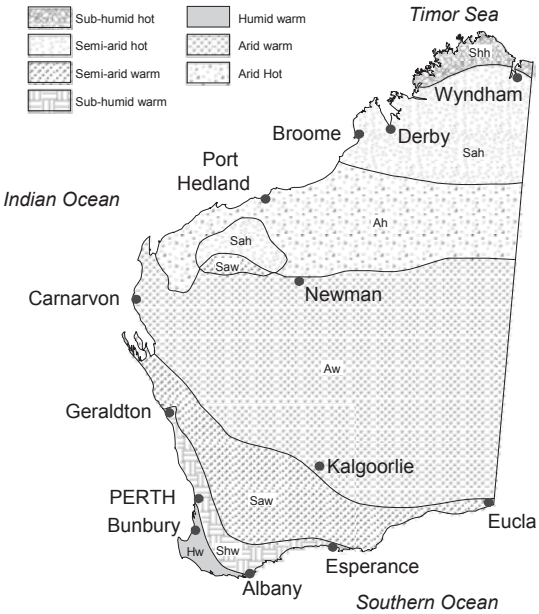

Figure 13: Climatic regions of Western Australia Thornthwaite's method (After Gentilli 1972).

Table 18: Selection criteria for lateritic gravels compared to crushed granite road base[1]

Type of Material	*Lateritic Gravel*			*Crushed Granite Rock* [2]
Designation(3)	Lt6	Lt10	Lt16	
Particle size distribution				
Sieve Size(mm)	% Passing	% Passing	% Passing	% Passing
37.5	100(4)	100(4)	100(4)	
26.5				100
19.0	71-100	95-100	95-100	95-100
13.2				70-90
9.5	50-81	50-100	50-100	60-80
4.75	36-66	36-81	36-81	40-60
2.36	25-53	25-66	25-66	30-45
1.18	18-43	18-53	18-53	20-35
0.60				13-27
0.425	11-32	11-39	11-39	11-23
0.30				8-20
0.15				5-14
0.075	4-19	4-23	4-23	5-11
0.0135	2-9	2-11	2-11	
Classification limits				
Dust Ratio(6)	0.3-0.7	0.3-0.7	0.3-0.7	0.35-0.6
Liquid Limit (7) (%)	≤ 35	≤ 35	≤ 35	≤ 25
Plasticity Index (%)	≤ 6	≤ 10	≤ 16	NS
Linear Shrinkage (%)	≤ 3	≤ 5	≤ 8	0.4-2.0
$P_{0.425}$ x LS(8)	≤ 150	≤ 200	≤ 250	NS(11)
Other test limits and minimum dry back requirements				
Maximum Dry Compressive Strength[9] (kPa)	≥ 1700	≥ 1700	≥ 1700	≥ 1700
Particle Toughness	Note [10]	Note [10]	Note [10]	LAA ≤ 35%
Dry back (%)	≤ 85[12]	≤ 85[12]	≤ 85[12]	≤ 60[12]

Notes:

(1) Selection criteria apply to basecourse for roads with a thin bituminous surfacing with a 20 year design traffic loading of up to $5x10^6$ ESA. For higher traffic loadings, specialist advice should be sought, or Table 19, should be used.
(2) Non lateritic, included for comparison purposes only. MRWA Specification 501.
(3) See Table 17 for applicable climate zones and traffic loading.
(4) Most deposits of lateritic gravel contain same oversize material which must be broken down by grid rolling.
(5) Dry sieving and decantation, Test Method WA 115.1.
(6) Dust Ratio is the percentage passing 0.075mm divided by percentage passing 0.425mm. Material with a low dust ratio is likely to be harsh and the achievement of a satisfactory surface may prove difficult.
(7) Liquid Limit (using the cone apparatus), Plasticity Index and Linear Shrinkage tests on samples air dried at 50°C.
(8) For materials approaching the upper limit for plasticity index or P 0.425 x LS confirmation of suitability by strength testing is recommended.
(9) Maximum Dry Compressive Strength, Test Method WA 140.1
(10) No particular test for particle toughness is specified for lateritic gravel at this time. However the lateritic pebble must be hard and durable. LAA is Los Angeles Abrasion.
(11) NS = not specified.
(12) Basecourse should be dried back to a moisture content of less than 85% (60% for Crushed Rock) of OMC prior to application of bituminous surfacing.

Table 19: Selection criteria lateritic gravels for heavy duty pavements (1)

Particle size distribution		
Sieve Size(mm)	*Target Value % passing*	*Range % passing*
37.5	100	100
19.0	80	72-100
9.50	57	50-78
4.75	43	36-58
2.36	31	25-44
1.18	23	18-35
0.600	18	13-28
0.425	15	11-25
0.300	13	9-22
0.150	9	6-17
0.075	7	4-13
0.0135	4	2-9
Classification limits		
Liquid Limit (4) (%)	≤ 35	
Linear Shrinkage (%)	≤ 2	
Plasticity Index	≤ 5	
Strength tests and minimum dry back requirements		
MDCS (5) (kPa)	≥ 2300	
Soaked CBR (6) (%)	≥ 80	
Dust Ratio(7)	0.3-0.7	
Dry back (8) (%)	≤ 85	

Notes:

(1) Selection criteria applies to basecourse roads with a thin bituminous surfacing with a 20 year design traffic loading of up to 2 $x10^7$ ESA. For higher traffic loadings, specialist advice should be sought. Main Roads WA does not permit the use of lateritic gravel on freeway pavements.
(2) The particle size distribution must conform as closely as possible to the target grading. Gap graded materials should not be used.
(3) Dry Sieving and Decantation, Test Method WA 115.1.
(4) Liquid Limit, (using the cone apparatus) and Linear Shrinkage tests on samples air dried at 50°C.
(5) Maximum Dry Compressive Strength, Test Method WA 140.1
(6) California Bearing Ratio in accordance with Test Method WA 141.1 or AS1289.6.1.1. Specimen for soaking prepared at 96% of MDD and 100% of OMC
(7) Dust Ratio is the percentage passing 0.075mm divided by percentage passing 0.425mm.
(8) Basecourse should be dried back to a moisture content of less than 85% of OMC prior to application of bituminous surfacing

Table 20: Selection criteria lateritic gravel for light duty pavements.(1)

Climate and Traffic Designation(2)	Lt6	Lt10	Lt16
CBR (3) Soaked (%)	≥ 80	≥ 60	≥ 60
CBR Unsoaked (%)	≥ 80	≥ 80	≥ 80
Maximum Size(4) (mm)	37.5	37.5	37.5
Grading Modulus(5)	≥ 1.5	≥ 1.5	≥ 1.5
Dust Ratio(6)	0.3-0.7	0.3-0.7	0.3-0.7
Plasticity Index (%)	≤ 6	≤ 10	≤ 16
Linear Shrinkage (%)	≤ 3	≤ 5	≤ 8
$P_{0.425}$ x LS	≤ 150	≤ 200	≤ 250
Particle Toughness	Note(7)	Note(7)	Note(7)
Dry back (8) (%)	≤ 85	≤ 85	≤ 85

Notes:

(1) Selection criteria apply to basecourse roads with a thin bituminous surfacing with a 20 year design traffic of up to $5x10^6$ ESA. For traffic exceeding $5x10^6$ ESA, specialist advice should be sought or Table 19 should be used.

(2) See Table 17 for applicable climate zones and traffic loading.

(3) CBR specimens compacted to OMC to the specified density for the project and tested at design unsoaked moisture conditions. Test method WA 141.1 or AS1289.6.1.1

(4) Most deposits of lateritic material include some oversize material which must be broken down by grid rolling.

(5) Grading Modulus = $\frac{300-(P_{2.36}+P_{0.425}+P_{0.075})}{100}$

(6) Dust Ratio is the percentage passing 0.075 mm divided by percentage passing 0.425 mm.

(7) No particular test for particle toughness is specified at this time. However the lateritic pebble must be hard and durable.

(8) Basecourse should be dried back to a moisture content of less than 85% of OMC prior to application of bituminous surfacing.

9.2 CRUSHED LATERITIC CAPROCK

Since the early 1990's, the practice of crushing massive lateritic caprock for use as a basecourse is common in the south-west region of WA. In addition to providing a resource as a road construction material, the crushing practice has had the advantage of cleaning up stockpiles of rock, left over from gravel pushing up operations, which were unsightly and had become a refuge for vermin such as foxes and rabbits.

A typical winning and crushing operation involves ripping of the caprock with a large dozer, breaking up the larger boulders with a hydraulic rock breaker and loading the material into an impact crusher. The feedstock into the crusher may or may not include some naturally occurring lateritic gravel and sand. The resulting product looks similar to lateritic gravel except that the particles are angular rather than rounded (pisolitic).

Crushed lateritic caprock and laterite gravel to the heavy-duty basecourse specification (Table 17) has been successfully used as basecourse on major State Roads carrying high volumes of heavy traffic (20 year design traffic up to 5 x 10^7 ESAs) such as Great Eastern Highway, Great Northern Highway, Albany Highway, Mandjoogoordap Road, South Western Highway, Coalfields Highway and Bussell Highway.

Naturally occurring laterite gravel and crushed laterite caprock complying with the heavy-duty basecourse specification has been successfully used to construct sealed roads for heavy duty mine trucks at Premier Coal Mine, Griffin Coal Mine and the Alcoa Bauxite Mine. The gross tonnage of the mine trucks is 300 to 400 tonnes on six wheels hauling up to one million tonnes of product per month. The basecourse gravel is 300 mm thick, placed on a coarse rock sub base material 500 mm to 1500 mm thick depending upon the strength of the subgrade. The subbase is coarse, pit run gravel, ripped by large dozers then hauled to site, placed and compacted. The subbase contained boulder size rock to about 500 mm in diameter, but predominantly 200 mm to 300 mm size.

Particular care must be taken in the production of crushed lateritic caprock, to remove roots and other sources of wood from the material prior to crushing. It will generally be necessary to do this by hand. Once material is crushed, the resulting wood chips are hard to locate and remove. Due to expansion and/or contraction with moisture changes, wood chips near the surface are likely to initiate the formation of potholes.

The selection of bituminous surfacing is critical to the successful use of crushed massive laterite. The crushed material can produce a very tight surface, which is difficult to penetrate with conventional primer seals. Opinions on the appropriate surfacing vary. Some practitioners recommend the use of a 40:60 (bitumen:MC cutter) prime prior to primersealing. Others recommend brooming the ferricrete surface to achieve a stone mosaic finish and then applying an initial single coat hot bitumen seal (10 mm aggregate) or a double/double seal (5 mm/10 mm) with bitumen emulsion followed by a 14 mm hot seal within two years. For heavy-duty mine roads, a two coat emulsion primerseal followed within 12 months with a single coat 20 mm aggregate with granulated rubber bitumen binder, has been used.

Selection criteria for crushed lateritic caprock are the same as for natural lateritic gravel with additional requirements that the Los Angeles Abrasion value be less than or equal to 50% and the Flakiness Index be less than or equal to 20%. A minimum point load index of 1 MPa may be used in lieu of Los Angeles Abrasion. Basecourse should be dried back to a moisture content of less than 85% of OMC prior to application of bituminous surfacing. Background to the development of crushed lateritic caprock as a basecourse material has been described by Coffey (1997).

RLT testing was carried out on crushed laterite caprock (ferricrete) used as basecourse on a trial section on Kwinana Freeway. Modulus (240 kPa mean stress and 120 kPa shear stress) of samples compacted to about 98% MMDD and cured for 28 days, varied from about 750 MPa at a moisture ratio close to 85% up to about 1700 MPa at a moisture ratio close to 65%. There was a strong negative correlation ($R^2 = 0.95$) between modulus and moisture ratio.

9.3 CALCRETE IN WESTERN AUSTRLALIA

One of the difficulties with terminology in relation to calcrete and calcarenite is the lack of consistency both within Australia, between geologists and pedologists, for example, and internationally. A number of other terms such as caliche, kankar, soil travertine, calcareous duricrust and limestone are also in common use, though with varying definitions.

In this report the terms calcrete is used as a general term. A system based on Netterberg and Caiger's (1983) has been adopted to describe the texture of various calcrete materials.

Calcareous Soil: Soils containing some carbonate but with little cementation or development of nodules.

Calcified Soil: A soil horizon that is very weakly to well cemented by carbonate. The carbonate is evenly distributed. Most aeolianite could be classified (for pavement engineering purposes) as calcified sands. Only after excavation can such materials be said to have a particle size distribution.

Powder Calcrete: Loose silt or fine sand sized carbonate with no visible nodules. It may be cemented but breaks down completely when worked.

Nodular Calcrete: Natural mixtures of silt sized to gravel sized particles with more than 50% coarser than 2 mm or a grading modulus greater than 1.5.

Honeycomb Calcrete: As the nodules in nodular calcrete grow, they may become cemented together to form honeycomb calcrete.

Hardpan Calcrete: Deposits of indurated and strongly cemented, usually massive calcrete forming a rock like horizon typically about 1 m thick.

Calcrete Boulders and Cobbles: Formed by the weathering of hardpan and honeycomb calcrete. The boulders and cobbles are generally very strong.

9.3.1 Occurrence of Calcrete in Western Australia

The mapping of calcrete in WA is not complete. A preliminary map based on current knowledge is presented in

.

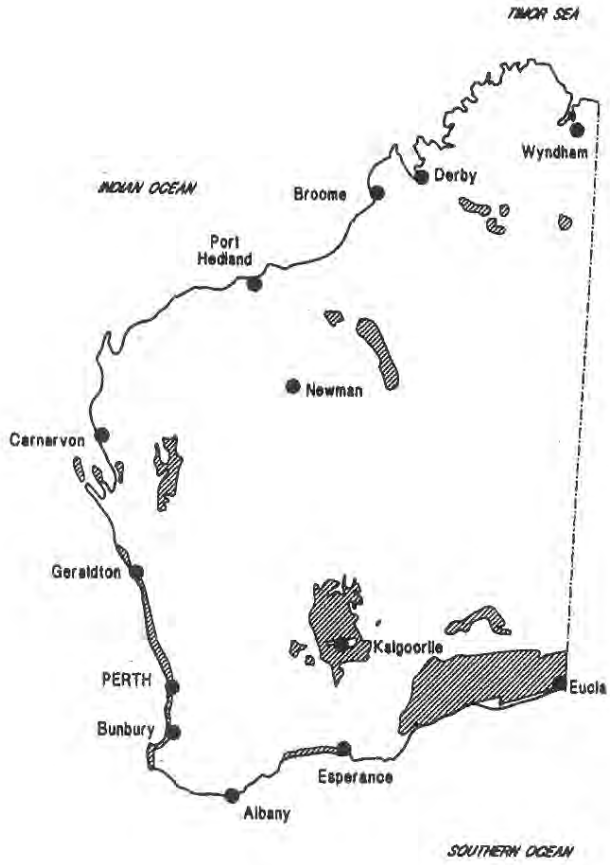

Figure 14: Occurrence of calcrete in Western Australia

9.3.2 Tamala Limestone

In terms of economic significance to the road building industry, Tamala Limestone is the most important of the calcrete materials in WA. This material is also known as coastal limestone and is found in coastal regions from the north of Geraldton to Augusta. Tamala Limestone consists of calcarenite in the form of aeolianite (dune limestone) and beach rock. Deposits suitable for road building occur up to 15 metres thick. A typical deposit includes residual quartz sand at the surface underlain by calcrete caprock (hardpan calcrete) which may be in the form of pinnacles. This calcarenite must be ripped by dozer and crushed under the dozer tracks or in a jaw crusher to produce a grading suitable for road construction. Beneath the caprock layer is typically a thick layer of calcarenite (calcified soil) that becomes less cemented with depth and therefore weaker and friable as a results. The resulting material is non-plastic.

Crushed Tamala Limestone appears to gain strength when compacted at OMC and dried back achieving a higher CBR than if compacted at the lower moisture content. A series of tests on limestone used as subbase on Kwinana Freeway gave results as set out in Table 21. This strength gain explains why this material can be used successfully on very heavily trafficked roads.

Table 21: California bearing ratios of crushed Tamala Limestone

Test Conditions	*CBR*		*Number of Tests*
	Mean (%)	*Standard Deviation (%)*	
Soaked	72	1	2
Unsoaked (Target Moisture = OMC)	94	12	4
Unsoaked (Target Moisture = 80% OMC)	113	7	4
Unsoaked (Compacted at 100% OMC and Dried Back to about 80% OMC)	121	29	4

Note: OMC= Optimum Moisture Content, modified compaction.

For 64 Clegg Impact Values (CIV) measurements on compacted limestone subbase on various roads the average CIV was 53.2 with a standard deviation of 12.8, i.e. a 15th percentile value of 39.9. A CIV of 39.9 infers a field CBR of about 117%. Field measurements of CIV indicate that the CBR of compacted Tamala Limestone material (of reasonable quality) some years after construction, almost always exceeds 80%.

South African dynamic cone penetrometer tests were carried out on crushed Tamala Limestone subbase on a series of trial sections constructed as part of the Kwinana Freeway project in 2008. The average penetration rate (DN) was 2.7mm/blow with a standard deviation of 1.0mm/blow. The equivalent CBR values were all 70% or greater. These tests were only a few months after construction before the full strength gain had time to develop.

RLT testing was carried out on a sample of crushed Tamala Limestone used as subbase on several trial sections on Kwinana Freeway. Modulus (94 kPa mean stress and 30 kPa shear stress) of samples compacted to about 94% MMDD and cured for 2 years, varied from about 420 MPa (at 80% OMC) to about 450 MPa (at 60% OMC). There was a very weak trend of decreasing modulus with increasing moisture ratio.

A limited set of field permeability measurements on compacted limestone subbase on a project in Guildford found an average value of about 0.1 m/day.

The effect of various parameters, including $CaCO_3$ content on the soaked CBR of Tamala Limestone has been investigated by Coffey (1993, 1994). The resulting regression equation with a correlation coefficient of 0.89 was:

$$\mathrm{CBR} = \frac{180500\ \mathrm{MDCS}^{0.32}\left(\frac{P_{0.075}}{P_{0.425}}\right)^{0.096}}{\left(72 + |72 - CaCO_3|\right)^{2.23}} \qquad \textbf{(12)}$$

where MDCS is maximum dry compressive strength (kPa).
$P_{0.075}$ is percentage passing a 0.075 mm sieve
$P_{0.425}$ is percentage passing a 0.425 mm sieve
$|72\text{-}CaCO_3|$ is the absolute value of the difference in $CaCO_3$ content (as a percentage) and the "ideal" value of 72%

Crushed Tamala Limestone has been successfully used as a subbase on a large number of roads including most freeways in Perth. Lightly trafficked residential subdivision streets in Perth are now commonly constructed with about 60 mm thickness of asphalt directly on the limestone subbase.

While untreated limestone has been used successfully as a basecourse, some problems of seal adhesion have occurred. For this reason, the material is usually modified with bitumen when used as a basecourse under sprayed surface treatments, (Hamory and Ladner, 1976; Sparkes and Hamory, 1980; Hamory and Cocks, 1988). Bitumen modified limestone has been used successfully on roads carrying in excess of 5 x 10^6 ESA. Performance problems with bitumen stabilised limestone basecourse are extremely rare and it is expense rather than performance that has led to reduced use in recent years. A limited set of RLT tests on bitumen-modified limestone (98% MMDD, 70% moisture ratio, 240 kPa mean stress, 120 kPa shear stress) gave moduli in the range of 570 to 630 MPa.

Typical selection criteria are presented in Table 22.

Table 22: Typical specification for crushed Tamala Limestone.

Pavement Layer	*Sub Base*	*Basecourse*
Material Type	*Crushed Limestone*	*Bitumen Stabilised[7] Crushed Limestone*
Particle size distribution		
Sieve Size (mm)	*% passing[1]*	*% passing[1]*
75.0[6]	95-100	
26.5		100
19.0	55-85	90-100
4.75		60-90
2.36	35-65	
1.18		35-75
0.075[8]	≤ 20	≤ 20
Other limits		
MDCS[2] (kPa)	≥ 650	≥ 650
$CaCO_3$[3] (%)	60-85	60-85
Los Angeles Abrasion [4] (modified test)	20-60	20-60
Bitumen Content (%)		≥2
Dry Density Ratio [5] (%)	≥ 94-95	≥ 95-96
Dry Back (%OMC)	≤ 85	≤ 85

Notes:

(1) Dry sieving and decantation Test Method WA 115.1 except that a 75.0 mm sieve should be used.
(2) Maximum Dry Compressive Strength, Test Method WA 140.1.
(3) $CaCO_3$ by solution in HC1, Test Method WA 915.1.
(4) Modified Los Angeles Abrasion test uses, half size test portion, four balls and 250 revolutions. Softer material is likely to break down excessively under rollers. Harder material may be difficult to work.
(5) Dry density ratio refers to modified compaction. Test Method WA 141.1. Required density to be selected by the designer in the range stated.
(6) All material should be smaller than 100 mm.
(7) The bitumen emulsion used for modification should be slow setting anionic with Vinsol Resin as the emulsifying agent.
(8) Limit on percent passing 0.075 mm is not always included.

For the New Perth Bunbury Highway project, the MDCS requirement was varied with $CaCO_3$ content: 1500 kPa for a $CaCO_3$ between 35% and 45%, and 1100 kPa for $CaCO_3$ between 45% and 60%.

Despite the non-plastic nature of the subbase material it is possible to cut intact cores from the pavement. Hamory and Cocks (1988) reported on average indirect tensile strength of 81 kPa for 55 cores of limestone subbase cut from Leach Highway in 1986 (9 years after construction). As the material is non plastic without any clay binder, this suggests some form of self-cementation.

9.3.3 Kimberley Calcrete

A number of small shallow deposits of nodular calcrete are located near Great Northern Highway south east of Fitzroy Crossing. These have not been assigned a type name by the Geological Survey of WA but are referred to as "Kimberley Calcrete" in this paper for convenience. The use of this material for construction of Great Northern Highway in 1981 has been described by Kilvington and Hamory (1986). The section of these trials constructed with a calcrete basecourse has carried (to December 2013) about 5 x 10^6 ESA with two reseals and has required only minor pavement repair mainly due to silted up drains causing water to pond close to the pavement after heavy rain. Elsewhere, the pavement shape is still generally good, with only minor rutting in the wheel paths.

While no measurements of salt content or electric conductivity were made, the vegetation and terrain at the various pit sites suggests that the base had a low salt content. Fresh water was used for compaction. No significant problems were encountered with sealing this calcrete basecourse.

The properties of calcrete used in trial sections near Gogo Station together with Netterberg & Pinard's (1991) selection criteria are presented in Table 23.

Table 23: Selection criteria with the properties of some WA calcrete and calcarenite with comparison to Netterberg & Pinard (1991).

Parameter	*Kimberley Calcrete (Pit 2584, Gogo Trials)*	*Roe Calcarenite (Hampton Pit)*		*Netterberg & Pinard (1991) Suggested Limits*	
		Costean samples	*After grid rolling*	*<500vpd*	*500 - 1000vpd* (1)
	Maximum size (mm)				
	Not stated	Not stated	Not stated	19-30	38-53
Particle size distribution					
Sieve Size (mm)	*Max % retained*				
Percentage retained on 19.00	7-15	22	11	Not stated	Not stated
	% passing				
19.00	100	100	100		
9.50	72-81	87	90		
4.75	54-63	75	76	(4)	(4)
2.36	41-55	66	66		
1.18	37-47	61	60		
0.60	36-45	55	55		
0.425	34-43	52	52	15-55	15-55
0.30	32-40	49	48		
0.15	27-34	32	34		
0.075	23-29	17	21		
0.0135	12-15	8	12		
Classification limits					
Liquid Limit (%)(5)	23-24	28	NA(16)	< 40	< 35
Plasticity Index (%)(5)	11-13	7	NA	< 12	< 12
Linear Shrinkage (%)	5-7	3	NA	< 6	< 4
$P_{0.425}$ x PI	374-516	378	NA	NS	NS(17)
$P_{0.425}$ x LS	185-258	168	NA	< 320	< 170
Grading Modulus(4)	1.7-2.0	1.7	1.6	1.5	1.5
Dust Ratio (6)	0.62-0.68	0.33	0.40	NS	NS
Other limits					
CBR (7)	NA	100(8)	NA	≥60	≥80
CBR Swell	NA	NA	NA	≤ 0.5	≤ 0.5
Max. Dry Density (t/m^3)	2.31-2.36	NA	1.95	NS	NS
Optimum Moist. Content (%)	4.5-5.7	NA	11.8	NS	NS
Particle Density (t/m^3)	2.70-2.72	NA	NA	NS	NS
Specified Dry Density Ratio (%)(10)	≥ 95	≥ 95		≥ 98	≥ 98
$CaCO_3$ on material passing 2.36 mm sieve (%)	37-38	55	NA	NS	NS
Saturated Paste Electric Conductivity (S/m at 25°)	(11)	(12)		≤ 0.15	≤ 0.15
Dry Aggregate Pliers Value(18)	(13)	(14)		≥50	≥60

Notes:

(1) Less than 25% vehicles> 3 tonnes.

(2) Samples from costeans ex pit.

(3) After grid rolling on road.

(4) Particle size distribution requirements are expressed as a grading modulus:

$$\frac{300-(P_{2.00}+P_{0.425}+P_{0.075})}{100}$$

Main Roads WA uses 2.36 mm sieve in lieu of 2.00mm.

(5) Differences in soil preparation and equipment mean that liquid limits and plasticity indices determined by South African methods are an average 3.5 units lower than those determined by Australian methods.

(6) Dust Ratio = $\frac{P_{0.075}}{P_{0.425}}$ desirably between 0.3 and 0.7

(7) Netterberg criteria refers to samples compacted at 98% MDD (modified compaction) and 4 day soak.
(8) Standard deviation was 35. CBR soaked at 95% MDD (modified compaction.)
(9) Netterberg limits based on material sampled from the road.
(10) Main Roads WA Specifications express dry density ratio requirements as a characteristic value $\overline{x}$ - ks. Actual characteristic dry density ratios for Gogo trial sections were closer to 98% (c.f. 95% specified). On subsequent construction 97% was specified.
(11) Not tested, but probably less than 0.15
(12) Not tested, but almost certainly greater than 0.15
(13) Not tested, but nodules were hard and strong.
(14) Not tested, but nodules and shell fragments were of medium strength.
(16) NA not available
(17) NS not specified.
(18) Netterberg includes other requirements on stone toughness.

9.3.4 Roe Calcarenite

The geology of the Roe Calcarenite has been described in detail by Lowry (1970). Briefly, the Roe Calcarenite is a marine Pleistocene deposit overlying a plain carved in Tertiary limestone by marine erosion. It is found on the Roe Plain, which is traversed by the Eyre Highway between Madura Pass and Eucla Pass.

Extensive use was made of Roe Calcarenite, from the Hampton pit, for reconstruction and widening of the Eyre Highway during 1988. The deposit at Hampton pit was about 3 metres thick and consisted of a thin discontinuous layer of honeycomb and hardpan calcrete near the surface underlain by nodular calcrete transitioning to calcified sands at depth. After stripping of the thin layer of topsoil, the full depth of material was mixed by dozer during the pushing up operations. A significant proportion of the gravel sized particles were whole shells and shell fragments filled with calcified sand.

The results of particle size distribution, consistency and strength tests are presented in Table 23.

In terms of particle size distribution and consistency tests, the material complied with Netterberg's (1982) selection criteria. The WACCT and soaked CBR results were also satisfactory bearing in mind the arid, well drained environment.

Extensive Clegg impact testing was carried out during construction leading to the following relationship between field impact value, density and moisture:

$$CIV = (1.012\ RD) - (0.890\ RW) + 19.4 \quad \textbf{(13)}$$

with a standard error of estimate SIV = 7.3

Where CIV = Clegg impact value
RD = Dry Density Ratio expressed as a percentage (modified compaction)
RW = moisture content expressed as % of OMC (modified compaction)

For the specified characteristic dry density ratio of 95% (minimum) and dry back to 85% of OMC equation 10 gives an impact value of 40, which is adequate.

No saturated paste electric conductivity tests were carried out. However Lowry (1970) reported that "the soil is rich in cyclic salts". The water used for compaction had a high salt content (about 30,000 ppm). It is highly probable that in an "as constructed condition" Netterberg's limit on electric conductivity would have been exceeded.

While the basecourse constructed of Roe Calcarenite from the Hampton pit had adequate strength, significant problems of "fluffing" were encountered with the primes and seals. Special priming and sealing techniques had to be developed.

9.3.5 Gascoyne Calcrete

In coastal areas between Shark Bay and Northwest Cape, calcareous materials are commonly used for base and subbase. These materials include shelly sands, limestone gravels, coral gravels, calcrete and massive limestone (suitably crushed). Many of the calcareous gravels are very coarse and require breaking down in size during stockpiling, on the road by construction machinery or by crushing and screening.

Geologically these materials occur as aeolian coastal sands, beach ridge deposits, as duricrust calcrete and as part of the Cape Range Group of marine deposits.

In the more inland Gascoyne, calcareous gravels also occur and are used for road construction. These gravels commonly have more plastic fines than Tamala Limestone.

Table 24 illustrates the range of selection criteria applied to these gravels.

Table 24: Typical selection criteria for Gascoyne calcrete.

Material	*Limestone Gravel Minilya Exmouth Road 0-60 km*	*(Popcorn) Limestone Gravel Minilya Exmouth Road 60-170 km*	*Pebbly Lime clay sand Shark Bay Road*
Particle size distribution			
Sieve Size (mm)	*Max % retained*	*Max % retained*	*Max % retained*
19	10	40	5
	% passing	*% passing*	*% passing*
2.36	35-55	35-50	85-100
0.425	28-48	30-45	50-70
0.0135	$\leq$ 10	$\leq$ 5	$\leq$ 16
Classification limits			
LL max (%)	25	20	25
PI max (%)	12	4	10
LS max (%)	6	1	5
Other criteria			
MDCS (kPa)		$\geq$ 5000	
MMDD (t/m^3)	2.050-2.250	1.85-2.00	2.10-2.25
OMC (%)	6.5-8.0	9.0-12.0	5.5-7.5

9.3.6 Selection Criteria for Calcrete in Western Australia

Based on the information available to date, Netterberg's (1982) and Netterberg & Pinard's (1991) selection criteria appear suitable for application to nodular calcrete in WA. An additional requirement that the dust ratio lie between 0.3 and 0.7 is recommended to ensure workability and low permeability. Netterberg's criteria on stone hardness and toughness have not been verified for WA conditions.

Special techniques may be required for sealing of some calcrete basecourse materials, particularly if the natural material has a significant salt content and/or salt water is used for construction.

9.4 SAND CLAY

The term sand clay is defined in AS 1348-2002 as "a mixture of sandy and clayey soils suitable for pavement construction". In terms of the Unified Soil Classification system (AS 1726-1993), sand-clay materials used successfully as basecourse in WA would be described as well graded silty sands (SW-SM) or well graded clayey sands (SW SC).

The sand clay materials used in WA are typically red in colour (e.g. Pindan sand). However, some yellow (Wodgil) sands have been used. The process by which the reddening of desert sands occurs has been described by Folk (1976).

Only limited chemical testing has been carried out on the fine material. However it seems likely that the red colour is due to iron oxide coatings on the quartz sand particles. Results of chemical tests on a sample of red sand clay basecourse from the Hamelin-Denham Road are set out in Table 25.

Table 25: Chemical composition of red sand clay from Hamelin-Denham Road.

Chemical	$A1_20_3$ (1)	Fe_20_3 (1)	SiO_2 (1)	$CaCO_3$ (2)
% by mass	6.2	3.3	8.4	6.1

Notes:
(1) Tests on fraction passing 0.425 mm sieve, determined by wet chemistry/inductivity coupled plasma spectrometry.
(2) Tests on fraction passing 0.425 mm sieve, determined by solution in HCl

A characteristic of sand clay is significant strength gain through dry back. The results of indirect tensile strength tests on samples of sand clay from the Hamelin-Denham Road are presented in Table 26.

Table 26: Indirect tensile strength tests on red sand clay from the Hamelin-Denham Road.

Sample Preparation	*Test Condition*	*Indirect Tensile Strengths (kPa)*
Cores cut from road	field moisture (~ 50% OMC)	281-573
Compacted in laboratory at 50% OMC	50% OMC	25-45
Compacted in laboratory at 100% OMC	dried back to50% OMC	252-288

9.4.1 Occurrence of Sand Clay in Western Australia

Mapping of sand clays in a form compatible with the "suitable for pavement construction" definition is not available. However the Atlas of Australian Soils indicates that red sands (Uc5), red earths (Gn 2.1) and brown calcareous earths (Gc) are common in coastal areas between the Murchison River and Derby. Yellow sands (Uc5.22) and earths (Gn 2.2) are less wide spread and are found generally south of the latitude 28°S. Not all sands within these soil units are suitable for pavement construction. Experience in the Gascoyne Region suggests that the most suitable sand clays have come from the Mx 13-14 soil units. Suitable material for basecourse has also been found in the units DD 18-20 and OC 40.

9.4.2 Selection criteria for Sand Clay

Little use has been made of yellow sand clay for road construction by Main Roads WA in recent years. No recommendations on selection criteria are therefore made.

For red sand-clays, selection criteria have been proposed by Ladner and Hamory (1974) for a section of North West Coastal Highway near Port Hedland. Emery *et al*. (2003) provided information on sand-clay proposed for use on a new airport near Broome, WA. International literature on the use of these materials for road construction is sparse. However selection criteria have been published for fine sandy soils for use in road construction in Brazil by Utiyama *et al*. (1977).

Suggested selection criteria for red sand-clay for use in well drained sites in arid areas of WA are set out in Table 27. It should be noted that edge wear is a significant problem with roads constructed with sand-clay basecourses. An increase in seal width by one metre is recommended to reduce wear.

An alternative basis of selection of sand-clay using the Φ scale of particle size distribution has been proposed by Metcalf and Wylde (1984).

Table 27: Interim selection criteria for red clayey or silty sand for basecourse on well drained sealed roads.

Design traffic ESA *Rainfall deficit (1) (mm)*	$< 10^5$ > 2500	$< 3 \times 10^4$ > 2500
Particle size distribution		
Sieve Size (mm)	*% Passing*	*% Passing*
4.75	100	100
2.36	70-100	70-100
1.18	50-79	50-100
0.60	36-63	36-100
0.425	30-56	30-84
0.300	25-50	25-71
0.150	18-40	18-50
0.075	13-31	13-35
0.0135	5-15	5-15
Classification limits		
Liquid Limit (%)	≤ 20	≤ 20
Plasticity Index (%)	≤ 8	≤ 8
Linear Shrinkage (%)	1-3	1-3
Dust Ratio (2)	0.2-0.6	0.2-0.6
Other test limits		
CBR (%) (3) Unsoaked	≥ 80	≥ 80
WACCT Class No (4) (5)	≤ 2.0	≤ 2.3
Cohesion (kPa)	≤ 85	≤ 85
Tensile Strength (kPa)	≥ 55	≥ 55
Horizontal Separation of Class No Contours (%)	≥ 1.3	≥ 1.3
Maximum Dry Density(6) kg/m^3	≥ 2100	≥ 2100
Optimum Moisture Content	5-7	5-7
$A1_2O_3 + Fe_2O_3$ (%)(7)	> 8?	> 8?

Notes:

(1) Rainfall deficit = potential evaporation - annual rainfall.

(2) Dust Ratio $= \frac{P_{0.075}}{P_{0.425}}$

(3) California Bearing Ratio carried out on specimens compacted at 90-100% OMC to 95% MDD (modified).

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

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

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

(7) Al_2O_3 and Fe_2O_3 determined by ICP on the fraction passing a 0.425 mm sieve and oxides extracted by boiling in dilute sulphuric acid.

9.5 SCREE GRAVEL FROM THE PILBARA REGION

The lateritic and calcrete materials so favoured elsewhere in the state are less common in the Pilbara Region and more use is made of colluvial or scree gravel. Typically the coarsest material is found near the head of a scree slope with the material becoming finer towards the toe. The climate throughout the Pilbara is warm to hot arid and the natural gravel used is typically relatively coarse with low fines content. As a result, the upper limit on plasticity index, even on relatively heavily trafficked highways is usually higher compared to other regions in WA.

Developing selection criteria for pavement gravels for use in the Pilbara region has proved challenging for the following reasons:

There is a wide range of geological origins of the parent rock from which scree gravels have developed.

The range of materials that have been used successfully is wide.

There have been no formally documented pavement trials in the region.

There has not always been a consistent link between material properties and performance.

Some examples of Pilbara gravel are set out in Table 28. Each of these examples had a sprayed seal surface.

Dampier Highway connects Dampier and Karratha and is one of the more heavily trafficked roads in the Pilbara region with a significant proportion of its length being on a causeway across solar salt ponds. The highway connects the heavy industrial area on Burrup Peninsula to the main land. Despite these adverse conditions a material with an average soaked CBR of less than 10 gave adequate performance as a basecourse for more than 30 years.

The borrow Pit 108 material was used on Karratha Tom Price Road from Western Creek to Barowanna Hill. Construction was completed in 2008. Shortly after opening, the highway was subject to an extreme rainfall event. Traffic volume is about 75 vehicles per day with some heavy vehicles. Between 2008 and 2013 about 5% of the pavement has developed ruts and required cement stabilisation.

The mine access road was constructed in 2012. Estimated traffic volume is 100 vehicles per day (one way). The initial surface was a single coat seal followed by a second coat one year later.

In view of the lack of formally documented pavement trials in the Pilbara, interim selection criteria have been derived based on anecdotal information from experienced Main Roads WA geotechnicians who worked in the region.

Proposed selection criteria for Pilbara scree gravel is set out in Table 29.

Table 28: Examples of Pilbara scree gravel.

	Mine Access Pilbara Region	*Dampier Highway*	*Karratha Tom Price Road Stage 2, Pit 108*
Traffic Loading	*200 vpd*	*Circa 10^7 ESA since construction*	*75 vpd*
	Particle size distribution		
Sieve size (mm)	*% passing (1)*	*% passing (1)*	*% passing (1)*
37.5	90	100	98
2.36	31	72	27
0.425	21	52	15
0.075	10	34	11
	Other grading limits		
Maximum Size (mm)	75	37.5	63
Grading Modulus(2)	2.38	1.42	2.47
Dust Ratio(3)	0.49	0.65	0.71
	Classification limits		
Plasticity Index (%)	7.5	22	17
Linear Shrinkage (%)	3.7	12.4	7.8
$P_{0.425}$ x LS	85	648	121
	Other limits		
CBR (%) Soaked	65 (4)	6 (5)	75 (6)

Notes:

(1) Particle size distribution and other data are average values.

(2) Grading Modulus $= \frac{300-(P_{2.36}+P_{0.425}+P_{0.075})}{100}$

(3) Dust Ratio $= \frac{P_{0.075}}{P_{0.425}}$

(4) Soaked CBR at 95% MMDD

(5) Soaked CBR at 94.5% MMDD

(6) Soaked CBR at 98% MMDD.

Table 29: Selection criteria for Pilbara scree gravel.

	Proposed Selection Criteria (1)				
Traffic Loading	$< 10^5$ ESA	10^5 to 10^6 ESA	10^6 to 10^7 ESA	$\leq 10^7$ ESA	$< 10^6$ ESA,
	Well Drained Conditions	*Well Drained Conditions*	*Well Drained Conditions*	*Cement Modified Floodway(2)*	*Special Coarse Grading*
	Particle size distribution				
Sieve size (mm)			% passing	% passing	% passing
37.5	75 to 100	75 to 100	75 to 100	75 to 100	50 to 100
2.36	15 to 55	15 to 50	15 to 45	15 to 45	≤ 35
0.425	< 30	< 30	5 to 25	5 to 25	≤ 15
0.075	< 15	< 15	< 12	< 12	≤ 8
	Other grading limits				
Maximum Size(3) (mm)	63	63	63	63	75
Grading Modulus(4)	≥ 1.5	≥ 1.75	≥ 2.0	≥ 2.0	≥ 2.5
Dust Ratio(5)	0.2 to 0.7	0.2 to 0.7	0.25 to 0.7	0.25 to 0.7	0.2 to 0.7
	Classification limits				
Liquid Limit	≤35	≤35	≤35	≤35	≤35
Plasticity Index (%)	≤ 20	≤ 18	≤ 12	≤ 15	≤ 20
Linear Shrinkage (%)	≤ 10	≤ 9	≤ 6	≤ 8	≤ 10
$P_{0.425}$ x LS	≤ 250	≤ 225	≤ 200	≤ 200	≤ 200
	Other limits				
CBR (%) (6) Soaked	≥ 50	≥60 for PI ≤ 12 ≥80 for PI ≥ 15	≥60 for PI ≤ 8 ≥80 for PI ≥ 10		≥ 60
UCS (MPa) (7)				0.6 to 1.0	
Particle Toughness(8)	Note(9)	Note(9)	Note(9)	Note(9)	Note(9)
Dry back (%)	≤ 85(9)	≤ 85	≤ 85	≤ 90(10)	≤ 85

Notes:

(1) Selection criteria apply to basecourse roads with a thin bituminous surfacing with a 20 year design traffic of up to 10^7 ESA. For greater traffic loading specialist advice should be sought.

(2) Limits on PI and Linear Shrinkage for floodways apply to the material before the addition of cement.

(3) Such coarse material will require extra effort and high skill by the grader operator to achieve a good finish. Replacement of oversize in the CBR test is recommended. Sealed shoulders required to limit lateral infiltration under the pavement.

(4) Grading Modulus $= \frac{300-(P_{2.36}+P_{0.425}+P_{0.075})}{100}$

(5) Dust Ratio $= \frac{P_{0.075}}{P_{0.425}}$

(6) CBR specimens compacted at OMC to 98% MMDD, and tested in a soaked condition.

(7) Sample prepared with GP cement, compacted to 95% MMDD and cured for 7 days then soaked for 2 hours before crushing.

(8) No particular test for particle toughness is specified at this time. However the aggregate must be hard and durable and not break down when alternatively wet and dried. Material with flaky and elongated particles should be avoided.

(9) Basecourse should be dried back to a moisture content of less than 85% of OMC prior to application of bituminous surfacing.

(10) Cement treated pavement to be sealed as soon as practicable after cement stabilisation. Minimum surface treatment for floodways is prime and double/double seal.

(11) For town streets, irrigation of verges and medians may result in an environment that does not fit the condition of "well drained".

10 REGIONAL AIRPORTS

WA has a network of regional aerodromes servicing towns, mines and oil and gas projects. These are almost totally built with flexible pavements. Naturally occurring granular materials are an important source for basecourse and subbase courses for these aerodromes.

The following comments apply to less busy aerodromes served by turbo-prop and small passenger jet aircraft. At these aerodromes, the number of aircraft movements is small (less than an average of 5 landings per day), the planes are typically passenger aircraft with a maximum of 100 seats (e.g. BAe146 or Fokker F100), and have tyre pressures of 1000 kPa or less. The comments do not apply to larger airports serving medium or large capacity passenger jets such as Boeing 737/Airbus A320, because there is a step change in pavement design philosophies above 45.5 tonne aircraft weight.

While tyre loads from even the smaller aircraft are higher than for regulation highway vehicles, there are offsetting factors for regional aerodrome runways, taxiway and apron pavements:

- The bituminous surfacing is very wide and in the normal operation aircraft are several metres from the edge zone affected by lateral infiltration of water.
- Load repetitions are very low (commonly less than 10 per day), and fatigue life is much less of a concern on aerodromes compared to roads.
- Wander and differences in gear track width are greater for aircraft and loads are distributed over the pavement.
- Sprayed seals on runways, taxiways and aprons have much higher binder application rates than highways with the same size aggregate and are therefore more waterproof.
- There are no drive axles on aircraft and tyres in a dual pair can rotate independently.
- Grades on runways and taxiways are always 1.5% or less.

10.1 DIFFERENCES BETWEEN AERODROME AND ROAD WORKS

The use of naturally occurring granular materials is similar to that on roads, but there are a number of aspects in which regional aerodromes vary from road works. These are:

- Safety and wet weather operations.
- Limited closure time.
- Proof rolling.
- Choice of materials.

10.1.1 Safety and Wet Weather Operation

Safety is a concern on aerodromes because of risk factors such as normal operating speeds up to 225 kph and engine vulnerability to damage. The consequences are potentially more severe on aerodromes than roads due to the high number of passengers/occupants in an aircraft and the high aircraft fuel capacity (up to 13,000 litres).

Wet weather is a particular issue on runways because of the high speeds. It is mandatory on aerodromes that the surface of a paved runway shall be so constructed as to provide good friction characteristics when the runway is wet (ICAO, 2009). There are ICAO (International Civil Aviation Organization) requirements for minimum macro texture, minimum limits for runway slopes and cross fall to ensure drainage, and limits on irregularities and ruts to reduce ponding of water. For regional aerodromes, this requires close attention to gradient and surface finish for the basecourse. Good macro texture is provided by using a bituminous seal or, more rarely, asphalt with grooving.

10.1.2 Limited Closure Time

There is usually less chance of closing a runway or taxiway to work on it than closing a lane of a road. On aerodromes, works must be planned to ensure that restrictions on available operating lengths or full closure of runways are kept to an absolute minimum. It is more restrictive than roads because detours are usually not possible and it is rare to have an alternate pavement available. While aerodrome works are often done at night in multiple stages, there is usually still a requirement to accommodate Royal Flying Doctor Service flights 24 hours/day. The availability requirement for an aerodrome often constrains the choice of materials and methods for maintenance and rehabilitation.

10.1.3 Proof Rolling

The tyre loads of aircraft are higher than for regulation highway vehicles, and their influence extends to a greater depth. A Fokker F-100 requires a pavement thickness of 925 mm over subgrade soaked CBR 3% (20 years, 1,200 annual

departures), compared to a pavement thickness of 380 mm to 520 mm for a road over subgrade CBR 3% (10^5 to 10^6 design traffic ESA). Proof rolling is commonly used on aerodromes to ensure the pavement and the foundation is sound to depth.

Before any pavement materials are placed on a prepared subgrade, including fill areas, the subgrade is proof rolled with six coverages of a pneumatic tyre roller. Any soft spots or areas, which become unstable under this rolling, should be excavated and replaced with compacted stable material. Proof rolling is a visual test, which covers the entire subgrade surface (unlike density tests that are at specific locations). A second series of proof rolling is carried out using a suitable roller on the basecourse prior to sealing to ensure adequate pavement strength is achieved. Pneumatic tyre rollers should be related to aircraft and thickness of layer to be compacted. For most aerodrome work, tyre pressures of 700 kPa and wheel loads of 2250-4500 kg would be appropriate for the roller.

10.1.4 Pavement Materials for Aerodromes

Base and subbase selection criteria applicable to roads with design traffic of 10^6 ESA or greater can usually be applied to regional aerodrome runway, taxiway and apron pavements. Note that care must be taken if stabilising runway base materials with cement; cracking may occur leading to surface disintegration. Loose material, referred to as foreign object debris (FOD), may be created. Ingestion of loose material (FOD) is a serious issue for jet engines.

10.2 BITUMINOUS SURFACING OF AERODROMES

A full explanation of the selection and design of bituminous surfacing for natural gravel basecourse materials is beyond the scope of this document. However there are some particular principles relevant to bituminous surfacing of aerodromes which are noted here.

10.2.1 Bituminous Surfacing Types for Aerodromes

In Australia, over 200 regional aerodromes have sealed runways, and bituminous seals are almost universally used on runway, taxiway and apron pavements carrying aircraft up to Fokker F100 size. They have also been used on pavements carrying Boeing 737 aircraft, although asphalt is more common (Emery, 2008).

Seals are better suited to locations with lower horizontal stresses. Although much of a runway or taxiway experiences low horizontal stresses from aircraft traffic, the sections with higher horizontal stresses are the runway turning nodes, runway ends (if these are used for 180 degree turning), intersections, and (to a much lesser extent) the touchdown zone. Seals are still used in these areas but some special treatment may be necessary. It is very rare to see damage due to aircraft braking, and this is therefore not considered a stress problem. Aprons are generally medium stress areas, although the parking bays are a special case. Viscous flow of bitumen in surfacing under the wheels of a parked aircraft in a hot climate can be a problem and asphalt or concrete can be used. Protective coatings to seals and asphalt or concrete pads may be required on apron bays where jet engines may drip fuel when parked or refuelling.

In the low stress areas, a primed base course with a double/double seal (10 or 14 mm stone on the lower layer and 5 or 7 mm stone on the upper layer) has proved very successful for new construction. The single seal has been used occasionally for general aviation aircraft <5700 kg, but it is not suitable for airline aircraft. In the warmer parts of WA, Class 320 bitumen is often used for seals to better cope with the horizontal stresses, although Class 170 is used in the cooler areas.

In the high stress areas, the triple seal (double seal with a thin sand seal on top to fill the voids) over a primed base has been used with success, or even a Cape Seal (single seal with a 13 mm or a 19 mm stone, and thin slurry on top which almost fills the voids and creates a strong mosaic) can be used. However the surface macro texture of triple seals, sand seals, Cape Seals, and asphalt can easily be less than the requirements for runways, and attention must be given to ensure adequate macro texture. Sand seals seem to extend the life of the underlying aggregate seal by improving stone retention and delaying binder oxidation.

Experience has shown that the stone on the upper layer should have a maximum nominal size of 7 mm (maximum size - not average least dimension which is smaller). The use of larger stone may lead to tyre shredding or excessive tyre wear on wheel spin-up in the touchdown zone. In the early stages of introducing airline jets to runways with seals, larger stones were experimented with. At Karratha Airport, a 10 mm top stone gave very high surface texture but caused unacceptable tyre wear in just four movements of a 30 tonne jet aircraft. The runway had to be re-rolled with a steel wheel roller and the touchdown area resealed with a smaller size aggregate.

Good penetration of the prime is essential as a strong bond between the seal and the base is required to reduce the FOD risk.

Helicopters are a particular problem for the surfacing. Small helicopters with skids commonly damage seals and even asphalt when parked in hot weather; the skids tend to move slightly apart over time as the bituminous surfacing yields

in creep. Large helicopters with wheels are less damaging but create significant downwash over large areas and minimisation of FOD requires large areas to be bituminous surfaced. It is common to use asphalt, concrete or block paving for busy helicopter aprons, although triple seals (14/7/sand) have been used with some success. Inset block paving has also been used for helicopter stands.

10.2.2 Friction and Macro Texture

The surfacing is required to provide adequate wet weather friction and must provide an average surface macro texture at least 1.0 mm over the full runway width and length. The same requirements are not applied to taxiways and aprons.

A seal can usually meet the regulatory requirements in terms of wet weather friction and macro texture depth; it is a friction treatment in itself. Other surfacing such as asphalt may have reasonable wet weather friction, but are inherently smoother and do not provide enough macro texture; friction treatment such as grooving is always needed for asphalt. Care is needed when using seal types, which have low macro texture, such as overfilled triple seals and Cape Seals, or sand seals to lock in the stone seal. It is important not to reduce average macro texture below 1.0 mm and thus breach the airport regulatory requirements on runways. Such seal types may not be suited for use along an entire runway; however they could be useful for limited high stress areas such as runway ends and turning nodes where their use will not reduce the average macro texture of the runway below 1.0 mm.

Fog sprays and dilute emulsion sprays have been used on bituminous seals on aerodromes, but on the runway it is essential that there is sufficient texture to accommodate them and that the average macro texture is not reduced to below 1.0 mm. The use of proprietary rejuvenation products can lead to slippery runways when wet. They reduce micro texture and macro texture, and do not reduce wheel ruts or correct drainage problems. They can be a particular problem for asphalt surfaces; even grooved runways can become slippery when wet after rejuvenation. Such concerns do not apply to taxiways and aprons, where they have been used with some success.

10.2.3 Foreign Object Damage

The issue of damage to aircraft by stones and/or bitumen must be addressed on aerodromes. The main problem is that loose stone can be ingested into engines or damage propeller blades, although minor problems also occur if bitumen or stone adheres to the wheels and are flung into the wheel wells or along the underside of the aircraft causing a cleaning problem.

Foreign object debris (FOD) is obviously more of a potential problem with seals than with asphalts, and a specific pavement maintenance capability is essential to deal with it. This includes regular inspections, maintenance rolling and sweeping (or regular use of the proprietary FOD Boss).

There have been occasional problems with bitumen thrown into aircraft wheel wells and on to the aircraft in the first week after a new seal or reseal, usually when the work has been done in hot weather and the airport is opened to traffic within an hour of completing each stage. These have not caused safety problems.

10.2.4 Maintenance Rolling on Aerodromes

A newly constructed seal or reseal needs to be rolled to embed the stone into the bitumen, and to 'work' the bitumen around the stone. Typically on a road, 20% of the necessary rolling is done at the time of construction, and the remaining 80% is provided by traffic. On aerodromes, the traffic is very much less, and so much more rolling has to be done at construction. It is common to have a specification for rolling at construction of one roller hour per 500 litres of bitumen.

The remainder of the rolling should be provided by maintenance rolling during the first few years of life of the seal. For sealed runways used by jet-engine aircraft, ongoing maintenance rolling is an essential part of runway maintenance and safe operating practice. As a maintenance specification, 1 roller hour is needed per 1000 litres of bitumen. For a double/double new seal, this is equivalent to 1 roller hour per 250 m^2. For a single/single reseal, it is equivalent to 1 roller hour per 500 m^2.

The best rollers are multi-tyre rollers as used in normal seal construction. Rolling should be done when the pavement is hot. Rolling is usually done on a part-time basis, which might be 4 hours per day for 30 days per year for a typical runway. Lack of rolling can be quickly seen by stone loss. In extreme cases – such as a double seal or reseal with almost no traffic – the onset of deterioration can be as short as 6 months if rolling has been inadequate.

11 LOCAL STREETS

Application of the selection criteria set out in the previous sections will generally result in a pavement with adequate strength. However pavements constructed with lateritic gravel, crushed massive laterite caprock and natural gravels with significant plastic fines will commonly develop block cracking. The tendency to develop block cracks is

exacerbated by the addition of cement. Block cracking typically commences with transverse cracks at about 6m spacing. With time cracks appear at about 3 m spacing both transversely and longitudinally and in the long term block cracks at about 1.5m spacing may develop as shown in Figure 15. These cracks do not indicate a structural weakness and are in fact evidence of the development of strength (self stabilisation). Provided the cracks are repaired to maintain waterproofing and regular resealing or asphalt surfacing applied, then local streets may last for many decades with these cracks.

Figure 15: Block cracking on local streets.

The block cracking may be aesthetically displeasing to land developers and Local Government bodies. If block cracking is to be minimized, then non plastic base materials must be used.

Elevations on local streets are relatively fixed due to the presence of kerbing and driveways. The opportunity to rectify problems with a thick overlay is limited. Despite the low traffic loading selection criteria for base materials can be restrictive.

12 UNSEALED ROADS

Australia has about 800,000 km of roads, of which about two-thirds are unsealed (Austroads, 2009). Unsealed roads generally carry light traffic (except for mine haul roads) and due to economic considerations, are often by necessity built from locally available materials, which may be of marginal quality.

Austroads (2009) Guide to Pavement Technology Part 6: Unsealed Pavements contains a considerable volume of information to guide the developer of unsealed roads.

Some specific considerations with unsealed roads are:

- Braking and skidding associated with loose gravel on the road surface.
- Visibility issues associated with the generation of dust.
- Damage to vehicles (e.g. windscreens) associated with flying stones.
- Impact on roadside habitat from dust generation.

There may be several pavement layers associated with unsealed roads and these may require different properties:

- Wearing course or ‘sheeting layer’ that is maintained with patrol grading and periodic replenishment.
- Base layer providing structural support to the wearing course and thickness required to minimise subgrade deformation.
- Subbase adding to structural capacity making the remaining contribution to thickness required over the subgrade.

Thus it is apparent that marginal materials of differing qualities may be applied at different levels in the pavement depending on the properties required. Higher quality materials may need to be conserved for higher performance requirements in upper parts of the pavement.

The wearing course material needs to provide resistance to generation of loose surface material, corrugations and not be excessively slippery in wet conditions. A degree of plasticity to provide cohesion is required, but excessive plasticity leads to potholes, rutting and shear. Shape loss in unsealed roads is not as important a consideration as for sealed roads, as maintenance grading is always required, and shape can be reinstated at this time.

South Africa has developed a useful guide to the selection of materials suitable for the wearing surface of an unsealed road based on grading coefficient and shrinkage product. (Paige-Green, 2006)

- Grading coefficient:

$$(Gc) = (P_{4.75} \times (P_{26.5} - P_{2.0}))/100 \quad \textbf{(14)}$$

- Shrinkage product

$$(Sp) = \text{Linear Shrinkage} \times (P_{0.425}) \quad \textbf{(15)}$$

In WA the percentage passing a 2.36 mm sieve may be used in lieu of the percentage passing a 2.0 mm sieve. Proposed limits on Shrinkage Product and Grading Coefficient of unsealed local roads are shown in Figure 16. The proportion of particles larger than 37.5 mm should not exceed 5%. This chart is based on that published by Paige-Green (2007) but has been modified to allow for differences in test methods and the use of the 2.36 mm sieve in lieu of the 2.00 mm sieve,

On mine haul roads, vehicles are larger, grades steeper, speeds lower and watering for dust control more frequent than for local roads. There is therefore more emphasis in controlling slipperiness when wet. Thompson (2011) has proposed the following limits for unsealed mine haul road wearing course in WA:

- Shrinkage product between 85 and 200.
- Grading Coefficient between 20 and 35.

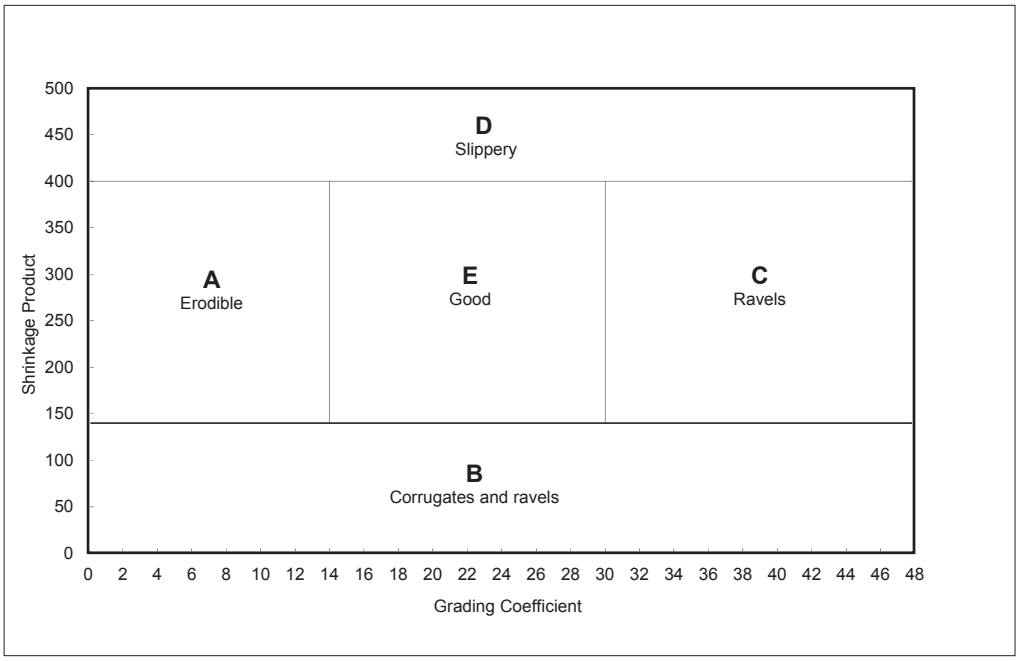

Figure 16: Selection of materials for wearing course of unsealed local roads (Paige Green 2006, 2007) modified for differences in test methods.

13 CONCLUDING COMMENTS

The information contained in this document reflects current practice in WA (and some similar international work) and is intended to provide practical guidelines for those involved in making decisions on utilizing natural materials for road construction.

These materials are a valuable resource for WA and enable low cost construction of road, highway and regional airport pavements. High quality natural materials suitable for pavement construction, are scarce and it is important that their use is managed well to ensure that good pavement materials are preserved for construction of pavements and not used where lower quality materials would suffice.

It has been demonstrated over the years that there is no substitute for the gradual implementation of marginal materials as they need to be proved and tested and the requirement for good understanding of fundamental road engineering principles. The authors wish to emphasise the need for careful, well-managed studies as the basis for the development of these now scarce resources. The recording of material properties, the monitoring and analysis of performance and comparison with selection criteria are essential to increase knowledge and confidence in the use of marginal materials.

Research and field studies of performance of pavement materials have had resurgence in WA in the last five years. There is a need for sustained effort and sharing of knowledge so that the gains of the last few years are not lost.

14 DISCLAIMER

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