

SELECTION AND USE OF LOCALLY AVAILABLE PAVEMENT MATERIALS FOR LOW VOLUME ROADS IN WESTERN AUSTRALIA

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ABSTRACT

Locally available granular materials are an important source for the basecourse and subbase in the construction of flexible pavements in many countries. On low volume roads, these 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. Many materials have been used that do not meet conventional selection criteria but still give satisfactory performance. Their choice is based on locally developed selection criteria, non-standard testing, and attention to construction technique.

Selection criteria are presented here for lateritic basecourse gravels (covering a wide range of climates), crushed lateritic caprock, limestone, and sand-clay. These have been adapted from research in several countries and refined through experience and field research in Western Australia. Some techniques are suggested for low volume roads to cope with large truck combinations using the experience of Western Australia; where some road trains have more than 20 axles. Non-standard test methods include the WACCT accelerated triaxial test and chemical composition tests. Suggestions are made on the bitumen surfacing of some materials to improve seal adhesion.

Adequate strength by itself is insufficient for the successful use of locally available materials, and attention to construction technique and workability is needed. Guidelines are given on issues such as winning [mining], curing, mixing, oversize and compaction, dryback, and surface preparation before sealing.

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INTRODUCTION

Naturally occurring granular materials are an important source for basecourse and subbase materials in the construction of flexible pavements in many countries. 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. These materials are widely used on, but are not limited to, low volume roads, which are surfaced with bitumen seals.

Guidelines for the use of such materials have been adapted from research in several countries and refined through experience and field research in Western Australia (WA). Over many years, the experience of the State road authority, Main Roads Western Australian (MRWA), has been that with careful selection, many materials successfully used in the past did not meet conventional selection criteria but still gave satisfactory performance (1). Development of guidelines for the use of locally occurring materials has continued, and this forms the basis for the paper. The objective of this paper is to present these guidelines which have relevance for low volume roads in other countries with similar climates and materials, and which have been proven in practice on the WA road network.

PAVEMENT DESIGN AND MATERIALS

The predominant pavement type in WA for low volume roads is untreated gravel on a granular subbase with subgrades of various soils or gravels. The surfacing is usually a bituminous seal (chip and spray) or for extremely low volume roads the road is left unsealed. Bituminous sealing of the road is usually justified for traffic volumes as low as 50 vehicles per day. Less commonly used pavement types for low volume roads are cement-stabilised natural gravel (modification level, target 7-day UCS < 1 MPa or 145 psi) or crushed rock. Floodways are typically cement stabilised.

Unsaturated design

Normal practice in WA is for unsaturated pavement design. Strength testing of materials is undertaken in either unsoaked conditions or at the predicted insitu moisture content. To maintain unsaturated conditions, new and reconstructed road pavements are raised above the surrounding terrain. The road shoulder is constructed of the same material as the basecourse (full width construction). The climate generally supports unsaturated design; WA is predominately arid warm and semi-arid warm, although the extremes range from humid warm to sub-humid hot according to Thornthwaite's method (2). There are no cold or tropical rainy climates.

For the purpose of assessing probable moisture content, WA pavements have been subdivided into four drainage units. The design moisture content for the basecourse and subbase of each unit is assessed in terms of a percentage of Modified AASHTO optimum moisture content (Table 1). The units are:

Unit 1 (Well Drained) Deep water table, good external drainage, moisture deficit climate and permeability of base less than permeability of underlying layers.

Unit 2 (Permeability Inversion) Deep water table, good external drainage, moisture deficit climate and permeability of base greater than permeability of underlying layer.

Unit 3 (High Water Table) High water table or moisture surplus climate, good external drainage and permeability of base less than permeability of underlying layer.

Unit 4 (Pavement Saturation) Pavement subject to inundation (such as in a floodway), or very high water table, or poor external drainage, or a moisture surplus climate.

Traffic

A common traffic mix is 85% light and 15% heavy vehicles on low volume rural roads, although up to 35% heavy vehicles can be encountered. The legal axle load is 9 metric tonnes (9.9 short tons) on (typically) dual tyres, inflated at 700 kPa (100 psi). There is a slight difference in weight between this, and the equivalent standard axle (ESA) of 8.2 tonnes (9 short tons) in the empirical design tables in this paper. Up to 2003, 16½ tonnes (18 short tons)

were legally permitted on a dual axle group, and 20 tonnes (22 short tons) on a triple axle group. This increased slightly in 2003 for vehicles with road-friendly axle groups. The larger heavy vehicles were historically 6 axle articulated trucks consisting of a truck and trailer, but in the last 10 years trucks with double trailers totalling 11 axles and with a legal mass of 79 tonnes (87 short tons) have become common on certain routes.

Road trains

In WA, road trains are also permitted on some low volume roads, with 16 to 20+ axles. The horizontal contact stresses under the drive axles are large, particularly when the road train is turning or on a steep grade. These cause significant horizontal shear forces at the tyre-road interface, which thin bituminous surfacings and marginal basecourse materials are poorly equipped to handle. The damaging effect to thin bituminous surfacings caused by road trains can be significant (3).

This can be accommodated for some designs by increasing the traffic class, as discussed later in the paper. Alternatively at specific high stress locations such as intersections and small radius curves (less than 100 m or 300 ft), design traffic is multiplied by an empirical factor of ten (4), and localised measures might be used such as cement modification of the basecourse, and/or localised surfacing with asphalt. Applying 40mm (1.6") of asphaltic concrete at intersections will generally cover the additional basecourse thickness required, although not necessarily the total pavement thickness.

Selection of materials

The selection of naturally occurring granular materials is initially by classification tests, which include the common particle size distribution, particle durability and fines plasticity. The practices derive from British and American Standards, and in this paper are described as 'conventional' as they are the basis for selection of road pavement materials in various parts of the world including Australia.

An indication of the probable suitability of a material for use as a basecourse is initially inferred from its grading curve. As well as the usual grading envelope, there are maximum size limits imposed since experience shows that material with a high proportion greater than about 37.5 mm (1.5") may be difficult to work and shape, particularly if the coarse material is larger than about 50 mm (2"). The use of material with sizes larger than 37.5mm (1.5") 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 75mm (3") or 100mm (4") maximum size can be considered particularly where the stone is sufficiently friable for some break down during construction. A maximum size of 75mm (3") is commonly specified for crushed limestone subbase, with a minimum allowable layer thickness of 150mm (6") when using that material.

Specification limits on liquid limit (LL), plasticity index (PI) and linear shrinkage (LS) follow conventional practice, except that they can generally be relaxed in well drained arid areas where loss of strength due to moisture is less likely. Basecourse gravels with PI of 12-16, and in extreme cases 18, have been used with success in semi-arid and arid regions.

Common natural materials

In Western Australia, natural materials used for basecourse and subbase in road pavements are most commonly pedocretes which were formed in previous geological times when the climate was tropical (5) (6). Some colluvial scree gravels are also used but suitable alluvial (river) gravels are uncommon.

The use of conventional limits with pedogenic materials results in selection of materials that give highly satisfactory performance. However conventional limits may be unnecessarily restrictive when applied to soils such as pedocretes. 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

Western Australia Confined Compression Test (WACCT) and California Bearing Ratio (CBR) if applied immediately after compaction do not measure the potential for strength gain with time. Evidence of self-bonding with time has been seen in monitoring calcrete and lateritic base course sections in the north west of WA (7) and in the red clayey-sand used as a basecourse in the central coast of WA (8).

The best naturally occurring gravel for basecourse and subbase in Western Australia is generally considered to be lateritic gravel. Many laterites are well graded and have similar strength characteristics and durability as a basecourse manufactured from crushed rock. Laterite 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 gravels, a variety of scree gravels, decomposed granite, hardpan, sand-clay and river shingle.

Dryback

It is almost always specified that the compacted basecourse be partly dried out before it is sealed. Compaction is usually carried out with the moisture content close to 100% Mod AASHTO optimum moisture content (OMC), but many of these naturally occurring materials have inadequate strength at a moisture content of 100% OMC. This is not of serious concern provided adequate strength at the generally lower design moisture content is available, which occurs over time as the pavement moves to its equilibrium moisture content.

It is strongly preferred that the basecourse material has sufficient strength from the first day of opening to traffic, and MRWA specifications typically require "dry back", to a moisture content less than 85% of OMC before sealing. The same requirement is applied to the subbase and subgrade, to improve initial strength and so as not to provide a source of moisture to move into the basecourse. For crushed rock materials, the required dry back value is 60% of OMC.

SELECTION CRITERIA FOR LATERITIC GRAVELS (FERRICRETES)

Indurated ferricretes and associated pisolitic gravels (lateritic gravels) are the most common surface rocks found in Western Australia and have been used extensively for road construction.

The general approach to the properties and behaviour of lateritic gravels used for road construction 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 base 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 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.

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 Western Australia satisfy this definition. The term "lateritic gravel" as used in this paper includes all natural gravels of lateritic origin including both insitu pisolitic and transported material. The sesquioxide content of most WA "laterites" exceeds 10%. The results of chemical analysis tests for a range of Western Australian laterites are presented in Table 2. Results of tests on crushed granite are included for comparative purposes.

Selection of lateritic gravel for use as basecourse in sealed roads needs to take into consideration the factors of climate, drainage, traffic loading, the presence of road trains, and road geometry. The criteria limits are given in Table 3, and the selection criteria are shown in Tables 4 and 5.

Stabilisation of Lateritic Gravels with Coastal Limestone

The blending of laterite gravels with coastal limestone (Tamala Limestone) is a common practice in the warm humid and warm sub humid climates in the south west of Western Australia. The method reduces the cost of sourcing and winning (locating and mining) the gravel, and it reduces the effect of clays in the gravel, thus improving performance under moist conditions. The ratio of the limestone/gravel blends varies between 1:10 to 1:3, with 1:10 being typical. The blend is typically determined by testing various ratios and selecting the mix that produces the required reduction in plasticity. Caution is needed to avoid excess surface dust with high limestone blends when bituminous surfacing.

Because of the difference in particle density between laterite and limestone (typically 3.2 t/m³ and 2.4 t/m³ respectively) (200 and 150 lb/ft³) the particle size distribution should be determined on the basis of volume rather than mass, however this does not usually 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.

CALCRETES AND CALCARENITES

Calcretes and calcarenites occur in various coastal and inland arid regions of Western Australia. Some have been used successfully for basecourse and subbase. Based on the information available to date, Netterberg's (9) selection criteria appear suitable for application to nodular calcretes in Western Australia. An additional requirement that the dust ratio lie between 0.3 and 0.7 is recommended here to ensure workability and low permeability. Netterberg's criteria on stone hardness and toughness have not been verified for Western Australian conditions.

Tamala Limestone

In terms of economic significance to the road building industry, Tamala Limestone is the most important of the calcretes and calcarenites. Also known as coastal limestone, it is found in the populous coastal regions in the south west of WA. Tamala Limestone consists of calcarenite in the form of eolianite (dune limestone) and beachrock. Deposits suitable for road building occur up to 15 metres (50 ft) thick. A typical deposit includes residual quartz sand at the surface, underlain by calcrete caprock that 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 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. The results of a series of tests for limestone used as subbase on a freeway in Perth are set out in Table 6. Field measurements indicate that the CBR of compacted Tamala Limestone material (of reasonable quality) some years after construction, almost always exceeds 80.

The soaked CBR of Tamala Limestone can be predicted by various parameters, including CaCO₃ using the following regression equation with a correlation coefficient of 0.89:

$$CBR = \frac{180500 * MDCS^{0.32} \left[\frac{P0.075}{P0.425} \right]^{0.096}}{\left[72 + |72 - CaCO_3| \right]^{2.23}}$$

where MDCS is maximum dry compressive strength in kPa.
P0.075 is percentage passing a 0.075mm sieve

P0.425 is percentage passing a 0.425mm 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. Typical selection criteria for its use in the subbase and base are presented in Table 7. While it has been used successfully as a basecourse, problems of seal adhesion have occurred. Limestone can be very dusty after surface finishing; the use of cationic emulsion as a prime is not recommended due to premature breaking on the limestone. For this reason, the material is usually modified with bitumen when used as a basecourse under sprayed surface treatments (10). This is plant mixed by the addition of a minimum of 2% residual bitumen by mass to crushed limestone material.

Despite the non-plastic nature of the subbase material it is possible to cut intact cores from the pavement. Hamory and Cocks (10) reported on average indirect tensile strength of 81 kPa (12 psi) for 55 cores of limestone subbase cut from a highway in Perth in 1986 (9 years after construction). This suggests some form of self-cementation.

SAND CLAY

Sand-clay materials have been used successfully as basecourse and subbase in Western Australia. In terms of the Unified Soil Classification system, sand-clay materials used successfully as basecourse in Western Australia would be described as well graded silty sands (SW-SM) or well graded clayey sands (SW-SC).

The sand-clay materials used in Western Australia are typically red in colour (e.g. Pindan sand). The red sands and brown calcareous earths are common in north west coastal areas of WA. Not all sands within these soil units are suitable for pavement construction. Only limited chemical testing has been carried out on the fine material. However it seems likely that the red colour is due to iron and aluminium hydroxide coatings on the quartz sand particles. Results of chemical tests on a sample of sand clay used as a basecourse in the central coast of WA are set out in Table 8.

Some Pindan and other sand-clays show significant strength gain through dry back. The results of indirect tensile strength tests on samples of the sand-clay shown in Table 8 are presented in Table 9. They show the self-cementation property after dryback during construction, with a substantial strength gain in dry moisture conditions. This strength is lost upon re-wetting, and it is not re-gained if compacted significantly dry of OMC. However, if the material is wetted then compacted and dried back, self-cementation can re-occur. The higher strength of cores from the road compared to the laboratory is thought to be due to long-term cementation strength increase as well as increased suction from reduced void geometry after traffic compaction of the material in service.

The self-cementation was previously thought to be due to the bridging effect of clay in the Pindan, and the bridges were due simply to iron oxide. Laboratory testing by scanning electron microscopy and x-ray diffraction suggests that the bridges also form from Fe-Kaolinite which contains both iron and aluminium (hydr)oxides. Suction testing showed a gain in shear strength upon drying out (an increase in suction). This suggests that the strength gain of the Pindan upon dryback is not just due to the cementing action of the bridges, but is also due to increased suction from the changed void geometry after the bridges have formed (11).

Basecourse

For red sand-clays used as a basecourse, selection criteria have been proposed based on unpublished research on a road in the north west of WA (Table 10). It should be noted that edge wear is a significant problem with roads constructed with a sand-clay basecourse. An increase in total seal width of one metre (3 feet) is recommended to allow for losses, on a typical low volume width of two lanes of 3.1 metres (10 feet) each.

Subbase

For red sand-clays used as a subgrade, selection criteria can be relaxed but caution is needed to ensure strength gain through dryback is achieved, and to control any potential collapse behaviour. Some Pindans can be collapsing soils, with low-density in-situ that may densify under load at high moisture contents. If not properly compacted, this could result in differential movement within the pavement structure, leading to general unevenness or even failure.

The criteria for red sand-clay for use as subbase in well drained sites in arid climatic areas are set out in Table 11, and derive from research on roads and airport pavements in the north west of WA. The materials typically fall into the clayey sand (SC) and silty sand (SM) classifications, and the SC materials are preferred for the subbase. Typically soaked CBR tests on disturbed materials yield results in the range CBR 3-7. Under a sealed surface, where a low equilibrium moisture content can be achieved (such as in drainage unit 1), properly compacted and dried-back Pindan sand-clay materials may exhibit field CBRs in the range 20-80.(11)

The laboratory testing discussed above and construction experience with Pindan suggest that the material has two divergent behaviours. When wet, Pindan sand-clay behaves like wet loose sand, so compaction can be done by inundation and rolling, just like sand. This process must be followed, and be thorough, to collapse the material. After compaction the Pindan is "dried back" or "baked up" and then it displays the characteristics of clay, with a high cohesion and strength. There is some experience to suggest that destruction of dry clay bridges by vibratory compaction in a dry condition severely impairs their ability to reform later on drying back. In construction there is a preference for static compaction, and good results have been had using a 35 ton static pneumatic roller. If vibrating, low frequency and low amplitude vibration is preferable to high frequency and high amplitude vibration, and soil moisture content is typically kept at 1 percentage point above standard compaction OMC.

Strength testing of the compacted and dried back material is used during construction to identify weak patches of Pindan sand-clay, which can occur with the more sandy (silty-sand SM) Pindan or with low sesquioxide levels in the kaolinite clay. The better materials typically have a cation exchange capacity (CEC) of 30-35 meq/100ml, compared to the weak patches with a CEC of 20-25, although this test is not considered routine. The weak patches are usually removed and replaced with more clayey sand material. For less important roads with a Pindan sub-base, strength is indirectly checked through density testing of the basecourse. If the Pindan sub-base strength is low, its support of the basecourse during compaction will be inadequate, and the basecourse above will usually fail to reach density. For more important pavements, direct insitu strength testing of the Pindan is preferred and in practical terms, the dynamic cone penetrometer (DCP) is easy to use. Typical target values after dryback and before covering are >40% DCP-CBR for Pindan used as a subbase (11).

CONCLUSIONS

Many locally available granular materials can be used for the basecourse and subbase of low volume roads that do not meet conventional selection criteria. Although considered marginal, they can be selected to give satisfactory performance. Selection criteria are presented here for lateritic basecourse gravels (covering a wide range of climates), crushed lateritic caprock, limestone, and sand-clay. These have been developed through experience and field research. Some techniques are suggested for low volume roads to cope with road trains such as are experienced in Western Australia; some of which have more than 20 axles.

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TABLE 1 Typical Design Moisture Content for natural gravel

Drainage Unit	Design Moisture Content Typical ^a
Unit 1	85% of OMC
Unit 2	110% of OMC
Unit 3	100% of OMC
Unit 4	120% of OMC (soaked)

Note: (a) Does not apply to humid climate condition (which is only of limited extent in WA) or to floodways (road sections across creeks designed to overtopped during high rainfall). Crushed rock basecourse materials are typically assessed in repeated load triaxial testing at 60% Mod AASHTO OMC representing the dry condition, and 85% Mod AASHTO OMC representing the wet condition (13).

TABLE 2 Chemical Composition of some Western Australian Lateritic Gravels used in Road Construction ^a

Property	Lateritic Gravel			Crushed Granite Rock Base		
	Al ₂ O ₃	Fe ₂ O ₃	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	SiO ₂
Mean	12.8	6.8	16.8	2.0	1.6	6.8
Std. dev.	7.1	2.6	6.4	0.4	0.2	1.1
No. samples	13	13	13	3	3	3

Note: (a) Expressed as a percentage by mass of the fraction passing a 0.425 mm sieve. Determined by wet chemistry/inductively coupled plasma spectrometry. SiO₂ is the non-quartz SiO₂. Testing by Department of Geology and Geophysics at University of Western Australia.

TABLE 3 Required Classification Numbers for Lateritic Gravel (Well Drained ^a)

Climate ^b	Required material classification number ^c				
	Design Traffic (equivalent standard axles) ^{d e}				
	≤5 E6 ^f	≤1 E6	≤5 E5	≤1 E5	≤5 E4
Sub humid hot	Lt6	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: (a) The matrix in Table 3 is for well drained (drainage unit 1) conditions only. The effect of a permeability inversion (drainage unit 2) or a high water table (drainage unit 3) can be accommodated in Table 3 by assigning a more adverse climatic zone. Drainage units have been described elsewhere in this paper.

(b) Thornthwaite's classification (2)

(c) The designation of Lt6, Lt10 and Lt 16 relate to plasticity index limits of 6, 10 and 16 respectively.

(d) 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.

(e) Design traffic refers to equivalent number of standard axle load repetitions (ESAs) in one direction during the design life of the pavement. ESA is a single axle load of 8.2 tonnes on dual tyres. The standard design life is typically 20 years.

(f) The use of lateritic gravel for roads carrying more than 5 E6 ESA is not precluded. However specialist advice should be obtained, particularly in relation to the selection of bituminous surfacing.

TABLE 4 Typical Selection Criteria for Lateritic Gravels based on grading and Classification Tests ^a

Type of Material	Lateritic Gravel				Crushed Rock ^b
Designation ^c		Lt6	Lt10	Lt16	-
Grading ^e	Sieve Size mm	% Passing	% Passing	% Passing	% Passing
	37.5	100 ^d	100 ^d	100 ^d	-
	19.0	71-100	95-100	95-100	95-100
	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
	0.425	11-32	11-39	11-39	11-23
	0.075	4-19	4-23	4-23	5-11
	0.0135	2-9	2-11	2-11	
Dust Ratio ^f		0.3-0.7			0.35-0.6
Liquid Limit ^g %		≤ 25	≤ 30	≤ 35	≤ 25
Plasticity Index %		≤ 6	≤ 10	≤ 16	Note (m)
Linear Shrinkage %		≤ 3	≤ 5	≤ 8	0.4-2.0
P0.425 x LS % ^h		≤ 150	≤ 200	≤ 250	-
Dry Compressive Strength ⁱ kPa		≥ 1700			
Particle Toughness		Note j			35
Dryback % (k)		≤ 85			≤ 60

Notes: (a) Selection criteria apply to basecourse roads with a thin bituminous surfacing with a 20 year design traffic loading of up to 5 E6 ESA. ESA is a single axle load of 8.2 tonnes on dual tyres.

(b) Non lateritic, included for comparison purposes only (14).

(c) See Table 3 for applicable climate zones and traffic loading.

(d) Most deposits of lateritic gravel contain some oversize material which must be broken down by grid rolling.

(e) Dry Sieving, MRWA Test Method WA 115.1 Particle Size Distribution: Sieving and Decantation Method.

(f) Dust Ratio = P0.075/P0.425 where P0.075 is the percentage passing the 0.075 mm sieve and P0.425 is the percentage passing the 0.425 mm sieve. Material with a low dust ratio is likely to be harsh and the achievement of a satisfactory surface prove difficult.

(g) Liquid Limit (using the cone apparatus; MRWA Test Method WA 120.2 Liquid Limit: Cone Penetrometer Method), MRWA Test Method 122.1 Plasticity Index and MRWA Test Method 123.1 Linear Shrinkage tests on samples air dried at 50°C.

(h) For materials approaching the upper limit of either plasticity index or the product of % passing 0.425mm by LS, the confirmation of suitability by strength testing is recommended.

(i) MRWA Test Method WA 140.1 Maximum Dry Compressive Strength

(j) No particular test for particle toughness is specified at this time. However the lateritic pebble must be hard and durable. The Los Angeles Abrasion Value of crushed rock base shall be determined using MRWA Test Method WA 220.1 Los Angeles Abrasion Value.

(k) Target moisture content as % Mod AASHTO OMC

(m) Controlled by linear shrinkage

- not applicable

TABLE 5 Typical Selection Criteria for Lateritic Gravel Based on Strength and Classification Tests ^a

Designation ^b	Lt6	Lt10	Lt16
WACCT Class No. ^c	≤ 2.0		≤ 2.3
WACCT Cohesion kPa Tensile strength kPa	≤ 85 ≤ 55		
CBR soaked	≥ 80	≥ 60	≥ 60
CBR unsoaked ^d	≥ 80		
Maximum Size ^e mm	37.5		
Grading Modulus ^f	≥ 1.5		
Dust Ratio ^g	0.3-0.7		
Plasticity Index	≤ 6	≤ 10	≤ 16
%Linear Shrinkage	≤ 3	≤ 5	≤ 8
P0.425 x LS (%)	≤ 150	≤ 200	≤ 250
Particle Toughness	Note ^h		
Dryback % ⁱ	≤ 85		

Notes: (a) Selection criteria apply to basecourse roads with a thin bituminous surfacing with a 20 year design traffic of up to 5x10⁶ ESA.

(b) See Table 3 for applicable climate zones and traffic loading.

(c) West Australian Confined Compressive Test, MRWA Test Method Test Method WA 142.1. This is a modified triaxial test in which the lateral stress is kept constant, and the vertical stress increased until failure occurs. Usually assessed at specified density for project and at the design moisture content for the site. Cohesion and tensile strength may be reduced to 45 kPa and 30 kPa respectively provided the angle of shearing resistance is $>60^\circ$. These parameters are not as critical where the shoulders are sealed. The criteria for the assessment by WACCT may not be met when testing specimens immediately after compaction. In these cases the specimens should be compacted at 100% OMC, dried to design moisture content and cured for three weeks without further loss of moisture prior to testing.

(d) CBR specimens compacted at Mod AASHTO OMC and the specified density for the project and tested at design unsoaked moisture conditions.

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

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

$$(g) \quad \text{Dust Ratio} = P_{0.075}/P_{0.425}$$

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

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

TABLE 6 California Bearing Ratios of Crushed Limestone

Test Conditions	CBR ^b		Number of Tests
	Mean	Standard Deviation	
Soaked (4 day)	72	1	2
Unsoaked (Target Moisture = OMC) ^a	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: (a) OMC = Mod AASHTO optimum moisture content

(b) MRWA test method WA 141.1 California Bearing Ratio

TABLE 7 Typical Specification for Crushed Tamala Limestone

Pavement Layer	Sub Base	Basecourse
Material Type	Crushed Limestone	Bitumen Stabilised Crushed Limestone ^f
Sieve Size mm	% Passing ^a	% Passing ^a
75.0	95-100	
26.5	-	100
19.0	55-85	90-100
4.75	-	60-90
2.36	36-65	-
1.18	-	35-75
MDCS ^b kPa	≥ 650	≥ 650
CaCO ₃ ^c %	60-85	60-85
Los Angeles Abrasion ^d (modified test)	20-60	20-60
Bitumen Content %	n.a.	≥ 2
Dry Density Ratio ^e %	≥ 94-95	≥ 95-96
Dry Back	≤ 85% OMC	≤ 85% OMC

Notes: (a) Dry sieving and decantation MRWA Test Method WA 115.1, except that a 75.0mm sieve should be used.

(b) Maximum Dry Compressive Strength, MRWA Test Method WA 140.1.

(c) CaCO₃ by solution in HCl acid, MRWA Test Method WA 915.1.

(d) 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.

(e) Dry density ratio refers to modified AASHTO compaction. MRWA Test Method WA 141.1. Required density to be selected by the designer in the range stated.

(f) The bitumen emulsion used for modification should be slow setting anionic with Vinsol Resin as the emulsifying agent.

- Not applicable

TABLE 8 Chemical Composition of Red Sand-Clay from Hamelin-Denham Road in the central coast of Western Australia

Property	Al_2O_3^a	Fe_2O_3^a	SiO_2^a	CaCO_3^b
% by mass	6.2	3.3	8.4	6.1

Notes: (a) Tests on fraction passing 0.425 mm sieve, determined by wet chemistry/inductivity coupled plasma spectrometry.

(b) Tests on fraction passing 0.425 mm sieve, determined by solution in HCl acid

TABLE 9 Indirect Tensile Strength Tests on Red Sand Clay from Hamelin-Denham Road in the central coast of Western Australia

Sample Preparation	Test Condition	Indirect Tensile Strengths kPa
Cores cut from road	Field moisture (~ 50% OMC)	281-573
Compacted in laboratory at 50% OMC ^a	50% OMC	25-45
Compacted in laboratory at 100% OMC	Dried back to 50% OMC	252-288

Notes: (a) Mod AASHTO compaction

TABLE 10 Suggested Interim Selection Criteria for Red Clayey or Silty Sand for use as a Basecourse on Well Drained Sealed Roads in an Arid Climate ^a

DESIGN TRAFFIC ESA		< 1 E5	< 3 E4
Grading	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
Dust Ratio ^b		0.2-0.6	
Liquid Limit %		≤ 20	
Plasticity Index %		≤ 8	
Linear Shrinkage		1-3	
CBR ^c	Unsoaked	≥ 80	
WACCT ^d	Class No	≤ 2.0	≤ 2.3
	Cohesion kPa	≤ 85	
	Tensile Strength kPa	≥ 55	
	Horizontal separation of Class No Contours %	≥ 1.3	
Mod AASHTO Maximum Dry Density ^e kg/cu.m		≥ 2100	
Mod AASHTO Optimum Moisture Content		5-7	
Al ₂ O ₃ + Fe ₂ O ₃ % ^f		>8 ?	

Notes: (a) Arid climate with a rainfall deficit > 2500mm, where rainfall deficit = potential evaporation - annual rainfall.

(b) Dust Ratio = P_{0.075}/P_{0.425}

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

(d) 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. 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.

(e) Maximum dry density. MRWA Test Method WA 133.1, similar to Modified AASHTO method.

(f) Al₂O₃ + Fe₂O₃ determined by /inductively coupled plasma spectrometry on the fraction passing a 0.425 mm sieve.

TABLE 11 Suggested Interim Selection Criteria for Red Clayey or Silty Sand for use as a Subbase or Selected Subgrade on Well Drained Sealed Pavements in an Arid Climate^a

Layer	Subbase	Select subgrade
Field DCP-CBR after dryback ^a	≥ 40	≥ 20
Grading % passing P0.425 sieve	30-100	
Grading % passing P0.075 sieve	15? – 40 ^c	
PI x P0.075	≥ 150	
Liquid limit %	≤ 25	
Plasticity index %	4-12	
MDD modified tonnes/cu.m.	≥ 2.0	
OMC	5-10	
$Al_2O_3 + Fe_2O_3$ % ^b	>8 ?	

Notes: (a) For low volume roads with design traffic < 1 E6 ESA

(b) $Al_2O_3 + Fe_2O_3$ determined by /inductively coupled plasma spectrometry on the fraction passing a 0.425 mm sieve.

(c) The lower limit is tentative. Some of the better performing Pindan sand-clays had $P0.075 > 25$, and this may be a guide for subbase quality. However it is suggested that characterisation of suitable sand-clay should be done by field strength testing or the use of the $PI \times P0.075$ product rather than grading alone.