Selection and Use of Locally Available Pavement Materials for Low-Volume Roads in Western Australia

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In many countries locally available granular materials are an important source for the base course and subbase in the construction of flexible pavements. 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, nonstandard testing, and attention to construction technique. Selection criteria are presented here for lateritic base course gravels (covering a wide range of climates), crushed lateritic caprock, limestone, and sand–clay. The criteria 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 on the basis of the experience of Western Australia, where some road trains have more than 20 axles. Nonstandard test methods include the Western Australia confined compression test accelerated triaxial test and chemical composition tests. Suggestions are made concerning 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, dry-back, and surface preparation before sealing.

Naturally occurring granular materials are an important source in many countries for base course and subbase materials in the construction of flexible pavements. They include fine-grained materials such as well-graded silty and clayey sands (sand–clay) and 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 that 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 of the materials that were successfully used in the past that did not meet conventional selection criteria still gave satisfactory performance (*1*). Development of guidelines for the use of locally occurring materials has continued, and that forms the basis for this paper. The objective is to present guidelines that have relevance for low-volume roads in other countries with similar climates and materials and that have been proved in practice on the WA road network.

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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-stabilized natural gravel (modification level, target 7-day UCS < 1 MPa or 145 psi) or crushed rock. Floodways are typically cement stabilized.

Unsaturated Design

Normal practice in WA is for unsaturated pavement design. Strength testing of materials is undertaken either in unsoaked conditions or at the predicted in situ 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 base course (full-width construction). Generally, the climate supports unsaturated design; WA is predominately arid warm and semiarid warm, although the extremes range from humid warm to subhumid 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 base course and subbase of each unit is assessed in regard to a percentage of modified AASHTO optimum moisture content (Table 1). Descriptions of the units follow:

- 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;

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)

[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 base course 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 (*3*).

- 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; and
- 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 tons (9.9 short tons) on (typically) dual tires, inflated at 700 kPa (100 psi). There is a slight difference in weight between this and the equivalent standard axle (ESA) of 8.2 metric tons (9 short tons) in the empirical design tables in this paper. Up to 2003, 16.5 metric tons (18 short tons) were legally permitted on a dual-axle group and 20 metric tons (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 six-axle articulated trucks consisting of a truck and trailer, but in the past 10 years trucks with double trailers totaling 11 axles and with a legal mass of 79 metric tons (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 tire–road interface, which thin bituminous surfacings and marginal base course materials are poorly equipped to handle. The damaging effect to thin bituminous surfacings caused by road trains can be significant (*4*).

For some designs this can be accommodated 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 10 (*5*), and localized measures might be used such as cement modification of the base course or localized surfacing with asphalt. Generally, applying 40 mm (1.6 in.) of asphaltic concrete at intersections will cover the additional base course 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" because they are the basis for the 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 base course is initially inferred from its grading curve. As well as the usual grading envelope, there are maximum size limits imposed because experience shows that material with a high proportion greater than about 37.5 mm (1.5 in.) may be difficult to work and shape, particularly if the coarse material is larger than about 50 mm (2 in.). The use of material of a size larger than 37.5 mm (1.5 in.) is usually restricted to subbase layers in which surface finish requirements are not as critical. The use of subbase material with large particles, for instance, up to 75 mm (3 in.) or 100 mm (4 in.) maximum size, can be considered, particularly in cases in which the stone is sufficiently friable for some breakdown during construction. A maximum size of 75 mm (3 in.) is commonly specified for crushed limestone subbase, with a minimum allowable layer thickness of 150 mm (6 in.) when that material is used.

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

Common Natural Materials

In Western Australia, natural materials used for base course and subbase in road pavements are most commonly pedocretes, which were formed in previous geological times when the climate was tropical (*6, 7*). 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-stabilization under the influence of wetting and drying cycles.

In instances in which strength is inferred on the basis of particle size distribution and plasticity tests only, this potential for self-stabilization is not taken into account. Similarly, if applied immediately after compaction, tests such as the Western Australia confined compression test (WACCT) and California bearing ratio (CBR) 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 northwest area of WA (*8*) and in the red clayey sand used as a base course in the central coast of WA (*9*).

In general, the best naturally occurring gravel for base course and subbase in Western Australia is considered to be lateritic gravel. Many laterites are well graded and have strength characteristics and durability similar to a base course 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 base course in WA. These include limestone gravels, a variety of scree gravels, decomposed granite, hardpan, sand–clay, and river shingle.

Dry-Back

It is almost always specified that the compacted base course 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 base course material have 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 base course. 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 WA and have been used extensively for road construction.

The general approach to the properties and behavior 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 characterize 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. Judgment 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 because 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 of less than 2. However, not all materials regarded as lateritic in WA satisfy this definition. The term lateritic gravel as used in this paper includes all natural gravels of lateritic origin, including both in situ pisolitic and transported material. The sesquioxide content of most WA laterites exceeds 10%. The results of chemical analysis tests for a range of WA laterites are presented in Table 2. Results of tests on crushed granite are included for comparative purposes.

Selection of lateritic gravel for use as base course in sealed roads needs to take into consideration the factors of climate, drainage, traffic loading, 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.

TABLE 2 Chemical Composition of Some Western Australian Lateritic Gravels Used in Road Construction

	Lateritic Gravel			Crushed Granite Rock Base		
Property	Al_2O_3	Fe_2O_3	SiO_2	Al_2O_3	Fe_2O_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: Expressed as a percentage by mass of the fraction passing a 0.425-mm sieve. Determined by wet chemistry/inductively coupled plasma spectrometry. SiO_2 is the nonquartz SiO_2. Testing by Department of Geology and Geophysics at University of Western Australia.

STABILIZATION 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 subhumid climates in the southwest area 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 and 1:3, with 1:10 being typical. Typically, the blend is 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 the bituminous surfacing is applied.

Because of the difference in particle density between laterite and limestone (typically 3.2 t/m^3 and 2.4 t/m^3, respectively) (200 and 150 lb/ft^3), the particle size distribution should be determined on the

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

	Design Traffic (equivalent standard axles)[b,c]				
Climate[a]	≤5 E6[d]	≤1 E6	≤5 E5	≤1 E5	≤5 E4
Subhumid hot	Lt6	Lt6	Lt6	Lt10	Lt10
Semiarid hot	Lt10	Lt10	Lt10	Lt16	Lt16
Arid hot	Lt10	Lt10	Lt16	Lt16	Lt16
Arid warm	Lt10	Lt10	Lt16	Lt16	Lt16
Semiarid warm	Lt10	Lt10	Lt10	Lt10	Lt16
Subhumid warm	Lt6	Lt6	Lt6	Lt10	Lt10
Humid warm	Lt6	Lt6	Lt6	Lt6	Lt6

NOTE: 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. The designations of Lt6, Lt10, and Lt16 relate to plasticity index limits of 6, 10, and 16, respectively.

[a]Thornthwaite's classification (2).

[b]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.

[c]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 tires. The standard design life is typically 20 years.

[d]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

	Lateritic Gravel, Designation[a]			Crushed Rock[b]
	Lt6	Lt10	Lt16	
Grading:[c] percent passing for sieve size (mm)				
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[e]	0.3–0.7	0.3–0.7	0.3–0.7	0.35–0.6
Liquid limit[f] (%)	≤25	≤30	≤35	≤25
Plasticity index (%)	≤6	≤10	≤16	[g]
Linear shrinkage (%)	≤3	≤5	≤8	0.4–2.0
P0.425 × LS[h] (%)	≤150	≤200	≤250	—
Dry compressive strength[i] (kPa)	≥1,700	≥1,700	≥1,700	≥1,700
Particle toughness[j]				35
Dry-back[k] (%)	≤85	≤85	≤85	≤60

NOTE: Selection criteria apply to base course 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 tires. A dash indicates not applicable.
[a]See Table 3 for applicable climate zones and traffic loading.
[b]Nonlateritic; included for comparison purposes only (*10*).
[c]Dry sieving, MRWA Test Method WA 115.1 Particle Size Distribution: Sieving and Decantation Method.
[d]Most deposits of lateritic gravel contain some oversize material, which must be broken down by grid rolling.
[e]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 a satisfactory surface will prove difficult to achieve.
[f]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.
[g]Controlled by linear shrinkage.
[h]For materials approaching the upper limit of either plasticity index or the product of percent passing 0.425-mm 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 with MRWA Test Method WA 220.1 Los Angeles Abrasion Value.
[k]Target moisture content as % Mod AASHTO OMC.

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

	Designation[a]		
	Lt6	Lt10	Lt16
WACCT			
Class No.[b]	≤2.0	≤2.0	≤2.3
Cohesion (kPa)	≤85	≤85	≤85
Tensile strength (kPa)	≤55	≤55	≤55
CBR soaked	≥ 80	≥60	≥60
CBR unsoaked[c]	≥80	≥80	≥80
Maximum size[d] (mm)	37.5	37.5	37.5
Grading modulus[e]	≥1.5	≥1.5	≥1.5
Dust ratio[f]	0.3–0.7	0.3–0.7	0.3–0.7
Plasticity index	≤6	≤10	≤16
% linear shrinkage	≤3	≤5	≤8
P0.425 × LS (%)	≤150	≤200	≤250
Particle toughness[g]			
Dry-back[h] (%)	≤85	≤85	≤85

NOTE: Selection criteria apply to base course roads with a thin bituminous surfacing with a 20-year design traffic of up to 5×10^6 ESA.
[a]See Table 3 for applicable climate zones and traffic loading.
[b]West Australian Confined Compressive Test, MRWA 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°. These parameters are not as critical where the shoulders are sealed. The criteria for the assessment by WACCT may not be met when specimens are tested immediately after compaction. In these cases the specimens should be compacted at 100% OMC, dried to design moisture content, and cured for 3 weeks without further loss of moisture prior to testing.
[c]CBR specimens compacted at Mod AASHTO OMC and the specified density for the project and tested at design unsoaked moisture conditions.
[d]Most deposits of lateritic material include some oversize material, which must be broken down by grid rolling.
[e] $\text{Grading modulus} = \dfrac{300 - (P2.36 + P0.425 + P0.075)}{100}.$
[f]Dust ratio = P0.075/P0.425.
[g]No particular test for particle toughness is specified at this time. However, the lateritic pebble must be hard and durable.
[h]Base course should be dried back to a moisture content of less than 85% of OMC prior to application of bituminous surfacing.

basis of volume rather than mass; however, that does not usually occur. In cases in which 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 WA. Some have been used successfully for base course and subbase. On the basis of the information available to date, Netterberg's (*11*) selection criteria appear suitable for application to nodular calcretes in WA. 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.

Of the calcretes and calcarenites, Tamala limestone is the most important because of its economic significance to the road building industry. Also known as coastal limestone, it is found in the populous coastal regions in the southwest area 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 m (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. Typically, beneath the caprock layer is a thick layer of calcarenite that becomes less cemented with depth and therefore weaker and friable as a result. The resulting material is nonplastic.

Crushed Tamala limestone appears to gain strength when compacted at OMC and dried back. Results of a series of tests for limestone used as subbase on a freeway in Perth are shown 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 $CaCO_3$ using the following regression equation with a correlation coefficient of 0.89:

$$CBR = \frac{180{,}500 * MDCS^{0.32}\left[\frac{P0.075}{P0.425}\right]^{0.096}}{\left[72+|72-CaCO_3|\right]^{2.23}}$$

where

MDCS = maximum dry compressive strength (kPa),
P0.075 = percentage passing 0.075-mm sieve,
P0.425 = percentage passing 0.425-mm sieve, and
$|72 - CaCO_3|$ = 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. Although it has been used successfully as a base course, 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 because of premature breaking on the limestone. For this reason, the material is usually modified with bitumen when used as a base course under sprayed surface treatments (*12*). This is plant mixed by the addition of a minimum of 2% residual bitumen by mass to crushed limestone material.

Despite the nonplastic nature of the subbase material, it is possible to cut intact cores from the pavement. Hamory and Cocks (*12*) reported an 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.

TABLE 6 California Bearing Ratios of Crushed Limestone

	CBR[a]		
Test Condition	Mean	Standard Deviation	Number of Tests
Soaked (4 days)	72	1	2
Unsoaked (target moisture = OMC)[b]	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

[a]MRWA Test Method WA 141.1 California Bearing Ratio.
[b]OMC = Mod AASHTO optimum moisture content.

TABLE 7 Typical Specifications for Crushed Tamala Limestone

	Pavement Layer and Material Type	
	Subbase, Crushed Limestone	Base Course, Bitumen-Stabilized Crushed Limestone[a]
Percent passing[b] for sieve size (mm)		
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[c] (kPa)	≥650	≥650
$CaCO_3$[d] (%)	60–85	60–85
Los Angeles abrasion[e] (modified test)	20–60	20–60
Bitumen content (%)	—	≥2
Dry density ratio[f] (%)	≥94–95	≥95–96
Dry-back	≤85% OMC	≤85% OMC

NOTE: Dash indicates not applicable.
[a]The bitumen emulsion used for modification should be slow-setting anionic with Vinsol resin as the emulsifying agent.
[b]Dry sieving and decantation MRWA Test Method WA 115.1, except that a 75.0-mm sieve should be used.
[c]Maximum dry compressive strength, MRWA Test Method WA 140.1.
[d]$CaCO_3$ by solution in HCl acid, MRWA Test Method WA 915.1.
[e]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.
[f]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.

SAND–CLAY

Sand–clay materials have been used successfully as base course and subbase in WA. According to the Unified Soil Classification system, sand–clay materials used successfully as base course 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 typically are red (e.g., pindan sand). The red sands and brown calcareous earths are common in northwest coastal areas of WA. Not all sands in these soil units are suitable for pavement construction. Only limited chemical testing has been carried out on the fine material. However it appears likely that the red color is due to iron and aluminum hydroxide coatings on the quartz sand particles. Results of chemical tests on a sample of sand–clay used as a base course in the central coast of WA are shown in Table 8.

TABLE 8 Chemical Composition of Red Sand–Clay from Hamelin–Denham Road on 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

[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 on Central Coast of Western Australia

Sample Preparation	Test Condition	Indirect Tensile Strength (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

[a]Mod AASHTO compaction.

Some pindan and other sand–clays show significant strength gain through dry-back. 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 dry-back during construction, with a substantial strength gain in dry moisture conditions. This strength is lost on rewetting, and it is not regained if compacted significantly dry of OMC. However, if the material is wetted and then compacted and dried back, self-cementation can recur. The higher strength of cores from the road compared with 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 aluminum (hydr)oxides. Suction testing showed a gain in shear strength on drying out (an increase in suction). This suggests that the strength gain of the pindan on dry-back is not due just to the cementing action of the bridges, but is due also to increased suction from the changed void geometry after the bridges have formed (*13*).

Base Course

For red sand–clays used as a base course, selection criteria have been proposed based on unpublished research on a road in the northwest area of WA (Table 10). It should be noted that edge wear is a significant problem with roads constructed with a sand–clay base course. On a typical low-volume width of two lanes of 3.1 m (10 ft) each, an increase in total seal width of 1 m (3 ft) is recommended to allow for losses.

Subbase

For red sand–clays used as a subgrade, selection criteria can be relaxed, but caution is needed to ensure that strength gain through dry-back is achieved and to control any potential collapse behavior. Some pindans can be collapsing soils, with low density in situ that may densify under load at high moisture contents. If not properly compacted, differential movement in the pavement structure could result, 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 (Table 11) derive from research on roads and airport pavements in the northwest area 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 CBR 3 to 7 range. 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 20 to 80 range (*13*).

The laboratory testing discussed above and construction experience with pindan suggest that the material has two divergent behaviors. When wet, pindan sand–clay behaves like wet, loose sand, so compaction can be done by inundation and rolling, just as is done for sand. This process must be followed, and must 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 re-form later on drying back. In construction there is a preference for static compaction, and good results have been

TABLE 10 Suggested Interim Selection Criteria for Red Clayey or Silty Sand for Use as a Base Course on Well-Drained Sealed Roads in an Arid Climate

		Design Traffic ESA	
		<1 E5	<3 E4
Grading: percent passing for sieve size (mm)			
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[a]		0.2–0.6	0.2–0.6
Liquid limit (%)		≤20	≤20
Plasticity index (%)		≤8	≤8
Linear shrinkage		1–3	1–3
CBR[b]	Unsoaked	≥80	≥80
WACCT[c]	Class no.	≤2.0	≤2.3
	Cohesion (kPa)	≤85	≤85
	Tensile strength (kPa)	≥55	≥55
	Horizontal separation of class no contours (%)	≥1.3	≥1.3
Mod AASHTO maximum dry density[d] (kg/m³)		≥2,100	≥2,100
Mod AASHTO optimum moisture content		5–7	5–7
$Al_2O_3 + Fe_2O_3$[e] (%)		>8?	>8?

NOTE: Arid climate with a rainfall deficit > 2,500 mm, where rainfall deficit = potential evaporation – annual rainfall.
[a]Dust ratio = P0.075/P0.425.
[b]California bearing ratio carried out on specimens compacted at 90–100% OMC to 95% MDD (modified AASHTO).
[c]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 3 weeks without further loss of moisture prior to testing.
[d]Maximum dry density. MRWA Test Method WA 133.1, similar to Modified AASHTO method.
[e]$Al_2O_3 + Fe_2O_3$ determined by wet chemistry/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

	Subbase	Select Subgrade
Field DCP-CBR after dry-back[a]	≥40	≥20
Grading % passing P0.425 sieve	30–100	30–100
Grading % passing P0.075 sieve	15[b]–40	15[b]–40
PI × P0.075	≥150	≥150
Liquid limit (%)	≤25	≤25
Plasticity index (%)	4–12	4–12
MDD modified (tonnes/m^3)	≥2.0	≥2.0
OMC	5–10	5–10
$Al_2O_3 + Fe_2O_3$[c] (%)	>8?	>8?

[a]For low-volume roads with design traffic < 1 E6 ESA.
[b]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 characterization of suitable sand–clay should be done by field strength testing or the use of the PI × P0.075 product rather than grading alone.
[c]$Al_2O_3 + Fe_2O_3$ determined by wet chemistry/inductively coupled plasma spectrometry on the fraction passing a 0.425-mm sieve.

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 to 35 meq/100 mL, compared with the weak patches with a CEC of 20–25 meq/100 mL, 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 subbase, strength is indirectly checked through density testing of the base course. If the pindan subbase strength is low, its support of the base course during compaction will be inadequate, and the base course above will usually fail to reach density. For more important pavements, direct in situ strength testing of the pindan is preferred, and in practical terms, the dynamic cone penetrometer (DCP) is easy to use. Typical target values after dry-back and before covering are >40% DCP-CBR for pindan used as a subbase (*13*).

CONCLUSIONS

Many locally available granular materials that do not meet conventional selection criteria can be used for the base course and subbase of low-volume roads. Although they are considered marginal, they can be selected to give satisfactory performance. Selection criteria are presented here for lateritic base course 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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