Engineering Road Note 9
January 2010

PROCEDURE FOR THE DESIGN OF FLEXIBLE PAVEMENTS

1. INTRODUCTION

This Note outlines the procedure to be used for the design of flexible road pavements under the control of the Commissioner of Main Roads, Western Australia.

Concrete pavements must be designed using the procedure for rigid road pavements in AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2008).

The design, construction and maintenance of road pavements involves the management of risks. The risks are particularly high for unbound granular or modified granular pavements with asphalt wearing courses designed for a high design traffic loading.

Complying with the requirements detailed in the Austroads guidelines, Main Roads guidelines (including this Note) or specified in Main Roads construction specifications may not ensure that the performance of the pavement will meet all obligations.

Pavement designs should only be undertaken by experienced practitioners that understand the risks involved and can use their skill, knowledge and experience to determine whether it is necessary to exceed the requirements:

(a) detailed in Austroads guidelines;

(b) detailed in Main Roads guidelines (including this Note); and

(c) specified in the Main Roads construction specifications.

DEFINITIONS

Where this Note refers to "**nominal thicknesses**" or "**nominal total thickness**" it is referring to the thicknesses shown in the drawings or otherwise specified (i.e. if there are no drawings).

Where this Note refers to "**the Principal**" it means the Commissioner of Main Roads Western Australia or his nominated representative.

"**asphalt wearing courses**" is that part of a pavement upon which traffic travels including any 10mm dense graded asphalt course below a course of open graded asphalt.

"**asphalt intermediate course**" is that part of a pavement which rests on the asphalt base course, subgrade, improved subgrade or subbase and is below the asphalt wearing courses.

"**asphalt base course**" is one or more layers of asphalt within a full depth asphalt pavement immediately below the asphalt intermediate course.

1.1 **General**

1.1.1 Minimum Design Life

Unless specified otherwise by the Principal, the permanent deformation of the pavement must have a minimum design life of 40 years.

Unless specified otherwise by the Principal, the asphalt design fatigue life must greater than or equal to the values in Table 1.

TABLE 1 Minimum Asphalt Design Fatigue Life

Asphalt Nominal Total Thickness	
60 mm or less	Greater Than 60 mm
15 Years	40 Years

1.1.2 Verification of Design Assumptions

At the time that a pavement design is being undertaken, the properties of the pavement material or the properties of the subgrade may not be known and it may be necessary to assign properties to these materials. All properties assigned in the pavement design must be verified by undertaking laboratory testing of the materials to be used. If necessary the proposed pavement materials and/or the proposed pavement design must be amended after these test results become available.

1.1.3 Design Procedures

As a minimum, the thickness of the granular base material and the total thickness of granular pavement material must both be greater than or equal to the thicknesses that would be determined by using the Main Roads Western Australia Empirical Pavement Design Procedure (empirical procedure) detailed in this Note, even when the pavement has asphalt wearing courses. The thickness of the granular pavement layers, determined by using the empirical procedure in this Note, must not be reduced by any amount to compensate for the thickness of any asphalt wearing courses or other bituminous surfacing.

Except that, where asphalt is greater than 60 mm nominal total thickness and the asphalt has a design fatigue life of forty years or more, then the pavement design only has to comply with the mechanistic procedure in this Note.

1.1.4 Adjustment for Granular Materials Construction Tolerances

The granular pavement material layer thicknesses determined from this Note are the design minimum thicknesses. In practice, due to construction tolerances, the actual constructed thickness will vary. To ensure that the constructed thickness will not be less than the design minimum thickness, the thickness of granular base material and total thickness of granular pavement material specified by the pavement designer (i.e. shown in the drawings) must both be at least 10 mm greater than the minimum thicknesses determined from this Note.

1.1.5 Adjustment for Asphalt Construction Tolerances

When the asphalt nominal total thickness is 60 mm or less, the thickness of dense graded asphalt used in the mechanistic procedure must be at least 10 mm greater than the dense graded asphalt nominal thickness.

When the asphalt nominal total thickness is greater than 60 mm, the thickness of asphalt intermediate course used in the mechanistic procedure must be at least 10 mm less than the asphalt intermediate course nominal thickness.

These adjustments are necessary to allow for the adverse effect that a change in the asphalt thickness (i.e. due to the effect of as-constructed level variances) has on the asphalt fatigue life.

1.1.6 Maximum Asphalt Nominal Thicknesses

Unless specified otherwise by the Principal, the asphalt nominal thickness (i.e. thicknesses shown in the drawings) must not exceed the values in Table 2

TABLE 2 Maximum Asphalt Nominal Thickness

Asphalt Wearing Courses			Asphalt Intermediate Course	
10 mm Open Graded Mix	10 mm Dense Graded Mix	14 mm Dense Graded Mix	14 mm Mix	20 mm Mix
30 mm	30 mm	40 mm	70 mm	No Maximum

1.1.7 Asphalt Mix Design

The asphalt mix design must ensure that rutting, ravelling or cracking of the asphalt, including any open graded asphalt wearing course, does not exceed the specified requirements.

1.1.8 Modified Granular Materials and Cemented Materials

The pavement must not incorporate cemented materials.

Cemented materials may be used as a working platform below the design subgrade surface. An unbound granular layer must be placed above the cemented material to provide transverse drainage. In order to limit reflection cracking, a minimum thickness of 230 mm of pavement material must be placed over the top of cemented material. The cemented material working platform must be constructed as a single layer and must not be less than 150 mm or greater than 230 mm in thickness.

No reduction in the pavement thickness can be made in response to using a cemented material working platform. The primary function of a cemented material working platform is to assist in construction. The California Bearing Ratio (CBR) of the cemented material used in the pavement design must not exceed the CBR of the unbound granular material used to manufacture the cemented material.

The pavement must not incorporate any modified granular material that satisfies one or more of the following criteria when tested at its in-service conditions: -

(a) 7-day unconfined compressive strength (UCS) of the material exceeds 1.0 MPa;
(b) 28-day UCS of the material exceeds 1.5 MPa; or
(c) vertical modulus of the material exceeds 1500 MPa.

In addition, where a granular pavement layer is modified by insitu stabilisation, the stabilised material must comply with the Guidance Notes in Specification 501 PAVEMENTS (e.g. the 7-day UCS of cement stabilised pavement layers must be in the range of 0.6MPa – 1.0MPa).

1.2 Mechanistic Procedure

1.2.1 General

In addition to complying with Section 1.1 of this Note, the design of pavements with asphalt courses must also as a minimum comply with the mechanistic procedure detailed in AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2008). The designer must use a project reliability level of not less than 95%.

1.2.2 Design Subgrade California Bearing Ratio

The design subgrade California Bearing Ratio (CBR) used in the mechanistic procedure must be less than or equal to the value determined in accordance with the empirical procedure detailed in this Note. The vertical modulus (in MPa) of the subgrade used in the mechanistic procedure must not exceed the lesser of 10 times the design subgrade CBR or 150 MPa.

1.2.3 Tyre-Pavement Contact Stress

Unless specified otherwise by the Principal, the tyre-pavement contact stress (which is related to the tyre inflation pressure) used in the mechanistic procedure must not be less than 750 kPa.

1.2.4 Laboratory Testing to Verify Design Assumptions

At the time that a mechanistic pavement design is being undertaken, the source of the pavement material may not be known and it may be necessary to assign moduli to these materials. All moduli assigned in the mechanistic procedure must be verified by undertaking laboratory repeated load triaxial testing of the granular materials and indirect tensile testing of the asphalt mixes, once the sources of the materials to be used in the pavement are known. If necessary the proposed pavement materials and/or the proposed pavement design must be amended after these test results become available.

1.2.5 Modulus of Granular Materials

When the asphalt nominal total thickness is 60 mm or less, the vertical moduli assigned to granular materials in the mechanistic procedure must not exceed the “typical” vertical modulus (MPa) presumptive values in Table 6.3 of the AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2008), unless higher material parameters are proven to be applicable by carrying out laboratory repeated load triaxial testing. Table 3 details the stress limits for determining the vertical modulus of granular pavement materials from laboratory repeated load triaxial testing.

TABLE 3
STRESS LIMITS

Minimum Octahedral Shear Stress (kPa)		Maximum Mean Normal Stress (kPa)	
Top Sublayer of Granular Base	Top Sublayer of Granular Subbase	Top Sublayer of Granular Base	Top Sublayer of Granular Subbase
120	30	240	94

In addition, when the asphalt nominal total thickness is 60 mm or less, the vertical modulus used in the mechanistic procedure for the top sublayer of granular base (including for modified granular materials), must not exceed 800 MPa.

When the asphalt nominal total thickness is greater than 60 mm, the vertical modulus used in the mechanistic procedure for granular base or granular subbase material (including for modified granular materials), must not exceed the values in Table 6.4 of the AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2008)..

1.2.6 Modulus of Asphalt

The design asphalt modulus must be calculated from the indirect tensile test asphalt modulus in accordance with AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2008).

The indirect tensile test asphalt modulus assigned to an asphalt mix in the mechanistic procedure must be verified by measuring the modulus of at least 3 laboratory prepared specimens of the asphalt mix to be used, in accordance with AS 2891.13.1 “Determination of the resilient modulus of asphalt - indirect tensile method”. The indirect tensile tests must be undertaken at the Standard Reference Test Conditions in the AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2008) on specimens prepared from the constituents mixed in the laboratory at the design binder content and grading and gyratory compacted to about 5% air voids.

Unless specified otherwise by the Principal, the indirect tensile test asphalt modulus used in the mechanistic procedure to calculate the design asphalt modulus for dense graded asphalt must comply with Table 4 and Table 5 in this Note (i.e. at the Standard Reference Test Conditions in Section 6.5.3 of the AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2008) for laboratory prepared specimens at the design binder content and grading and gyratory compacted to 5% air voids).

TABLE 4 – ALL DENSE GRADED ASPHALT MIXES

Asphalt Nominal Total Thickness 60 mm or less		Asphalt Nominal Total Thickness Greater Than 60 mm	
the indirect tensile test asphalt modulus used must exceed the greater of the following		the indirect tensile test asphalt modulus used must be less than the lower of the following	
The “typical” Australian dense-graded asphalt modulus values in Table 6.12 of the AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2008)	When 3 to 9 tests are undertaken on the mix to be used, the highest individual result obtained; or When 10 or more tests are undertaken on the mix to be used, the value that 85% of test results are lower than	The “typical” Australian dense-graded asphalt modulus values in Table 6.12 of the AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2008)	When 3 to 9 tests are undertaken on the mix to be used, the lowest individual result obtained; or When 10 or more tests are undertaken on the mix to be used, the value that 85% of test results are higher than

TABLE 5 - PERTH DENSE GRADED ASPHALT MIXES (ADDITIONAL REQUIREMENTS)

Asphalt Nominal Total Thickness 60 mm or less	
10 mm Perth dense graded asphalt with Class 170 binder	14 mm Perth intersection mix dense graded asphalt with Class 320 binder
the indirect tensile test asphalt modulus used must exceed 5,000 MPa	the indirect tensile test asphalt modulus used must exceed 5,500 MPa

Unless specified otherwise by the Principal, the design asphalt modulus used for Perth open graded asphalt mixes in the mechanistic procedure must comply with Table 6. Higher values should be used if the WMAPT is less than 29^0C:

TABLE 6 - PERTH OPEN GRADED ASPHALT MIXES

10 mm Perth Open Graded Asphalt Mixes with Class 320 binder		
Asphalt Nominal Total Thickness 60 mm or less		Asphalt Nominal Total Thickness greater than 60 mm
Posted Speed Limit	Design Asphalt Modulus used must be greater than	Design Asphalt Modulus used must be less than
60 km/hr or lower	1500 MPa	800 MPa
70 km/hr	1800 MPa	
80 km/hr	2000 MPa	
90 km/hr	2200 MPa	
100 km/hr or higher	2500 MPa	

1.2.7 Representative Value for the Heavy Vehicle Traffic Speed

Unless specified otherwise by the Principal, the representative value for the heavy vehicle traffic speed used in the mechanistic procedure must comply with Table 7.

TABLE 7 Representative Values for the Heavy Vehicle Traffic Speed

Asphalt Nominal Total Thickness 60 mm or less	Asphalt Nominal Total Thickness greater than 60 mm	
all locations	all roundabouts and intersection controlled by traffic control signals (including approaches and exits). Intersection where a high proportion of the heavy vehicle traffic undertake turning movements	all other locations
not less than 10 kilometres per hour below the posted speed limit	not to exceed 10 km/hr	Not to exceed 10 kilometres per hour below the posted speed limit

1.2.8 Asphalt in Service Air Voids and Binder Content

Unless specified otherwise by the Principal, the in service air voids and the in service binder content used in the mechanistic procedure must comply with Table 8.

TABLE 8
IN SERVICE AIR VOIDS AND BINDER CONTENT

Asphalt layer	Air Voids (by volume)		Binder Content (by volume)
	Asphalt 60 mm Nominal Total Thickness or Less	Asphalt Greater than 60 mm Nominal Total Thickness	
10 mm Class 170 Perth dense graded asphalt	7.8% Maximum	7.8% Minimum	11.8% Maximum
14 mm Class 320 Perth intersection mix	8.3% Maximum	8.3% Minimum	10.3% Maximum
14 mm Class 320 Perth asphalt intermediate course		8.3% Minimum	10.3% Maximum
20 mm Class 320 Perth asphalt intermediate course		7.4% Minimum	10% Maximum
20 mm Class 320 Perth asphalt base course		5.4% Minimum	11.7% Maximum

1.2.9 Asphalt Base Course

Unless specified otherwise by the Principal, Main Roads does not permit the use of asphalt base course in pavements on its network due to concerns that this layer may be impermeable and may inhibit moisture from draining from the asphalt intermediate course.

1.3 **Empirical Procedure**

The remainder of this Note details the Main Roads Western Australia Empirical Pavement Design Procedure (empirical procedure). Except that Section 3 “Design Traffic Loading” is applicable to both the empirical procedure and the mechanistic procedure.

The empirical procedure, determines the minimum pavement layer thicknesses required to contain the permanent deformation and the travel comfort of the pavement to acceptable limits.

The empirical pavement design procedure is limited in its application to road pavements that:

- Carry normal road and highway traffic;
- Are composed of layers of unbound granular material or modified granular material; and
- Are surfaced with a sprayed bituminous seal

The Main Roads Western Australia empirical procedure is based on the empirical procedure detailed in AUSTROADS Guide to Pavement Technology Part 2 – Pavement Structural Design (2008) but incorporates the results of some research and investigations, which relate specifically to Western Australia.

The thickness required to produce a specified level of performance for a pavement will be dependent upon the:

- Environmental conditions controlling the behavior (strength) of the pavement and subgrade materials
- Amount of traffic the road is required to carry; and
- Standard of travel comfort that is desired during the period for which the road is required to perform

The empirical procedure determines the minimum thicknesses of granular pavement layers that are required to support the design traffic loading over the subgrade. The strength of the subgrade is assessed in terms of the California Bearing Ratio (CBR) at the expected critical moisture condition and density. The design traffic loading is measured as the number of equivalent standard axles (ESA’s).

MAIN ROADS WESTERN AUSTRALIA EMPIRICAL PAVEMENT DESIGN PROCEDURE

2. ASSESSMENT OF SUBGRADE STRENGTH

The strength of the subgrade is measured at its critical design condition in terms of its California Bearing Ratio value (CBR value). This value is primarily dependent on the following three variables:

- Type of subgrade material
- Subgrade moisture content; and
- Subgrade density

The design value of the subgrade CBR will also be dependent on the degree of confinement provided by the pavement. However, the effects of confinement vary with the thickness of the pavement, which is in turn controlled by the subgrade material type and its moisture content. Thus, the estimation of the design subgrade CBR for a particular subgrade material requires the selection of a design moisture content, a design density and an appropriate degree of confinement (i.e. surcharge).

2.1 Subgrade Design Units

The length of the route for which the pavement thickness is required to be determined is divided into sections within which the condition and type of subgrade material is essentially constant. These units of constant material type, drainage conditions and in situ density are termed “subgrade design units”.

The formation type affects the design pavement moisture. As a minimum, the types of formation, which should be considered when selecting subgrade design units, are raised formation, cuts and floodways.

It is important to appreciate that although each subgrade design unit has nominally constant material type, there will be a natural degree of variability of its strength properties throughout the unit. It is necessary that this variability be quantified and accommodated when selecting a pavement thickness.

2.2 Subgrade Design Moisture Content

For a normal sealed pavement on a raised formation and for which provision is made for the effective removal of both storm water and ground water, the subgrade moisture will vary with the position across the width of the sealed portion of the pavement. With the exception of two strips about one metre wide inside each edge of the sealed area, the moisture content of the subgrade can be assumed constant. However, the moisture content within the one metre edge strips varies seasonally. Generally the moisture contents within the edge strips are higher on occasions than those of the central area. Thus the critical design moisture content commonly occurs within an area of one metre width inside the edge of the seal. Except for roads with wide sealed shoulders, the outer wheel path falls in this zone.

When an existing sealed road is being reconstructed, the design moisture content must not be less than the moisture content of the subgrade of the existing road, measured at any point within the traffic lanes of the existing road.

Adequate provision must be made for drainage to ensure that moisture cannot be trapped and unable to drain from the pavement. The provision of effective drainage is as important for asphalt pavements as it is for granular pavements. The importance of making an adequate provision for drainage increase when the pavement is constructed below ground level and increases when the permeability of the subgrade decreases.

The subgrade design moisture content (MC_{85}) for a sealed pavement on a raised formation or in cuts provided with adequate drainage works, must not be less than values estimated using the following equations:-

$$\frac{MC_{85}}{LL} = 0.54 - 0.08V \qquad \text{Equation (1)}$$

where MC_{85} = 85th percentile value of the subgrade moisture content under the outer wheel path (% mass of dry soil)

LL = Liquid Limit (%) of the subgrade material

V = Annual Average Evaporation (metres/year) (Figure 1)

Where the road shoulders are sealed for a width of at least one metre and suitably delineated from the trafficked pavement, the subgrade design moisture content may be reduced and must not be less than the value estimated using Equations 2:-

$$\frac{MC_{85}}{LL} = 0.50 - 0.08V \qquad \text{Equation (2)}$$

For well drained clean sand subgrades, the subgrade design moisture content may be assumed to be independent of climatic factors and must not be less than a value estimated using the following equation:

$$\frac{MC_{85}}{OMC} = 0.75 \qquad \text{Equation (3)}$$

OMC = Subgrade Optimum Moisture Content for Modified Compaction (%)

Equations 1 to 3 must not be used in the following areas, even when subsoil drainage or drainage layers are installed to protect the pavement from moisture ingress. In these locations the subgrade evaluation must be based on a soaked CBR test (minimum 4 days soaking):

(a) Subgrade is below the water table or the drainage backwater level;

(b) Subgrade is above the water table or the drainage backwater level, but the subgrade is still within the zone affected by capillary rise;

(c) Floodways and floodway approaches;

(d) Pavement subject to inundation during flooding; and

(e) Cutting with inadequate longitudinal drainage in table drains (slope is less than 1 in 500).

Consideration should also be given to adopting a soaked CBR test where high permeability pavement material in combination with a low permeability subgrade is to be used.

NOTE: Equations 1 to 3 have been derived for Western Australia only and should not be used elsewhere without due consideration of climatic differences.

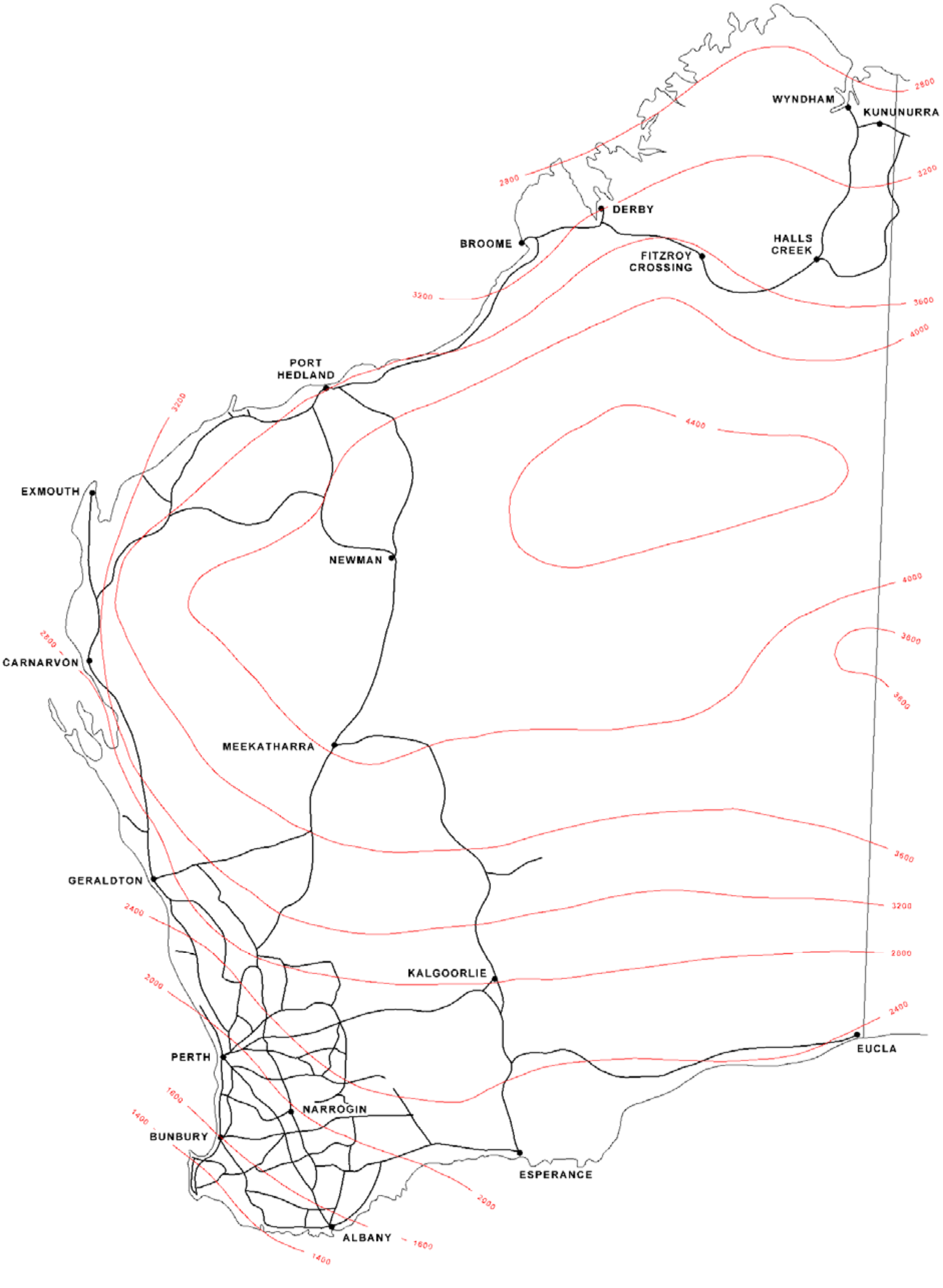

FIGURE 1 ANNUAL AVERAGE EVAPORATION (mm)
(WITH 7% BIRD SCREEN CORRECTION)

Source: Bureau of Meteorology (1975)

2.3 In Situ Density

The density to which a subgrade material is compacted can have a significant effect upon its strength. Variability in density is also a matter of concern, as it will result in differential deformation due to traffic compaction.

Consideration should be given to the depth to which effective compaction can be achieved. In some circumstances the strength of the material below the subgrade layer may be a critical consideration in the design of the total pavement system.

2.4 Method of Determining the Design Subgrade California Bearing Ratio (CBR)

The design subgrade CBR must be determined from laboratory measurement in accordance with MRWA Test Method WA 141.1.

The results of laboratory subgrade CBR strength tests within each design unit will exhibit a natural variability. If the lowest CBR result were chosen as the basis for design, most of the road pavement would be over-designed and could represent an uneconomical investment of capital. A percentile value should be chosen. Desirably, the appropriate percentile should be selected on the basis of a thorough economic analysis taking into account, construction, maintenance and road user costs. Yoder (1969) reported the results of such an analysis. He found that:

- For low volume roads the optimum value approaches the average value
- For high volume roads, the optimum value approaches the minimum value
- For arid climates, the optimum value approaches the average value; and
- For high rainfall climates, the optimum value approaches the minimum value

The design subgrade CBR must not exceed a value calculated using the following expression:

$$\text{Design CBR} = \bar{c} - ks \qquad \text{Equation (4)}$$

where $\bar{c}$ = Mean of all CBR determinations within a single design unit

s = Standard deviation of all CBR determinations within a single design unit

k = A multiplier from Table 9

TABLE 9
k VALUES FOR DESIGN

Traffic (AADT)	Arid and Well Drained	High Rainfall or Poorly Drained
< 100	0.25	0.50
100 - 5000	0.50	0.85
> 5000	0.85	1.0

A minimum of four (4) determinations of subgrade CBR must be made for each design unit. Isolated extremely high values of CBR should not be used for the determination of design CBR using equation (4).

2.4.1 Laboratory Measurement of CBR

The design subgrade CBR must be assessed in the laboratory on specimens prepared at the design moisture content and the characteristic dry density ratio that does not exceed the value specified for the project. It may be difficult to achieve the specified value of density in a single specimen. Specimens may be prepared at slightly under and over the specified density and the CBR at the specified density can be interpolated.

Where drainage conditions are poor, for example floodways, it is necessary to conduct 4-day soaked CBR tests on samples compacted at not less than 100% of OMC.

As a minimum, MRWA Test Method WA 141.1 Determination of the California Bearing Ratio of a Soil: Standard Laboratory Method for a Remoulded Specimen must be used.

When preparing a sample for testing it is necessary to apply a surcharge to the specimen to simulate the overlying pavement layers. The surcharge is to be applied during soaking and testing. The amount of surcharge required expressed in terms of layer thickness and number of 2.25 kg surcharge units must not exceed the values shown in Figure 2.

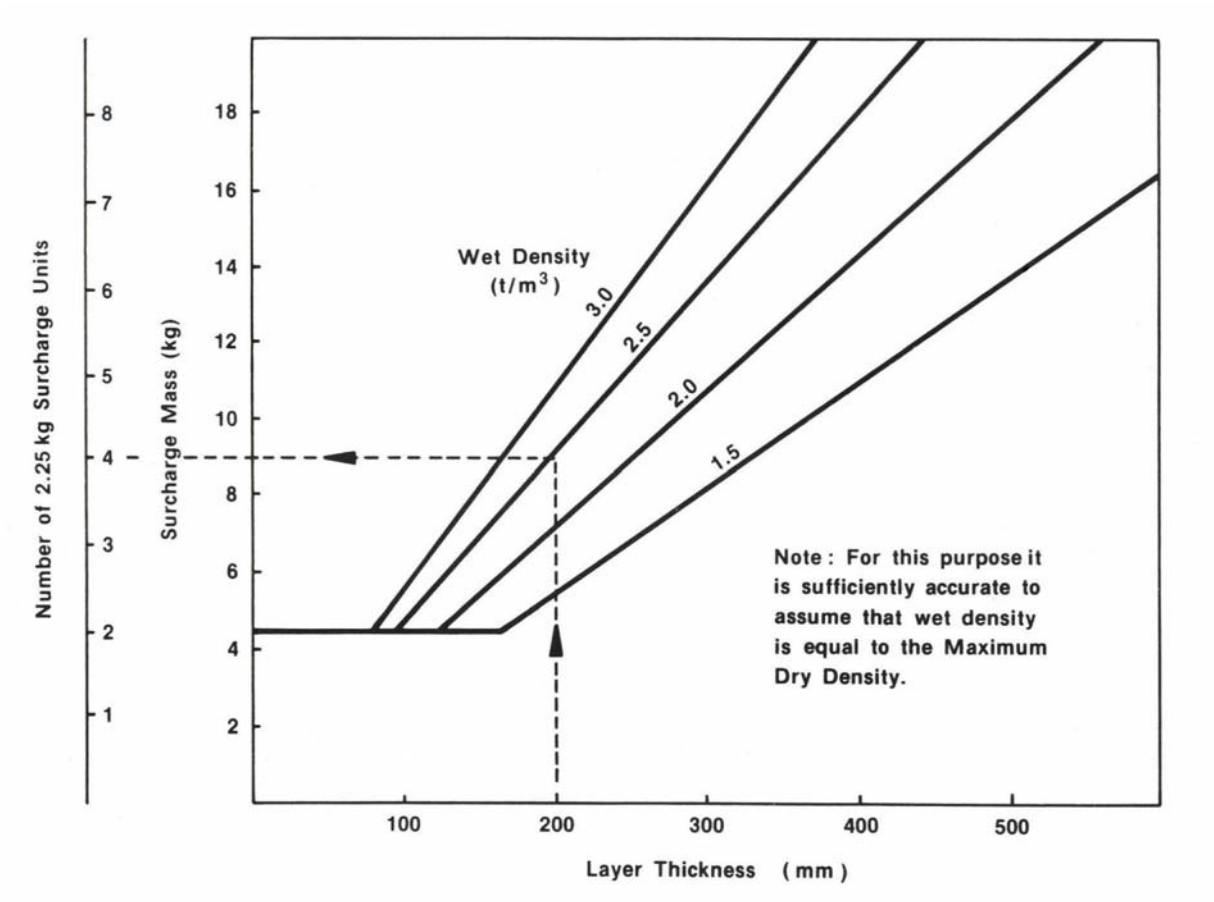

FIGURE 2 LABORATORY CBR: SELECTION OF SURCHARGE

2.4.2 Empirically Correlated In Situ Measurements

2.4.2.1 General

The California Bearing Ratio test is a fairly time consuming and expensive test. Empirical correlations have been established between in situ CBR and some "quick” tests (e.g. dynamic cone penetrometer, Clegg Impact Soil Tester, etc). These “quick” tests provide a means of obtaining a large number of measurements in an economical manner.

<u>These empirical correlations must not be used to design the pavement but may be used during the construction phase to check whether further laboratory CBR testing is required to verify assumptions made about subgrade design units during the design phase (e.g. when an area of weak subgrade is identified during construction). In special circumstances where there is insufficient time to carry out additional laboratory CBR testing, the Principal may permit the dynamic cone penetrometer or another “quick” test to be used during the construction phase to design an increase in the pavement thickness.</u>

2.4.2.2 Dynamic Cone Penetrometer

This test is particularly useful for investigating the variation in subgrade strength with depth. A correlation between dynamic cone penetrometer results and in situ CBR for cohesive materials (Scala 1956) is shown in Figure 3. This correlation is not suitable for use with cohesionless sands. Australian Standard AS 1289 Method 6.3.2: Soil strength and consolidation tests - Determination of the penetration resistance of a soil - 9 kg dynamic cone penetrometer, must be used.

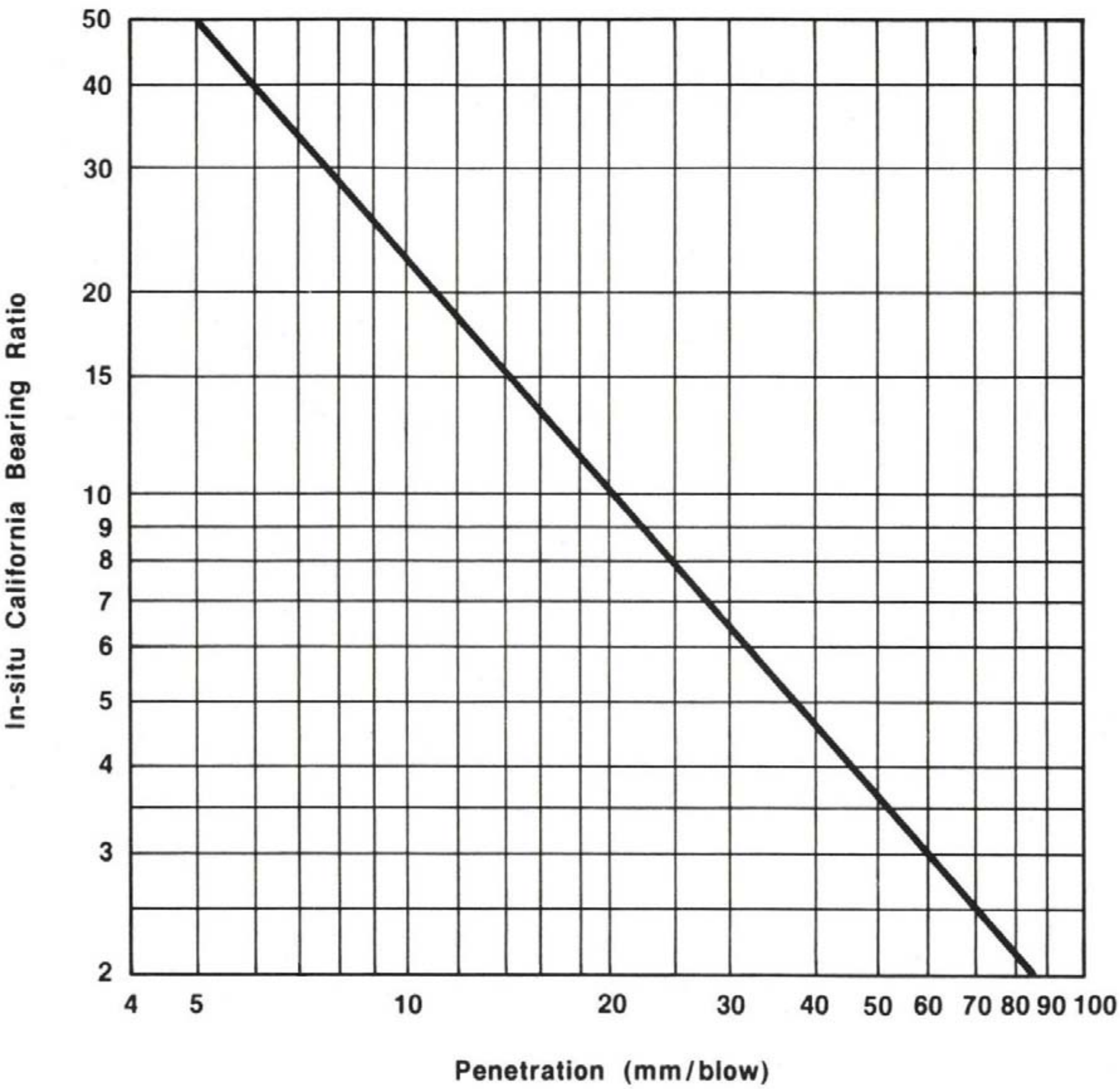

FIGURE 3 CORRELATION OF CONE PENETRATION (mm/blow) AND CALIFORNIA BEARING RATIO

2.4.2.3 Static Cone Penetrometer

Correlations have been carried out by Scala (1956) on a number of different pavement materials to assess the relationship between field CBR and ultimate cone resistance measured by a Static Cone Penetrometer.

A correlation of field CBR (for 2.5 mm penetration) and static cone resistance is shown in Figure 4. Australian Standard AS 1289.Method 6.5.1: Soil strength and consolidation tests – Determination of the static cone penetration resistance of a soil – Field test using a mechanical and electrical cone or friction - cone penetrometer, must be used.

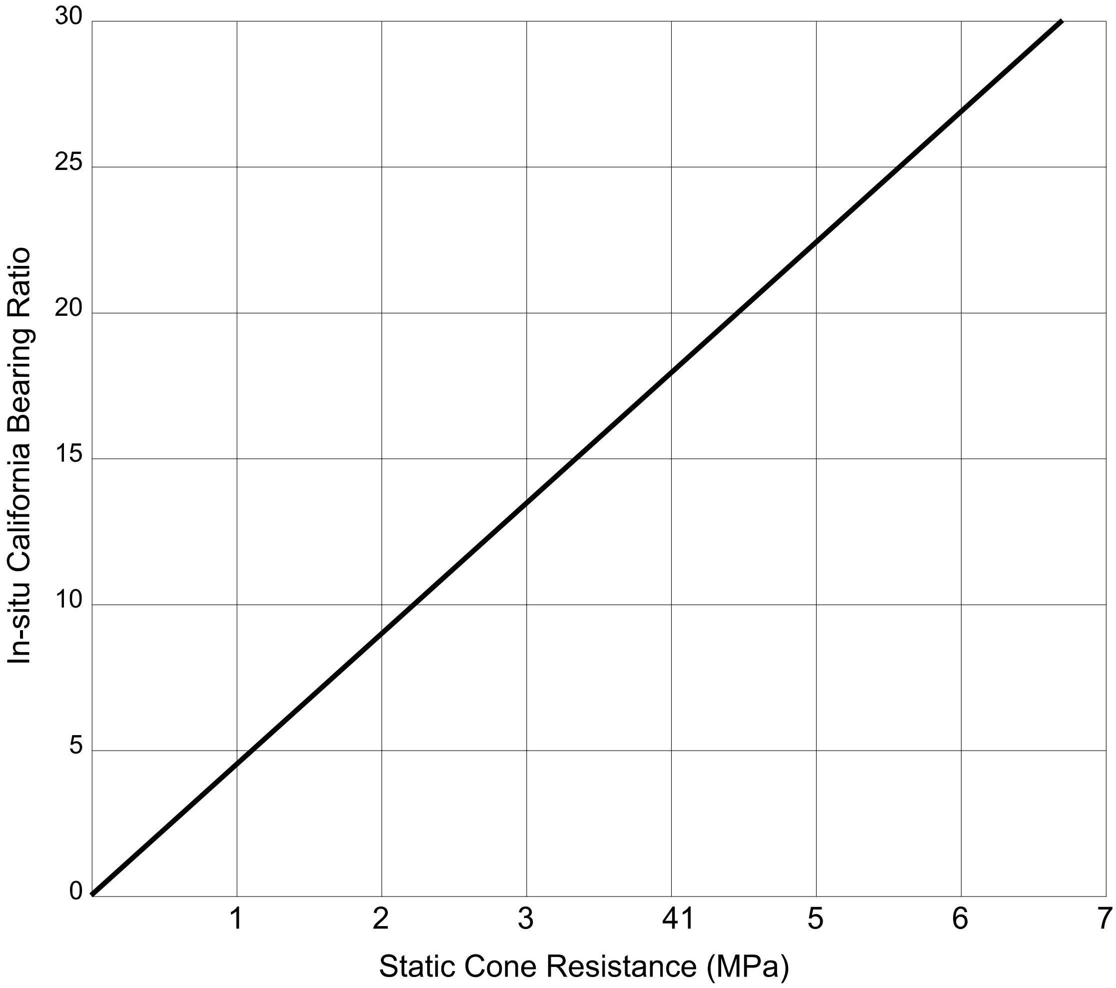

FIGURE 4 CORRELATION OF STATIC CONE RESISTANCE (MPa) AND CALIFORNIA BEARING RATIO

2.4.2.4 Clegg Impact Soil Tester

A correlation between impact value and CBR has been reported by Clegg (1986). The relationship derived is:

$$CBR = 0.06(CIV)^2 + 0.52 CIV + 1 \qquad \text{Equation (5)}$$

where CBR = California Bearing Ratio

CIV = Clegg impact value

The correlation in equation 5 must not be used for cohesionless sands without verification of its appropriateness. AS 1289 Method 6.9.1: Soil strength and consolidation test – Determination of stiffness of soil-Clegg impact value (CIV) must be used.

2.4.2.5 Calculation from Classification Tests

A convenient means of characterising a soil is by the use of classification tests such as particle size distribution, Atterberg Limits and linear shrinkage. Empirical correlations have been established between the CBR value and these parameters for a fine grained soil at a moisture condition represented by four days soaking for a sample compacted to a dry density ratio of 95% of the maximum dry density, at optimum moisture content (OMC) and as determined by MRWA Test Method WA 134.1: Dry Density Ratio (percent) and MRWA Test Method WA133.1: Dry Density/Moisture Content Relationship - Modified Compaction – Fine and Medium Grained Soils. The method is limited to soils having more than 75% passing the 2.36 mm sieve. The soaked CBR value may be estimated from equations 6 and 7 developed by the Country Roads Board of Victoria (1980).

$$\log_{10}CBR = 1.668 - 0.00506\, P_{0.425} + 0.00186\, P_{0.075} - L\left(0.0168 + 0.000385\, P_{0.075}\right) \quad \text{Equation (6)}$$

$$\log_{10}CBR = 1.886 - 0.00372\, P_{2.36} - 0.00450\, P_{0.425} + \frac{P_{0.075}}{P_{0.425}}\left(5.15 - 0.0456\,\frac{P_{0.075}}{P_{0.425}}\right)10^{-3} - 0.0143\, PI$$

Eq(7)

where CBR = Estimate of the soaked value

$P_{2.36}$ = Percentage by mass of particles passing the 2.36 mm sieve

$P_{0.425}$ = Percentage by mass of particles passing the 0.425 mm sieve

$P_{0.075}$ = Percentage by mass of particles passing the 0.075 mm sieve

L = Percentage Linear Shrinkage

PI = Plasticity Index

The values from equations (6) and (7) are combined as follows:

$$CBR_1 = 0.25\left(3\, CBR_{min} + CBR_{max}\right) \quad \text{Equation (8)}$$

where CBR_1 = Calculated estimate of the subgrade CBR at the soaked/standard condition

CBR_{min} = Lesser of the two values calculated by equations (6) and (7)

CBR_{max} = Greater of the two values calculated by equations (6) and (7)

The value calculated from equation 8 (CBR_1) is an estimate of the soaked CBR. For well drained conditions this should be corrected for probable field moisture conditions by multiplying by the rainfall correction factor shown in Figure 6. Rainfall distribution throughout Western Australia is shown in Figure 5.

The “calculated CBR” method should only be used in those regions of the state where experience with local soil and environmental conditions has verified its appropriateness. As a general guide it should not be used for projects located north of latitude 26°S without specific investigation and verification.

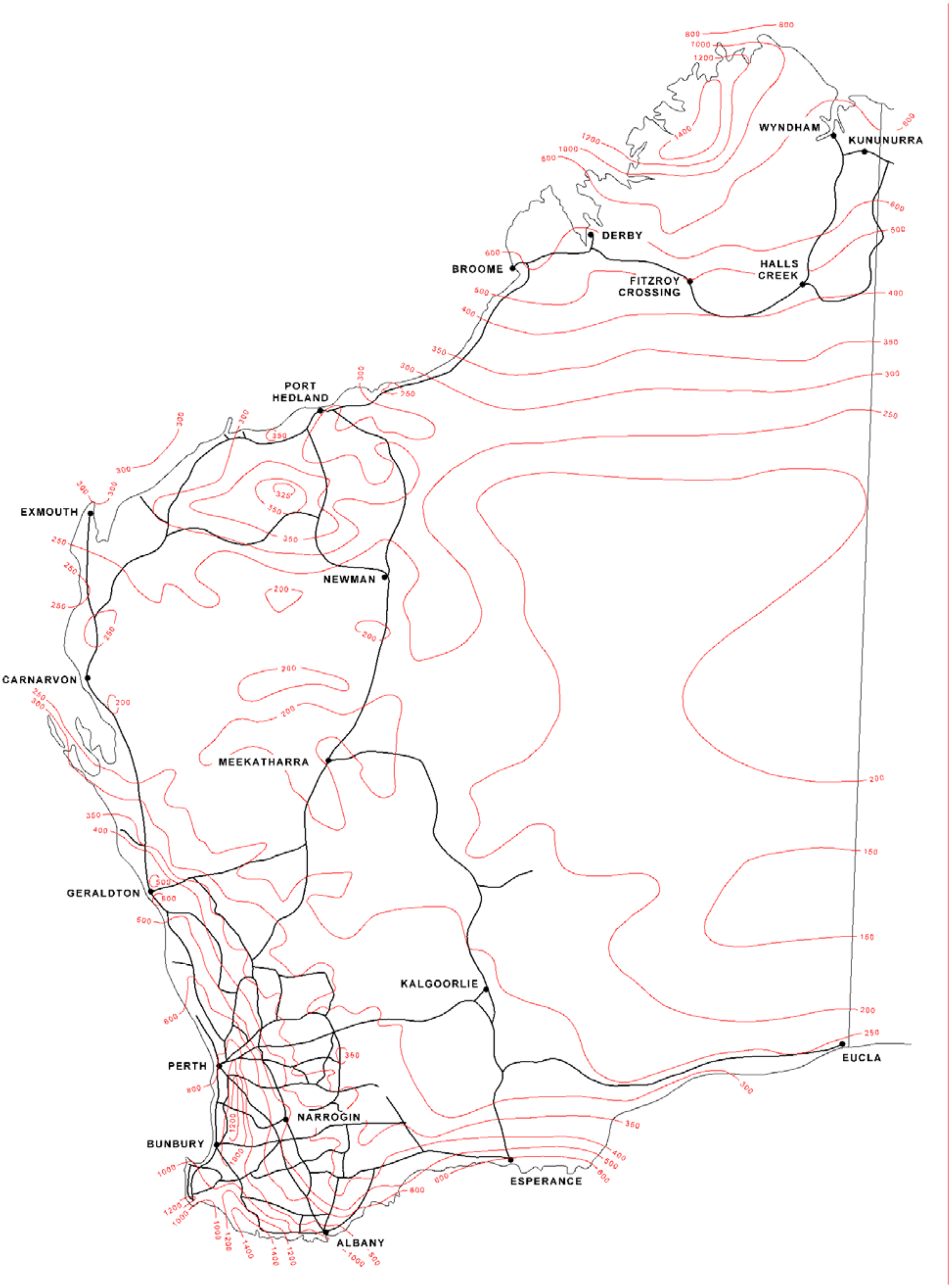

FIGURE 5 AVERAGE ANNUAL RAINFALL (mm)
Source: Bureau of Meteorology (1979)

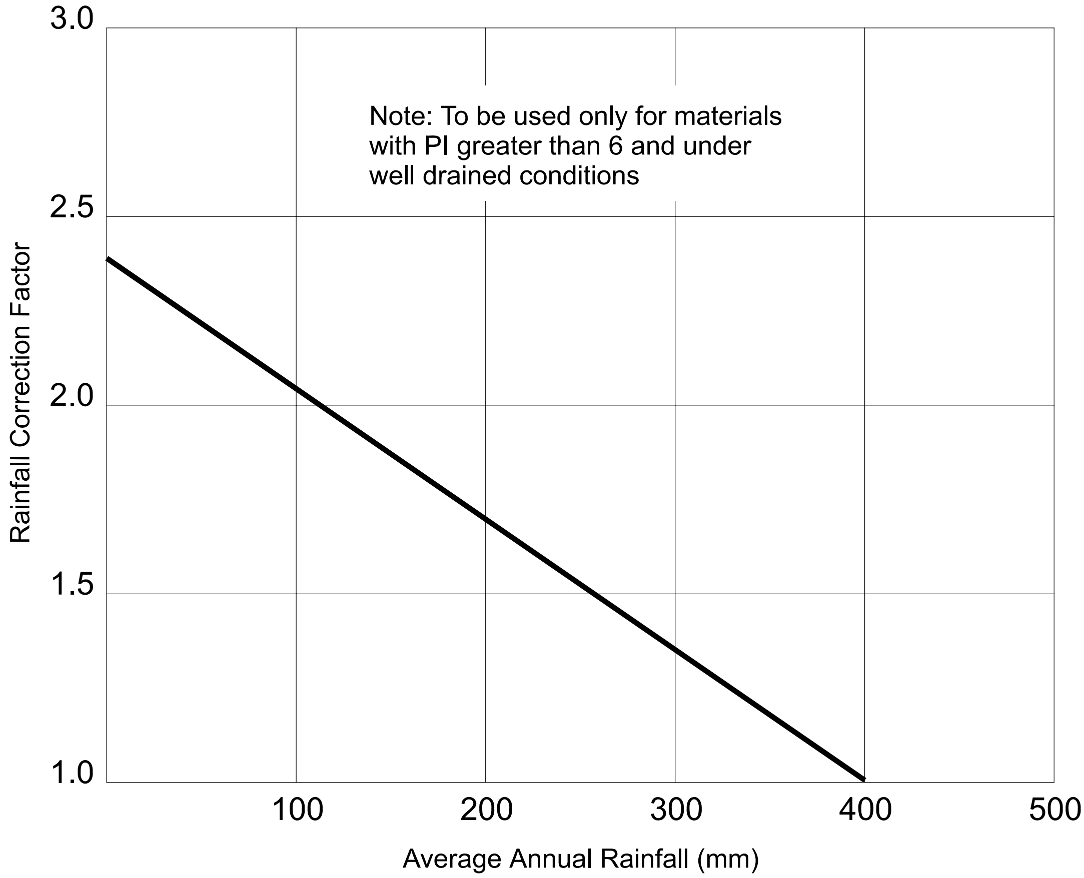

FIGURE 6 RAINFALL CORRECTION FACTOR
(Adapted for W.A Conditions)

3. DESIGN TRAFFIC LOADING

3.1 Number of Equivalent Standard Axles (ESA)

In the empirical procedure, the design traffic loading is expressed as the number of equivalent standard axle load repetitions in the design lane during the design life of the pavement. The standard axle to which all others are related is a dual wheeled single axle applying a load of 80kN.

There are a number of procedures by which this design traffic loading can be estimated depending on the form of the traffic data available. The design traffic loading must not be less than estimated by the following methods.

- Method 1 Traffic data in terms of the heavy vehicles by class;
- Method 2 Traffic data in terms of the proportion of heavy vehicles

Regardless of whether Method 1 or Method 2 is used, the pavement designer must decide whether a road is rural or urban in nature. This decision must be based on the function of the road, rather than its geographic location. Sections of roads that carry a large proportion of heavy vehicles that are going to or are coming from rural locations should be classed as rural, even where those roads pass through urban areas. Major inter regional routes will be affected (i.e. particularly through routes and sections of inter regional roads that pass through the outskirts of urban areas). Examples of roads in the Perth metropolitan area affected by this issue include sections of Kwinana Freeway, Great Eastern Highway and Great Northern Highway.

As the proportion of each vehicle class is used in Method 1, a more accurate estimate of the design traffic loading can be made using Method 1 than can be made by using Method 2.

Method 1 requires that a vehicle classification count be undertaken for the project.

3.2 Design Traffic Loading Method 1 – Traffic Data in Terms of the Heavy Vehicles by Class

As a minimum, this method must be used in estimating the design traffic loading for all significant projects, except where a new link is being constructed and it is not possible to undertake a vehicle classification count because the road does not yet exist.

The data required for this method is:

- Annual average number of vehicles daily in one direction in the first year n
- Each heavy vehicle class as a percentage of the total traffic $c_3, c_4, c_5, c_6, c_7, c_8, c_9, c_{10}, c_{11}, c_{12}$ (%)
- Annual heavy vehicle growth rate[1] expressed as a ratio (e.g. 0.03 for 3%) r
- Percentage of heavy vehicles using the design lane (refer to Section 3.8) d (%)
- Number of equivalent standard axles per vehicle for each heavy vehicle class – Axle Equivalency Factors (refer to Section 3.6) $F_3, F_4, F_5, F_6, F_7, F_8, F_9, F_{10}, F_{11}, F_{12}$
- Pavement design life in years P
- Cumulative growth factor (refer to Section 3.4) R

In this method the design traffic loading, expressed as the number of equivalent standard axle repetitions (N) is given by:

$$N = 365\, n \cdot d \cdot R\left(c_3F_3 + c_4F_4 + c_5F_5 + c_6F_6 + c_7F_7 + c_8F_8 + c_9F_9 + c_{10}F_{10} + c_{11}F_{11} + c_{12}F_{12}\right) \times 10^{-4}$$

Equation (9)

[1] If the heavy vehicle growth rate is not available, the growth rate for all traffic could be used as an approximate estimate of the heavy vehicle growth rate.

3.3 Design Traffic Loading Method 2 – Traffic Data in Terms of the Proportion of Heavy Vehicles

This method may only be used for minor projects where vehicle classification count data is not available or where a new link is being constructed and it is not possible to undertake a vehicle classification count because the road does not yet exist.

The required data for this procedure is as follows:

- Annual average number of vehicles daily in one direction in the first year — n
- Heavy vehicles as a percentage of the total traffic — c (%)
- Annual heavy vehicle growth rate[1] expressed as a ratio (e.g. 0.03 for 3%) — r
- Percentage of heavy vehicles using the design lane (refer to Section 3.8) Design Lane Distribution Factor — d (%)
- Number of equivalent standard axles per heavy vehicle – Axle Equivalency Factor (refer to Section 3.6) — F
- Pavement design life — P (years)
- Cumulative growth factor (refer to Section 3.4) — R

In this method the design traffic loading, expressed as the number of equivalent standard axle repetitions (N), is given by:

$$N = 365n \cdot d \cdot R \cdot c \cdot F \times 10^{-4} \qquad \text{Equation (10)}$$

(1) If the heavy vehicle growth rate is not available, the growth rate for all traffic could be used as an approximate estimate of the heavy vehicle growth rate.

3.4 Cumulative Growth Factor R

If the annual heavy vehicle growth rate r is constant during the design life P years, the cumulative growth factor R can be calculated from Equation (11). It can also be read from Figure 7.

$$R = \frac{(1+r)^P - 1}{r}, \quad r \neq 0 \qquad \text{Equation (11)}$$

When a change in the annual heavy vehicle growth rate is expected during the design life, with the annual growth rate being r_1 for the first Q years and then r_2 for the remainder of the design life P years, then the cumulative growth factor R can be determined from Equation (12).

$$R = \frac{(1+r_1)^Q - 1}{r_1} + (1+r_1)^{Q-1}(1+r_2)\left[\frac{(1+r_2)^{P-Q} - 1}{r_2}\right], \quad r_1, r_2 \neq 0 \qquad \text{Equation (12)}$$

When the annual heavy vehicle growth rate is r_1 for the first Q years and then the annual heavy vehicle growth rate is equal to zero for the remainder of the design life P years, the cumulative growth rate R can be calculated from Equation (13).

$$R = \frac{(1+r_1)^Q - 1}{r_1} + (P-Q)(1+r_1)^{Q-1}, \quad r_1 \neq 0$$

Equation (13)

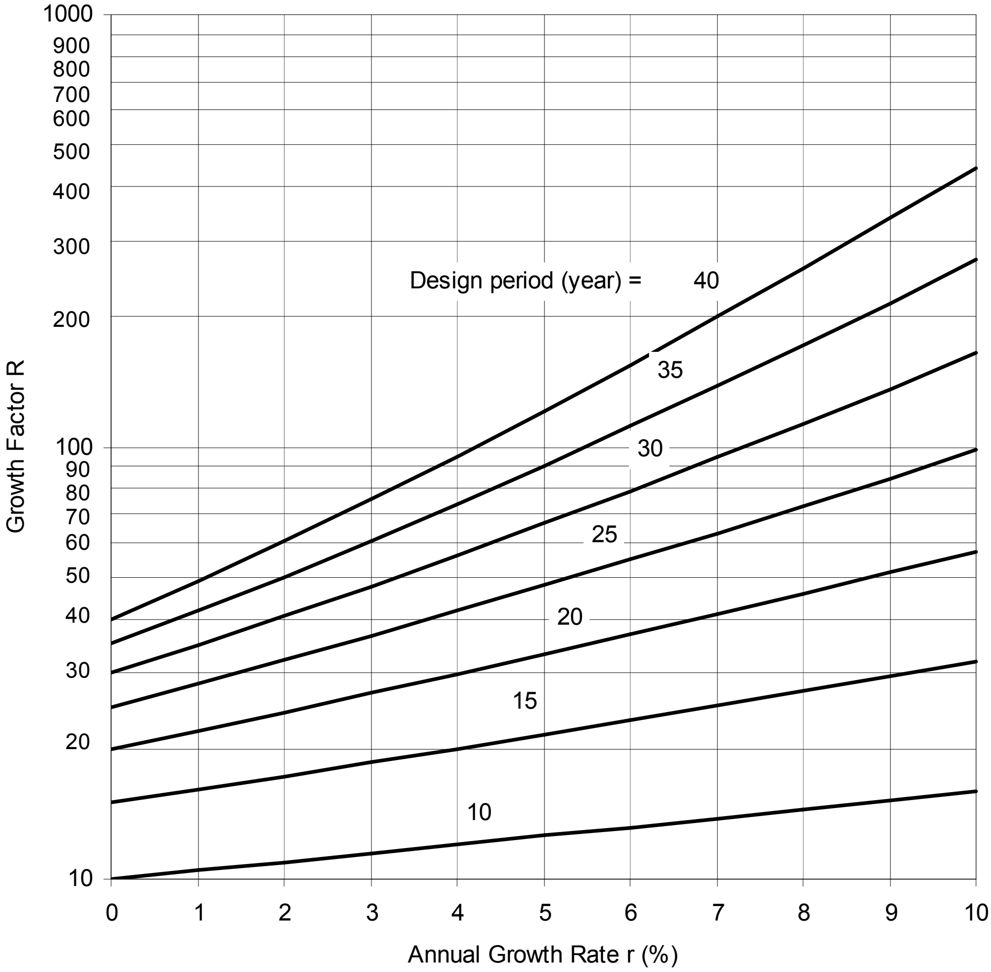

FIGURE 7 DETERMINATION OF GROWTH FACTOR FROM ANNUAL GROWTH RATE AND DESIGN LIFE

3.5 Heavy Vehicle

A heavy vehicle is defined in accordance with the Austroads Vehicle Classification System detailed in AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2008) (i.e. Class 3 to Class 12 inclusive).

3.6 Axle Equivalency Factors (F)

Axle equivalency factors are derived from traffic data at CULWAY sites in Western Australia collected between 2003 and 2007.

3.6.1 Method 1 Axle Equivalency Factors

Axle equivalency factors for Method 1 are provided in Table 10 by Main Roads WA road classification.

Axle equivalency factors at 12 CULWAY sites in WA are shown in Table 11. These values must only be used when the sections being designed are in the close vicinity of these CULWAY sites.

TABLE 10 AXLE EQUIVALENCY FACTORS FOR **GENERAL** USE IN DESIGN TRAFFIC LOADING METHOD 1

Main Roads WA Road Classification*	Axle Equivalency Factors by Vehicle Class									
	F_3	F_4	F_5	F_6	F_7	F_8	F_9	F_{10}	F_{11}	F_{12}
Rural National Highways	0.34	2.00	1.85	0.51	0.86	2.31	3.16	4.04	5.14	8.04
Rural Highways	0.48	2.45	2.02	0.80	1.33	2.61	3.31	5.22	8.39	8.53
Rural Main and Secondary Roads	0.20	0.85	2.09	0.15	0.34	1.92	2.21	3.60	5.09	6.56
Urban Freeways & Highways	0.47	1.44	1.77	0.73	0.91	1.75	2.31	3.42	4.33	8.00
Other Important Urban Arterial Roads	0.47	1.44	1.77	0.73	0.91	1.75	2.31	3.42	4.33	8.00

TABLE 11 AXLE EQUIVALENCY FACTORS AT CULWAY SITES IN WA FOR **SPECIFIC** USE IN DESIGN TRAFFIC LOADING METHOD 1

CULWAY Site	Axle Equivalency Factors by Vehicle Class									
	F_3	F_4	F_5	F_6	F_7	F_8	F_9	F_{10}	F_{11}	F_{12}
H018 Roe Highway, East of Karel Av, Perth	0.47	1.44	1.77	0.73	0.91	1.75	2.31	3.42	4.33	8.00
H006 Great Northern Highway SLK 35, Muchea	0.38	2.24	1.91	0.56	0.88	2.36	3.24	4.08	5.31	8.04
H006 Great Northern Highway SLK 1163.55, Newman	0.30	1.46	1.40	0.50	0.48	2.18	2.97	3.95	4.73	8.04
H003 Eyre Highway SLK 8.08, Norseman	0.16	0.94	1.34	0.38	0.65	1.23	2.31	2.88	3.74	8.04
H005 Great Eastern Highway SLK 102.66, Northam	0.26	1.37	1.71	0.41	0.79	1.57	2.21	3.19	3.89	8.04
H007 North West Coastal Highway SLK 760.4, Nanutarra	0.37	2.12	1.00	0.26	1.12	2.47	3.42	4.00	4.70	8.53
H009 South Western Highway SLK 79.29, Waroona	0.38	1.78	1.99	0.80	0.76	2.31	2.59	3.43	4.13	8.53
H009 South Western Highway SLK 204.79, Kirup	0.48	1.83	1.77	0.57	0.80	2.25	3.21	5.86	8.10	8.53
H001 Albany Highway SLK 102, Bannister	0.48	2.78	1.72	0.80	1.53	2.66	3.04	4.58	6.12	8.53
H052 Brookton Highway SLK129, Brookton	0.39	1.46	1.84	0.58	0.66	2.57	2.66	4.41	7.33	8.53
H008 South Coast Highway SLK 468.4, Esperance	0.28	2.00	2.05	0.30	0.84	1.99	2.89	4.12	8.68	8.53
H050 Geraldton-Mt Magnet Road SLK 8.43, Geraldton	0.20	0.85	2.09	0.10	0.34	1.86	2.21	3.60	5.09	6.56

*Section 3.1 claries that the pavement designer must consider the function of the road not just its geographic location.

3.6.2 Method 2 Axle Equivalency Factor

The axle equivalency factors for use in design traffic loading Method 2 are provided in Table 12.

The axle equivalency factor at CULWAY sites in WA for use in the design traffic loading Method 2 are shown in Table 13. These values must only be used when the sections being designed are in the vicinity of these CULWAY sites.

TABLE 12 AXLE EQUIVALENCY FACTOR FOR **GENERAL** USE IN DESIGN TRAFFIC LOADING METHOD 2

Main Roads WA Road Classification*	Axle Equivalency Factor
Rural National Highways	3.02
Rural Highways	3.74
Rural Main and Secondary Roads	2.32
Urban Freeways & Highways	1.51
Other Important Urban Arterial Roads	1.51

TABLE 13 AXLE EQUIVALENCY FACTORS AT CULWAY SITES IN WA FOR **SPECIFIC** USE IN DESIGN TRAFFIC LOADING METHOD 2

CULWAY Site	Axle Equivalency Factor
H018 Roe Highway, East of Karel Av, Perth	1.51
H006 Great Northern Highway SLK 35, Muchea	3.19
H006 Great Northern Highway SLK 1163.55, Newman	1.62
H003 Eyre Highway SLK 8.08, Norseman	2.63
H005 Great Eastern Highway SLK 102.66, Northam	2.31
H007 North West Coastal Highway SLK 760.4, Nanutarra	3.28
H009 South Western Highway SLK 79.29, Waroona	2.07
H009 South Western Highway SLK 204.79, Kirup	4.00
H001 Albany Highway SLK 102, Bannister	3.19
H052 Brookton Highway SLK129, Brookton	3.47
H008 South Coast Highway SLK 468.4, Esperance	2.04
H050 Geraldton-Mt Magnet Road SLK 8.43, Geraldton	2.32

*Section 3.1 claries that the pavement designer must consider the function of the road not just its geographic location.

3.7 Design Life (P)

Unless specified otherwise by the Principal, the permanent deformation of the pavement must have a minimum design life of 40 years.

3.8 Design Lane Distribution Factor (d)

As a minimum, Table 14 must be used to determine the percentage of heavy vehicles using the design lane. If project-specific information is available that indicates that a higher factor should be used, then that value must be used.

TABLE 14
MINIMUM LANE DISTRIBUTION FACTORS (d)

Location	Lanes each direction	Minimum Design Lane Distribution Factor for Design lane d (%)
Rural	1 lane	100
Rural	2 lane	90
Rural	3 lane	70
Urban	1 lane	100
Urban	2 lane	80
Urban	3 lane	65

3.9 Roundabouts and other Small Radius Curves

At roundabouts and other small radius curves load transfer on turning vehicles increases the damaging effect of the traffic loading. To compensate for these effects, as a minimum, the design traffic loading in ESA's must be multiplied by a factor of 3 at these locations.

3.10 Ratios of SAR to ESA

When the mechanistic procedure is used for designing a flexible pavement, the pavement is analysed to determine the allowable number of Standard Axle Repetitions (SAR's) for each of the relevant damage types, which are listed in Table 15 (refer to Section 7.6.2 in AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2008)).

The design traffic loading in terms of ESA's needs to be expressed as a design traffic loading in terms of SAR's for each damage type.

The SAR/ESA ratio for each damage type must not be less than the estimated values in Table 15. The estimated values in Table 15 have been derived from traffic load distribution data collected using weigh-in-motion facilities. The design traffic loading in SAR's for each damage type must not be less than calculated by multiplying the design traffic loading in ESA's by the ratios of SAR/ESA in Table15.

TABLE 15 MINIMUM RATIOS OF SAR TO ESA FOR THREE DAMAGE TYPES

Pavement type	Damage type	Damage Index	Minimum Ratio of SAR to ESA
Flexible pavements	Fatigue of asphalt	SAR5/ESA	1.1
	Rutting and shape loss	SAR7/ESA	1.6
	Fatigue of cemented materials	SAR12/ESA	12

4.0 PAVEMENT THICKNESS (EMPIRICAL PROCEDURE)

4.1 General

In the empirical procedure the minimum thickness of granular pavement materials required over the design subgrade must not be less than the upper value determined from Figure 8 and from the following equation:

$$t = \left[219 - 211(\log CBR) + 58(\log CBR)^2\right]\log\frac{N}{120} \qquad \text{Equation (14)}$$

where t = Minimum thickness in millimetres

N = Design traffic loading in equivalent standard axles (ESA's)

CBR = Design subgrade CBR (equation 4)

4.2 Pavement Composition

The total minimum thickness of granular pavement materials required above the design subgrade in the empirical procedure may be made up of a granular base and any number of granular subbase layers.

The soaked design CBR value of the subbase must be greater than 30.

Except that in some situations the Principal may expressly give approval for a sand drainage subbase layer, with a soaked design CBR value of less than 30, to be installed above the design subgrade. The total minimum thickness of granular pavement material required over a sand drainage layer must not be less than the upper value determined from Figure 8 and from equation 14 (i.e. using the soaked design CBR of the sand drainage layer instead of the design subgrade CBR).

It is necessary to provide a minimum thickness of a granular base material with a soaked CBR of 80 or above over the subbase. This minimum granular base thickness must not be less than shown in Figure 8.

For soaked CBR tests the period of soaking must not be less than 4 days.

Equation 14 must also be used to ensure that an adequate thickness of granular materials is provided over any layer of weaker material below the design subgrade. A change of strength below the design subgrade may result from a change in the quality of the material, the density of the material and/or the in-service moisture condition.

No reduction in thickness requirements can be made for pavements incorporating granular material modified with cement, lime, bitumen or other similar materials.

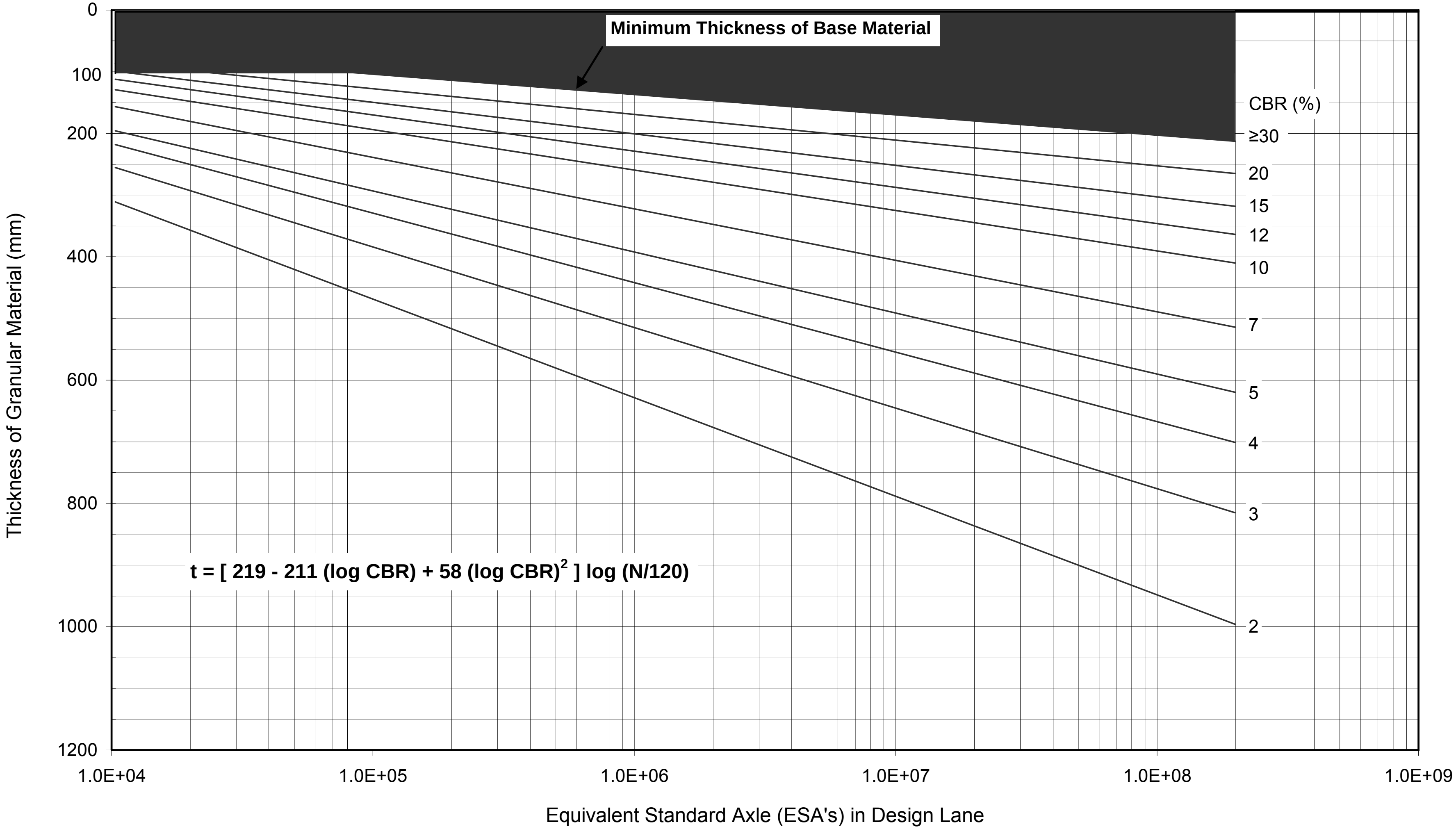

FIGURE 8 CBR DESIGN CHART

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REVISION STATUS RECORD OF THIS ISSUE

Page No.	Section	Revision Description / Reference
2 - 7	1.1 to 1.2	Added sub headings
3	1.1.5	Adjustment for asphalt construction tolerance
3	1.1.6	Included 14 mm asphalt intermediate course
4	1.2.5	Stress limits for repeated load triaxial testing
4	1.2.5	Reference to Tables 6.3 and 6.4 of the Austroads guide
5	1.2.6	Reference to section 6.5.3 of the Austroads guide
5	1.2.6	Reference to Tables 6.12 of the Austroads guide
5	1.2.6	Perth asphalt modulus reduced from 6000 MPa (Table 5)
7	1.2.8	Included 14 mm asphalt intermediate course
28	4.2	Included sand drainage layer