Engineering Road Note 9

May 2013

PROCEDURE FOR THE DESIGN OF ROAD PAVEMENTS

Western Australian Supplement to the Austroads Guide to Pavement Technology Part 2: Pavement Structural Design

MAJOR REVISION STATUS RECORD OF THIS ISSUE

Page No.	Section	Major Revision Description / Reference
1	Cover	ERN9 is supplement to Austroads Guide
3 - 5	Contents	New Table of Contents and section numbering
6	1.1	ERN9 only be used for pavements with a bituminous or concrete wearing course
		Restrictions placed on the arrangements that can be entered into by the party undertaking the pavement design
		Removed the asphalt wearing course, asphalt intermediate course and asphalt base course definitions
7	1.2	Asphalt fatigue life reduced to 5 years under specific circumstances
8	1.4	HCTCRB must only be used in pavements where Main Roads carries the design risk
8	1.5	When concrete pavement is required in floodways, continuously reinforced concrete must be used
8	1.6	New clause on widening existing pavements
9	1.12	Details the circumstances where it will be necessary to increase the pavement thickness when using PMB
10	1.15	New clause on improved subgrade
11	1.17	Requirements on slope and exit points for sub-soil drainage
12	2.5	Requirement for flexural beam testing of cemented materials
13	2.7	Direction provided about how to select the appropriate combination of particle size distribution and binder content to undertake indirect tensile tests on asphalt
24	3.7.1	Empirical CBR correlations may be used to increase the pavement thickness but must not be used to decrease the pavement thickness
27	3.7.5	CBR calculated from classification tests limited to soils having PI greater than 6
		Removal of the section containing the correction factor for calculating the CBR from classification tests for well drained conditions
31	3.8.4	In some circumstances it may be more appropriate to use linear traffic growth rather than compound traffic growth
37	3.9.1	Requirement to repeat CBR testing under certain conditions
		Upper limits on design subgrade CBR
39	4	New location for section on design traffic loading for rigid pavements
33 - 55	Table 11 – Table A13	Nanutarra weigh in motion site removed from tables

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1 INTRODUCTION

1.1 Responsibilities of the Pavement Designer

Engineering Road Note 9 is the Western Australian supplement to the Austroads Guide to Pavement Technology Part 2: Pavement Structural Design.

Engineering Road Note 9 outlines the procedure to be used for the design of road pavements that have a bituminous or concrete wearing course and are under the control of the Commissioner of Main Roads, Western Australia.

The design, construction and maintenance of road pavements involve the management of risks.

Complying with the requirements detailed in the Austroads guidelines, Main Roads guidelines (including Engineering Road Note 9) or Main Roads construction specifications may not ensure that the performance of the pavement will meet all obligations.

Pavement designs must 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 in Engineering Road Note 9); and

(c) specified in the Main Roads construction specifications.

In addition, the party undertaking the pavement design must not enter into arrangements that:

(a) limit an experienced pavement design practitioner from using their skill, knowledge and experience to determine when it is necessary to exceed the above guidelines and specifications; and
(b) impose damages when an experienced pavement design practitioner uses their skill, knowledge and experience to exceed the requirements in the above guidelines and specifications.

DEFINITIONS

Where Engineering Road Note 9 refers to "**nominal thickness**" or "**nominal total thickness**" it is referring to the thicknesses shown in the drawings (or otherwise specified if there are no drawings).

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

1.2 Minimum Design Life

Unless specified otherwise by the Principal: -

(a) the permanent deformation of flexible pavements must have a minimum design life of 40 years;

(b) concrete pavement must have a minimum design life of 40 years for fatigue and erosion damage; and

(c) the asphalt fatigue design life must greater than or equal to the values in Table 1. Except that, the asphalt fatigue design life can be reduced to 5 years when; the asphalt nominal total thickness is 60 mm or less, the design traffic loading for 40 years is less than or equal to $3x10^7$ ESA, the pavement is well drained, the subgrade is Perth sand, the sub-base is crushed limestone and the basecourse material is either crushed rock base or bitumen stabilised limestone.

Table 1. Minimum Asphalt Design Fatigue Life

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

1.3 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.

1.4 Flexible Pavement Design Procedure

As a minimum, the thickness of the granular basecourse 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.

In a pavement with a bituminous surfacing thickness of 60 mm or less, designed using the mechanistic procedure, the top three sub-layers of the granular pavement must be constructed from basecourse material. Except that, when the thickness of the top three sub-layers exceeds 250 mm, the basecourse nominal thickness can be reduced to 250 mm.

Except that, when pavement materials have been modified with cement, bitumen and/or lime, the stiffness of these materials may continue to increase for 12 months or longer after construction (e.g. hydrated cement treated crushed rock base, crushed recycled concrete and bitumen stabilised materials). The design of pavements using these materials must consider the risk that these materials may fail prematurely in fatigue and that the basecourse thickness may need to exceed 250 mm to manage this risk.

Hydrated cement treated crushed rock base (HCTCRB) must be designed to have a minimum fatigue life of 40 years. Until Main Roads has verified that its current design approach for fatigue of HCTCRB is adequate and shared that information with industry, HCTCRB must only be used in pavements where Main Roads carries the design risk (e.g. construct only contracts).

1.5 Concrete Pavement Design Procedures

Concrete pavements must be designed using the procedure for rigid road pavements in AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2012). Concrete pavements must also satisfy the requirements set out in RMS QA Specification R83 and R84, RMS R83-R84 User Guide and RMS standard concrete pavement detail drawings. Concrete roundabouts must be constructed from steel fibre reinforced concrete and must also satisfy the requirements set out in the RMS guide “Concrete Roundabout Pavements – A Guide to their Design and Construction”.

Refer to Section 4 of Engineering Road Note 9 when calculating the design traffic loading for rigid pavements.

When it is decided to use a concrete pavement in a floodway because the risk of scouring of the bituminous surfacing is too high (e.g. the water flow velocity over a floodway exceeds 2.4 m/sec) a continuously reinforced concrete pavements must be used.

Concrete pavements must incorporate a lean mix concrete sub-base. Lean mix concrete sub-base must satisfy the requirements set out in RMS QA Specification R82 and RMS R82 User Guide.

The designer must use a project reliability level of not less than 95%.

1.6 Widening an Existing Pavement

Unless specified otherwise by the Principal, longitudinal joins between an existing pavement and a new pavement must be located on a traffic lane edge line or in the middle of a traffic lane.

1.7 Adjustment for Granular Materials Construction Tolerances

The granular pavement material layer thicknesses determined from Engineering Road Note 9 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 nominal thickness of granular basecourse material and nominal 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 Engineering Road Note 9.

1.8 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.

1.9 Adjustment for Concrete Construction Tolerances

The concrete layer thicknesses determined from Engineering Road Note 9 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 and to allow for future maintenance profiling to the surface, the nominal thickness of concrete specified by the pavement designer (i.e. shown in the drawings) must be at least 20 mm greater than the minimum thicknesses determined from Engineering Road Note 9.

1.10 Asphalt Nominal Thicknesses

Unless specified otherwise by the Principal, the asphalt nominal thickness (i.e. thicknesses shown in the drawings) must comply with Table 2

Table 2. Asphalt Nominal Total 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
30mm	30mm	40mm	50mm	No maximum nominal total thickness. Maximum layer thickness is 100mm. Minimum layer thickness 60mm

1.11 Asphalt Mix Design

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

1.12 Polymer Modified Binders

Polymer modified binders must be used when necessary to reduce the risk of premature cracking, rutting and/or fatigue of asphalt wearing courses and structural layers.

As a minimum the top 100 mm of dense graded asphalt in full depth asphalt pavements, must use a polymer modified binder (e.g. A15E). The depth of polymer modified binder in a full depth asphalt pavement may need to be increased at signalized intersections on heavy vehicle routes and at other high stress locations (e.g. roundabouts).

Whilst polymer modified binders must be used to manage the risk of premature shear cracking, fatigue cracking and/or rutting, the pavement thickness must not be less than would be determined had the unmodified binder been used, unless approved otherwise by the Principal. It will be necessary to increase the pavement thickness when using a polymer modified binder that adversely affects pavement thickness (e.g. when A15E is used in a full depth asphalt pavement).

1.13 Cemented Materials

With the following exceptions, flexible pavements must not incorporate cemented materials: -

(a) Insitu cement stabilised crushed rock base (2% by mass of Type LH cement) or crushed recycled concrete may be placed as sub-base under a full depth asphalt pavement or under a hydrated cement treated crushed rock base basecourse. The cement stabilised crushed rock base used in the sub-base must comply with Main Roads specification for crushed rock base basecourse. The vertical modulus used in the mechanistic procedure for the cement stabilised crushed rock base sub-base or crushed recycled concrete sub-base, must not exceed 500 MPa; and

(b) Cemented materials may be used as a working platform below the design subgrade surface. 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.

In order to limit reflection cracking, a minimum nominal thickness of 230 mm of granular pavement material or 175 mm of dense graded asphalt must be placed over the top of cemented material.

1.14 Modified Granular Materials

The pavement must not incorporate modified granular material where the 7-day unconfined compressive strength (UCS) of the material exceeds 1.0 MPa, when tested at its in-service density in accordance with Test Method WA 143.1 or Test Method WA 143.2.

In addition, where a granular pavement layer is modified by in-situ 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).

1.15 Improved Subgrade

When the naturally occurring material in a cutting or the material used to construct an embankment has a low CBR, consideration should be given to using a layer of imported better quality material below the pavement to reduce the cost of constructing the pavement.

The thickness of the imported layer must not be less than 150 mm. The combined thickness of the imported layer plus the thickness of the pavement must be adequate to provide the required cover over the underlying low CBR material.

A design subgrade CBR value greater than 15% should generally not be used, even when a layer of selected material with a high soaked CBR has been imported and placed below the subgrade surface.

1.16 Minimum Cover over Reactive Material

Sufficient cover thickness of non-reactive material over a reactive material can assist in limiting the amount of shape loss evident at the pavement surface. The required thickness of non-reactive material may include the fill, improved subgrade, working platform and drainage layer.

The cover thickness between the reactive material and the subgrade surface must not be less than detailed in Table 3 for all pavements where the untreated reactive material has a swell greater than or equal to 0.5%. The swell must be measured in accordance with Test Method WA141.1 on a specimen soaked for a minimum of 4 days:

Table 3. Minimum Cover over Reactive Material

Untreated Material Swell (%)	Minimum Cover Over Reactive Material (mm)
>5.0	1000
>2.5 to ≤ 5.0	600
≥0.5 to ≤ 2.5	150

A geotechnical assessment must be carried out when the swell exceeds 7.0%. The specified requirements of the geotechnical assessment must be applied in addition to the minimum requirements specified in Table 3. A geotechnical assessment means assessment and advice from a geotechnical engineer. Their assessment is likely to include shallow boreholes, with continuous undisturbed sampling, to allow the extents of expansive material, shrink swell index testing, moisture content variations, suction testing and x-ray diffraction testing to be determined. A review of the maintenance history and condition of existing pavements and structures should also be undertaken.

1.17 Sub-Soil Drainage

A drainage layer not less than 300 mm in depth must be constructed above the subgrade surface for granular pavements in cuttings under the following circumstances: -

a) The subgrade surface is below the water table;
b) The subgrade surface is above the water table, but the performance of the pavement material may be adversely affected by capillary rise; and
c) Rock cutting where there is evidence of seepage within rock fractures. Particular care is required in northern regions when seepage may only be evident after torrential downpours associated with tropical lows and cyclones.

The subgrade surface below a drainage layer must be designed with adequate slope (minimum slope 1 in 500) and exit points so that water can drain freely out of the drainage layer without any risk of ponding.

Drainage layers must not be installed in floodways and floodway pavements must be capable of withstanding inundation for the anticipated flooding duration (e.g. by using cement stabilized basecourse or a continuously reinforced concrete pavement).

Full depth asphalt pavements must not be constructed below the water table, even when sub-soil drainage has been installed. Full depth asphalt pavements must not be constructed within the zone where capillary rise may adversely affect the performance of the asphalt. Concrete pavements may be used at these locations.

2 MECHANISTIC PAVEMENT DESIGN PROCEDURE

2.1 Project Reliability Level

In addition to complying with Section 1 of Engineering Road Note 9, the design of pavements with asphalt courses or other bound layers (e.g. cemented layers) must also as a minimum comply with the mechanistic procedure detailed in AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2012). The designer must use a project reliability level of not less than 95%.

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 Engineering Road Note 9. 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.

2.3 Design Traffic Loading

Section 3.8 “Design Traffic Loading for Flexible Pavements” is applicable to both the empirical procedure and the mechanistic procedure.

2.4 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.

2.5 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 unbound and modified granular materials, flexural beam testing on cemented 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.

2.6 Modulus of Unbound and Modified Granular Materials

When the asphalt nominal total thickness is 60 mm or less, the vertical moduli assigned to unbound or modified 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 (2012), unless higher material parameters are proven to be applicable by carrying out laboratory repeated load tri-axial testing.

Table 4 details the maximum or minimum stress limits that may be used for determining the vertical modulus of the top sublayer of unbound or modified granular basecourse materials from laboratory repeated load tri-axial testing.

Table 4. STRESS LIMITS

Minimum Octahedral Shear Stress (kPa)	Maximum Mean Normal Stress (kPa)
Top Sublayer of Granular Basecourse	Top Sublayer of Granular Basecourse
120	240

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 basecourse (including for modified granular materials), must not exceed 1,000 MPa.

When the asphalt nominal total thickness is greater than 60 mm, the vertical modulus used in the mechanistic procedure for granular basecourse or granular subbase material (including for modified granular materials, but excluding cemented materials), must not exceed the values in Table 6.4 of the AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2012).

2.7 Asphalt Modulus

The asphalt design modulus must be calculated from the indirect tensile test asphalt modulus in accordance with AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2012). Except that when the asphalt design modulus is less than 1,000 MPa an asphalt design modulus of 1,000 MPa may be used.

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 2891.13.1-1995 "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 (2012) on specimens prepared from the constituents mixed in the laboratory and gyratory compacted to about 5% air voids.

The indirect tensile tests must be undertaken at the combination of particle size distribution and binder content conditions within the approved range for the mix design that result in the greatest pavement thickness.

As a minimum three indirect tensile tests must be undertaken at the following test conditions: -

(a) Upper limit of the binder content range and at the coarse grading limit of the grading envelope when the asphalt nominal total thickness is greater than 60 mm; and
(b) Middle of the binder content range and middle of grading envelope for asphalt wearing courses where the asphalt nominal total thickness is 60 mm or less.

If the indirect tensile test asphalt modulus obtained from testing at a combination of grading and binder content other than the minimum test conditions mandated above yields a modulus that requires an increase to the pavement thickness, then that increased pavement thickness must be used in the pavement.

Unless specified otherwise by the Principal, the indirect tensile test asphalt modulus used in the mechanistic procedure to calculate the asphalt design modulus for dense graded asphalt must comply with Table 5 and Table 6 in Engineering Road Note 9 (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 (2012) for laboratory prepared specimens at the design binder content and grading and gyratory compacted to 5% air voids).

Table 5. 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 6.13 of the AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2012)	When 3 to 9 tests are undertaken on the mix to be used at the combination of particle size distribution and binder content that results in the greatest pavement thickness, the highest individual result obtained; or When 10 or more tests are undertaken on the mix to be used at the combination of particle size distribution and binder content that results in the greatest pavement thickness, the value that 85% of test results are lower than	The “typical” Australian dense-graded asphalt modulus values in Table 6.13 of the AUSTROADS Guide to Pavement Technology Part 2 - Pavement Structural Design (2012)	When 3 to 9 tests are undertaken on the mix to be used at the combination of particle size distribution and binder content that results in the greatest pavement thickness, the lowest individual result obtained; or When 10 or more tests are undertaken on the mix to be used at the combination of particle size distribution and binder content that results in the greatest pavement thickness, the value that 85% of test results are higher than

Table 6. 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 asphalt design modulus used for Perth open graded asphalt mixes in the mechanistic procedure must comply with Table7. Higher values should be used if the WMAPT is less than 29^0C:

Table 7. 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	Asphalt Design Modulus used must be greater than	Asphalt Design 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	

2.8 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 8.

Table 8. 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

2.9 Asphalt in Service Air Voids and Binder Content

Unless specified otherwise by the Principal, the maximum or minimum in service air voids and the in service binder content used in the mechanistic procedure must comply with Table 9.

Table 9. 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	8.8% Maximum	8.8% Minimum	11.8% Maximum
14 mm Class 320 Perth intersection mix	8.8% Maximum	8.8% Minimum	10.3% Maximum
14 mm Class 320 Perth asphalt intermediate course		8.8% Minimum	10.3% Maximum
20 mm Class 320 Perth asphalt intermediate course		5.4% Minimum	10.3% Maximum
20 mm Class 320 Perth asphalt base course		3.4% Minimum	12.1% Maximum

2.10 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.

3 MAIN ROADS WESTERN AUSTRALIA EMPIRICAL PAVEMENT DESIGN PROCEDURE

This Section of Engineering Road Note 9 details the Main Roads Western Australia Empirical Pavement Design Procedure (empirical 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 (2012) 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).

3.1 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).

3.2 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, moisture content 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.

3.3 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:-

$$\frac{MC_{85}}{LL} = 0.54 - 0.08V$$ 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:-

$$\frac{MC_{85}}{LL} = 0.50 - 0.08V$$ 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:

$$\frac{MC_{85}}{OMC} = 0.75$$ 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 performance of the subgrade may be adversely affected by capillary rise;

c) Rock cutting where there is evidence of seepage within rock fractures. Particular care is required in northern regions when seepage may only be evident after torrential downpours associated with tropical lows and cyclones;

d) Floodways and floodway approaches;

e) Pavement subject to inundation during flooding; and

f) 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.

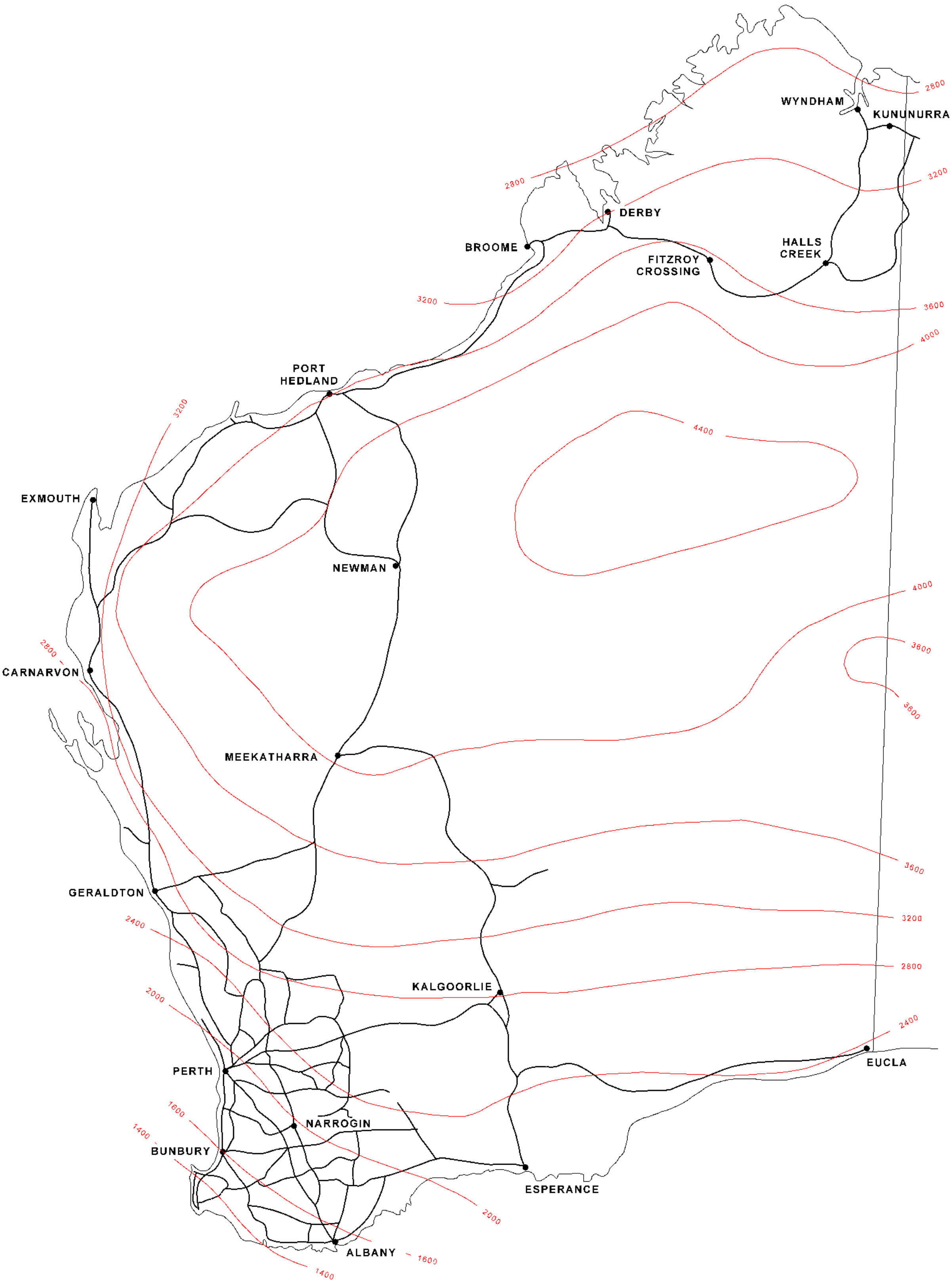

Figure 1. FIGURE 1 ANNUAL AVERAGE EVAPORATION (mm)
(WITH 7% BIRD SCREEN CORRECTION)
Source: Bureau of Meteorology (1975)

3.4 In Situ Subgrade 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.

3.5 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:

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

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 10

Table 10. 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 high or low values of CBR should not be used for the determination of design CBR using equation (4). When an isolated high or low value is encountered the test should be repeated. If the repeat testing confirms that the original result is correct, then the area from which the sample was taken must form a separate subgrade design units.

3.6 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.

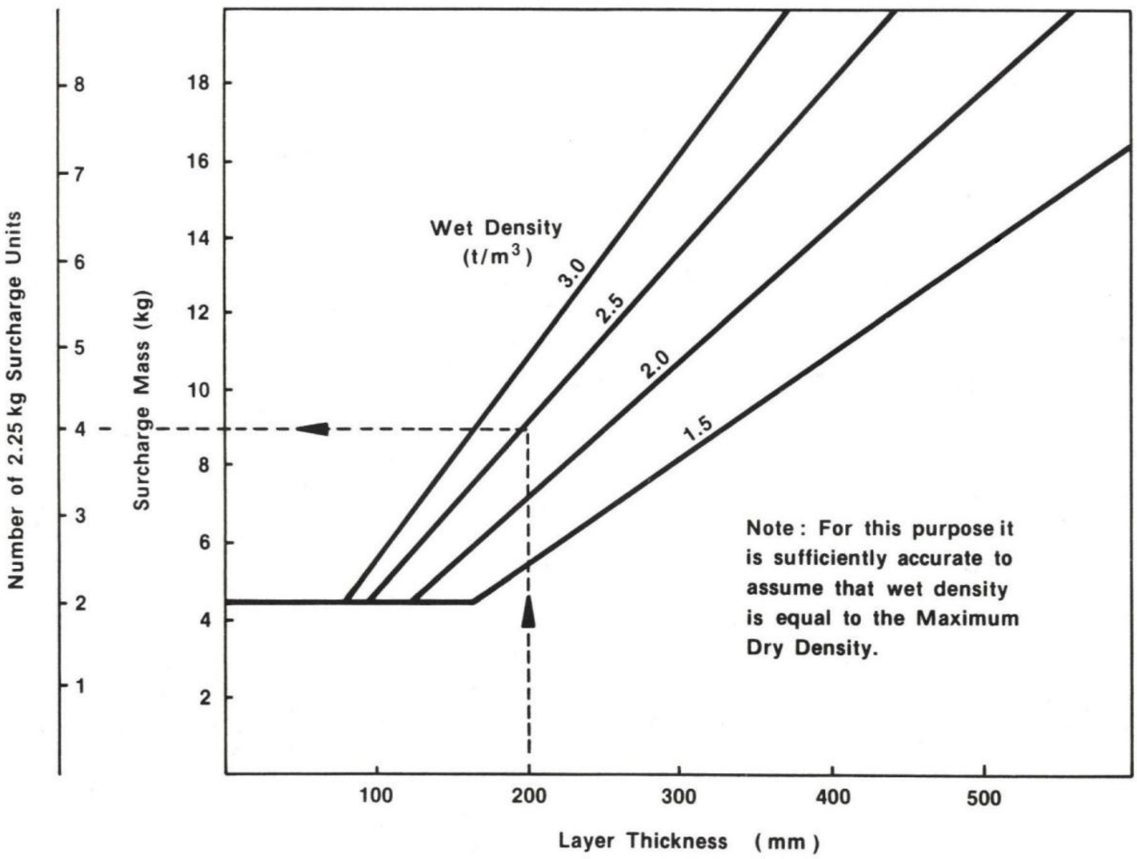

Figure 2. LABORATORY CBR: SELECTION OF SURCHARGE

3.7 CBR Empirically Correlation With In Situ Measurements

3.7.1 When Use of Empirical Correlations Permitted

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.

Some practitioners have found that calculating the subgrade CBR of fine grained soils using classification tests (section 3.7.5 refers) can provide a better indication of the in-service subgrade CBR in circumstances where the soaked CBR appears to be too low and the un-soaked CBR appears to be too high.

These empirical correlations may only be used to increase the pavement thickness determined from using the design subgrade CBR determined from laboratory measurements. These empirical correlations must not be used to decrease the pavement thickness determined from using the design subgrade CBR determined from laboratory measurements.

These empirical correlations may also 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.

3.7.2 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.

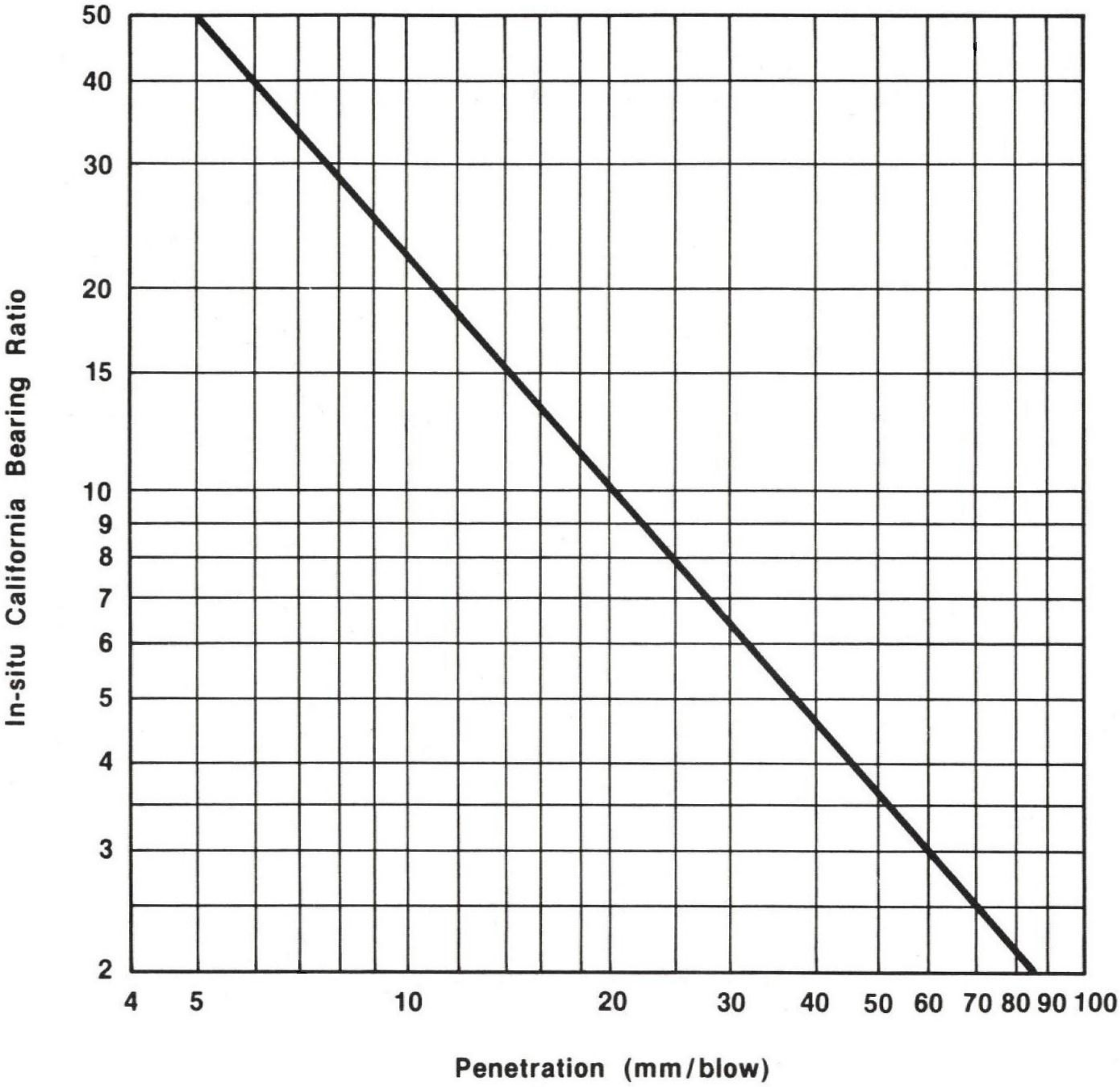

Figure 3. CORRELATION OF CONE PENETRATION (mm/blow) AND CALIFORNIA BEARING RATIO (%)

3.7.3 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.

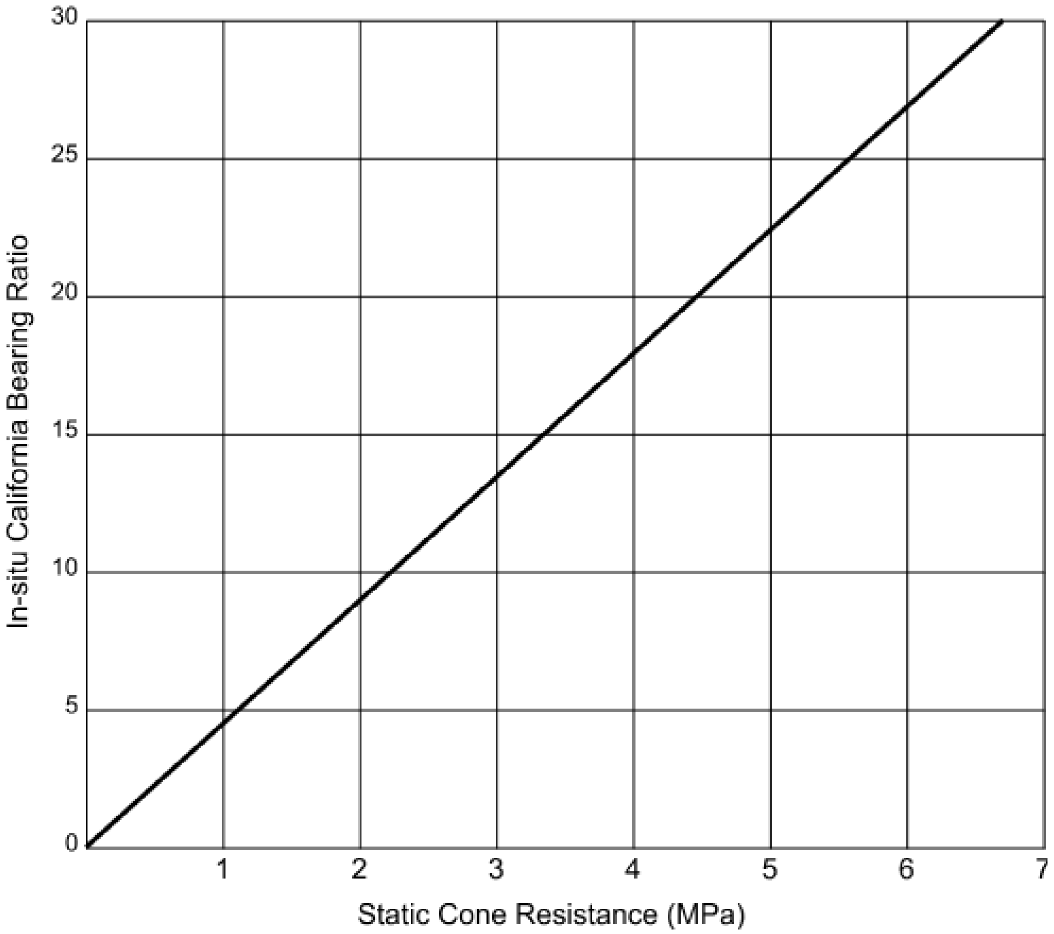

Figure 4. CORRELATION OF STATIC CONE RESISTANCE (MPa) AND CALIFORNIA BEARING RATIO (%)

3.7.4 Clegg Impact Soil Tester

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

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

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.

3.7.5 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 soaked CBR value may be estimated from equations 6 and 7 developed by the Country Roads Board of Victoria (1980).

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

$$\log_{10}\mathrm{CBR} = 1.886 - 0.00372\,\mathrm{P}_{2.36} - 0.00450\,\mathrm{P}_{0.425} + \frac{\mathrm{P}_{0.075}}{\mathrm{P}_{0.425}}\left(5.15 - 0.456\frac{\mathrm{P}_{0.075}}{\mathrm{P}_{0.425}}\right)10^{-3} - 0.0143\,\mathrm{PI} \quad \text{Eq(7)}$$

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:

$$\mathrm{CBR}_1 = 0.25\left(3\,\mathrm{CBR}_{min} + \mathrm{CBR}_{max}\right) \quad \text{Equation (8)}$$

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. Use of this method should be limited to soils having more than 75% passing the 2.36 mm sieve and a plasticity index greater than or equal to 6.

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.

3.8 Design Traffic Loading For Flexible Pavements

3.8.1 Number of Equivalent Standard Axles (ESA)

In the empirical procedure, the design traffic loading is expressed as the number of equivalent standard axles 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.

3.8.2 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 3.8.8) — d (%)
- Number of equivalent standard axles per vehicle for each heavy vehicle class – Axle Equivalency Factors (refer to Section 3.8.6) — 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
- Compound cumulative growth factor (refer to Section 3.8.4) — R

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

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

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.

3.8.3 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 3.8.8) Design Lane Distribution Factor — d (%)
- Number of equivalent standard axles per heavy vehicle – Axle Equivalency Factor (refer to Section 3.8.6) — F
- Pavement design life — P (years)
- Compound cumulative growth factor (refer to Section 3.8.4) — R

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

$$N = 365\,n \cdot d \cdot R \cdot c \cdot F \times 10^{-4}$$ 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.

3.8.4 Compound Cumulative Growth Factor R

If the annual heavy vehicle growth rate r is constant during the design life P years, the compound cumulative growth factor R can be calculated from Equation (11). It can also be read from Figure 5.

$$R = \frac{(1+r)^P - 1}{r}, \quad r \neq 0$$ 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 compound cumulative growth factor R can be determined from Equation (12).

$$R = \frac{(1+r_1)^Q - 1}{r_1} + (1+r_1)^{Q-1}(1+r_2)\left[\frac{(1+r_2)^{P-Q} - 1}{r_2}\right], \quad r_1, r_2 \neq 0$$ 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 compound cumulative growth rate R can be calculated from Equation (13).

$$R = \frac{(1+r_1)^Q - 1}{r_1} + (P-Q)(1+r_1)^{Q-1}, \quad r_1 \neq 0, r_2 = 0 \qquad \text{Equation (13)}$$

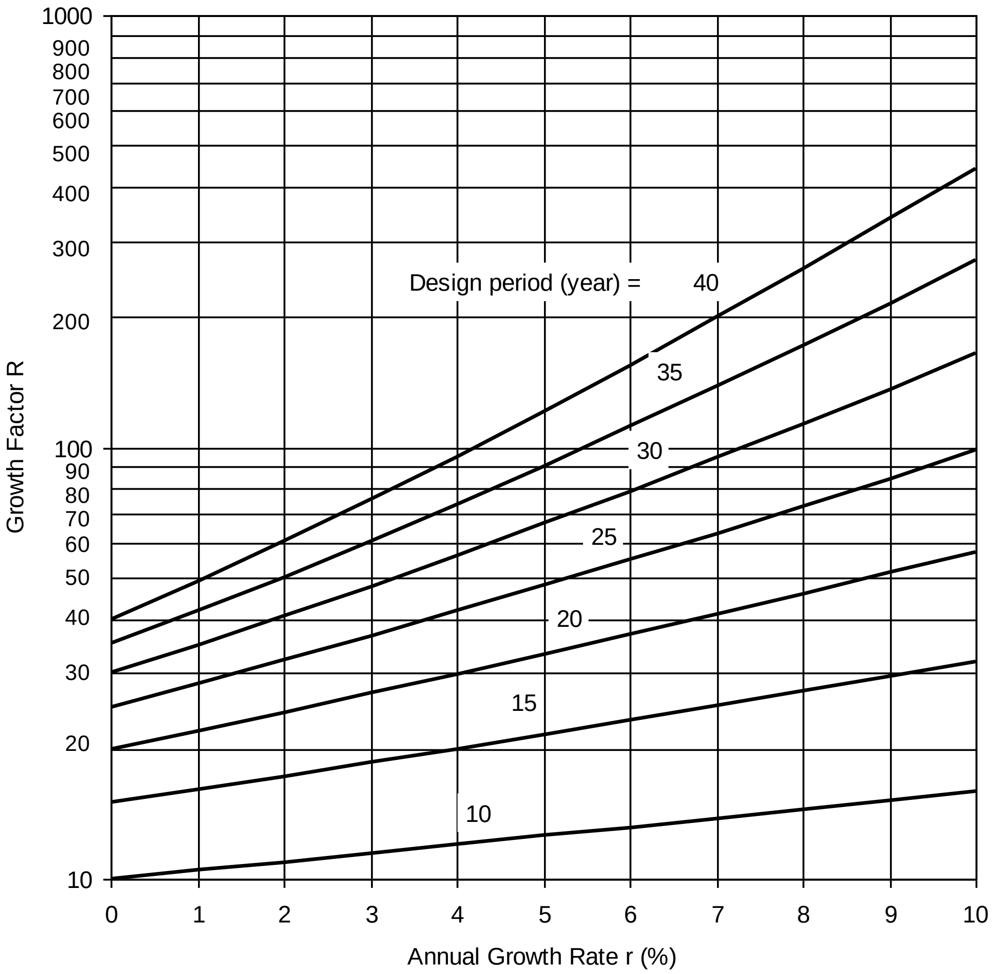

Figure 5. DETERMINATION OF GROWTH FACTOR FROM ANNUAL GROWTH RATE AND DESIGN LIFE

Not all sections of road exhibit compound heavy vehicle growth. Some locations exhibit growth that is linear (i.e. the number of vehicles using the section increases by a similar number each year). Historical traffic data should be analysed in conjunction with modeling predictions of future traffic to determine whether it is more appropriate to use linear or compound growth to predict the design traffic loading.

3.8.5 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 (2012) (i.e. Class 3 to Class 12 inclusive).

3.8.6 Axle Equivalency Factors (F)

Axle equivalency factors are derived from traffic data from 9 WIM sites in regional areas of Western Australia collected between 2003 and 2007, 1 WIM site in Metropolitan area collected in 2006, and 3 WIM sites in Metropolitan area collected in 2011.

3.8.6.1 Method 1 Axle Equivalency Factors

Axle equivalency factors for Method 1 are provided in Table 11 by Main Roads WA road classification.

Axle equivalency factors at 13 WIM sites in WA are shown in Table 12. These values must only be used when the sections being designed are in the close vicinity of these WIM sites.

Table 11. AXLE EQUIVALENCY FACTORS FOR GENERAL USE IN DESIGN TRAFFIC LOADING METHOD 1

Main Roads WA Road Classification*	Axle Equivalency Factor 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	0.77	2.57	4.23	2.29	1.59	3.40	4.20	8.08	10.03	11.54
Rural Highways	0.77	2.57	4.23	2.29	1.59	3.40	4.20	8.08	10.03	11.54
Rural Main and Secondary Roads	0.23	1.09	3.24	0.96	0.60	2.90	3.13	5.64	7.26	7.87
Urban Freeways and Highways	0.68	2.30	2.67	1.30	1.48	2.95	3.42	4.74	5.84	6.98
Other Important Urban Arterial Roads	0.68	2.30	2.67	1.30	1.48	2.95	3.42	4.74	5.84	6.98

Table 12. AXLE EQUIVALENCY FACTORS AT WIM SITES IN WA FOR **SPECIFIC** USE IN DESIGN TRAFFIC LOADING METHOD 1

Road and Location of WA WIM Site	Axle Equivalency Factor By Vehicle Class									
	F_3	F_4	F_5	F_6	F_7	F_8	F_9	F_{10}	F_{11}	F_{12}
Great Eastern Hwy (H005) SLK102.66, Northam	0.49	2.05	2.67	0.90	1.48	2.40	3.24	4.38	5.33	9.08
Great Northern Hwy (H006) SLK30, Bullsbrook	0.49	2.32	4.16	0.70	1.06	3.01	3.82	6.63	6.89	9.08
Great Northern Hwy (H006) SLK35, Muchea	0.61	2.30	3.53	0.78	1.03	2.95	3.42	4.74	5.84	9.08
Victoria Hwy (40Km East of WA Border)	0.33	1.60	2.50	0.15	0.55	1.91	4.35	4.84	6.23	9.08
Brookton Hwy (H052) SLK129, Brookton	0.64	2.08	3.10	2.29	1.38	3.40	3.46	5.04	8.25	11.54
South Coast Hwy (H008) SLK468.4, Esperance	0.44	1.89	1.77	0.52	1.10	2.09	2.85	5.07	10.03	11.54
South Western Hwy (H009) SLK204.79, Kirup	0.77	2.22	2.82	0.86	1.11	3.07	4.10	8.08	9.95	11.54
South Western Hwy (H009) SLK79.29, Waroona	0.73	2.47	4.23	1.39	1.55	3.37	3.51	4.61	5.33	11.54
Geraldton-Mt Magnet Rd (H050) SLK8.43, Geraldton	0.23	1.09	3.24	0.96	0.60	2.90	3.13	5.64	7.26	7.87
Kwinana Freeway (H015) SLK56.84, Mandurah	0.49	2.63	2.80	0.68	1.49	3.73	4.69	6.79	8.88	11.54
Kwinana Fwy (H015) SLK 69.05, Pinjarra	0.48	2.30	2.66	0.55	1.33	3.05	3.74	5.16	6.88	11.54
Reid Highway (H021) SLK22.65) Middle Swan	0.40	2.62	3.66	0.66	0.76	3.31	4.52	3.72	5.67	6.98
Roe Hwy (H018) SLK13.03, Jandakot	0.68	1.96	2.06	1.30	1.32	2.48	3.06	4.28	5.13	6.98

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

3.8.6.2 Method 2 Axle Equivalency Factor

The axle equivalency factors for use in design traffic loading Method 2 are provided in Table 13 by Main Roads WA road classification.

The axle equivalency factor at 13 WIM sites in WA for use in the design traffic loading Method 2 are shown in Table 14. These values must only be used when the sections being designed are in the vicinity of these WIM sites.

Table 13. AXLE EQUIVALENCY FACTOR FOR GENERAL USE IN DESIGN TRAFFIC LOADING METHOD 2

Main Roads WA Road Classification*	Axle Equivalency Factor
	F
Rural National Highways	5.81
Rural Highways	5.81
Rural Main and Secondary Roads	3.75
Urban Freeways and Highways	1.99
Other Important Urban Arterial Roads	1.99

Table 14. AXLE EQUIVALENCY FACTORS AT WIM SITES IN WA FOR SPECIFIC USE N DESIGN TRAFFIC LOADING METHOD 2

Road and Location of WA WIM Site	Axle Equivalency Factor
	F
Great Eastern Hwy (H005) SLK102.66, Northam	3.27
Great Northern Hwy (H006) SLK30, Bullsbrook	4.04
Great Northern Hwy (H006) SLK35, Muchea	3.60
Victoria Hwy (40Km East of WA Border)	4.06
Brookton Hwy (H052) SLK129, Brookton	4.14
South Coast Hwy (H008) SLK468.4, Esperance	5.81
South Western Hwy (H009) SLK204.79, Kirup	5.21
South Western Hwy (H009) SLK79.29, Waroona	2.88
Geraldton-Mt Magnet Rd (H050) SLK8.43, Geraldton	3.75
Kwinana Freeway (H015) SLK56.84, Mandurah	2.86
Kwinana Fwy (H015) SLK 69.05, Pinjarra	2.86
Reid Highway (H021) SLK22.65) Middle Swan	1.90
Roe Hwy (H018) SLK13.03, Jandakot	1.99

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

3.8.7 Design Life (P)

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

3.8.8 Design Lane Distribution Factor (d)

As a minimum, Table 15 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 15. 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	95
Rural	3 lane	95
Urban	1 lane	100
Urban	2 lane	80
Urban	3 lane	65

3.8.9 Roundabouts and other Small Radius Curves

At roundabouts and other small radius curves (e.g. right turning and/or left turning traffic streams at an intersection; loop-shaped ramps on or off a freeway, etc.) 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.

3.8.10 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 (SARs) for each of the relevant damage types (i.e. Fatigue of asphalt, Rutting and shape loss, Fatigue of cemented materials). The design traffic loading in terms of ESAs needs to be expressed as a design traffic loading in terms of SARs for each damage type, so that the design traffic loading can be compared with the allowable traffic loading of the candidate pavement.

The SAR/ESA ratios for each damage type are determined from WIM survey data. Table 16 lists the SAR/ESA for each damage type by Main Roads WA Road Classification.

Table 17 provides the SAR/ESA ratios for each damage type for 13 WIM sites in Western Australia. These values must only be used when the sections being designed are in the close vicinity of these WIM sites.

The SAR/ESA ratios for each damage type must not be less than the estimated values in Table 16 or Table 17. The design traffic loading in SARs for each damage type must not be less than calculated by multiplying the design traffic loading in ESAs by the ratios of SAR/ESA in Table15 or Table 16.

Table 16. MINIMUM RATIOS OF SAR TO ESA FOR GENERAL USE IN DESIGN

Main Roads WA Road Classification*	Asphalt fatigue	Rutting and shape loss	Cemented material fatigue
	SAR5/ESA	SAR7/ESA	SAR12/ESA
Rural National Highways	1.26	2.31	21.40
Rural Highways	1.22	1.93	9.42
Rural Main and Secondary Roads	1.13	1.53	4.66
Urban Freeways and Highways	1.13	1.64	9.78
Other Important Urban Arterial Roads	1.13	1.64	9.78

Table 17. MINIMUM RATIOS OF SAR TO ESA AT WIM SITES IN WA FOR SPECIFIC USE IN DESIGN

Road and Location of WA WIM Site	Asphalt fatigue	Rutting and shape loss	Cemented material fatigue
	SAR5/ESA	SAR7/ESA	SAR12/ESA
Great Eastern Hwy (H005) SLK102.66, Northam	1.12	1.51	4.68
Great Northern Hwy (H006) SLK30, Bullsbrook	1.24	2.11	14.21
Great Northern Hwy (H006) SLK35, Muchea	1.19	1.83	10.64
Victoria Hwy (40Km East of WA Border)	1.26	2.31	21.40
Brookton Hwy (H052) SLK129, Brookton	1.22	1.93	7.85
South Coast Hwy (H008) SLK468.4, Esperance	1.18	1.70	4.94
South Western Hwy (H009) SLK204.79, Kirup	1.22	1.90	7.66
South Western Hwy (H009) SLK79.29, Waroona	1.18	1.82	9.42
Geraldton-Mt Magnet Rd (H050) SLK8.43, Geraldton	1.13	1.53	4.66
Kwinana Freeway (H015) SLK56.84, Mandurah	1.25	2.08	9.85
Kwinana Fwy (H015) SLK 69.05, Pinjarra	1.19	1.82	6.94
Reid Highway (H021) SLK22.65) Middle Swan	1.19	1.86	8.22
Roe Hwy (H018) SLK13.03, Jandakot	1.13	1.64	9.78

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

3.9 Pavement Thickness Using Empirical Pavement Design Procedure

3.9.1 Pavement Thickness

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 6 and from the following equation:

$$t = \left[219 - 211(\log CBR) + 58(\log CBR)^2\right]\log\frac{N}{120} \qquad \text{Equation (16)}$$

where t = Minimum thickness in millimetres

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

CBR = Design subgrade CBR (equation 4)

Proficiency testing has demonstrated that the repeatability of laboratory CBR testing, especially between laboratories, is sometimes poor. When CBR test results do not appear to be representative of the material tested, the testing must be repeated, preferably by another laboratory.

A significant amount of local knowledge about the performance of similar material is required to use a design subgrade CBR value that is greater than 10%. Perth sands are an example of where there is a significant amount of local knowledge and a design subgrade CBR of up to 12% can be used for Perth sands. Using a design subgrade CBR value greater than 15% should generally not occur, even when a layer of selected material with a high soaked CBR has been imported and placed below the subgrade surface.

3.9.2 Pavement Composition

The total minimum thickness of granular pavement materials required above the design subgrade surface in the empirical procedure may be made up of a granular basecourse 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 surface. 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 6 and from equation 16 calculated using the soaked design CBR of the sand drainage layer.

It is necessary to provide a minimum thickness of a granular basecourse material with a soaked CBR of 80% or above over the subbase. This minimum granular basecourse thickness must not be less than shown in Figure 6.

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

Equation 16 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.

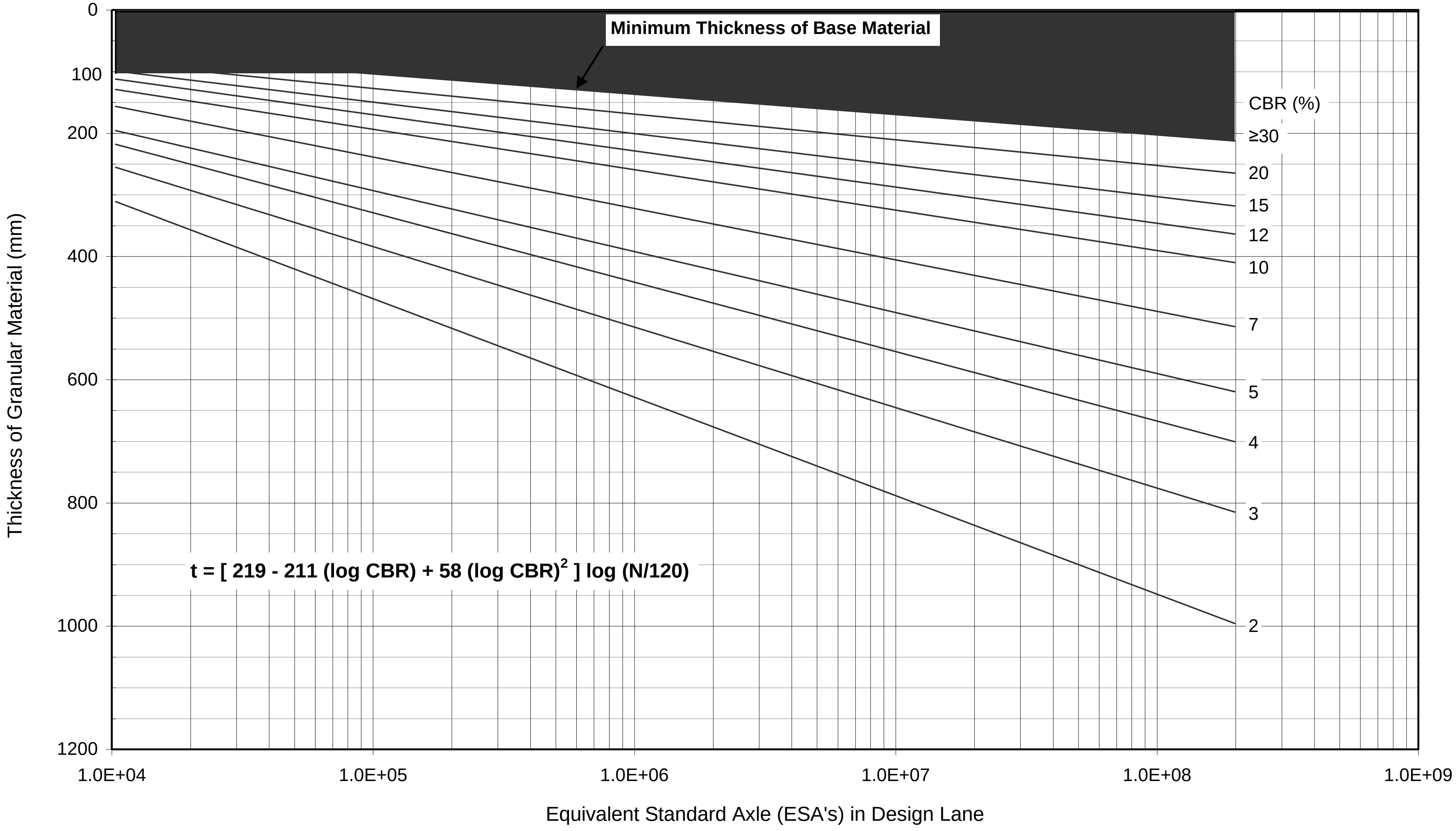

Figure 6. CBR DESIGN CHART

4 DESIGN TRAFFIC FOR RIGID PAVEMENTS

4.1 Number of Heavy Vehicle Axle Groups (HVAG)

The design traffic loading for rigid pavement design is expressed as the number of heavy vehicle axle groups (HVAG) in the design lane during the design life of the pavement.

The required data for calculating design traffic for rigid pavement 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 3.8.8) Design Lane Distribution Factor d (%)
- Average number of axle groups per heavy vehicle (refer to Section 4.2) N_{HVAG}
- Pavement design life (years) P
- Compound cumulative growth factor (refer to Section 3.8.4) R

The cumulative number of heavy vehicle axle groups N traversing the design lane during the specified period is calculated using the following equation:

$$N = 365\, n \cdot d \cdot R \cdot c \cdot N_{HVAG} \times 10^{-4}$$ Equation (14)

(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.

4.2 Percentage Distribution of Axle Group Types and Average Number of Axle Groups per Heavy Vehicle N_{HVAG}

The percentage distribution of axle group types i.e. Single Axle Single Tyre (SAST), Single Axle Duel Tyre (SADT), Tandem Axle Single Tyre (TAST), Tandem Axle Duel Tyre (TADT), Tri-Axle Duel Tyre (TRDT), Quad-Axle Duel Tyre (QADT) can be estimated from the following survey data either collected for the project or recently collected for other purposes:

- Weight-in-motion (WIM) survey data
- Vehicle classification counter survey data

Average number of axle groups N_{HVAG} per heavy vehicles can be calculated from the percentage distribution of axle group types using the following equation:

$$N_{HVAG} = \frac{100}{SAST\% + TAST\%}$$ Equation (15)

Where:
SAST% = percentage of single axle single tyre including single steer axle (e.g. input 24 for 24%),

TAST% = percentage of tandem axle single tyre including tandem steer axle (e.g. input 3 for 3%)

Classification data can be collected for projects in Western Australia. Designers should request the percentage distribution of axle group types from those who provide traffic data.

A list of percentage distribution of axle group types and N_{HVAG} values for 13 WIM sites in Western Australia are presented in Table 18. These values must only be used when the sections being designed are in the close vicinity of these WIM sites.

Table 18. NUMBER OF AXLE GROUPS PER HEAVY VEHICLE AND PERCENTAGE DISTRIBUTION OF AXLE GROUP TYPE AT WIM SITES IN WA

Road and Location of WA WIM Site	N_{HVAG}	Percentage distribution of axle group types (%)					
		SAST	SADT	TAST	TADT	TRDT	QADT
Great Eastern Hwy (H005) SLK102.66, Northam	3.54	27.09%	9.10%	1.14%	32.65%	29.97%	0.04%
Great Northern Hwy (H006) SLK30, Bullsbrook	3.60	24.53%	9.72%	3.24%	32.85%	29.59%	0.06%
Great Northern Hwy (H006) SLK35, Muchea	3.64	24.33%	10.34%	3.13%	32.33%	29.81%	0.07%
Victoria Hwy (40Km East of WA Border)	4.04	24.52%	11.72%	0.24%	33.79%	29.71%	0.03%
Brookton Hwy (H052) SLK129, Brookton	3.51	25.69%	11.49%	2.84%	29.97%	29.95%	0.05%
South Coast Hwy (H008) SLK468.4, Esperance	4.14	20.08%	8.26%	4.05%	24.52%	43.02%	0.07%
South Western Hwy (H009) SLK204.79, Kirup	3.57	24.81%	10.44%	3.24%	35.72%	25.67%	0.13%
South Western Hwy (H009) SLK79.29, Waroona	3.13	29.49%	14.53%	2.49%	30.72%	22.66%	0.11%
Geraldton-Mt Magnet Rd (H050) SLK8.43, Geraldton	3.81	23.08%	10.91%	3.14%	25.89%	36.94%	0.04%
Kwinana Freeway (H015) SLK56.84, Mandurah	2.76	33.80%	20.73%	2.41%	24.57%	18.48%	0.01%
Kwinana Fwy (H015) SLK 69.05, Pinjarra	2.98	30.60%	16.06%	2.96%	27.14%	23.23%	0.01%
Reid Highway (H021) SLK22.65) Middle Swan	2.45	38.13%	24.87%	2.63%	23.47%	10.86%	0.03%
Roe Hwy (H018) SLK13.03, Jandakot	2.63	35.53%	20.03%	2.51%	25.81%	16.10%	0.02%

4.3 Traffic Load Distribution (TLD)

In addition to the cumulative number of HVAG discussed in Section 4.1, the traffic load distribution (TLD) for the project is required for rigid pavement design.

WIM data either collected for the project or other purposes may be used to estimate the TLD for a project. Appendix A provides tables of TLD obtained from 13 WIM sites in Western Australia. Electronic copies of these TLD tables can be downloaded from the website of Main Roads Western Australia, Designer should use engineer judgment to select appropriate TLD table on the basis of project location, road classification, similarity of heavy vehicle characteristics, etc.

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APPENDIX - A TRAFFIC LOAD DISTRIBUTION

(The electronic copies of the TLD tables are available at Main Roads WA Website)

Table A-1 Traffic Load Distribution for WIM Site on Great Eastern Highway (H005) SLK102.66, Northam

Axle Group Load (kN)	Axle Group Type					
	SAST	SADT	TAST	TADT	TRDT	QADT
10	0.2700	5.8912	0.6894	0.2792		
20	9.9510	23.3747	0.2909	0.6582	0.0200	
30	8.5209	16.6933	0.1293	0.7978	0.0500	1.1406
40	5.8106	16.2132	0.0108	1.0472	0.1001	2.2811
50	16.6617	11.5923	0.0323	3.5504	0.7804	1.9010
60	46.5347	9.1718	0.3770	5.2258	3.8919	3.0415
70	11.8212	6.9414	3.6195	6.1035	6.3232	0.3802
80	0.3900	4.9910	8.7579	6.1832	5.1026	0.7604
90	0.0400	3.1606	19.7350	6.3628	4.4322	1.9010
100		1.3703	24.3564	6.8315	4.0320	1.5208
110		0.4501	25.4120	7.8986	3.8119	3.4217
120		0.1100	16.5895	8.8660	4.0520	5.7029
130		0.0300		10.2341	4.6223	6.8434
140		0.0100		10.8764	4.9625	9.1346
150				10.5753	5.5028	3.8019
160				7.5467	6.2031	3.8019
170				3.9021	7.0635	1.1406
180				1.6965	8.3042	2.2811
190				0.7196	9.2346	0.7604
200				0.3291	8.4542	2.2811
210				0.1511	6.0530	2.2811
220				0.0798	3.6318	2.6613
230				0.0403	1.8009	5.7029
240				0.0211	0.8104	3.8019
250				0.0111	0.3702	0.7604
260				0.0111	0.2001	3.4217
270					0.0900	3.0415
280					0.0500	2.2811
290				0.0011	0.0300	3.0415
300				0.0004	0.0100	1.9010
310					0.0100	1.9010
320						2.2811
330						1.5208
340						1.5208
350						2.6613
360						1.5208
370						1.5208
380						0.7604
390						0.3802
400						0.3802
410						0.3802
420						1.9010
430						1.5208
440						0.7604
450						
460						
470						
480						
490						
500						
Total	100.00	100.00	100.00	100.00	100.00	100.00
Proportion of Each Axle Group (%)	27.09	9.10	1.14	32.65	29.97	0.04
N_{HVAG}	ESA/HVAG	ESA/HV	SAR5/ESA	SAR7/ESA	SAR12/ESA	
3.54	0.92	3.27	1.12	1.51	4.68	

Table A-2 Traffic Load Distribution for WIM Site on Great Northern Highway (H006) SLK30, Bullsbrook

Axle Group Load (kN)	Axle Group Type					
	SAST	SADT	TAST	TADT	TRDT	QADT
10	7.1021	26.0374	0.2655	0.9936	0.0400	
20	10.0871	18.8181	0.0379	0.7695	0.0800	
30	6.7114	15.0685		1.0326	0.1000	0.2715
40	7.2724	12.1288	0.0632	4.0133	1.1303	
50	17.2293	8.6991	0.2908	7.6955	5.1215	
60	30.1513	6.4894	1.4791	9.1274	9.2828	0.2715
70	17.1492	4.7195	4.9305	8.2410	8.7226	1.3674
80	3.7764	3.5796	11.4918	6.5752	6.0018	5.1880
90	0.5209	2.1798	14.2225	5.1433	4.0712	4.9165
100		1.1899	17.7244	4.1302	2.7708	3.5492
110		0.5799	25.5247	3.7016	2.0406	6.0125
120		0.2800	23.9697	3.8088	1.6105	5.7410
130		0.1400		5.9365	1.5105	7.1084
140		0.0500		6.7130	1.6905	7.1084
150		0.0200		7.5114	2.0406	4.1022
160		0.0200		7.6308	2.8308	3.5492
170				6.0215	4.3013	2.4633
180				4.2254	5.7917	3.8206
190				2.8312	6.6620	2.7348
200				1.8071	6.8020	3.2777
210				1.0351	6.6920	4.1022
220				0.5005	6.1018	1.0959
230				0.2460	4.8615	2.4633
240				0.1181	3.6811	3.2777
250				0.0609	2.5508	3.0062
260				0.0292	1.5505	3.5492
270				0.0292	1.0803	1.3674
280				0.0195	0.5802	1.6388
290				0.0207	0.2001	2.4633
300				0.0097	0.0600	2.1818
310				0.0110	0.0200	1.3674
320				0.0110	0.0100	0.5429
330					0.0100	1.9103
340						1.9103
350						0.5429
360						0.8245
370						1.0959
380						0.8245
390						0.5429
400						1.3674
410						0.2715
420						0.5429
430						0.2715
440						0.5429
450						0.5429
460						
470						0.2715
480						
490						
500						
Total	100.00	100.00	100.00	100.00	100.00	100.00
Proportion of Each Axle Group (%)	24.53	9.72	3.24	32.85	29.59	0.06
N_{HVAG}	ESA/HVAG	ESA/HV	SAR5/ESA	SAR7/ESA	SAR12/ESA	
3.60	1.12	4.04	1.24	2.11	14.21	

Table A-3 Traffic Load Distribution for WIM Site on Great Northern Highway (H006) SLK35, Muchea

Axle Group Load (kN)	Axle Group Type					
	SAST	SADT	TAST	TADT	TRDT	QADT
10	4.6051	23.5500	0.2324	0.9848	0.0500	0.1006
20	9.6907	18.5000	0.1057	0.8356	0.0800	0.1006
30	6.2869	13.2000		0.9948	0.1400	0.3018
40	6.0967	11.3600		3.4220	1.7000	0.1006
50	14.0254	9.9300	0.0740	8.7141	7.7900	
60	36.1698	7.5800	0.8030	9.4304	11.1000	0.2012
70	19.6816	6.1500	5.2404	8.5649	7.6600	1.2875
80	3.1334	4.1000	16.5452	6.9932	5.2000	3.5707
90	0.3103	2.8900	17.4749	5.5011	3.7600	3.7719
100		1.5900	20.2853	4.6058	2.5100	6.1557
110		0.7100	25.6630	4.1681	1.7700	9.5353
120		0.2800	13.5763	4.0089	1.5100	8.1372
130		0.1200		4.7967	1.4000	6.3569
140		0.0300		6.0079	1.5300	3.0779
150		0.0100		8.2434	2.0200	2.8767
160				9.0736	3.1300	1.6898
170				7.1098	5.2400	2.4844
180				3.9125	7.5300	2.0821
190				1.5649	8.7500	2.7761
200				0.5800	9.7200	2.6856
210				0.1811	9.1200	2.5850
220				0.0717	5.3300	3.5707
230				0.0418	2.0600	2.4844
240				0.0408	0.6500	1.9815
250				0.0298	0.1700	2.5850
260				0.0309	0.0400	3.4802
270				0.0209	0.0200	2.7761
280				0.0209	0.0100	3.5707
290				0.0199	0.0100	2.8767
300				0.0099		2.7761
310				0.0099		2.3838
320				0.0099		1.7904
330						1.4886
340						1.5892
350						1.4886
360						0.8952
370						0.8952
380						0.3018
390						0.2012
400						0.4929
410						0.2012
420						0.2012
430						0.3018
440						0.2012
450						0.3018
460						0.4929
470						0.6940
480						0.1006
490						
500						
Total	100.00	100.00	100.00	100.00	100.00	100.00
Proportion of Each Axle Group (%)	24.33	10.34	3.13	32.33	29.81	0.07
N_{HVAG}	ESA/HVAG	ESA/HV	SAR5/ESA	SAR7/ESA	SAR12/ESA	
3.64	0.99	3.60	1.19	1.83	10.64	

Table A-4 Traffic Load Distribution for WIM Site Victoria Highway (40km East of WA Border) Kununurra

Axle Group Load (kN)	Axle Group Type					
	SAST	SADT	TAST	TADT	TRDT	QADT
10	3.8631	25.3875	3.7754	1.2844	0.1501	
20	22.3379	34.1266	1.8877	1.6926	0.1401	
30	4.8339	11.1489		0.6173	0.0901	
40	5.8447	10.3390		0.9359	0.1701	
50	14.2014	6.1194	1.8877	2.5190	1.3408	
60	23.2986	4.7195		5.8346	8.1249	16.6683
70	18.9151	3.1197	1.8877	6.7208	10.0360	
80	5.8447	2.0298	3.7754	7.6866	7.0642	
90	0.8607	1.6398	22.6362	9.7775	6.0536	
100		0.8599	28.2994	9.0108	5.8635	16.6683
110		0.3900	22.6362	7.0195	5.3432	16.6683
120		0.1200	13.2141	6.9099	4.4927	
130				6.7749	3.5821	
140				6.3759	4.0324	33.3267
150				5.5295	3.5721	
160				5.4001	3.9224	
170				4.4650	4.4527	
180				3.6477	4.7529	
190				2.9778	4.1725	
200				1.8854	4.7228	
210				0.9702	4.8529	
220				0.7539	4.4627	
230				0.4851	2.9818	
240				0.3023	2.0812	
250				0.1529	1.5409	
260				0.0498	0.6504	
270				0.0434	0.5803	
280				0.0968	0.2301	
290				0.0135	0.2502	
300				0.0235	0.0800	
310				0.0434	0.1401	
320					0.0500	16.6683
330					0.0200	
340						
350						
360						
370						
380						
390						
400						
410						
420						
430						
440						
450						
460						
470						
480						
490						
500						
Total	100.00	100.00	100.00	100.00	100.00	100.00
Proportion of Each Axle Group (%)	24.52	11.72	0.24	33.79	29.71	0.03
N_{HVAG}	ESA/HVAG	ESA/HV	SAR5/ESA	SAR7/ESA	SAR12/ESA	
4.04	1.00	4.06	1.26	2.31	21.40	

Table A-5 Traffic Load Distribution for WIM Site on Brookton Highway (H052) SLK129, Brookton

Axle Group Load (kN)	Axle Group Type					
	SAST	SADT	TAST	TADT	TRDT	QADT
10	0.1000	3.2700		0.4549	0.0600	
20	12.5325	25.9700		1.6415	0.1800	
30	11.1422	20.1600		1.7701	1.0400	
40	5.6011	13.0700		3.4809	4.1700	
50	8.9218	8.7200		5.9432	8.6100	
60	35.7772	7.9300	0.4353	9.4636	9.2400	
70	23.1746	8.2700	3.0469	9.4141	4.4500	
80	2.5905	4.6300	8.2031	6.7837	2.4200	8.0000
90	0.1600	3.2000	11.8304	3.7577	1.6300	
100		2.4200	13.0580	3.4314	1.3800	4.0000
110		2.0500	34.8326	3.8665	1.4100	4.0000
120		0.2900	28.5938	4.1335	1.1400	8.0000
130		0.0200		5.6056	1.1700	12.0000
140				6.6441	1.6000	16.0000
150				8.6816	1.9800	8.0000
160				9.7581	2.5800	4.0000
170				8.0099	2.5500	
180				4.0445	3.0000	
190				1.9580	5.1500	
200				0.7318	8.3700	
210				0.2176	9.7300	
220				0.0989	8.3800	
230				0.0593	6.8100	8.0000
240				0.0297	6.0300	4.0000
250				0.0099	4.2400	
260					1.8300	8.0000
270				0.0099	0.5700	4.0000
280					0.1800	
290					0.0600	8.0000
300					0.0100	4.0000
310					0.0100	
320					0.0100	
330						
340						
350						
360						
370					0.0100	
380						
390						
400						
410						
420						
430						
440						
450						
460						
470						
480						
490						
500						
Total	100.00	100.00	100.00	100.00	100.00	100.00
Proportion of Each Axle Group (%)	25.69	11.49	2.84	29.97	29.95	0.05
N_{HVAG}	ESA/HVAG	ESA/HV	SAR5/ESA	SAR7/ESA	SAR12/ESA	
3.51	1.18	4.14	1.22	1.93	7.85	

Table A-6 Traffic Load Distribution for WIM Site on South Coast Highway (H008) SLK468.4, Esperance

Axle Group Load (kN)	Axle Group Type					
	SAST	SADT	TAST	TADT	TRDT	QADT
10	0.4001	5.9100	0.0996	0.2554		
20	13.5427	29.1300	0.5424	0.8643	0.0100	
30	9.7520	15.0600		0.8545	0.0300	
40	6.2813	14.4500		0.5696	0.0400	
50	12.1524	12.2700	0.0111	1.1098	0.1701	
60	41.2683	9.9100	0.0111	2.7304	0.7202	
70	16.1332	5.9500	1.7047	3.9875	1.4504	
80	0.4701	4.3900	4.0735	4.5473	1.6605	0.8503
90		2.4100	7.3279	4.3804	1.8005	
100		0.4700	23.7215	3.7714	1.5205	6.7820
110		0.0300	35.2004	3.1134	1.1003	14.4143
120		0.0100	27.3079	3.4768	0.9303	5.9318
130		0.0100		6.6595	0.7702	10.1731
140				10.6789	0.7402	8.4725
150				17.0376	0.9003	4.2413
160				17.1691	1.3504	5.9318
170				11.8778	2.4107	3.3910
180				4.9813	5.0115	5.0815
190				1.4634	9.9930	1.6905
200				0.3634	17.6753	3.3910
210				0.0786	21.3164	1.6905
220				0.0098	17.0751	2.5408
230				0.0098	8.6426	3.3910
240					3.1810	2.5408
250				0.0098	1.0403	1.6905
260					0.3501	2.5408
270					0.0800	3.3910
280					0.0300	2.5408
290						1.6905
300						0.8503
310						
320						1.6905
330						3.3910
340						0.8503
350						
360						
370						
380						
390						
400						
410						0.8503
420						
430						
440						
450						
460						
470						
480						
490						
500						
Total	100.00	100.00	100.00	100.00	100.00	100.00
Proportion of Each Axle Group (%)	20.08	8.26	4.05	24.52	43.02	0.07
N_{HVAG}	ESA/HVAG	ESA/HV	SAR5/ESA	SAR7/ESA	SAR12/ESA	
4.14	1.40	5.81	1.18	1.70	4.94	

Table A-7 Traffic Load Distribution for WIM Site on South Western Highway (H009) SLK204.79, Kirup

Axle Group Load (kN)	Axle Group Type					
	SAST	SADT	TAST	TADT	TRDT	QADT
10	0.2901	3.3207	0.2726	0.1168		
20	8.4834	18.3637	0.3504	0.3697	0.0100	
30	9.7139	15.2631	0.1168	0.5059	0.0200	
40	12.5850	13.4827		1.0800	0.0900	
50	19.4578	14.0028	0.0130	1.8097	0.5099	
60	37.1749	12.7225	0.5062	2.7535	1.4399	
70	11.8948	9.0018	1.5834	2.8216	1.9998	
80	0.3601	5.9412	6.9955	2.7730	2.0498	
90	0.0400	4.0508	8.3712	3.2108	2.2298	1.4619
100		2.3005	16.9371	3.0649	2.1898	3.8650
110		1.1002	31.2005	3.2011	1.5598	8.7714
120		0.3401	33.6535	3.6681	1.1499	10.9643
130		0.0900		6.3671	1.0999	8.5611
140		0.0200		6.6662	1.4799	7.4096
150				10.3624	2.0098	6.5786
160				14.3183	2.3098	5.2168
170				14.7877	3.2597	2.9238
180				11.5928	5.4595	2.5033
190				6.2706	9.9690	2.0927
200				2.7497	15.2485	2.1929
210				0.9948	17.1183	2.0927
220				0.3332	13.5886	3.3443
230				0.1168	8.4792	5.4271
240				0.0401	4.0996	5.1167
250				0.0109	1.6498	4.2856
260				0.0121	0.6499	3.1341
270				0.0012	0.2200	3.4445
280					0.0700	2.4031
290					0.0300	1.5620
300					0.0100	1.0414
310				0.0012		1.9826
320						0.5207
330						0.6308
340						0.5207
350						0.3104
360						0.2103
370						
380						0.1001
390						0.1001
400						0.2103
410						0.1001
420						0.1001
430						0.2103
440						0.1001
450						0.1001
460						0.2103
470						0.1001
480						0.1001
490						
500						
Total	100.00	100.00	100.00	100.00	100.00	100.00
Proportion of Each Axle Group (%)	24.81	10.44	3.24	35.72	25.67	0.13
N_{HVAG}	ESA/HVAG	ESA/HV	SAR5/ESA	SAR7/ESA	SAR12/ESA	
3.57	1.46	5.21	1.22	1.90	7.66	

Table A-8 Traffic Load Distribution for WIM Site on South Western Highway (H009) SLK79.29, Waroona

Axle Group Load (kN)	Axle Group Type					
	SAST	SADT	TAST	TADT	TRDT	QADT
10	3.2468	13.4673	0.8009	0.7741	0.0100	0.1203
20	11.4541	18.7463	0.5485	1.3597	0.2300	
30	10.2415	11.8876		0.7841	0.6601	
40	7.7362	11.4677		1.7269	1.0702	
50	11.5142	11.1178	0.1317	4.2876	2.5005	0.1203
60	32.3780	10.8278	1.2836	8.1782	5.6011	
70	21.4050	8.7083	2.9731	10.6098	9.4419	0.2406
80	1.8439	6.0688	12.2655	9.6471	11.0422	0.6014
90	0.1804	3.7992	16.5880	8.0293	7.8916	1.5636
100		2.0496	14.4707	6.0344	5.0010	3.4780
110		0.9398	29.6544	5.3595	3.7207	4.0794
120		0.5699	21.2836	5.6870	3.2507	6.3646
130		0.2300		6.5846	3.2006	5.6430
140		0.0800		7.0479	3.2807	3.8388
150		0.0200		7.4714	3.5407	3.3577
160		0.0100		7.0089	3.8508	2.7563
170		0.0100		4.7588	4.4209	2.6361
180				2.5104	5.1510	3.5983
190				1.0934	5.9412	3.2374
200				0.4411	6.7213	2.4055
210				0.1994	6.1912	3.2374
220				0.1001	3.9608	2.5158
230				0.0830	2.0304	3.3577
240				0.0730	0.8502	2.4055
250				0.0567	0.2601	4.4402
260				0.0433	0.0800	3.1172
270				0.0342	0.0300	3.3577
280				0.0135	0.0200	2.4055
290				0.0009	0.0100	3.8388
300				0.0009	0.0100	1.9244
310				0.0009	0.0100	2.9969
320						1.8041
330						2.2853
340					0.0100	1.5636
350						1.3230
360					0.0100	0.6014
370						0.8419
380						0.6014
390						1.0825
400						0.7217
410						0.9622
420						1.4433
430						2.1650
440						1.5636
450						2.8766
460						1.6839
470						0.6014
480						0.2406
490						
500						
Total	100.00	100.00	100.00	100.00	100.00	100.00
Proportion of Each Axle Group (%)	29.49	14.53	2.49	30.72	22.66	0.11
N_{HVAG}	ESA/HVAG	ESA/HV	SAR5/ESA	SAR7/ESA	SAR12/ESA	
3.13	0.92	2.88	1.18	1.82	9.42	

Table A-9 Traffic Load Distribution for WIM Site on Geraldton-Mt Magnet Rd (H050) SLK8.43, Geraldton

Axle Group Load (kN)	Axle Group Type					
	SAST	SADT	TAST	TADT	TRDT	QADT
10	7.6431	21.7143	0.6494	0.9065	0.0100	
20	18.2273	29.6159	0.3299	1.1854	0.0200	
30	9.3437	20.1240		1.7134	0.0800	0.5427
40	7.7931	11.5523		5.3892	2.0504	
50	20.5682	6.2412	0.1546	6.2858	6.6213	
60	28.3713	4.4409	0.7010	5.4091	4.4109	1.0853
70	7.6030	2.2605	4.8242	4.0942	1.6503	
80	0.4102	1.4203	7.6899	1.9624	1.0302	2.1606
90	0.0400	1.3603	12.2359	1.3050	0.5301	3.2459
100		0.9102	34.1924	1.2552	0.2100	10.2703
110		0.2901	28.5228	1.8130	0.1500	11.3456
120		0.0600	10.6999	3.7057	0.1600	11.8883
130		0.0100		8.3078	0.2601	4.3212
140				13.1121	0.8802	1.6179
150				15.7930	3.0806	0.5427
160				13.3985	6.0812	1.6179
170				8.3691	10.2420	2.7032
180				3.9025	15.3131	2.7032
190				1.3822	16.2432	0.5427
200				0.4234	13.3327	2.1606
210				0.1345	9.0718	3.2459
220				0.0511	5.1810	3.7785
230				0.0199	2.2905	3.2459
240				0.0199	0.7301	1.0853
250				0.0212	0.2601	2.7032
260				0.0199	0.0600	3.7785
270				0.0100	0.0300	4.3212
280				0.0100	0.0100	4.8638
290						3.2459
300					0.0100	1.0853
310						2.7032
320						2.1606
330						
340						0.5427
350						2.7032
360						0.5427
370						1.6179
380						
390						
400						
410						
420						0.5427
430						0.5427
440						
450						
460						
470						0.5427
480						
490						
500						
Total	100.00	100.00	100.00	100.00	100.00	100.00
Proportion of Each Axle Group (%)	23.08	10.91	3.14	25.89	36.94	0.04
N_{HVAG}	ESA/HVAG	ESA/HV	SAR5/ESA	SAR7/ESA	SAR12/ESA	
3.81	0.98	3.75	1.13	1.53	4.66	

Table A-10 Traffic Load Distribution for WIM Site on Kwinana Freeway (H015) SLK56.84, Mandurah

Axle Group Load (kN)	Axle Group Type					
	SAST	SADT	TAST	TADT	TRDT	QADT
10	5.8921	16.4096	0.5342	0.7423	0.0032	
20	25.6345	32.0016	0.1214	1.8700	0.0601	
30	10.5723	11.3824	0.2671	1.3799	0.2119	
40	7.1698	9.5948	0.8742	1.2514	0.4238	
50	9.7216	8.2274	0.5342	3.2950	1.1703	
60	20.4841	6.9558	0.5100	6.5116	5.8673	
70	17.1335	5.3176	2.3312	8.5243	10.1594	
80	3.1570	3.7105	5.0753	7.8986	7.2874	
90	0.2351	3.1917	13.0403	6.7732	4.4028	
100		1.9708	17.0471	5.4552	3.8209	8.0000
110		0.8797	28.9704	4.8747	3.3338	4.0000
120		0.2763	30.6945	4.9271	2.9605	8.0000
130		0.0648		6.7114	2.7454	16.0000
140		0.0113		6.1333	2.6474	16.0000
150		0.0056		6.7566	2.5304	
160				7.5322	2.3785	
170				7.8058	2.3216	
180				5.9334	2.6379	
190				3.4782	2.6189	
200				1.4370	3.2136	8.0000
210				0.4948	4.5673	
220				0.1451	5.9653	
230				0.0452	7.0059	
240				0.0190	6.8004	4.0000
250				0.0048	6.2563	4.0000
260					4.6938	8.0000
270					2.5209	
280					0.9837	
290					0.3226	
300					0.0664	4.0000
310					0.0127	8.0000
320					0.0095	4.0000
330						4.0000
340						
350						4.0000
360						
370						
380						
390						
400						
410						
420						
430						
440						
450						
460						
470						
480						
490						
500						
Total	100.00	100.00	100.00	100.00	100.00	100.00
Proportion of Each Axle Group (%)	33.80	20.73	2.41	24.57	18.48	0.01
N_{HVAG}	ESA/HVAG	ESA/HV	SAR5/ESA	SAR7/ESA	SAR12/ESA	
2.76	1.03	2.86	1.25	2.08	9.85	

Table A-11 Traffic Load Distribution for WIM Site on Kwinana Freeway (H015) SLK69.05, Pinjarra

Axle Group Load (kN)	Axle Group Type					
	SAST	SADT	TAST	TADT	TRDT	QADT
10	1.8630	9.0299	0.3691	0.6008	0.0031	
20	22.4850	34.3019	0.2461	1.4189	0.0407	
30	9.5698	12.2847		1.0675	0.0282	
40	7.0786	10.8160	0.0492	1.6684	0.2037	
50	13.9669	9.4968	0.2215	5.9493	2.3531	
60	30.0014	8.0734	0.2461	8.2775	10.4869	
70	13.8836	5.6618	1.7717	8.9292	10.0295	
80	1.0731	4.7008	7.5787	7.2877	5.3453	
90	0.0785	3.2729	15.8465	5.6596	4.2079	14.2857
100		1.5549	24.6309	5.1768	3.3807	
110		0.5712	35.0640	5.3779	3.1959	14.2857
120		0.1496	13.9764	5.6140	3.0455	14.2857
130		0.0635		6.4160	2.9828	
140		0.0181		6.8290	2.7604	
150		0.0045		7.9583	2.6946	
160				8.2131	2.4940	
170				6.7512	2.4658	
180				4.0690	2.8700	
190				1.8186	3.8539	
200				0.6035	5.3233	7.1429
210				0.2280	6.3949	
220				0.0483	6.4858	7.1429
230				0.0268	6.2696	14.2857
240				0.0080	5.1072	7.1429
250				0.0027	4.3708	
260					2.4627	7.1429
270					0.8773	
280					0.1723	7.1429
290					0.0689	
300					0.0125	
310					0.0031	
320						7.1429
330					0.0031	
340						
350					0.0063	
360						
370						
380						
390						
400						
410						
420						
430						
440						
450						
460						
470						
480						
490						
500						
Total	100.00	100.00	100.00	100.00	100.00	100.00
Proportion of Each Axle Group (%)	30.60	16.06	2.96	27.14	23.23	0.01
N_{HVAG}	ESA/HVAG	ESA/HV	SAR5/ESA	SAR7/ESA	SAR12/ESA	
2.98	0.96	2.86	1.19	1.82	6.94	

Table A-12 Traffic Load Distribution for WIM Site on Reid Highway (H021) SLK22.65, Middle Swan

Axle Group Load (kN)	Axle Group Type					
	SAST	SADT	TAST	TADT	TRDT	QADT
10	4.0886	12.6641	0.6964	1.2491	0.0230	
20	27.6176	33.4650	0.5065	1.5862	0.0383	
30	14.3079	15.7146	0.1266	1.7530	0.1380	
40	9.9419	11.4318	0.6648	3.4030	0.6747	
50	12.9297	9.3591	0.5065	8.3390	10.6119	
60	18.3615	6.4325	1.7727	11.6958	15.3657	
70	11.1322	4.5908	6.8693	9.5596	6.0497	2.9412
80	1.4917	3.0572	16.3659	6.8344	3.1284	
90	0.1289	1.8249	25.0079	4.6201	2.2466	
100		0.9443	17.2206	3.1262	1.6179	2.9412
110		0.3315	16.0810	3.0127	1.4645	8.8235
120		0.1339	14.1817	3.1794	1.5028	14.7059
130		0.0402		4.7408	0.8741	5.8824
140		0.0033		5.8586	1.0965	
150		0.0033		7.8493	1.2038	8.8235
160				8.2822	1.2805	5.8824
170				6.8876	1.6179	8.8235
180		0.0033		4.2369	2.5073	
190				2.1717	3.2510	2.9412
200				0.8942	5.7507	
210				0.3939	8.9173	2.9412
220				0.1561	10.7345	2.9412
230				0.0887	9.1627	8.8235
240				0.0461	5.9347	5.8824
250				0.0177	2.9290	2.9412
260				0.0071	1.2345	
270				0.0035	0.3910	2.9412
280				0.0071	0.0997	
290					0.0843	
300					0.0153	2.9412
310					0.0307	2.9412
320					0.0077	
330						
340						
350						
360					0.0077	2.9412
370						
380					0.0077	
390						2.9412
400						
410						
420						
430						
440						
450						
460						
470						
480						
490						
500						
Total	100.00	100.00	100.00	100.00	100.00	100.00
Proportion of Each Axle Group (%)	38.13	24.87	2.63	23.47	10.86	0.03
N_{HVAG}	ESA/HVAG	ESA/HV	SAR5/ESA	SAR7/ESA	SAR12/ESA	
2.45	0.77	1.90	1.19	1.86	8.22	

Table A-13 Traffic Load Distribution for WIM Site on Roe Highway (H018) SLK13.03, Jandakot

Axle Group Load (kN)	Axle Group Type					
	SAST	SADT	TAST	TADT	TRDT	QADT
10	0.9920	5.9095	0.1320	0.4295		
20	12.0341	15.5048	0.4263	0.8291	0.0400	
30	14.3988	14.5333	0.4060	0.5294	0.0500	1.0000
40	14.5291	15.7452	0.1929	1.3186	0.0800	3.0000
50	23.8176	15.6651	0.1117	4.0157	0.4202	
60	28.7876	13.0308	0.7917	6.9925	6.2025	
70	4.9499	7.6923	7.0747	9.1202	16.1265	
80	0.4208	4.6474	25.8323	12.5066	11.4246	
90	0.0701	3.7961	28.4003	12.0071	9.2837	
100		1.7528	22.0869	8.1512	5.7623	2.0000
110		0.9615	10.9521	5.1545	3.8415	2.0000
120		0.3606	3.5932	4.1555	2.7811	14.0000
130		0.1402		4.2787	1.9808	18.0000
140		0.0601		4.3101	1.6407	4.0000
150		0.0501		5.1583	1.8207	
160		0.0501		6.7886	2.0608	2.0000
170		0.0501		6.8766	2.6811	3.0000
180		0.0501		4.5461	4.3217	1.0000
190				1.8819	6.3225	5.0000
200				0.6063	7.6331	1.0000
210				0.1908	6.7327	4.0000
220				0.0729	4.5918	9.0000
230				0.0310	2.7011	3.0000
240				0.0220	1.1204	3.0000
250				0.0110	0.2801	2.0000
260				0.0010	0.0600	2.0000
270				0.0100	0.0300	
280					0.0100	
290				0.0039		
300						1.0000
310				0.0010		
320						
330						
340						
350						
360						1.0000
370						1.0000
380						4.0000
390						3.0000
400						3.0000
410						3.0000
420						3.0000
430						
440						1.0000
450						1.0000
460						
470						
480						
490						
500						
Total	100.00	100.00	100.00	100.00	100.00	100.00
Proportion of Each Axle Group (%)	35.53	20.03	2.51	25.81	16.10	0.02
N_{HVAG}	ESA/HVAG	ESA/HV	SAR5/ESA	SAR7/ESA	SAR12/ESA	
2.63	0.76	1.99	1.13	1.64	9.78	