SEALED UNBOUND GRANULAR PAVEMENTS

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AN IMPORTANT NOTE FOR THE READER

CONTENTS

EXECUTIVE SUMMARY

The objective of this project was to undertake a review of the international technical literature available in 1993, and of a limited number of 1995 documents, on the topic of sealed unbound granular pavements. The literature included technical papers, pavement design standards, and specifications for materials and construction from a number of roading authorities around the world.

Sealed unbound granular pavements comprise one or more layers of compacted unbound aggregate overlying a natural or filled subgrade. The surface is generally a sprayed bituminous seal coated with crushed rock chips.

A number of research findings were identified as being potentially beneficial to the design and construction of unbound granular pavements in New Zealand. These findings were ranked in terms of their benefit/cost (B/C) ratio. For the majority of the research findings it was not possible to assign a B/C ratio as it was impossible to reliably assess the benefits and/or costs.

Costing, including implementation, showed that the research findings for which a B/C ratio could be derived were as follows:

Item	**B/C Ratio**
Basing pavement design on shakedown theory	99
Relaxation of Sand Equivalent requirements for aggregates	93
Inclusion of sub-layering of aggregate layers	43
Investigation of use of steel mill slag for sub-base construction	20

A brief summary of these four top-ranked research findings is presented below:

- Development of shakedown theory for pavement design is considered to have the top priority because it has the potential to characterise pavement performance in a superior manner to the current mechanistic pavement design procedures. This will result in optimised pavement designs with significant reductions in life cycle costs.

- The literature suggests that relaxation of the Sand Equivalent requirement from the current value of 40 to a value of about 25 may be appropriate. This would result in a reduced aggregate cost as many producers have difficulty meeting the current specification.

- The current Transit New Zealand design method combines the basecourse and the sub-base layers into one layer for design. This does not resemble the construction sequence, nor the difference in quality of the basecourse and sub-base materials. Reduced pavement thicknesses may be achieved by the superior pavement characterisation provided by sub-layering the granular layers.

- In the future, the use of waste and marginal materials such as steel mill slag will become more important as resources diminish. Steel mill slag is currently being trialled in the South Auckland - Waikato regions, and its use should be encouraged.

ABSTRACT

This project involved a review of literature that was available in 1993 with additions from some 1995 documents, on the design and construction of unbound granular pavements. This type of pavement is that most frequently used on New Zealand roads as it generally performs well and minimises costs.

The technical literature, to 1995, and the standards and specifications used by a number of international roading authorities, have been reviewed. As a result a number of research findings have been identified as being potentially beneficial for further investigation and/or inclusion in future updates of the design and construction procedures of unbound granular pavements in New Zealand. Where possible, the research findings have been prioritised by determining their benefit/cost ratio.

1. INTRODUCTION

1.1 General

Most of New Zealand's 10,500 km state highway network comprises sealed unbound granular pavements (defined in Section 1.2 of this report). The popularity and success of this type of pavement in New Zealand can be attributed to a number of factors, e.g.

- a good supply of quality aggregates;
- relatively low levels of traffic loading; and
- generally favourable environmental conditions.

The standards adopted for the design, construction and materials appropriate for these pavements in New Zealand are administered by Transit New Zealand. However these standards could be updated.

Therefore a survey was carried out in 1995 with the objective to identify recent research findings which may be applied to improve these current Transit New Zealand design and construction practices for such pavements. This task has been approached in the following manner:

- Local and international standards of design, construction and materials for sealed unbound granular pavements were reviewed;
- The wider international technical literature was reviewed;
- Potentially beneficial research findings were identified for inclusion in future Transit standards and specifications or for further validation;
- Benefit/cost ratios were determined for these research findings to prioritise their usefulness.

This report presents the results of that survey of international literature available in 1993, and of an additional limited number of 1994-1995 documents.

1.2 Structure of Sealed Unbound Granular Pavements

In New Zealand, a sealed unbound granular pavement consists of a one or two coat bituminous surface seal overlying a layer of compacted unbound granular aggregate which forms the basecourse. The basecourse layer overlies another compacted unbound granular aggregate layer (usually of lower quality) termed the sub-base. Below this, the pavement structure is supported by a foundation material, i.e. the subgrade, which may comprise natural soil or imported fill. A sketch of the typical pavement structure is presented in Figure 1.1.

The surface seal is the only component of the pavement which does not contribute to its structural integrity. The surface seal does however play a critical role in the performance of the pavement because the seal provides a barrier which prevents water from entering or leaving the pavement structure. This seal is important because the presence of water can facilitate particle degradation, and water can interact with secondary minerals in the basecourse aggregate and this can lead to instability of the pavement. The surface seal also provides a resistant wearing course that protects the upper portion of the basecourse layer from the high stresses associated with the contact of vehicle tyres. The rough texture of the stone chips in the seal promotes skid resistance and minimises the risk of aquaplaning in wet conditions.

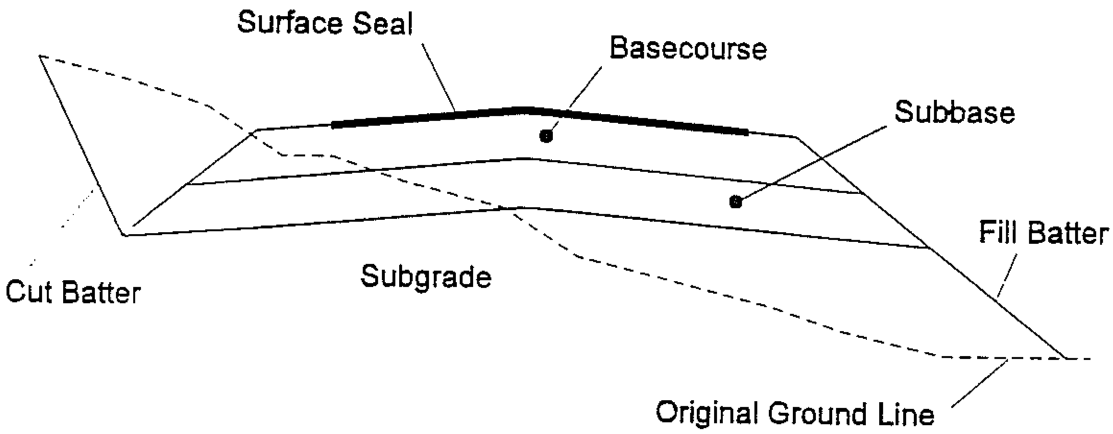


Figure 1.1 Structure of a typical sealed unbound granular pavement used in New Zealand roads.

The basecourse layer is the component primarily responsible for the structural integrity of the pavement. The basecourse spreads the applied wheel load so that the underlying materials are not over-stressed. This is achieved by transfer of load by way of particle interlock and inter-particle friction.

The sub-base layer also spreads the load in a similar way to protect the underlying subgrade. Because the sub-base is lower in the pavement structure the stresses imposed on the aggregate are not as high as those experienced by the basecourse layer. Consequently, the requirement for high aggregate quality is not as critical for sub-base aggregates. Ideally the sub-base layer should be more permeable than the basecourse. The sub-base also serves a useful practical function as a running course for plant and equipment during construction.

The subgrade is the foundation material on which the layered structure of the pavement is built. The subgrade may comprise the original in situ material in cut or at-grade areas, or be placed material in areas of fill. The design charts for flexible pavements with thin surfacings, in *Transit New Zealand Pavement Design and Rehabilitation Manual* (Transit New Zealand 1989), show that the elastic modulus of the subgrade has a significant effect on the required thickness of granular cover. For this reason a relatively common practice to improve poor quality subgrades is to stabilise them with admixtures such as cement or lime.

1.3 Characteristics of an Ideal Sealed Unbound Granular Pavement

Road pavements provide a carriageway essential for the transport of people and goods and are an important factor in the economic performance of a locality. A pavement should possess several characteristics and qualities if it is to perform adequately. The ideal pavement should:

- Provide a safe, relatively permanent medium for the efficient passage of vehicle types that are appropriate to the function and location of the road.
- Possess sufficient structural integrity to function without suffering significant distress.
- Be designed and constructed to achieve minimum costs over the life of the pavement, preferably utilising available local materials and the experience of local contractors.
- Have a surface that remains smooth throughout its design life that will minimise the cost of transportation and will promote safety and passenger comfort.
- Be resistant to the effects of climate.
- Be unobtrusive with respect to the adjacent environment.
- Be easily maintained using readily available materials and techniques.
- Be amenable to a staged construction approach if future loading conditions differ from those assumed at the time of design.

Many of the characteristics listed above are readily achieved by the adoption of sound geometrical design and traffic engineering principles. The characteristics relevant to pavement longevity are however significantly influenced by the nature of the pavement materials and the construction techniques employed. The relevant characteristics of pavement materials that promote good performance in sealed unbound granular pavements are discussed in detail in Section 1.4 of this report.

1.4 Pavement Performance

1.4.1 Achieving Pavement Performance

To achieve a high level of pavement performance the unbound aggregate must achieve a state of stability under the prevailing loading and environmental conditions. During construction and when first opened to traffic an aggregate layer experiences distortions with each cycle of load. The resulting shear strains cause the aggregate particles to relocate into a more stable packing regime. The amount of particle movement decreases with successive passages of load until permanent deformations become negligible and the pavement responds to loading in an elastic manner, i.e. strains induced by applied loads are fully recovered when the load is removed.

Some aggregate particles will suffer breakage in the period leading up to achieving this stability. This is simply a form of stress relief and has no significant detrimental effect providing that the fines produced are of low plasticity. Once in a stable state, the aggregate mass should survive indefinitely given that the loading and environmental conditions do not change and the material does not change with age or moisture cycling, i.e. repeated wetting and drying.

1.4.2 Factors Influencing Pavement Performance

The performance of a pavement depends on six factors, i.e.

- load;
- water content / permeability;
- particle constraint / confinement;
- aggregate properties;
- subgrade properties; and
- surface seal.

1.4.2.1 Load

The current Transit New Zealand design method (Transit New Zealand 1989) relates length of pavement life to the magnitude of the vertical compressive strain at the top of the subgrade, in that large strains correspond to a relatively short pavement life. Large values of subgrade strain are precipitated by high applied stresses, a thin pavement structure and low modulus aggregate layers.

1.4.2.2 Water content/permeability

Water in the aggregate mass may cause particles to become lubricated and thus to reduce the overall strength of the aggregate structure. The ground-water level should therefore be maintained well below the pavement level, and the surface seal coat should always be kept intact to ensure that water cannot enter the pavement.

A low ground-water level is conducive to the formation of negative pore pressures in the aggregate layers if the pore sizes permit capillary rise to occur. This negative pore pressure increases the effective strength of the aggregate and encourages pavement stability.

In the event of the pavement becoming saturated there is the potential for detrimental positive pore water pressures to develop if the permeability of the aggregate is sufficiently low.

1.4.2.3 Particle constraint/confinement

The shear strength of any unbound material is dependent on the confinement under which it exists. High confining stresses provide restraint to the particles which minimises the magnitude of horizontal displacement. High confining stresses are developed by the process of compaction and promoted by the establishment of edge structures such as kerbs and shoulders. The level of compaction must therefore be appropriate for the materials and loading conditions adopted in the design.

1.4.2.4 Aggregate properties

The properties of aggregates can be divided into two categories, those associated with the characteristics of the individual aggregate particles and those associated with the characteristics of the assemblage of particles constructed in a pavement layer.

- **Individual aggregate particles** The factors which influence individual aggregate particles are generally petrological. These include mineralogy, hardness, weathering, the presence of secondary minerals, and grain size and texture. An aggregate which has undergone weathering is likely to contain plastic fines which, in the presence of water, can lubricate the aggregate particles and reduce inter-particle friction. Petrological features, such as cleavage planes, may also result in potential planes of weakness or preferred failure.

- **Assemblage of aggregate particles** The strength of an aggregate mass is dependent on the ability of the particles to transfer load by inter-particle friction and particle interlock. To achieve this load transfer the aggregate must conform to an appropriate grading and receive adequate compaction. The grading should be continuous to promote high density and to provide good support for individual particles. Adjacent aggregate layers should have compatible gradings to prevent migration of particles from one layer to another.

Particle angularity, surface texture and the prevailing state of stress all influence the shear strength and elastic modulus of an assemblage of aggregate particles.

1.4.2.5 Subgrade properties

As the subgrade is situated at the lowest level in the pavement it must ultimately provide the support for the overlying pavement structure and the imposed axle loads. The elastic modulus of the subgrade also influences the magnitude of the compactive effort which can be imparted to the overlying aggregate layers.

The level of compactive effort affects the ability to achieve high densities, and hence the ability to achieve high elastic moduli, in the sub-base and basecourse layers.

If an adequate subgrade modulus cannot be achieved, high vertical compressive subgrade strains occur which lead to a short pavement life. Also, if large vertical deflections occur in the subgrade the pavement layers distort, the surface deforms and ultimately the seal coat cracks. A poor quality subgrade can either be improved by stabilisation, e.g. mechanical compaction, the addition of lime or cement, blending of superior material, or it can be excavated and replaced.

1.4.2.6 Surface seal

The surface seal provides a measure of waterproofing to minimise the potential for water- induced instability. It also provides a wearing course to protect the basecourse from being disturbed by the high stresses imposed on the pavement surface by vehicle tyres.

A typical one coat chipseal is effectively just over one chip thick. It comprises a thin layer of bituminous binder sprayed on to the top surface of the pavement over which stone chips are evenly spread and then pushed into the bitumen with heavy steel wheel or pneumatic tyred rollers.

A two-coat chipseal is used to extend the period before maintenance is necessary because a thicker layer of bitumen is applied. Also, the greater quantity of bitumen makes the seal layer less permeable and therefore protects the pavement structure from the ingress of water.

On more heavily loaded pavements an asphaltic concrete material is used to provide the surfacing. This material is manufactured by mixing a suitably sized and graded aggregate with hot bitumen in a revolving mixer. A layer of the hot material is then laid onto the prepared surface of the basecourse using a paving machine, and is compacted.

1.4.3 Assuring Pavement Performance

In addition to the factors discussed above, the performance of a pavement is enhanced by following appropriate construction procedures, including the adoption of quality assurance procedures during all aspects of the pavement design and construction process. Quality assurance procedures include regular testing of materials and verification of design assumptions at prescribed milestones of construction. They also include the development of contingency strategies which take effect if compliance with prescribed criteria is not achieved.

1.5 Structure of Report

The subject, sealed unbound granular pavements, has been divided into a number of relevant topics for detailed investigation. New Zealand and international standards and specifications, as well as published research into each of these topics is reviewed. The purpose of the review is to identify those factors which may be adopted into the current practice used in New Zealand for unbound granular pavement construction. The ultimate objective is to suggest possible changes to the current design, materials and construction specifications which will result in the production of superior and more cost-effective pavements for New Zealand roads.

The research about sealed unbound granular pavements has been reviewed under two headings: standards and specifications; technical literature.

The reviews of standards and specifications were then considered under the headings:

- Pavement Design
- Pavement Materials
- Pavement Construction

The reviews of technical literature were considered under the headings:

- Pavement Design
- Pavement Materials
- Pavement Construction
- Road Type
- Road Environment

Based on this review of standards and literature, changes and priorities that are recommended to be made to improve standards and specifications used for roading in New Zealand are summarised.

2. REVIEW OF STANDARDS AND SPECIFICATIONS FOR PAVEMENT DESIGN

2.1 Introduction

Pavement design standards from New Zealand, Australia, United Kingdom (UK), United States of America (US), Canada and Europe have been reviewed. The results of the review are presented in this Section 2 of this report.

2.2 New Zealand Standards and Specifications

2.2.1 Introduction

The current New Zealand pavement design method uses an analytical approach. The method is specified in the document entitled *State Highway Pavement Design and Rehabilitation Manual* (Transit New Zealand 1989a). The document was last updated in July 1989 and contains design charts for both flexible and rigid pavements and provides guidelines for pavement rehabilitation and drainage. The background of the design document is described in detail by Dunlop et al. (1983).

This design method is based on the *Shell Design Method* (Claessen et al. 1977) developed in 1977. It assumes that all constituent materials are homogeneous, isotropic and conform to a linear elastic stress/strain response.

The method recognises four categories of pavement, i.e. Group I to Group IV, where Group I corresponds to the highest level of serviceability and Group IV the lowest. Pavement serviceability conditions are quantified using a rating system adopted from the Canadian Good Roads Association which uses a scale of 0 to 100. A poor quality pavement, for example, has a high numerical rating. The serviceability rating criteria appropriate for each pavement category at the beginning (constructed) and end (terminal) of pavement life are presented in Table 2.1.

Table 2.1 Pavement serviceability rating criteria (from Dunlop et al. 1983).

Condition	**Group I**	**Group II**	**Group III**	**Group IV**
Constructed Rating	0-2	0-2	0-5	0-5
Terminal Rating	40	48	56	65

2.2.2 Characterisation of Pavement Layers

Characterisation of the pavement construction materials is related predominantly to the California Bearing Ratio (CBR) of the subgrade, because the design method combines the basecourse and sub-base layers into one layer. It also defines the overall aggregate layer stiffness using the empirical modular ratio relationship presented in Equation 1. The surface seal does not contribute to the pavement's structural capacity.

$$E_{AG} = k\,E_{SG} \qquad \text{Equation 1}$$

where E_{AG} = elastic modulus of combined aggregate layers
E_{SG} = elastic modulus of subgrade
k = modular ratio
= $0.2h_1^{0.45}$ (note $2 =< k =< 4$)
and h_1 = aggregate layer thickness (mm)

The elastic modulus of the subgrade is determined using an empirical relationship developed in South Africa by the National Institute for Transportation and Road Research (NITRR). The relationship is based on the CBR test and is presented in Equations 2 and 3. Note that, while laboratory CBR tests with soaked specimens are generally used, in situ tests may be adopted where subgrade saturation is not expected to occur.

$$E_{SG} = 20\,(\text{CBR})^{0.64} \quad \text{if CBR} < 13 \qquad \text{Equation 2}$$

$$E_{SG} = 8\,(\text{CBR}) \quad \text{if CBR} > 13 \qquad \text{Equation 3}$$

2.2.3 Design Traffic Loading

Assessment of the design traffic loading is achieved using a reference axle, called an *equivalent design axle* (EDA). EDA is defined as a dual wheel axle loaded to 8.2 tonnes. The tyre inflation pressure is 580 kPa. The damaging effect of loaded axles is characterised using the *fourth power law* to determine the design traffic loading in terms of EDA. Table 2.2 shows equivalent axle loads in terms of their damaging effect for a range of axles and configurations.

Table 2.2 Standard axle load for different axle configurations.

Axle Configuration	**Single**		**Tandem**	**Triple**
Wheel Configuration	**Single**	**Dual**	**Dual**	**Dual**
Load (kN)	66	80	142	196

2.2.4 Pavement Life

The Transit 1989 design method assumes that the life of the pavement is determined by the vertical compressive strain at the top of the subgrade. Two criteria are defined for use. These were adopted from the *Shell Design Manual* (Claessen et al. 1977) but have been modified slightly to be more conservative. The criteria are as follows:

$$\epsilon_v = 0.021\ N^{-0.23} \text{ for Groups I + II (premium) pavements} \qquad \text{Equation 4}$$

$$\epsilon_v = 0.025\ N^{-0.23} \text{ for Groups III + IV (2nd grade) pavements} \qquad \text{Equation 5}$$

where ϵ_v = allowable vertical compressive strain at top of subgrade (ϵ)
N = pavement design life (EDA)

Figure 5.1 (in Chapter 5 of this report) provides a comparison of subgrade strain criterion from a selection of roading authorities.

The multi-layer elastic program *BISAR* was used to generate the design charts in the Transit design manual for flexible pavements by considering a large number of pavement profiles and calculating the magnitude of the vertical compressive strain at the top of the subgrade with the model subjected to one half of a standard 8.2 tonne axle load. The design chart used for the design of *premium* unbound aggregate pavements is reproduced from the manual as Figure 2.1.

The design method does not consider other possible causes of pavement distress such as densification of aggregates after construction. Also, it does not differentiate between the basecourse and sub-base materials, which typically are of significantly different quality.

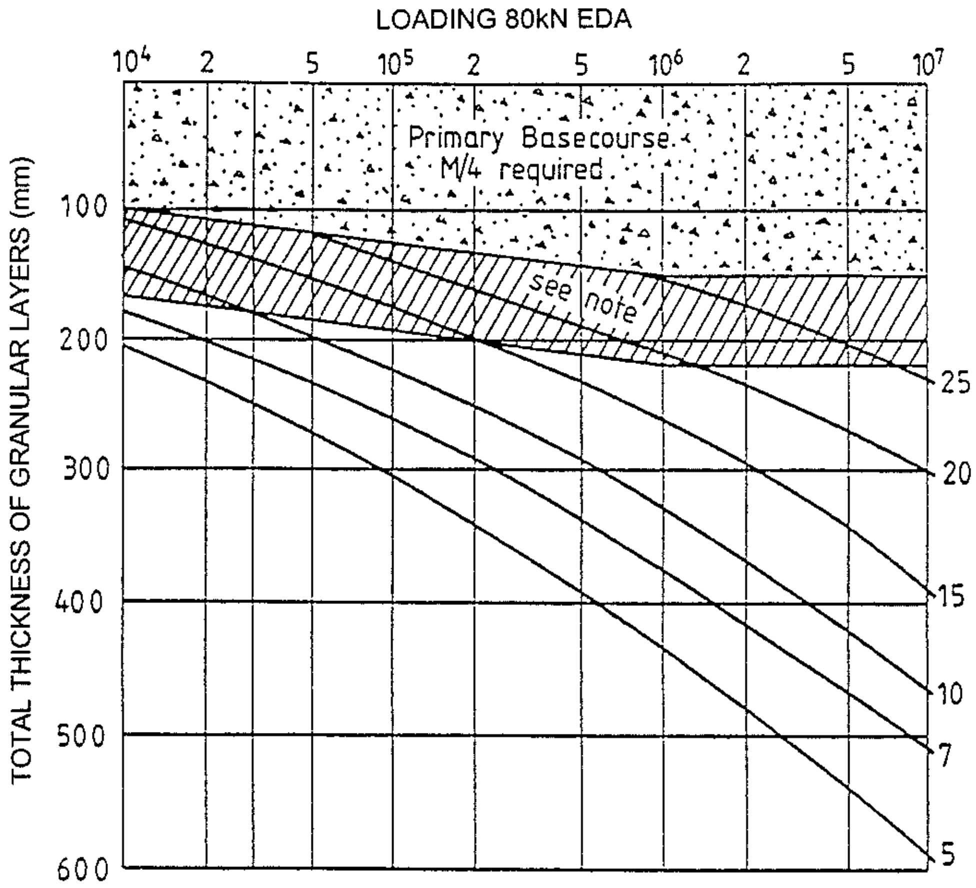
2.3 Australian Standards and Specifications

2.3.1 Design Procedure

The Australian state highway authority, AUSTROADS (formerly NAASRA), has produced an analytical pavement design method which is described in the document entitled *Pavement Design - A Guide to the Structural Design of Road Pavements* (AUSTROADS 1992). A summary of the AUSTROADS pavement design system is presented Figure 2.2.

The design procedure is based on the premise that a new pavement will have an AUSTROADS roughness value of approximately 50 counts per kilometre, whereas at the completion of its design life the terminal roughness will be approximately 150 counts per kilometre. The procedure allows the designer flexibility to control the terminal pavement condition by applying a modified design traffic value which is determined in the AUSTROADS Guide.

Figure 2.1 Design chart for premium flexible pavements with thin surfacings (reproduction of Figure 2 in *Transit Design Manual* (Transit New Zealand 1989a)).



1. The curves give minimum cover required above the sub-grade. Allowance must be made for loss of material through penetration into soft subgrades and construction tolerances. Materials for use in hatched zone must satisfy permeability requirements SE >40.

2. If subgrade CBR exceeds 25 it should be taken as 25.

3. Even when CBR is high, always take for the design no less than the full primary M/4 basecourse indicated for the EDA involved. For example, if CBR = 25 and EDA = 10^5 the pavement shall consist of 125mm M/4 basecourse.

4. Where a sub-base thickness less than 50mm would be indicated, use full depth M/4. For example, if CBR = 25 and EDA = 1.5×10^6 the pavement shall consist of 160mm M/4 basecourse.

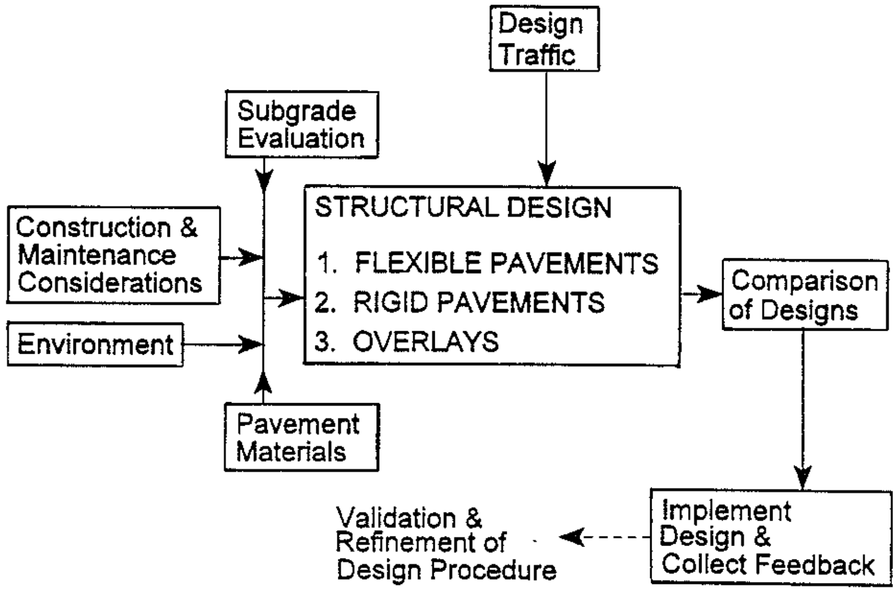


Figure 2.2 AUSTROADS design system for new pavements (reproduced from AUSTROADS 1992).

2.3.2 Characterisation of Pavement Layers

The AUSTROADS Guide allows the use of in situ tests to determine subgrade support using an existing pavement if it is near to the pavement proposed for construction, and the soil and moisture conditions at both locations are similar. The existing pavement must be sealed and have been in place for at least two years. Testing may be carried out using the in situ CBR or cone penetrometer (static or dynamic) methods. The results are analysed statistically to identify the 10 %ile value (10 %ile = mean − 1.3 x SD). When comparison with existing pavements is not possible, specimens of subgrade soil are prepared to the design density and moisture content conditions, and subjected to laboratory CBR tests.

The subgrade material is characterised as having an elastic stress/strain response and being cross-anisotropic with a vertical to horizontal elastic modulus ratio of two. A maximum vertical elastic modulus of 150 MPa is recommended with Poisson's ratios of 0.45 for cohesive materials and 0.35 for non-cohesive materials. The vertical elastic modulus of the subgrade (E_{SG}) is calculated using the following empirical relationship:

$$E_{SG} = 10\,(CBR) \qquad \text{Equation 6}$$

AUSTROADS allows the use of presumptive CBR values for lightly trafficked pavements and when no other information is available.

The analytical pavement design method is based on limiting the compressive strain occurring at the top of the subgrade. The AUSTROADS subgrade criterion is as follows:

$$\epsilon_v = 8511\ N^{-0.14} \qquad \text{Equation 7}$$

where ϵ_v = allowable vertical compressive strain at top of subgrade ($\mu\epsilon$)
N = pavement design life (equivalent standard axles, ESA)

Characterisation of the unbound granular pavement layers is carried out recognising the nonlinear stress/strain response of these materials, i.e. the elastic modulus is dependent on the prevailing state of stress. AUSTROADS recommends the use of repeated load triaxial tests at appropriate stress levels to determine accurate resilient modulus values.

If this is not possible, AUSTROADS suggests that the elastic modulus be determined by a process of back calculation from deflection tests or the selection of presumptive values.

The non-linear elastic behaviour of the granular materials is modelled by breaking down the granular layers into sub-layers with a thickness of 50 mm to 150 mm. A modular ratio is then used to calculate the elastic modulus of each sub-layer.

2.3.3 Traffic Loading

In assessing the design traffic loading, the AUSTROADS design method considers the number of axle passes, the axle loadings, and the axle configurations. An axle equivalence calculation is carried out for each vehicle type (excluding passenger cars and light commercial vehicles) to determine the total number of *equivalent standard axles* (ESA) over the design life of the pavement. The equivalence relationship is calculated from the ratio of the load on the axle group to the appropriate standard load, which is then raised to a power of four. The standard loads for each axle configuration are presented in Table 2.3.

Table 2.3 Standard axle loads for a range of axle configurations.

Axle Configuration	**Single**		**Tandem**	**Triple**
Wheel Configuration	**Single**	**Dual**	**Dual**	**Dual**
Load (kN)	53	80	135	181

2.3.4 Design Procedure

The AUSTROADS Guide does not provide design charts for unbound granular pavements but it does give a detailed description of the required design steps. Once the material properties and design traffic parameters have been established, the proposed pavement profile is analysed using elastic theory to determine the magnitude of the vertical compressive strain at the top of the subgrade. This is then compared with the allowable subgrade strain criterion. The pavement geometry is adjusted until the optimal solution is obtained. The multi-layer elastic computer program *CIRCLY* is used to analyse a range of pavement conditions allowing a series of design charts to be generated.

As an alternative to the analytical design procedure, the AUSTROADS guide provides a design chart for unbound granular pavements with thin bituminous surfacing (or chipseals). The chart, which is based on empirical relationships obtained from experience of Australian pavement performance, is presented in Figure 2.3.

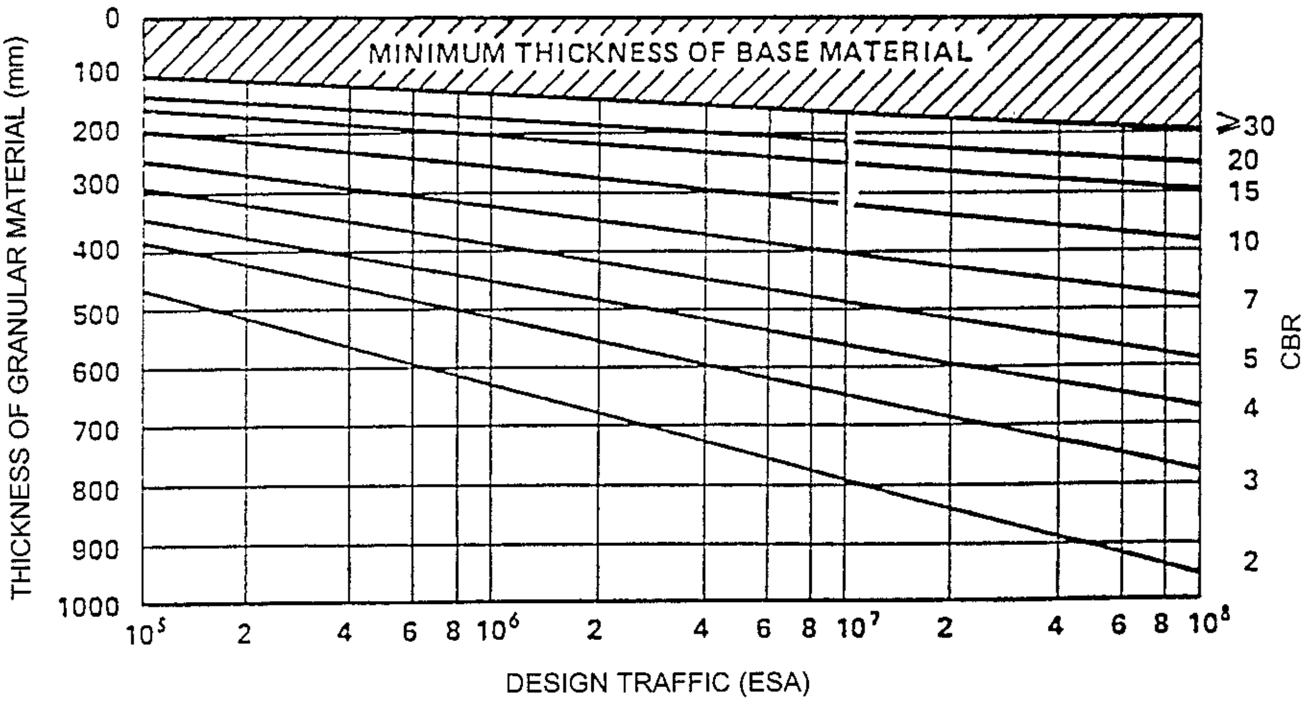


Figure 2.3 Design chart for granular pavements with sprayed seal surface (reproduced from AUSTROADS 1992).

2.4 Australian State Standards Supplements

Although the AUSTROADS Guide is the primary design document for all the Australian states, each state has prepared a supplement or adaptation to allow for local conditions and experience. The main adaptations are described in the following Sections 2.4.1 - 2.4.5 of this chapter.

2.4.1 New South Wales

The Roads and Traffic Authority (RTA) in New South Wales carries out pavement design in accordance with the AUSTROADS design guide. The RTA has also produced a supplement document which adapts the AUSTROADS method to the specific conditions and procedures experienced in the state of New South Wales.

The RTA allows the subgrade CBR of an existing unbound aggregate pavement to be determined by analysing the deflected shape of a pavement using elastic theory. The Benkelman beam apparatus is used to measure the deflected shape at offset distances of 0 mm, 600 mm and 900 mm. The subgrade CBR is determined from a chart using an intermediate parameter termed *spreadability*.

The AUSTROADS design guide adopts tyre pressures of 550 to 700 kPa, whereas the RTA recommends that a minimum of 700 kPa is more appropriate. The basis for this difference in loading configuration is that improvements in tyre construction technology allow greater inflation pressures with the benefit of improved fuel economy.

The RTA also stipulates that a subgrade with a CBR of 2 or less must be improved by stabilisation, sub-excavation or placement of geotextile to provide a stable platform for the compaction of sub-base material.

2.4.2 Victoria

The Roads Corporation of Victoria (VicRoads) has produced a supplement to the AUSTROADS design guide entitled *VicRoads Guide to Pavement Design* (VicRoads 1993a). Although the document is described as a supplement, it could stand alone as a design method in its own right.

The VicRoads design method is based on a 50% reliability level, i.e. there is a 50% probability that a given pavement will fail prematurely because mean values of material parameters have been used. This is considered to be not acceptable and the design of pavements incorporating bound layers are subject to a reliability factor of 3 to 6 depending on the type of road. The design traffic is multiplied by the reliability factor and the pavement is designed using the modified design traffic value so that the probability of premature failure is significantly less than the original 50%. The design chart used for unbound aggregate pavements with a thin surface seal is reported to have a built-in reliability of 95% based on the observations of pavement performance in the state of Victoria.

The limiting vertical compressive subgrade strain adopted by VicRoads is presented in the relationship shown in Equation 8. Note that this criterion is more conservative than that of the AUSTROADS design guide by approximately 25%.

$$\epsilon_v = 10520\, N^{-0.14} \qquad \text{Equation 8}$$

where ϵ_v = allowable vertical compressive strain at top of subgrade ($\mu\epsilon$)
N = pavement design life (ESA)

The VicRoads design guide provides a chart specifically for the design of granular pavements with a sprayed seal surface (Figure 2.4). The chart has been developed using a tyre pressure of 700 kPa. The hatched area of the chart refers to poor subgrade conditions (CBR < 3) which usually require some form of improvement to enable construction to proceed. The following subgrade improvement techniques are suggested:

- draining of wet areas;
- excavation and replacement of soft material;
- construction of a gravel fill platform;
- stabilisation with lime or cement; or
- use of geotextiles.

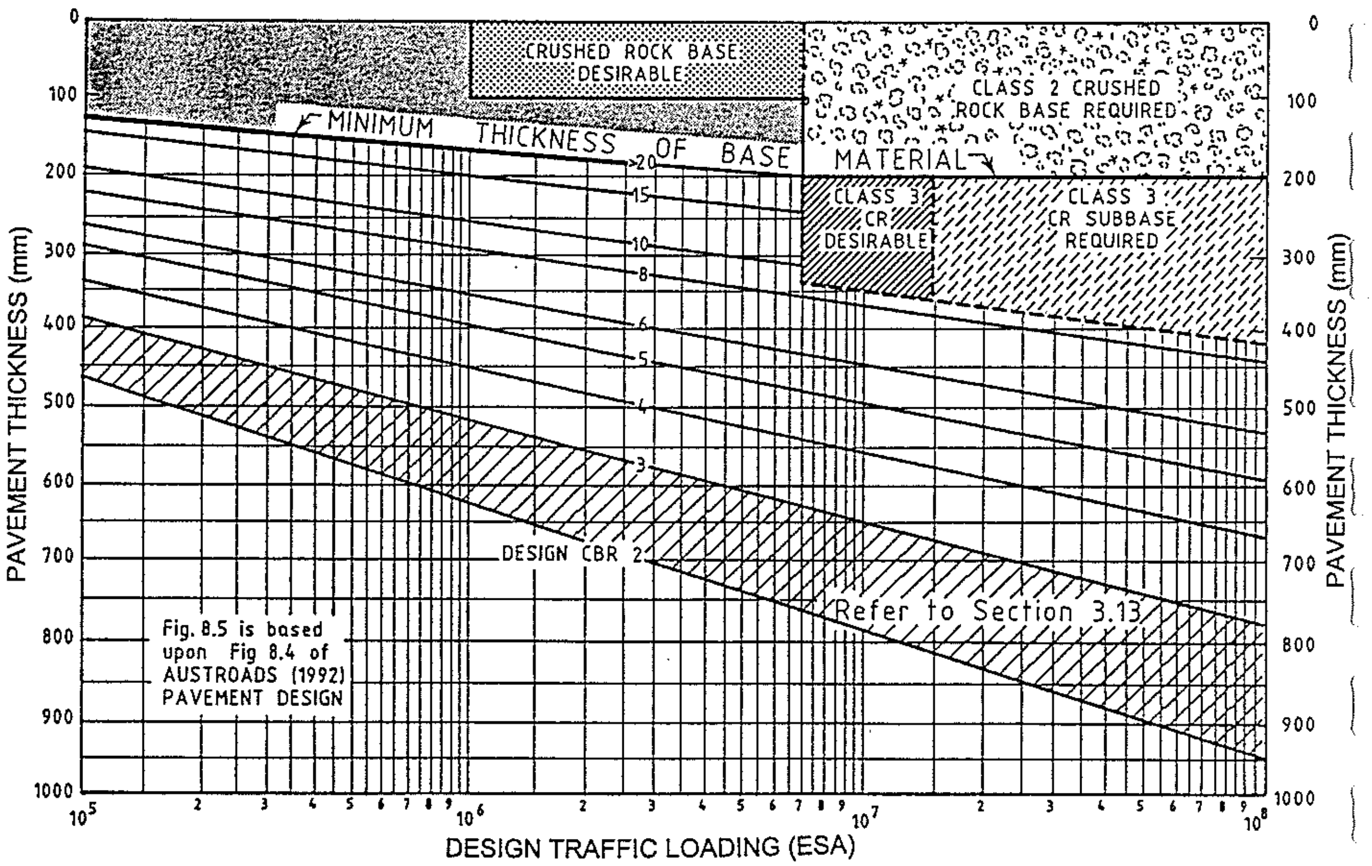


Figure 2.4 Design chart for granular pavements with sprayed seal surface (reproduced from VicRoads 1993a).

2.4.3 Queensland

The Queensland Department of Transport has produced its own pavement design method as described in the document entitled *Pavement Design Manual* (Queensland Department of Transport 1990a). The design methodology adopted in the manual closely parallels the analytical approach of the AUSTROADS Guide (Angell 1988). The AUSTROADS pavement design system (Figure 2.2) is retained but other aspects of design have been altered to take account of recent research findings, regional climatic conditions, and the experience of local authorities and consulting engineers.

The most significant variation from the AUSTROADS Guide is the relationship used between allowable subgrade strain and design traffic. The Queensland method adopts two design standards, a *first standard* and a *second standard*, where the latter allows for lower quality pavements. The subgrade strain criteria for the two pavement design standards are given in Equations 9 and 10. Note that, for the first standard pavement, the Queensland subgrade strain criterion is less conservative than the AUSTROADS criterion up to an ESA value of approximately 3×10^5, and more conservative above this value. Since most first standard pavements are highly trafficked (i.e. design traffic considerably greater than 3×10^5 ESA), the criterion is generally more conservative.

$$\epsilon_v = 34000 N^{-0.25} \text{ for first standard pavement design} \qquad \text{Equation 9}$$

$$\epsilon_v = 22000 N^{-0.20} \text{ for second standard pavement design} \qquad \text{Equation 10}$$

where ϵ_v = allowable vertical compressive strain at top of subgrade ($\mu\epsilon$)
N = pavement design life (ESA)

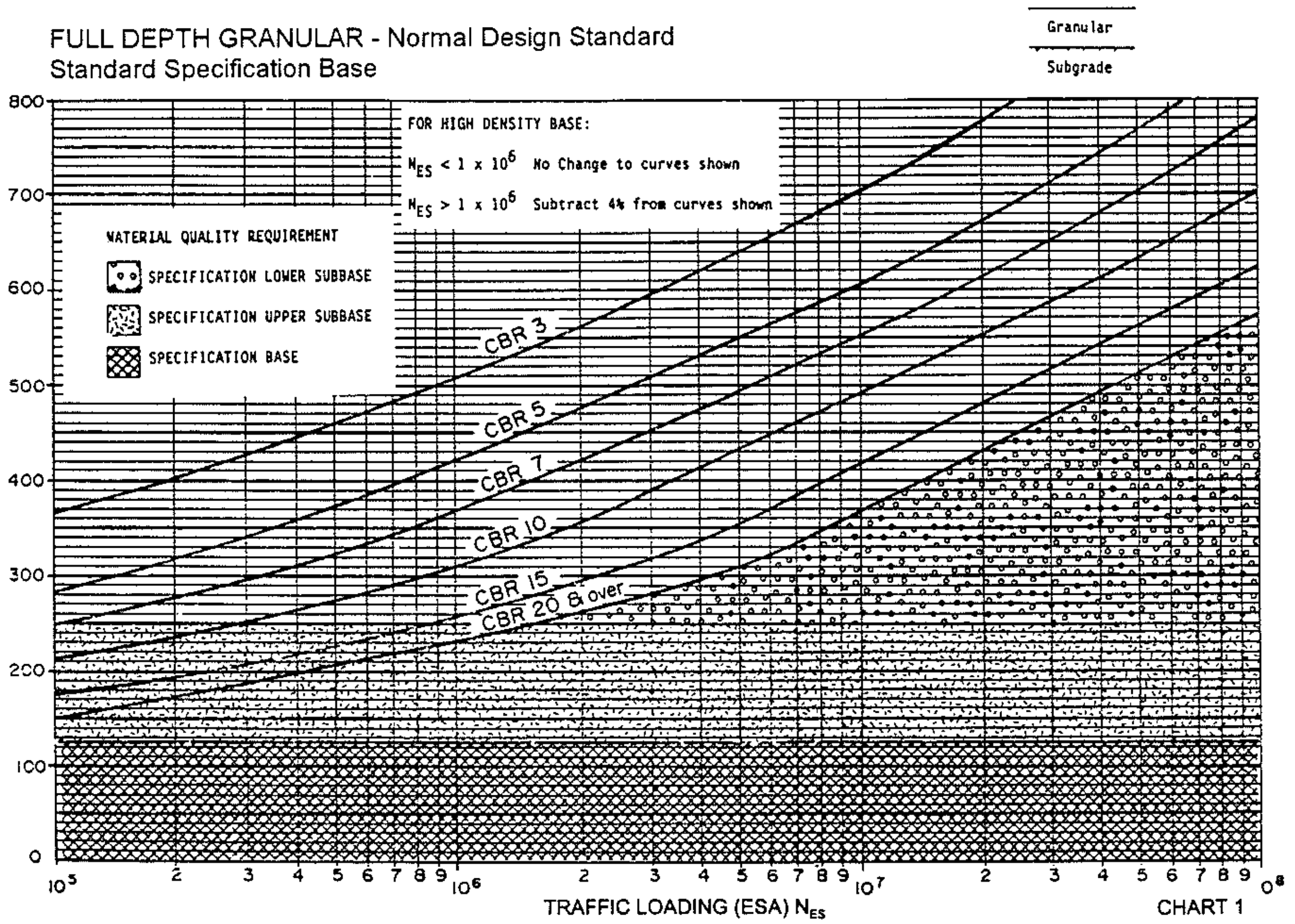
Figure 5.1 (in Chapter 5 of this report) provides a comparison of subgrade strain criteria from a selection of roading authorities.

The Queensland Department of Transport design manual contains 226 design charts produced for several combinations of pavement configuration and material properties. A computer program called *PADS* has also been developed. This can be used in place of the charts or for the design of pavements under unusual conditions. In the design of unbound aggregate pavements with a thin surface seal, the design charts specify the thicknesses required for the basecourse, upper sub-base and lower sub-base layers.

A single design chart is provided for *specification materials* (Figure 2.5), while three charts are provided for use with other materials with prescribed minimum CBR requirements. These include basecourse CBR values ranging from 40 to 50 %, upper sub-base CBR values ranging from 30 to 40 % and lower sub-base CBR values ranging from 25 to 35 %. The design chart designated for specification materials allows a 4% reduction in pavement thickness for traffic loadings in excess of 10^6 ESA if a *high density* condition is achieved, i.e. the aggregate layers are compacted to 102% of the standard compaction test density.

Each design chart in the Queensland design manual provides an indication of the distress mode which controls the pavement design life. This is beneficial as it provides the designer with information regarding the expected pavement configuration at the end of its design life.

Figure 2.5 Design chart for granular pavements with sprayed seal surface (reproduced from Queensland Department of Transport 1990a).



2.4.4 South Australia

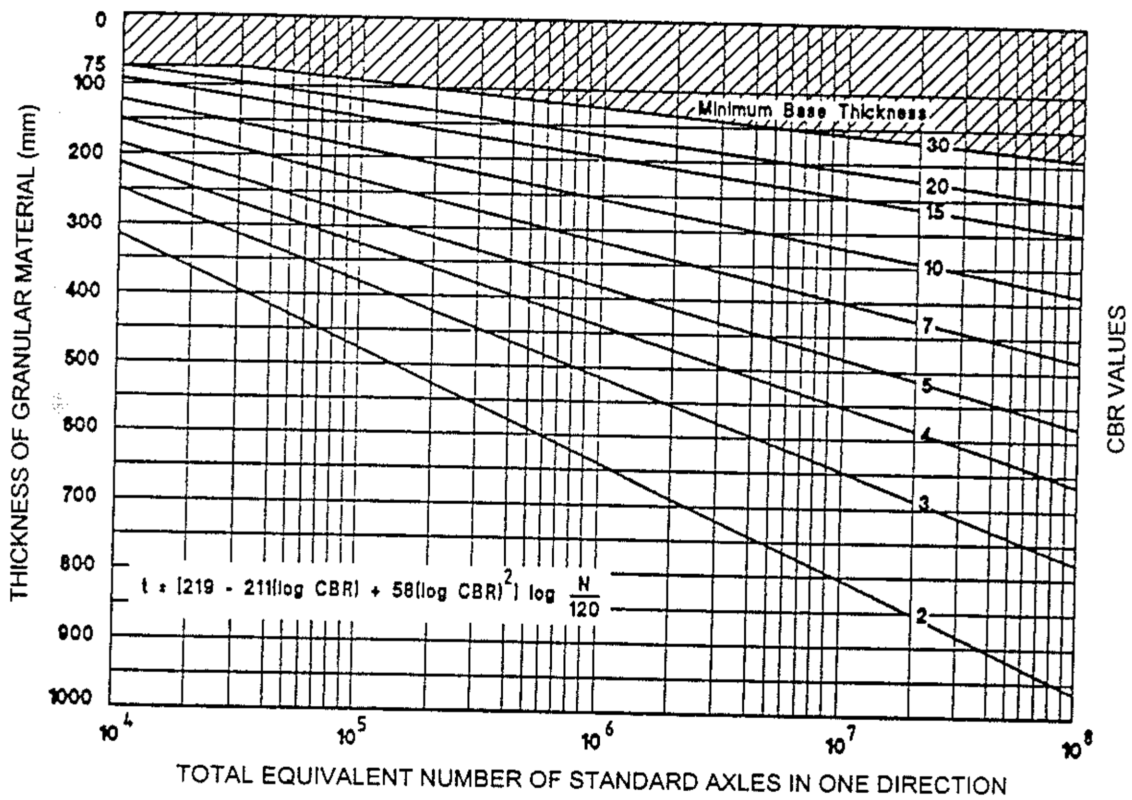
The South Australia Department of Road Transport utilises the AUSTROADS pavement design guide, and has no supplementary clauses that specifically relate to the South Australian conditions. They do however go to significant effort to establish aggregate moduli specific to each material source.

2.4.5 Western Australia

The design procedure for flexible pavements in Western Australia is described in their *Procedure for the Design of Flexible Pavements* (Main Roads Department Western Australia 1988). The note specifically discusses the design of pavements comprising unbound aggregate layers surfaced with a bituminous seal or up to 50 mm of asphaltic concrete. The procedures are based on the AUSTROADS design guide but have been adapted to take into account the Western Australian conditions.

The document provides a design chart (Figure 2.6) which is used to determine the total thickness of granular material over a subgrade with a given CBR. Unbound aggregate pavements must have a single basecourse layer with a minimum thickness of 75 mm and a maximum thickness of 195 mm depending on the design traffic loading. Any number of sub-base layers are permitted as long as the minimum granular cover corresponding to the sub-base material CBR is provided.

Figure 2.6 Design chart for granular pavements with sprayed seal surface (reproduced from Main Roads Department Western Australia 1988).



2.5 United Kingdom Standards and Specifications

2.5.1 Introduction

Pavement design in the UK is generally based on the procedure described in their document *TRRL LR1132, The Structural Design of Bituminous Roads* (Powell et al. 1984). This method adopts an analytical approach which is reinforced with research findings and empirical relationships.

The method is used predominantly for the design of pavements incorporating a bound base and/or surface layer, e.g. asphaltic concrete. However it can also be used for the design of unbound granular pavements. *TRRL LR1132* is a revision of the method of the preceding *TRRL Road Note 29, A Guide to the Structural Design of Pavements for New Roads*, which was published in 1970. *TRRL LR1132* is based on a terminal pavement condition corresponding to a 10 mm rut depth. This is relatively conservative given that the terminal condition adopted in *TRRL Road Note 29* was a 20 mm rut depth.

TRRL LR1132 generally specifies multi-layer pavement structures comprising a subgrade, capping layer, granular sub-base, granular or lean mix concrete base, and a thick asphaltic concrete surfacing. The method prescribes the placement of a capping layer to protect the subgrade in areas of poor support. This capping layer is typically a local gravel material, the thickness of which depends on the CBR of the subgrade. The capping layer is not intended to contribute in any significant way to the structural integrity of the completed pavement.

A second pavement design specification which originates from the UK is the method of *TRRL Road Note 31, A Guide to the Structural Design of Bitumen-surfaced Roads in Tropical and Sub-tropical Countries* (TRRL 1977) which is specifically intended for use in tropical and sub-tropical countries. The method is empirically based and provides the designer with a simple chart, from which to chose a suitable sub-base thickness depending on the design traffic and the subgrade CBR.

2.5.2 TRRL LR1132 Design Charts

TRRL LR1132 provides design charts which specify the required sub-base thickness based upon the subgrade CBR and the number of standard axle passes expected during the pavement's construction period. In the development of the design charts the stiffness of each lift of compacted sub-base was assumed to be three times that of the underlying material to a limit of 150 MPa (Salter 1988). If an unbound granular base layer is desired, *TRRL LR1132* contains design charts for *wet-mix macadam* basecourse. The thick asphaltic concrete surface layer specified in the design charts is simply substituted by an increased thickness of unbound aggregate in the proportion of 2:1, i.e. 100 mm of wet-mix unbound aggregate is considered to be equivalent to 50 mm of asphaltic concrete.

2.5.2.1 Subgrade characterisation

The *TRRL LR1132* document makes recommendations of appropriate CBR values for a range of subgrade material types and ground-water conditions. These values range from 1% for a saturated silt to 60% for a well graded sandy gravel. The empirical relationship in Equation 11 is used to calculate subgrade elastic modulus from the CBR test result:

$$E_{SG} = 17.6\,(CBR)^{0.64} \qquad \text{Equation 11}$$

TRRL LR1132 uses the widely adopted fourth power law to characterise the design traffic loading. This relates the damaging effect of each heavy commercial vehicle axle to a standard 8.2 tonne single axle with dual wheels.

2.5.2.2 Pavement life characterisation

With respect to unbound granular pavements, the basis of the *TRRL LR1132* design method involves limiting the vertical compressive strain occurring at the top of the subgrade to the criterion in Equation 12.

$$\epsilon_v = 22000\,N^{-0.20} \qquad \text{Equation 12}$$

where ϵ_v = allowable vertical compressive strain at top of subgrade ($\mu\epsilon$)
N = pavement design life (ESA)

2.5.3 TRRL Road Note 31

For tropical and sub-tropical conditions, *TRRL Road Note 31* (1977) specifies that an unbound aggregate pavement to carry design traffic values less than 500,000 standard axles (ESA) should have a 150 mm-thick base layer with a two-coat surface seal. Pavements carrying greater than 500,000 standard axles are specified to have a granular base thickness of 200 mm with a two-coat surface seal or 150 mm of base with 50 mm of bituminous surfacing (Carter 1983).

2.6 United States Standards and Specifications

2.6.1 Introduction

Pavement design in the US is generally based on the American Association of State Highway and Transportation Officials (AASHTO) (Salter 1988) pavement design method published in 1986. The method is purely empirical, being based on the American Association of State Highway Officials (AASHO) Road Test performed in the 1950s. The method incorporates a number of empirical equations which are solved using nomographs containing the various design parameters. The ultimate parameter in the process is the so-called *structural number* which represents the structural capacity of the pavement. Most flexible pavements designed in the US incorporate an asphaltic concrete surface course of at least 50 mm thickness. Although these pavements are outside the scope of the present study, a brief discussion of the design procedures is provided in the following paragraphs.

2.6.2 Design Traffic

The design traffic is characterised using the fourth power law for pavement damage and incorporates a standard axle load of 18,000 lb (8.2 tonnes). The design traffic value is also dependent on the structural number as shown by the axle load equivalence factors outlined in Table 2.4.

Table 2.4 Axle load equivalence factors (from Salter 1988).

Axle Load [kips]	Structural Number					
	1	2	3	4	5	6
2	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
4	0.002	0.003	0.002	0.002	0.002	0.002
6	0.01	0.01	0.01	0.01	0.01	0.01
8	0.03	0.04	0.04	0.03	0.03	0.03
10	0.08	0.08	0.09	0.08	0.08	0.08
12	0.16	0.18	0.19	0.18	0.17	0.17
14	0.32	0.34	0.35	0.35	0.34	0.33
16	0.59	0.60	0.61	0.61	0.60	0.60
18	1.00	1.00	1.00	1.00	1.00	1.00
20	1.61	1.59	0.56	1.55	0.57	1.60
22	2.49	2.44	2.35	2.31	2.35	2.41
24	3.71	3.62	3.43	3.33	3.40	3.51
26	5.36	5.21	4.88	4.68	4.77	4.96
28	7.54	7.31	6.78	6.42	6.52	6.83
30	10.38	10.03	9.24	8.65	8.73	9.17
32	14.00	13.51	12.37	11.46	11.48	12.17
34	18.55	17.87	16.30	14.97	14.87	15.63
36	24.20	23.30	21.16	19.28	19.02	19.93
38	31.14	29.95	27.12	24.55	24.03	25.10
40	39.57	38.02	34.34	30.92	30.04	31.25

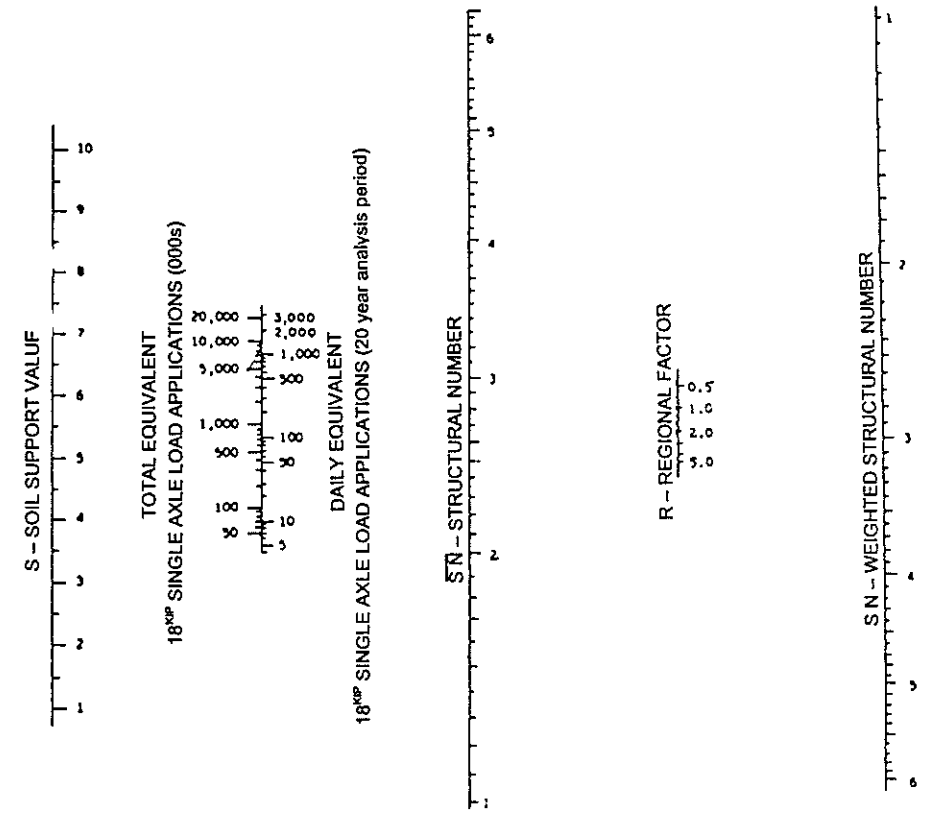
2.6.3 Subgrade

The strength of the pavement subgrade is assigned a parameter known as the *soil support value* (S). Soil support values are not exclusively related to any one test method, but several US roading authorities have determined correlations between the S parameter and their preferred subgrade test, e.g. the Utah Department of Highways uses the CBR test and has established that S is proportional to the logarithm of the CBR (Carter 1983). The flexibility allowed in the determination of S means that more sophisticated test methods such as the repeated load triaxial test can be utilised.

2.6.4 Design Charts

Design nomographs are provided in the AASHTO method for flexible pavements with terminal *present serviceability index* (PSI) values of 2.0 and 2.5 (Figures 2.7a and 2.7b respectively). The nomograph is entered at the left edge with the appropriate S-value and a straight line is extended through the scale of daily equivalent standard axle load applications to obtain the structural number.

(a)



(b)

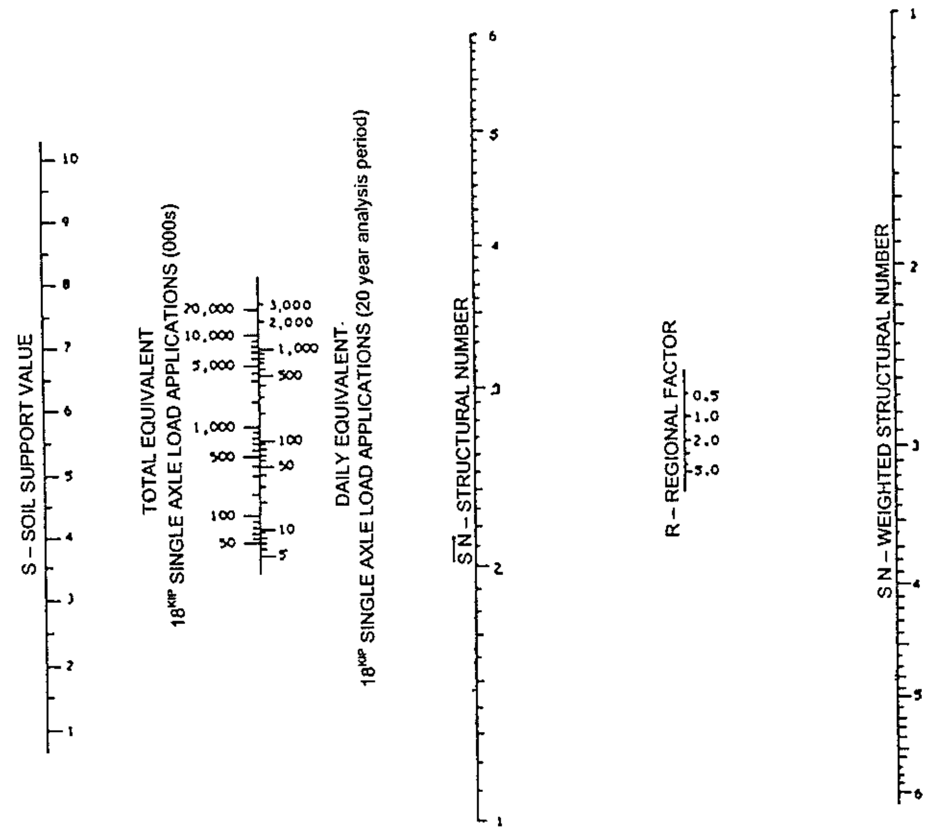


Figure 2.7 AASHTO pavement design nomographs for terminal present serviceability index (PSI) values of (a) 2.0 and (b) 2.5 (reproduced from Salter 1988).

A *regional factor* is introduced to take into account the effect of local conditions on the performance of the pavement. This is a subjective number which is dependent on a number of considerations, e.g. topography, rainfall, frost penetration, frequency of freeze-thaw cycles, gradient, vehicle manoeuvres and the similarity between the proposed site and the AASHO Road Test site (Carter 1983).

Establishment of the regional factor allows the designer to use the nomograph to obtain the *weighted structural number* (SN_W). This number is related to the thickness of the pavement layers and the nature of the constituent materials using the general Equation 13.

$$SN_W = a_1D_1 + a_2D_2 + a_3D_3 + \ldots.. \qquad \text{Equation 13}$$

where a_i = Layer coefficient for layer i
D_i = Layer thickness for layer i (mm)

Minimum thicknesses of 50 mm, 100 mm, and 100 mm are specified for the wearing course, basecourse, and sub-base layers respectively.

The *layer coefficients* are equivalence factors which allow for the difference in stiffness of different construction materials. These values are subjective and can only be based on local knowledge of material sources and experience of pavement performance. Typical layer coefficient values are presented in Table 2.5.

Table 2.5 Typical layer coefficient values (after Carter 1983).

Material	**Layer Coefficient**
Asphaltic concrete surfacing	0.018
Hard crushed rock basecourse	0.006
Well graded sandy gravel sub-base	0.004

2.6.5 State Specifications

Although the AASHTO pavement design method has been adopted by a number of states in the US, the California Department of Transportation (Caltrans) has preferred to develop its own method, details of which are described in the document entitled *Caltrans Highway Design Manual* (Caltrans 1990). The Caltrans design method is based on a combination of theory, test track studies, experimental sections, and research on materials, methods and equipment.

The Caltrans design method differs slightly from other common design methods in the way it treats the design traffic loading. Appropriate *Equivalent Single Axle Loads* (ESAL) are calculated, based on a standard 18 kip (8.2 tonne) axle, and summed over the design life of the pavement. This is then converted to a *traffic index* (TI) using the relationship in Equation 14:

$$TI = 9\,(ESAL / 10^6)^{0.119} \qquad \text{Equation 14}$$

The thickness of each pavement layer is calculated in terms of a *gravel equivalence* (GE) thickness. The GE is calculated using the relationship in Equation 15 based on the TI and the *stabilometer resistance value* (R) for the underlying pavement layer material:

$$GE = 0.0032\,(TI)\,(100 - R) \qquad \text{Equation 15}$$

where GE = gravel equivalent (in units of feet)
TI = traffic index
R = resistance value of underlying material

The overall GE required for a pavement to be supported on a subgrade of a given R value is calculated using the relationship presented in Equation 15. The actual layer thicknesses are then determined from the top of the pavement down using a table of *gravel factors* (G_f) which account for the performance of different construction materials when compared to gravel.

The requirement for adequate drainage is stressed in the Caltrans design method. This is achieved by constructing a highly permeable asphalt layer or cement-treated layer at an appropriate elevation in the pavement. This may be placed on a bituminous prime coat which acts as a membrane to exclude infiltrated water from the underlying layers. Recommended layer thicknesses are 75 mm and 105 mm for the asphalt materials and cement-treated materials respectively. Water discharged from the permeable layer must be fed into a collector system which can cope with the stormwater infiltration rate relevant to the pavement location.

2.7 Canadian Standards and Specifications

The Canadian practice of pavement design and construction is set out in the Roads and Transportation Association of Canada (RTAC) document entitled *Pavement Management Guide* (RTAC 1977). The pavement design philosophy applied in Canada is similar to that used in the US in that the pavements are generally designed with a surface course of at least 50 mm of asphaltic concrete. While these pavements are outside the scope of this study, a brief discussion of the design methodologies is presented in this Section 2.7 of this report.

In Canada, the *Riding Comfort Index* (RCI) is used as the measure of pavement performance. The RCI is a subjective, panel-type pavement rating system which uses a scale of zero to ten, where zero is very poor and ten is very good. Suggested minimum RCI values are 3.5 for minor rural roads, 4.5 for secondary rural highways, and 5.5 for freeways and primary highways.

The RTAC Pavement Management Guide describes three approaches to pavement design. These incorporate designs that are based on:

- experience;
- empirical relationships using pavement deflection data; and,
- theoretical stress/strain/deflection analyses.

Considerable variation in pavement structure exists between the Canadian provinces because of the nature of the pavement construction materials and the extremes of climatic conditions. For example, the experience-based design for a pavement on a subgrade of CBR 5%, with a design life of 5×10^5 ESA, ranges from a total pavement thickness of 330 mm in Alberta to 745 mm in New Brunswick. Similarly, the empirical and theoretical design methodologies vary significantly from province to province.

Pavement loadings are treated as equivalent numbers of standard 80 kN single axles and the RTAC Guide provides both graphical (Figure 2.8) and nomograph (Figure 2.9) procedures for the determination of load equivalence values. The designer generally breaks down the spectrum of axle loads into 9 kN intervals and obtains the load equivalence value for each load category from the plot in Figure 2.9. The total design traffic is then taken as the sum of the number of axles multiplied by the load equivalence factor for each load category. The nomograph method provides an estimate of the design traffic loading when limited axle weight data are available.

The RTAC Guide provides a list of material equivalences to allow for the substitution of asphaltic concrete with an equivalent thickness of unbound aggregate. It recommends that a minimum thickness of asphaltic concrete between 50 mm to 125 mm (depending on the traffic loading) is provided as a surface course. The equivalence values vary from province to province as presented in Table 2.6.

Figure 2.8 Load equivalency values for different axle configurations (reproduced from RTAC 1977).

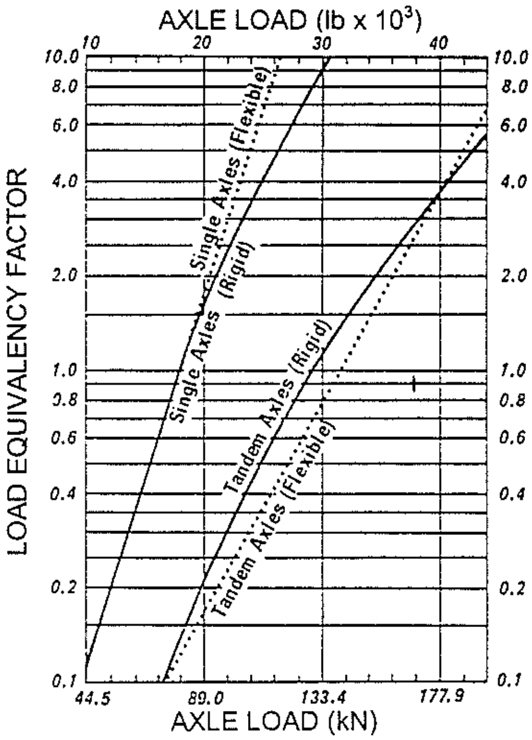


Table 2.6 Equivalence values (relative to 1 mm of asphaltic concrete) for pavement construction materials (adopted by the Canadian provincial roading authorities (from RTAC 1977).)

Province	**Equivalent Material**	**Equivalent Thickness (mm)**
Alberta	Crushed gravel	2.25
British Columbia	Gravel base Sandy gravel sub-base	2.00 2.50
Manitoba	Gravel base	2.00
New Brunswick	Crushed rock Gravel sub-base	2.00 3.00
Newfoundland	Graded crushed rock Gravel sub-base Sandy gravel	2.50 3.00 4.00
Ontario	Granular base Granular sub-base	2.00 3.00
Quebec	Crushed rock base Gravel base/sub-base Sand sub-base	2.00 2.50 5.00
Saskatchewan	Data not available	

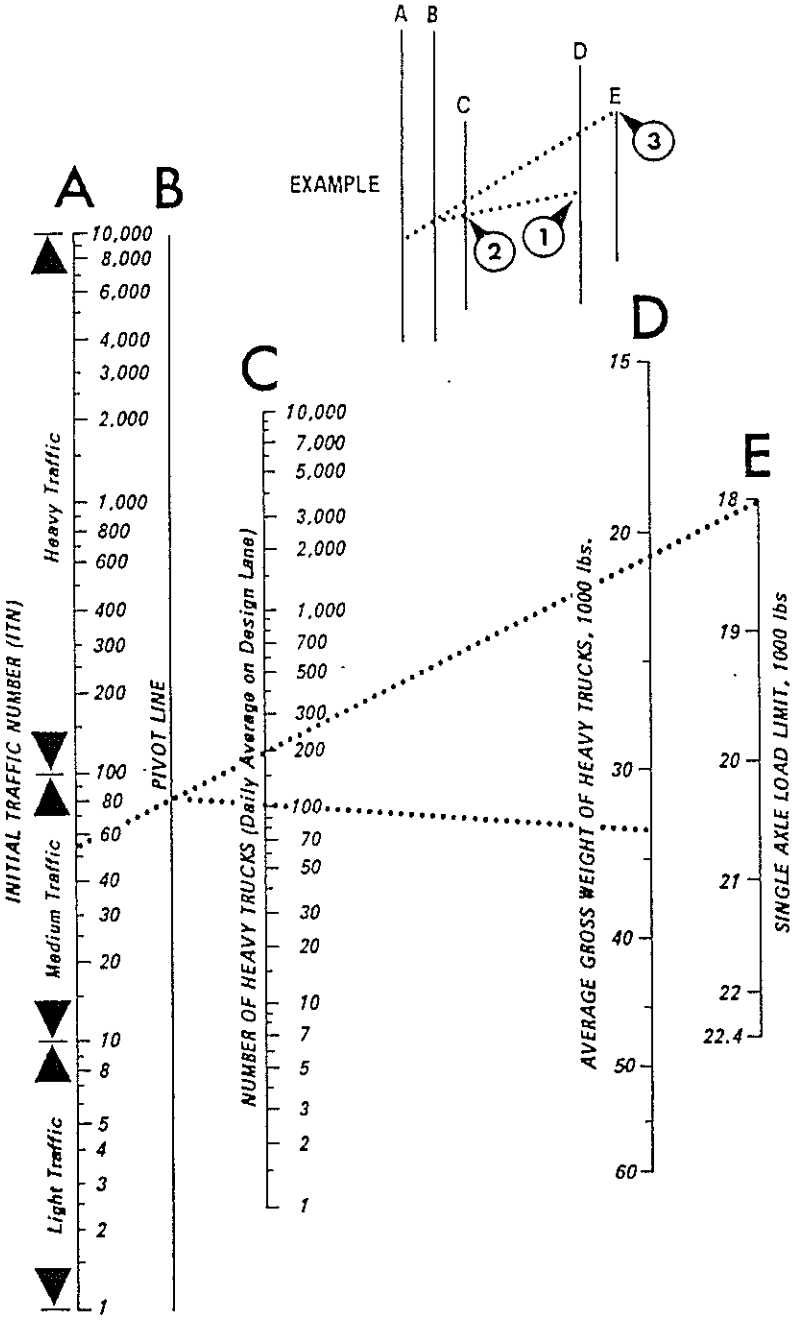


Figure 2.9. Nomograph for determining ESA values (reproduced from RTAC 1977).

2.8 European Standards and Specifications

Pavement design methodologies from a small number of European countries have been identified in the international technical literature. The method of pavement design used in Germany is empirical but is significantly different from the AASHTO method. The German method uses a standard set of pavement structures, one or more of which will be appropriate for a given set of environmental and material parameters (Sweere 1989). The pavement is categorised into one of six classifications depending on the design traffic. A standard design chart is then used for the appropriate pavement structure, e.g. unbound aggregate, structural asphaltic concrete (AC), full depth AC, rigid, etc.

The method relies heavily on the use of in situ testing to ensure that the design assumptions are met at each stage of construction. Initially the subgrade is tested to ensure that its support exceeds the minimum criteria specified in the design manual. If it does not measure up, it must be improved using an appropriate stabilisation technique. Similarly, the sub-base and base layers are tested using the plate bearing test to ensure that they comply with the specified bearing capacity standard. If they do not comply, extra compaction or an increased layer thickness is adopted until the standard is met. The method also allows for a reduction in layer thickness if the bearing capacity of the underlying layer is significantly greater than the value assumed in the design.

Similar approaches are used in France, Spain and Italy. Their pavement design methods comprise a catalogue of standard pavement structures which are selected on the basis of traffic loading, subgrade conditions and material parameters.

3. REVIEW OF STANDARDS AND SPECIFICATIONS FOR PAVEMENT MATERIALS

3.1 Introduction

Standards and specifications for materials used in the construction of sealed unbound pavements that are used in New Zealand, Australia, United Kingdom (UK), United States of America (US), Canada and Europe have been reviewed. The results of the review are presented in this Section 3.

3.2 New Zealand Standards and Specifications

In New Zealand, standards for pavement construction materials are prescribed in the Transit New Zealand specifications. The document titles which are particularly relevant to sealed unbound granular pavements are listed in Table 3.1. Some specifications have explanatory notes to assist the reader.

Table 3.1 Transit New Zealand construction material specification titles relevant to sealed unbound granular pavements.

Specification	Subject
TNZ M/3 (1986)	Sub-base aggregate
TNZ M/4 (1985)	Crushed basecourse aggregates
TNZ M/5 (1984-93)	Regional aggregates
TNZ M/6 (1993)	Sealing chips

3.1.1 Specification TNZ M/3

Specification TNZ M/3 (Notes 1986; Transit New Zealand 1986c) describes a recommended format for the specification of sub-base materials. A rigid specification is not provided since the range of materials suitable for use as sub-base is very large. The TNZ M/3 (Notes 1986) document presents information on the following items:

- material composition and testing;
- grading requirements;
- bearing strength; and,
- permeability.

The most pertinent clauses in the TNZ M/3 (Notes 1986) document make the following assertions:

- The material must be free from all non-mineral matter.
- The material does not need to be manufactured by crushing.
- Maximum particle size should not be greater than 100 mm, or 0.6 times the construction layer thickness. Grading control is relatively loose. However it is recommended that the general form of the TNZ M/4 (1985) grading envelope is followed.

  To achieve compatibility between the subgrade and sub-base, specific grading criteria should be met for a layer thickness of at least 75 mm. Alternatively, an intrusion zone can be designated at the bottom of the sub-base and the layer thickness amended accordingly.

- The sub-base layer should possess sufficient permeability to prevent the build-up of pore water pressure in the overlying basecourse. To achieve this, TNZ M/3 (Notes 1986) specifies a sand equivalent value of at least 40, or less than 10% of particles by weight passing the 4.75 mm sieve. It is desirable to attain a permeability of greater than 10^{-4} m/s in the top 100 mm of the sub-base.

3.1.2 Specification TNZ M/4

Specification TNZ M/4 (1985) is the relevant standard for crushed rock primary basecourse aggregate. The TNZ M/4 (1985) document presents information on the following items:

- compliance test specifications for proportion of broken rock, crushing resistance, weathering resistance, sand equivalence, and grading; and,
- basis of measurement and payment.

The most pertinent clauses in the TNZ M/4 (1985) document make the following assertions:

- At least 70% (by weight) of the aggregate fractions between the 37.5 mm and 4.75 mm sieves should have two or more broken faces.
- The aggregate's crushing resistance must not be less than 130 kN. The sample is deemed to comply with this specification if less than 10% of fines are produced when loaded up to the prescribed forces within a ten minute period.
- The weathering resistance quality index of the material must fall into one of the following categories - *AA, AB, AC, BA, BB or CA.*

- The material should have a sand equivalent not less than 40.
- The material must comply with the grading envelope limits presented in Table 3.2 and the grading envelope shape limits presented in Table 3.3.

Table 3.2 TNZ M/4 (1985) grading envelope limits.

Sieve Aperture Size	% Passing Limits	
	AP 40 (1)	AP 20 (2)
37.5 mm	100	100
19.0 mm	66 - 81	100
9.5 mm	43 - 57	55 - 75
4.75 mm	28 -43	33 - 55
2.36 mm	19 - 33	22 - 42
1.18 mm	12 - 25	14 - 31
600 μm	7 - 19	8 - 23
300 μm	3 - 14	5 - 16
150 μm	<= 10	<= 12
75 μm	<= 7	<= 8

Notes: (1) All passing 40 mm sieve.
(2) All passing 20 mm sieve.

Table 3.3 TNZ M/4 (1985) grading envelope shape control limits.

Sieve Aperture Size	% Passing Limits	
	AP 40	AP 20
19.0 mm - 4.75 mm	28 - 48	-
9.5 mm - 2.36 mm	14 - 34	20 - 45
4.75 mm - 1.18 mm	7 - 27	9 - 34
2.36 mm - 600 μm	6 - 22	6 - 26
1.18 mm - 300 μm	5 - 19	3 - 21
600 μm - 150 μm	2 - 14	2 - 17

3.1.3 Specification TNZ M/5

There are a number of versions of the TNZ M/5 (1984 - 1993) specification for regional aggregates (Transit New Zealand 1993a), i.e. appropriate documents for Rotorua, Gisborne, Napier, Wellington, Nelson and Christchurch regions. The TNZ M/5 specifications allow certain local materials to be used for basecourse aggregates instead of the more stringent TNZ M/4 (1985) specification aggregates. This allows better utilisation of local resources, e.g. use of partly crushed river gravels. Some of the TNZ M/5 materials have restrictions over their use that are based on experience of their past performance as basecourse aggregates.

Examples of compliance parameters which differ between the different TNZ M/5 specifications and the TNZ M/4 (1985) specification are as follows:

- crushing resistance (for Rotorua, Napier);
- sand equivalent value (for Gisborne, Wellington, Christchurch);
- weathering resistance (for Napier);
- proportion of broken rock (for Rotorua, Gisborne, Napier);
- grading envelope (for Napier); and,
- grading shape control (for Wellington, Christchurch).

3.1.4 Specification TNZ M/6

Specification TNZ M/6 (1993; Transit New Zealand 1993b) is the relevant standard for sealing chip aggregate. The TNZ M/6 document presents information on the following items:

- aggregate composition; and,
- compliance test specifications.

The most pertinent clauses in the TNZ M/6 document make the following assertions:

- The material must be broken or crushed from rock or waterworn gravel and be hard, sound and of uniform quality.
- The weathering resistance quality index should be in categories *AA or BA.*
- The material must display a cleanness value of 85 to 89 depending on the grade.
- The specification gives grading limits for each category of chip in terms of the average least dimension and average greatest dimension for Grades 2, 3 and 4 chips and sieve gradations for Grades 5 and 6 chips.

3.2 Australian Standards and Specifications

3.2.1 New South Wales

The specification of unbound pavement materials in New South Wales is described in the Roads and Traffic Authority (RTA) document entitled *RTA Specification 3051 Unbound and Modified Base and Sub-base Materials for Surfaced Road Pavements* (RTA 1991a). The specification recognises eight classes of unbound material with designations as outlined in Table 3.4.

The reader is referred to RTA Specification 3051 for information regarding grading requirements for each of the designated unbound aggregates.

Table 3.4 Unbound material designations and descriptions used in RTA Specification 3051 (RTA 1991a).

Designation	Description
DGB20	20mm top size, densely graded base
DGS20	20mm top size, densely graded sub-base
DGS40	40mm top size, densely graded sub-base
GMB20	20mm top size, graded macadam base
GMS40	40mm top size, graded macadam sub-base
GMS60	60mm top size, graded macadam sub-base
MS50	50mm top size, macadam sub-base
MS75	75mm top size, macadam sub-base

A number of other unbound aggregate specification requirements exist which differ depending upon the magnitude of the design traffic loading (expressed in ESA). These include modified Texas triaxial classification numbers ranging from 2 to 3, maximum liquid limit of 20 to 23%, maximum plastic limit of 20%, maximum plasticity index of 6 to 12%, aggregate wet strength ranging from 50 to 200 kN, and wet/dry strength variation of 30 to 60%.

The eight standard aggregate materials included in the specification show a wide variation of performance parameters reflecting a significant variation in their applicability.

3.2.2 Victoria

VicRoads has produced a series of specification documents for unbound basecourse and sub-base materials. These include specifications for gravel, sand and soft (or ripped) rock for unbound aggregate layers, crushed scoria for unbound aggregate layers and crushed rock for unbound aggregate layers. The document designated *Section 812Q* VicRoads (1993c) is discussed in detail in this Section 3.2.2. This specification excludes the use of sedimentary rock for the production of basecourse aggregates.

Table 3.5 Unbound aggregate classes used in Victoria (from VicRoads 1993c).

Material	Usage
Class 1 Base	Freeways, heavily trafficked urban arterials Annual average daily traffic > 15,000
Class 2 Base	Urban arterials, rural roads and highways, other urban streets
Class 3 Sub-base	Upper 150 mm sub-base
Class 4 Sub-base	Lower layers of sub-base

A number of aggregate gradings are specified depending on the aggregate class, the type of rock, the Los Angeles value and the nominal top size. The reader is referred to VicRoads (1993c) for grading details.

Table 3.6 Los Angeles Abrasion Loss test result specifications for different rock types (from VicRoads 1993c).

Rock Type	Los Angeles Abrasion Loss (max)			
	Base		Sub-base	
	Class 1	Class 2	Class 3	Class 4
ACID IGNEOUS				
Granite, Adamellite, Granodiorite	40	40	45	-
Granophyre, Rhyolite, Rhyodacite	25	25	30	-
INTERMEDIATE IGNEOUS				
Diorite, Porphyry	25	25	30	-
Trachyte	30	30	35	-
BASIC IGNEOUS				
Basalt, Dolerite, Limburgite	30	30	35	-
METAMORPHIC				
Hornfels	25	25	25	-
Quartzite, Schist, Phyllite, Gneiss, Greenstone	30	30	35	-
SEDIMENTARY				
Argillaceous - e.g. Mudstone, Shale, Claystone, Tillite, Calcareous Mudstone	-	-	25	-
Arenaceous - e.g. Sandstone, Greywacke, Arkose, Quartzite, Calcarenite	-	-	45	-

The different classes of unbound aggregates used in Victoria are outlined in Table 3.5. The hardness of the parent rock is determined using the Los Angeles abrasion test. Maximum Los Angeles values for each aggregate class and various rock types are presented in Table 3.6. Other requirements of the unbound aggregate materials that are stipulated in the VicRoads document are presented in Table 3.7.

Table 3.7 Other VicRoads aggregate specification requirements (from VicRoads 1993c).

Test	**Test Value**			
	Base		**Sub-base**	
	Class 1	**Class 2**	**Class 3**	**Class 4**
Liquid Limit % (max)	25	30	35	40
Plasticity Index % (max)	3	6	10	20
CBR % (min)*	-	-	-	15
PI % passing 425 μm sieve (max)	-	-	-	600
Sand Equivalent (min) **	55	50	-	-

Notes:

* Value applicable to material passing 19 mm sieve: initially at optimum moisture content, 95% of max dry density (modified compaction), and soaked for 4 days prior to CBR test.

** Other sand equivalent values may be specified depending on the test results for plasticity & grading.

\- Not applicable.

3.2.3 Queensland

The comprehensive specification designated *MRS 11.05 2/90* by Queensland Department of Transport (1990b) describes unbound aggregate materials to be used in pavement construction. The specification recognises four material types with each type divided into up to five subtypes. The different types correspond to the nature of the material and the mechanism by which it develops its strength, as shown in Table 3.8. The compliance standards presented in this Section 3.2.3 refer to Type 1 materials only. The aggregates are categorised according to their geological origin. This recognises the variation of property limits which are appropriate for different material sources.

The aggregate properties which are examined for compliance with the MRS 11.05 specification fall into three categories, i.e. particle size distribution, coarse fraction behaviour, and fine fraction behaviour. A strength category is included for material Types 2, 3 and 4, but not for Type 1.

Table 3.8 Strength mechanisms of Queensland material types (after Queensland Department of Transport 1990b).

Material Type	Strength Mechanism
Type 1 - Subtype 1.1 for basecourses - Subtype 1.2 for sub-bases	Internal friction
Type 2	Internal friction + cohesion
Type 3	Internal friction + cohesion
Type 4	Cohesion

The reader is referred to MRS 11.05 for the grading requirements. Other grading related clauses state that the ratio of the material percentage passing the 75 μm sieve and the material percentage passing the 425 μm sieve should be greater than 0.30 but less than 0.55. In addition, the grading curve must be smooth and not vary from the upper one third of the percent passing range to the lower third of the percent passing range (or vice-versa) between successive sieves.

The compliance specifications for the fine and coarse aggregate fractions are presented in Tables 3.9 and 3.10 respectively. Note that the coarse fraction is defined as being all particles retained on the 425 μm sieve and the fine fraction is all particles passing the 425 μm sieve.

Table 3.9 Fine fraction compliance standards for Type 1 aggregates (after Queensland Department of Transport 1990b).

Property	Subtype 1.1	Subtype 1.2
Liquid Limit	≤ 25	≤ 28
Plasticity Index	≤ 4	≤ 6
Linear Shrinkage	≤ 2.5	≤ 3.5

3.2.4 South Australia

The South Australia Department of Road Transport provides specifications for aggregate materials ranging from sands (d_{100} = 6.7 mm) to spalls (d_{100} = 300 mm). The criteria for these materials are set out in the document *Standard Specification for Supply and Delivery of Pavement Material* (South Australia Department of Road Transport 1993). The specifications for four crushed rock aggregates are outlined in Table 3.11.

Table 3.10 Coarse fraction compliance standards for Type 1 aggregates (after Queensland Department of Transport 1990b).

Property	**Subtype**	**Source Material Category**					
		Acid Igneous	**Intermediate Igneous**	**Basic Igneous**	**Metamorphic**	**Sedimentary**	**Natural Gravel**
10% Fines Value (Wet)	1.1 1.2	≥ 130kN ≥ 95kN	≥ 140kN ≥ 105kN	≥ 150kN ≥ 110kN	≥ 140kN ≥ 105kN	≥ 130kN ≥ 95kN	≥ 130kN ≥ 95kN
Wet/Dry Strength Variation	1.1 1.2	≤ 40% ≤ 45%	≤ 35% ≤ 40%	≤ 30% ≤ 35%	≤ 35% ≤ 40%	≤ 40% ≤ 45%	≤ 40% ≤ 45%
Washington Degradation	1.1 1.2	≥ 40 ≥ 30	≥ 45 ≥ 35	≥ 50 ≥ 40	≥ 45 ≥ 35	- -	- -
Crushed Particles	1.1 1.2	≥ 70% ≥ 70%	≥ 70% ≥ 70%	≥ 70% ≥ 70%	≥ 70% ≥ 70%	≥ 70% ≥ 70%	≥ 70% ≥ 70%
Flakiness Index	1.1 1.2	≤ 35% ≤ 35%	≤ 35% ≤ 35%	≤ 35% ≤ 35%	≤ 35% ≤ 35%	≤ 35% ≤ 35%	≤ 35% ≤ 35%

Note that the fine fraction compliance standards are independent of the geological origin of the aggregate.

Table 3.11 Aggregate specifications for Pavement Materials PM32, PM33, PM35 and PM36 (after South Australia Department of Road Transport 1993).

Test		**Permitted Test Values**			
		PM32	**PM33**	**PM35**	**PM36**
Grading	53.0 mm	-	-	100	100
	37.5 mm	-	100	95 - 100	95 - 100
	26.5 mm	100	95 - 100	79 - 91	78 - 90
	19.0 mm	95 - 100	79 - 93	65 - 83	62 - 80
	13.2 mm	77 - 93	-	-	-
	9.5 mm	63 - 83	53 - 73	44 - 64	46 - 60
	4.75 mm	44 - 64	36 - 56	29 - 49	26 - 46
	2.36 mm	29 - 49	25 - 43	20 - 38	16 - 34
	0.425 μm	13 - 23	10 - 21	8 - 18	6 - 16
	0.075 μm	5 - 11	4 - 10	3 - 9	2 - 7
Liquid Limit (LL)		≤ 25	≤ 25	≤ 25	≤ 25
Plasticity Index (PI)		1≤ PI ≤ 6	1≤ PI ≤ 6	1≤ PI ≤ 6	1 ≤ PI ≤ 6
Linear Shrinkage (LS)		≤ 3	≤ 3	≤ 3	≤ 3
Los Angeles Abrasion (LAA)		≤ 30%	≤ 30%	≤ 30%	LAA ≥ 30% & LAA ≤ 45%

3.2.5 Western Australia

A specification for basecourse aggregate material forming part of a construction project contract document has been obtained from the Main Roads Department Western Australia. The document allowed the use of both a crushed rock basecourse and a laterite gravel basecourse - limestone was specifically excluded. A selection of material requirements are specified in Table 3.12. The grading requirements for the two basecourse types are presented in Table 3.13.

Table 3.12 Specification requirements for basecourse materials used in a Western Australian highway construction project*.

Type of Test	Crushed Rock Basecourse	Laterite Gravel Basecourse
Soaked CBR	≥ 80%	≥ 80%
Dry Compression Strength	-	≥ 1700 kPa
Liquid Limit	≤ 25%	≤ 25%
Linear Shrinkage	LS≥1% & LS=<3%	≤ 2%
Plasticity Index	≤ 6%	≤ 4%

Table 3.13 Grading requirements for basecourse materials used in a Western Australian highway construction project*.

Sieve Size (mm)	% Passing	
	Crushed Rock Base	Laterite Gravel Base
53.00	100	100
37.50	100	95 - 100
19.00	90 - 100	71 - 100
9.50	63 - 82	50 - 82
4.75	45 - 67	36 - 67
2.36	32 - 54	25 - 54
1.18	23 - 44	18 - 44
0.60	15 - 35	12 - 35
0.425	13 - 32	10 - 32
0.300	11 - 28	8 - 28
0.150	7 - 22	5 - 22
0.075	5 - 19	5 - 19

* From Albany Highway Road Construction Technical Specifications, Contract No. 82/92 Narrikup to Albany

3.4 United Kingdom Standards and Specifications

The UK pavement materials specification recognises two classes of sub-base material, i.e. Type 1 and Type 2, and three classes of unbound basecourse material, i.e. wet-mix macadam, dry bound macadam (coarse) and dry bound macadam (fine). Type 2 sub-base is restricted to use in pavements with a design loading of less than 6×10^6 standard axles. Since publication of the specification, the UK Department of Transport no longer allows the use of unbound aggregates in the basecourse component of pavements (Biczysko 1989).

The reader is referred to Biczysko (1989) for details of the grading requirements. Other compliance test parameters are presented in Table 3.14.

A British Standard document for unbound aggregates is being planned, according to Biczysko (1989), which will include both prescriptive and performance compliance criteria. This standard will allow the engineer either to specify certain material criteria such as grading, crushing resistance, etc, or to specify end result performance properties and leave the selection of construction materials to the contractor.

Table 3.14 Compliance parameters for unbound aggregates used in the UK (after Biczysko 1989).

Compliance Parameter	Sub-base		Dry Bound Macadam		Wet-mix Macadam
	Type 1	Type 2	Coarse	Fine	
Agg. type					
- Rock	✓	✓	✓	✓	✓
- Slag	✓	✓	✓	✓	✓
- Concrete	✓	✓	x	x	x
- Shale	✓	✓	x	x	x
- Sand	x	✓	x	x	x
- Gravel	x	✓	x	x	x
Plasticity	Non Plastic	≤ 6	N/A	N/A	N/A
10% Fines	50 kN	50 kN	50 kN	N/A	50 kN
Flakiness Index	N/A	N/A	≤ 40	N/A	≤ 40
Min.Moisture	N/A	OMC-2%	N/A	N/A	OMC-0.5%
Max.Moisture	N/A	OMC+1%	N/A	N/A	OMC+0.5%

Key

✓	Allowable
x	Not allowable
N/A	Not applicable
OMC	Optimum moisture content

3.5 United States Standards and Specifications

In the US, each transportation authority specifies its own standards for materials and construction depending on the environmental and climatic conditions which are prevalent in that state. Generalised standards published by AASHTO and the American Society for Testing and Materials (ASTM) are available. However the wide variation in conditions and materials favours the use of regional specifications (Barksdale 1989). Table 3.15 presents basecourse aggregate specification data from eight American states. The reader is referred to Barksdale (1989) for aggregate grading requirements.

Table 3.15 Basecourse compliance specifications from eight states of the US (after Barksdale 1989).

State	Los Angeles Abrasion (%)	Comments
California	N/A	25% by weight (min) crushed particles for fraction retained on 4.76 mm sieve. R-value ≥ 78. Durability index ≥ 35. Sand equivalent ≥ 22 (25 minimum moving average)
Michigan	≤ 50	Stone quarry, gravel deposits or waste material; clay lumps and roots must be removed. 25% by weight (min) crushed particles for fraction retained on 9 mm sieve.
Arizona	40	30% by weight (min) retained on 2.38 mm sieve to have at least one rough angular surface from crushing. Stone gravel. PI ≤ 3%.
Maine	N/A	Hard durable particles, must be free from vegetable matter, lumps, balls of clay or other deleterious substances.
Oregon	≤ 35	Crushed gravel or crushed rock. One fractured face for 50% retained on 38 mm sieve and 70% between 38 mm & 6.35 mm sieves. PI = 0 for ≥ 25% passing 0.42 mm sieve. Sand equivalent ≥ 30.
North Carolina	≤ 55	Crushed or uncrushed gravel/stone. Hard durable particles free of adherent coatings. Sodium sulphate soundness ≤ 15% of agg. passing No. 40 sieve. LL 0-30%. PL 0-6%.
Illinois	≤ 45	Pit run gravel or crushed stone. CBR ≥ 80%. PI (agg. passing No. 4 sieve) : 2-6% gravel, 0-4% stone. Sodium sulphate soundness, 5 cycles ≤ 25% loss.
Georgia	≤ 60	Sand equivalent for agg. passing No. 10 sieve : ≥ 22 Group 1, ≥ 28 Group 2. Magnesium sulphate soundness ≤ 15% for 5 alterations.

3.6 Canadian Standards and Specifications

The RTAC Pavement Management Guide (1977) does not provide unbound aggregate specification information.

3.7 European Standards and Specifications

No unbound aggregate specifications from European countries have been identified in the international literature.

4. REVIEW OF STANDARDS AND SPECIFICATIONS FOR PAVEMENT CONSTRUCTION

4.1 Introduction

Standards and specifications for the construction of sealed unbound pavements that are used in New Zealand, Australia, United Kingdom (UK), United States of America (US), Canada and Europe have been reviewed. The results of the review are presented in this Section 4.

4.2 New Zealand Standards and Specifications

In New Zealand, standards for pavement construction procedures are generally prescribed in the set of Transit New Zealand standard specifications. These specifications are a series of documents which cover almost all aspects of road construction and material specifications. The titles of those specifications particularly relevant to the construction of sealed unbound granular pavements are presented in Table 4.1.

Table 4.1 Titles of Transit New Zealand construction specifications relevant to sealed unbound granular pavements.

Specification	Subject
TNZ G/1 (1986b)	Road construction general clauses
TNZ F/1 (1986a)	Earthworks construction
TNZ B/2 (1987)	Construction of unbound granular pavement courses
TNZ P/3 (1988)	First coat sealing
TNZ P/4 (1989b)	Second coat sealing and resealing

4.1.1 Specification TNZ G/1:1986

Specification TNZ G/1 (1986) is general in its content, with most of its clauses concerning common practice for public safety and courtesy. It defines the responsibilities of the contractor with respect to a programme of work, public safety, traffic control, utilities and authorisation for material usage.

4.1.2 Specification TNZ F/1:1986

Specification TNZ F/1 (1986) describes the methodology and terminology involved in earthworks construction. The specification presents information on the following items:

- definition of terms;
- clearing the site and removal of topsoil;
- surface drainage control;
- salvage of aggregate from existing pavements;
- classification of excavated material for payment;
- fencing requirements;
- excavation management and practice including undercutting, batters, dump areas, borrow areas, benching and drainage;
- filling practice and compaction;
- subgrade surface finishing, including tolerances, testing and drainage;
- intersecting roads and private ways;
- shaping and topsoiling;
- grassing and batter protection;
- maintenance of works; and,
- method of measurement and basis of payment for the above clauses.

The most pertinent clauses in the TNZ F/1 (1986) specification make the following assertions:

- The reduced level of a subgrade formation should be within 0 to –20 mm of the design level. In addition, all points along a 3-m straightedge should lie within 15 mm of the formation.

- When forming a subgrade by filling, the upper 400 mm should comprise material passing a 75-mm sieve compacted in layers not exceeding 135 mm. The underlying 600 mm should comprise material passing a 125-mm sieve compacted in layers not exceeding 200 mm.

- Compaction of granular fill should continue until the layer has attained a dense condition, which is considered to have been achieved when the surface impression left by the passage of a smooth wheel roller, that applies a load of 6250 kg per metre of roll width, is not greater than 5 mm.

  In the case of cohesive materials, the in situ wet density is expressed as a percentage of the wet density under standard compaction conditions and at the same moisture content. The compliance criterion states that, for five test locations, the average density should not be less than 98% of the maximum density (under standard compaction conditions) plus 0.3 times the density range. Note that compaction should not continue if the material shows signs of excessive heaving or weaving.

4.1.3 Specification TNZ B/2:1987

Specification TNZ B/2 (1987) describes the methodology for the construction of unbound granular pavement courses. The specification presents information on the following items:

- maintenance of existing surface;
- control of construction dimensions;
- tolerances for setting out and construction;
- spreading and compaction of aggregate, including supply, placement, compaction, use of water and surface shape;
- feather edges;
- defects to be remedied;
- crossroads and private ways;
- running course materials;
- maintenance; and,
- measurement and basis of payment for above clauses.

The most pertinent clauses in the TNZ B/2 (1987) specification require that:

- The maximum allowable geometrical tolerances should be as follows:

Width (unconstrained)	-20 mm to + 50 mm
Width (constrained)	zero
Vertical (sub-base)	-25 mm to + 5 mm
Vertical (base)	-5 mm to +15 mm

- The transportation and stockpiling of aggregate must be strictly controlled to avoid contamination and segregation. Aggregate placement must not cause segregation and the uncompacted layer thickness should not be less than twice the maximum particle size (when that size is greater than 20 mm) or greater than 250 mm or four times the maximum particle size, whichever is less.

- Compaction should be continued until the material attains a stable state and does not weave or creep under rolling. Primary compaction should be carried out using plant with a single or double vibrating drum of not more than 3.2 tonnes per metre of roll width and a vibrating frequency of not less than 37 Hz. Other roller configurations are permitted but are restricted to certain combinations of weight and amplitude of vibration. Final surface rolling is generally by controlled traffic loading but alternatively it can be carried out using three-wheel steel-tyred plant with rear rollers at least 500 mm wide and apply a load of not less than 4500 kg per metre of roll width. Pneumatic tyred rollers with an operating weight of at least 7 tonnes distributed over at least seven tyres may also be used.

- During compaction the aggregate should be kept moist by using a controlled application of water. The water should not disturb the surface fines, nor should it be allowed to adversely affect either the layer being compacted or the underlying layers.

4.1.4 Specification TNZ P/3:1988

Specification TNZ P/3 (1988) describes the methodology for the construction of first-coat surface seals. The specification presents information on the following items:

- edge definition;
- sealing period;
- preparation of the basecourse surface;
- sealing binder usage, i.e. type, temperature and application;
- spreading and rolling of chips;
- traffic control;
- rain damage;
- pattern of operations;
- surplus chip removal;
- cross-roads and private ways;
- maintenance; and,
- basis of payment.

Pertinent clauses in the TNZ P/3 (1988) define the grade of bitumen and proportion of kerosine to be used in the binder for various regions of the country. The spray temperatures are specified with respect to the proportion of diluting agents in the binder. The specification also requires that:

- The sealing operation take place on a reasonably dry surface which is free from frost and in conditions where the air temperature in the shade exceeds 10°C. The basecourse surface must be free of dust and other deleterious matter and should be tightly compacted with a mosaic appearance.

- Binder spraying be carried out using appropriate equipment and at the approved application rate.

- Sealing chips be spread so that the resulting layer is one stone thick with adjacent chips packed shoulder to shoulder.

- Rolling must follow chip spreading within 30 minutes with at least four passes of a pneumatic tyred roller. The roller must have a minimum weight of 10 tonnes and at least seven tyres. A steel wheel roller may be used for initial compaction. This plant must have a roller wheel at least 500 mm wide and apply a load of at least 2700 kg per metre of roll width.

4.1.5 Specification TNZ P/4:1989

Specification TNZ P/4 (1989) describes the methodology for the construction of second-coat surface seals and reseals. The content of the specification is generally similar to that for document TNZ P/3 (1989) and hence no further discussion is presented.

4.2 Australian Standards and Specifications

4.2.1 New South Wales

Document *CMS-QA-R35 Specification Part 35 Unbound Pavement Course (Normal Duty)*, produced by the Roads and Traffic Authority of New South Wales (RTA 1991b) sets out the standards required for the construction of unbound aggregates in pavements in New South Wales. The main clauses of the specification are summarised as follows:

- Aggregates must be spread and compacted in uniform layers so that the compacted thickness is within the range 100 to 200 mm.

- The moisture content of the material should be uniform and within the range 60 to 90 % of the standard optimum moisture content.

- Compaction should start at the low side of the pavement and work towards the high side.

- The deviation from a 3-m straightedge placed anywhere on the top of the basecourse layer should not exceed 7 mm. Other geometrical tolerances are as follows:
    - Top of basecourse : –0 to +15 mm
    - Top of sub-base : ± 15 mm

4.2.2 Victoria

The VicRoads (1993b) document, *Section 304Q, Flexible Pavement Construction*, provides details of the requirements for the construction of pavement courses in Victoria. The main clauses of the Section 304Q are summarised below:

- The level of each pavement course comprising crushed rock material should not differ from the specified level by more than 15 mm.

- The combined thickness of the sub-base and basecourse layers should not be less than the specified thickness by more than 15 mm.

- The top surface of each pavement layer must lie within 10 mm of a 3 m straightedge laid parallel to the centre-line of the pavement.

- For major highway pavements, the dry density of compacted sub-base layers should be not less than 98% of the maximum laboratory dry density under modified compaction conditions. The upper 100 mm of the basecourse should achieve 100% of maximum dry density and the remainder of the basecourse 99%.

- The minimum frequency of testing for compaction is as outlined in Table 4.2.

Table 4.2 Minimum testing frequency for compaction (after VicRoads 1993b)

Material	**Acceptable Lot Size (Single Layer)**
Upper basecourse	5,000 m^2 or one day's production
Lower basecourse	5,000 m^2 or one day's production
Sub-base	10,000 m^2 or one day's production
Lower sub-base	15,000 m^2 or one day's production

4.2.3 Queensland

The Queensland Department of Transport (1990b) document *MRS 11.05 2/90, Unbound Pavements,* sets out construction specifications for unbound aggregate pavements. The main clauses of the document requires that:

- All layer construction using premium aggregate must be carried out using a self-propelled spreading machine. There should be no visual signs of aggregate segregation.
- Compacted layer thicknesses must be within the range 75 mm to 250 mm.
- The in situ dry density should be 102% of the maximum laboratory dry density using standard compaction conditions.
- The deviation from a 3-m straightedge placed anywhere on the pavement should not exceed 5 mm. Other geometrical tolerances are as follows:
  - Horizontal : ± 50 mm
  - Vertical : ± 15 mm (any layer)
  - Crossfall : ± 0.5%
- The pavement surface should achieve a maximum roughness of 60 counts/km. Roughness counts significantly less than the specification may earn an additional payment to the contractor in recognition of the superior standard of rideability.

4.2.4 South Australia

An example of a technical specification for a road reconstruction contract was obtained from the Department of Road Transport, South Australia in order to analyse the relevant specifications for construction in that state. The main clauses in the document are summarised as:

- Subgrade fill should be placed in lifts not exceeding 200 mm thickness and compacted to at least 90% of the laboratory maximum dry density (under modified compaction conditions).

- Finished subgrade levels should be within +0 mm to –40 mm of the specified level.

- Compacted aggregate layer levels should be within ±15 mm for Type 2 base materials or ±20 mm for other aggregate materials.

- Aggregate layers must be compacted to give a 90% statistical assurance of achieving 95% of the maximum laboratory dry density for sub-base materials and 98% of the maximum laboratory dry density for basecourse materials in terms of modified compaction conditions.

4.2.5 Western Australia

An example of a technical specification for a road reconstruction contract was obtained from the Main Roads Department, Western Australia* to analyse the relevant specifications for construction in that state. The main clauses in the document are summarised as:

- Fill for subgrade formation should be placed in lifts not exceeding 300 mm and compacted to achieve at least 94% of the maximum laboratory dry density under standard compaction. The moisture content of subgrade materials should be within the range 85% – 100% of optimum moisture content.

- Basecourse aggregate must be compacted to at least 97% of the maximum laboratory dry density under standard compaction.

- The basecourse should be compacted in lifts not less than 80 mm thick or more than 175 mm thick. If a lift of less than 80 mm is required, the underlying material should be scarified to such a depth that the compacted layer is greater than 80 mm thick.

- The finished surface of the basecourse should be within –5 mm to +20 mm of the level specified on the construction drawings.

4.3 United Kingdom Standards and Specifications

No construction specifications from the UK have been identified in the literature.

4.4 United States Standards and Specifications

In the US, each state roading authority has its own set of construction standards, a selection of which are summarised in Table 4.3.

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* From Albany Highway Road Construction Technical Specifications, Contract No. 82/92 Narrikup to Albany

Most US construction specifications require aggregate to be handled and placed in an appropriate manner to avoid segregation. The use of spreaders is stipulated by some states although Georgia and Illinois require the additional assurance of using a central aggregate mixing plant.

Table 4.3 Construction standards from various US locations (after Barksdale 1989).

State	Density	Use of Spreader?	Allowable Vertical Tolerance (mm)
California	0.95M	✓	+15
Michigan	0.98M	✓	+19
Arizona	1.00S	x	+12
Maine	0.95M	x	+6
Oregon	0.95S	–	+12
North Carolina	1.00M	✓	–
Illinois	1.00S	✓	–
Georgia	1.00M	✓	+6

Key

✓ Allowable
x Not allowable
– Unknown
S Maximum density using standard compaction
M Maximum density using modified compaction

4.5 Canadian Standards and Specifications

The *RTAC Pavement Management Guide* (1977) does not provide detailed construction specifications, although it does discuss at some length the roles of the construction management personnel and the relationship between construction activities and the environment. While these factors are important they are not relevant to the present project.

4.6 European Standards and Specifications

No construction specifications from European countries have been identified in the literature.

5. REVIEW OF TECHNICAL LITERATURE FOR PAVEMENT DESIGN

5.1 Design Philosophy

Two main philosophies of pavement design are currently in use, namely analytical and empirical. The analytical design approach (often termed *rational* or *mechanistic*) attempts to characterise pavement performance on a rigorous theoretical basis while the empirical design approach is based on the identification of successful pavement structures from full-scale trials. Both approaches have their own particular merits and limitations, as discussed in Sections 5.1.1 and 5.1.2 of this chapter. A new analytical design method based on *shakedown theory* is discussed, as are statistically based methods that are emerging.

5.1.1 Analytical Design

The current Transit New Zealand pavement design method (Transit New Zealand 1989a) adopts an analytical approach. The design charts have been generated using a multi-layer, linear elastic computer program to calculate critical pavement stresses and strains for a large number of traffic loading and subgrade conditions.

The methodology does however include a number of assumptions and simplifications which may introduce errors of significant magnitude. For example, the simplified characterisation of material response parameters has particular effect since this factor is fundamental to the performance of the pavement model.

Most analytical design methods adopt a linear elastic constitutive model for all materials in the pavement. Although this may be justified for the sake of simplicity, it is contrary to the widely accepted theory that the stress/strain response of both fine-grained and granular materials is nonlinear. Barksdale (1984) found that in situ vertical subgrade stresses were understated by up to 50% when analysed assuming linear elastic material parameters. This was confirmed by Doddihal and Pandey (1984) who used the finite element method in a comparison with a linear elastic analysis.

A further disadvantage of using a linear elastic analysis is that it predicts the development of tensile stresses in the lower zone of unbound aggregate layers. Because unbound materials cannot sustain tensile stresses, the model is clearly inadequate. In practice, the predicted development of tension is represented by a reduction in the prevailing compressive stress provided by the pavement's lateral support. Other analysis methods have been investigated in an attempt to produce a superior pavement model, e.g. pseudo-nonlinear elastic analysis using iterative multi-layer elastic theory and the finite element method.

The analytical design method adopted by Transit New Zealand in 1989 is based on the concept that the pavement will reach a state of failure (i.e. a specified terminal level of serviceability) after a certain number of applications of a standard axle load. The design life of the pavement is assumed to be dependent on the magnitude of the vertical compressive strain occurring at the top of the subgrade when subjected to a static application of one half of a standard eight tonne axle load.

This fatigue type relationship is questionable since most well constructed sealed unbound granular pavements appear to improve with age rather than deteriorate. The pavement surface may deteriorate with time because of environmental effects and the nature of the tyre contact stresses, but it is suggested by the authors that the pavement's structural integrity either reaches a plateau or continues to increase with time.

The analytical design method incorporates a number of other vital assumptions such as the characterisation of the subgrade elastic modulus and the damaging effect of the spectrum of applied axle loads. These factors are generally taken into account using empirical relationships which were originally developed using a particular set of materials and environmental conditions. These may not be appropriate for all pavement types, materials or axle loading spectra.

Computer-based design analyses also have the risk of becoming black boxes where the software user simply presses the keyboard buttons without fully understanding the meaning of the results or investigating the sensitivity of the input data.

5.1.2 Empirical Design

An alternative to the analytical design approach is the empirical approach. This is the basis of the pavement design method specified by AASHTO and several other roading authorities throughout the world. The empirical approach is fundamental in that it is based on the performance of pavement structures and materials as indicated by the analysis of full scale pavement tests using in-service roads and test tracks. The AASHTO method is based on the earlier AASHO Road Test carried out in the 1950s in the state of Illinois, US.

An advantage of the empirical approach is that a set of successful pavement structures and materials can be established under a certain range of conditions. The designer can then simply adopt an appropriate pavement structure for a given combination of traffic loading, material parameters and environmental conditions for the design situation.

A problem arises however if the available materials or environmental conditions differ from those used in the development of the empirical model because extrapolation away from the standard base of experience cannot be justified. In an attempt to allow for such circumstances the AASHTO method has introduced a regional factor to allow for varying environmental conditions and layer coefficients to allow for varying material properties. These factors are subjective and may result in significant variations of design depending on the values adopted (Sweere 1989).

The AASHTO pavement model is criticised in the literature because the original formulation of the model excluded the effects of environmental conditions. Suggestions have been made that the most significant influence on the performance of the AASHO Road Test pavements was the spring thaw at the initiation of these particular tests rather than the effects of axle load magnitude, construction material properties, or pavement structure.

An empirical design method based on pavement deflection testing is used in Canada (RTAC 1977). This approach to pavement design follows sound engineering principles, although it requires a large database of deflection test results to establish relationships between applied load and pavement structure, as well as between pavement deflection and pavement performance.

The RTAC method involves deflection testing of a section of prototype pavement (i.e. a pavement with similar subgrade and environmental characteristics), followed by the selection of a design deflection based on the design loading of the new pavement. The required addition (or reduction) to the prototype pavement thickness to achieve the design deflection is then calculated to determine the new pavement structure.

5.1.3 Shakedown Theory

The development of the shakedown theory as a means of pavement design appears to have considerable benefits over both the current analytical and empirical methods as it provides a better basis for pavement performance characterisation. Although shakedown theory has been used in other realms of engineering design, e.g. structures and mechanical components, its potential for use in pavement design has not yet been fully developed.

Shakedown theory states that a structure subjected to repeated loading can conform to one of the following four modes of behaviour depending on the magnitude of the applied stress (Collins et al. 1993):

1. If the magnitude of the applied stress is very small the structure will respond elastically from the first application of stress. The maximum repeated stress which results in this behaviour is termed the *elastic limit stress*.

2. A higher level of stress may result in plastic deformations for the first finite number of load cycles, thereafter plastic deformations diminish to zero and all further loading response is purely elastic and the structure is deemed to have *shaken down*. The maximum repeated stress which results in this behaviour is termed the *shakedown limit stress*.

3. At a stress slightly higher than the shakedown limit stress described above, the pavement may achieve a steady elastic state but the response is non-linear. The result is that the stress/strain plot forms an elongated loop, the area of which is proportional to the work done (or energy absorbed) during the stress/strain excursion. This work can result in a fatigue type failure of some pavement components, e.g. cracking of a surface seal.

4. At repeated stress magnitudes well in excess of the shakedown limit stress the pavement structure suffers significant plastic deformations until it ultimately fails by a mechanism called *incremental collapse*.

The basis of pavement design using shakedown theory is to ensure that each component of the pavement is only subjected to applied stresses below the shakedown limit load of that material so that the long-term response of the pavement is purely elastic. The upper layers of the pavement are subjected to the greatest stresses and hence these materials must be of the highest quality and accordingly possess the highest shakedown limit load. With high quality materials located in the upper portion of the pavement, the underlying materials are afforded considerable protection by rapid dissipation of the applied stresses in the upper layers. The materials in the lower portion of the pavement can therefore be of reduced quality corresponding to the lower stresses and a lower shakedown limit load requirement.

The gradual reduction in the requirement for material quality with increasing depth continues to the level of the subgrade soil which can also be characterised in terms of its shakedown limit load. Lefebvre and LeBoeuf (1989) investigated the cyclic loading behaviour of a saturated clay and reported shakedown behaviour. Using slightly different terminology, they reported the existence of a *stability threshold* for the cyclic loading response of Grande Baleine clay (Hudson Bay, Quebec) at 60% to 65% of the undrained shear strength under monotonic (or static) loading. They also presented a summary of cyclic loading test results from a number of investigations on different clays which showed stability threshold values ranging from 18% to 96% of the undrained shear strength. Similar findings have been reported by Sangrey et al. (1969), Raymond et al. (1979) and Anderson et al. (1980).

The great advantage of the shakedown approach is that it characterises pavement performance in a more credible manner than either the current analytical or empirical design methods. It is consistent with the observations of Bartley and Cornwell (1993) that good quality aggregates generally do not fail by fatigue but rather they effectively have an indefinite life given favourable environmental conditions. Brett (1987) attributes shakedown behaviour to the stable performance of approximately 40% of a sample of pavements observed in New South Wales.

A pavement design based on shakedown theory is independent of the number of vehicle load applications (as long as they are below the shakedown limit load), thus eliminating the uncertainties of traffic counts, growth rates and axle load damaging effect. The pavement will therefore have an indefinite life provided that it is well constructed and maintained.

A design method incorporating shakedown theory has been developed by Sharp (1983). This method is based on a two dimensional, plane strain pavement model and utilises pavement performance data obtained from the AASHO Road Test and various pavement tests in Sydney. Significant advancement of the shakedown theory has been undertaken in New Zealand by Collins et al. (1993) and proposals for future mathematical development with laboratory verification are currently being considered by Transit New Zealand.

The concept of indefinite pavement life is not new. Several empirical pavement design methods are independent of the number of load repetitions, including the design chart developed by Porter (1942) which uses only the wheel load magnitude and the subgrade CBR to determine the thickness of granular cover required. The International Air Transport Association (IATA) Aircraft Classification Number - Pavement Classification Number (ACN-PCN) design method for airfield pavements is also independent of the number of load repetitions. In this method an airfield pavement is assigned a classification number depending on its structural capacity. The different aircraft types are also assigned a classification number depending on the damaging effect of their axle configuration and loading. Only aircraft with a classification number less than or equal to the pavement classification number are permitted to operate on that particular airfield pavement. Note that the ACN-PCN method has a nominal structural redundancy which allows for a limited number of overweight aircraft to use the airfield pavement for emergency situations.

5.1.4 Statistically Based Design Methods

The variability of the material parameters and environmental conditions associated with pavement construction means that the design process lends itself well to a statistical approach. The reliability of the parameters can be assessed in probabilistic terms to provide a realistic, consistent and transportable pavement design model. Bourdeau (1990) presented a probabilistic extension to the AASHTO design method which includes a sensitivity analysis on the influence of various design parameters, e.g. the design was found to be very sensitive to the subgrade support value but relatively insensitive to the design traffic loading.

RTAC (1977) described the application of a probabilistic approach to pavement design in terms of either the application of statistical confidence limits or the use of a reliability concept. The use of statistical confidence limits involves a weighting of the design by not using the mean values for design parameters. On the other hand, reliability refers to the probability of a pavement achieving the design objectives in terms of its pavement life and serviceability.

Reliability is closely related to cost, i.e. as the required reliability increases, so too does the design effort, the pavement thickness and the need for high quality materials and construction techniques. The increased cost of a high reliability pavement is however offset by the reduced requirement for pavement maintenance and significant savings in vehicle user costs. Hence, there is an optimum condition where total costs are minimised by the appropriate degree of design effort, and pavement material and construction quality.

The Transit New Zealand pavement design method adopts a confidence limit approach by specifying the use of the 10%ile value for the subgrade CBR, i.e. 90% of the subgrade should have a CBR greater than the design value. This is consistent with Australian practice (AUSTROADS 1992). The Caltrans design manual (Caltrans 1990) states that the minimum value of subgrade support should be adopted for use in design providing the range of values is relatively narrow. Engineering judgement should be exercised in this regard to ensure that excessively overconservative designs are not adopted.

5.2 Failure Criteria

5.2.1 Terminal Serviceability Condition

The analytical and empirical pavement design methods require a criterion which determines the condition of failure for a pavement. The criterion is generally based on a terminal serviceability condition, either in terms of a specified rut depth or a surface roughness measurement. A summary of terminal condition criteria from New Zealand and other countries is outlined in Table 5.1. Note that a terminal pavement condition is not observed in the shakedown approach to pavement design.

The Works Consultancy Services (WCS) serviceability rating cited in Table 5.1 is derived from the system used by the Canadian Good Roads Society in which a pavement is assigned a score of 0 to 100, where 0 is very good and 100 is very poor. The rating system takes into account surface condition, pavement structural condition and a subjective assessment of ride (TNZ 1989a).

Pavement serviceability rating systems can be significantly influenced by the roughness of the pavement surface. This may be misleading in some instances since a pavement with a poor serviceability rating may be successfully up-rated by applying a thin asphaltic concrete levelling course. On the other hand, a pavement with a similar rating but exhibiting distress caused by a problem at depth may require an expensive structural overlay or a complete reconstruction.

5.2.2 Subgrade Strain Criterion

Regardless of the dilemma described in Section 5.2.1 of this report, most analytical pavement design methods are based on an empirical relationship between the magnitude of the vertical compressive strain occurring at the top of the subgrade and the total design traffic required to achieve the relevant terminal condition criterion. The vertical compressive subgrade strain is calculated from the static application of one half of a standard eight tonne axle and the total design traffic is expressed as the number of equivalent standard axle passes. Various subgrade strain criteria from several sources are presented in Figure 5.1.

The terminal condition criteria presented in Table 5.1 are not specified in a consistent set of units, making comparisons difficult. However, from Figure 5.1 clearly the *TRRL LR1132* design method (Powell et al. 1984) is based on a relatively conservative failure criterion. Despite the differences in subgrade strain criteria the pavement design thicknesses do not vary significantly, e.g. for a subgrade CBR of 5% and 10^5 EDA loading, the difference between the most conservative and the least conservative subgrade strain criterion amounts to approximately 30 mm of aggregate.

Table 5.1 Summary of pavement terminal serviceability condition criteria from a number of roading authorities.

Design Method	Terminal Condition Criterion	Value
TNZ : Group I[1]	WCS Rating	40
TNZ : Group II[1]	WCS Rating	48
TNZ : Group III[1]	WCS Rating	56
TNZ : Group IV[1]	WCS Rating	65
AASHTO : Chart 400-1[2]	PSI	2.5
AASHTO : Chart 400-2[2]	PSI	2.0
AUSTROADS[3]	AUSTROADS Road Roughness	150 /km
Queensland DoT (First Standard)[4]	PSI, Rut Depth	2.5, 20 mm
TRRL LR1132[5]	Rut Depth	10 mm
TRRL Road Note 29[6]	Rut Depth	20 mm

Key

PSI Present Serviceability Index
WCS Works Consultancy Services
(1) TNZ - Transit New Zealand (1989a)
(2) Salter (1988)
(3) AUSTROADS (1992)
(4) Queensland Department of Transport (1990a)
(5) Powell et al. (1984)
(6) TRRL (1970)

The subgrade strain criterion approach was originally developed for UK conditions and constructions by Dorman and Metcalf (1965) using AASHO Road Test data which related the magnitude of compressive subgrade strains to the number of axle passes necessary to result in a given present serviceability index (PSI). It therefore accounts for permanent strains in the subgrade which accumulate with every axle pass to produce surface rutting.

The applicability of this type of criterion is questionable however since TRRL investigations have shown that most permanent pavement deformation occurs in the layers above the subgrade (Lister 1972). Surface rut depths are significantly influenced by the density of the unbound layers at the completion of construction and the amount of subsequent compaction provided by in-service traffic. In most pavement design methods no consideration is given to the behaviour of the materials in the basecourse and sub-base layers, to post construction compaction, or to the effect of water in the pavement.

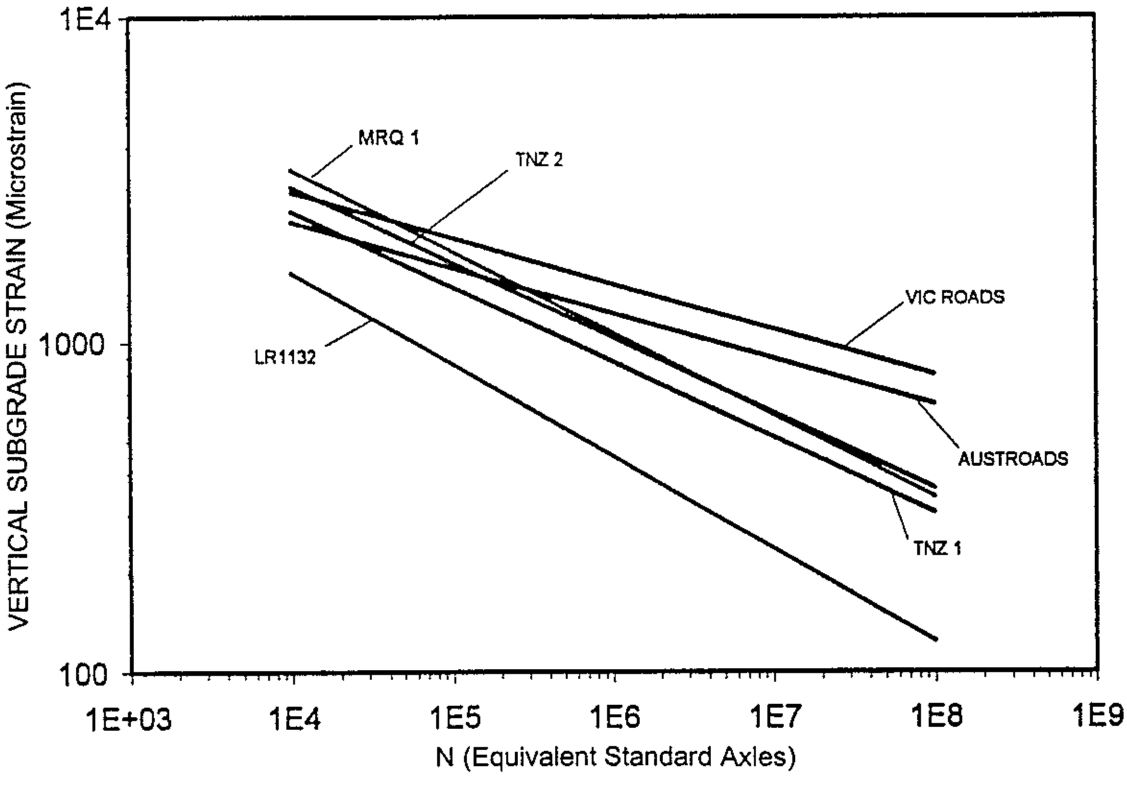


Figure 5.1 Subgrade strain criteria adopted by various roading authorities.

The subgrade strain criterion uses a fatigue concept to account for pavement deformation but it is based on the strain calculated from a single static application of one half of a standard axle load. This appears to be contradictory and includes several approximations and inaccuracies.

5.3 Characterisation of Axle Loading

5.3.1 Equivalent Design Axles

The current (1995) analytical and empirical pavement design methods require the design traffic loading to be an input parameter to the design procedure. If all axle loads were equal, the total design loading would simply be a summation of the expected number of axle passages over the pavement's design life. Since this is not true, a method of accounting for the wide spectrum of axle loads and configurations is required. Most design methods utilise an equivalent standard axle comprising an 8.2 t axle load carried on dual wheels or a 6.2 t axle load carried on single wheels.

The damaging effect of each group of axle loads is related to the number of EDA as the ratio of the applied axle load to the standard axle load, all raised to a suitable exponent known as the *load equivalency exponent*:

$$\text{EDA} = (\text{applied axle load} / \text{standard axle load})^{\text{load equivalency exponent}}$$

In New Zealand the standard EDA loading model is configured as shown in Figure 5.2.

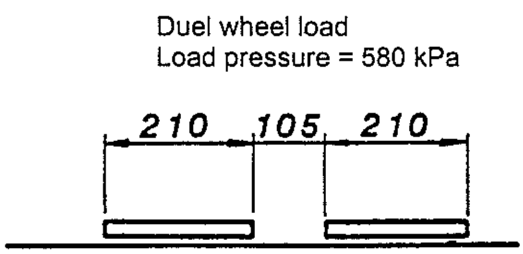


Figure 5.2 Configuration of the equivalent design axle used in New Zealand (from Transit New Zealand 1989a). Dimensions in mm.

5.3.2 Equivalency

Research shows that the appropriate value of the load equivalence exponent is dependent on the pavement structure, the predominant mode of pavement distress and the axle configuration (Pidwerbesky 1990). The AASHTO design method recognises the dependence on pavement structure by varying the load equivalence exponent according to the weighted structural number of the pavement. Investigators have found load equivalence exponent values ranging from about one to eight with most values within the range two to six. Extensive evaluation of load/deformation data from the AASHO Road Test produced an average exponent of 4.15. This has been modified slightly by most roading authorities (including Transit New Zealand) to a value of four, hence the widely known *fourth power law* for pavement damage. Research findings reported by Maree et al. (1982) suggest that a value somewhat less than four would be appropriate for unbound aggregate pavements due to a decreased sensitivity to loading for this type of pavement.

Since unbound aggregate pavements are commonly used in New Zealand, this finding would suggest that the present design traffic characterisation is over-conservative for axle loads greater than one EDA and non-conservative for axle loads less than one EDA. A summary of load equivalency exponent values from various researchers is presented in Table 5.2.

The variation of the load equivalency exponent values outlined in Table 5.2 can have a significant effect on the traffic loading adopted for design, and correspondingly influence the design solution. An approximation of the total number of axle passes (including an estimated growth rate) in each load category over the design life of the pavement must also be made.

Table 5.2 Load equivalency exponents used by several countries (from Pidwerbesky 1990).

Country of Research	Criterion	Exponent	Axle/Tyre Configuration
Australia	Cracking Rutting	2.0 3.3 - 6.0	Dual & Single Tyre Single Axle
Finland	Cracking Cracking	3.3 4.0	Dual Tyre, Single Axle Dual Tyre, Tandem Axle
France	Cracking Rutting	2.0 8.0	Dual Tyre, Single Axle Dual Tyre, Single Axle
Italy	Cracking	1.2 - 3.0	Various
USA	Loss of Serviceability Loss of Serviceability Rutting Rutting Cracking Cracking	4.4 4.9 4.2 4.8 1.3 - 1.7 1.9	Single Axle Tandem Axle Single Axle Tandem Axle Single Axle Tandem Axle

5.4 Characterisation of Materials

5.4.1 Subgrade Materials

The subgrade is an important component of the pavement structure because it supports the overlying pavement and the superimposed traffic loads. Therefore the subgrade must be appropriately characterised in the pavement design model. The Transit design method (Transit New Zealand 1989a) treats the subgrade as a homogeneous, isotropic material which displays a linear elastic stress/strain response. Unfortunately, none of these assumptions are true but they are adopted for the sake of simplicity.

Most subgrades comprise natural ground which can be highly variable transversely, vertically and along the pavement. In New Zealand, the relatively recent geological processes have produced a wide range of subgrade soils ranging from clays to gravels. The stress/strain response of these soils is nonlinear (Pyke 1978, Martin 1989), i.e. the elastic modulus of the materials is dependent on the prevailing state of stress (Figure 5.3).

The analytical design approach places critical importance on the elastic modulus of the pavement subgrade. The final design is very sensitive to this parameter because the subgrade elastic modulus not only influences the magnitude of the vertical subgrade strains but it also controls the elastic modulus of the overlying sub-base and basecourse layers by way of the modular ratio relationship

$$k = E_{\text{unbound layers}} / E_{\text{subgrade}} \qquad \text{Equation 16}$$

It is therefore extremely important that the elastic modulus of the subgrade *is* measured using a suitable and reliable testing technique. Three techniques that can be used are CBR, dynamic triaxial test, some in situ tests, and advantages of these are now discussed.

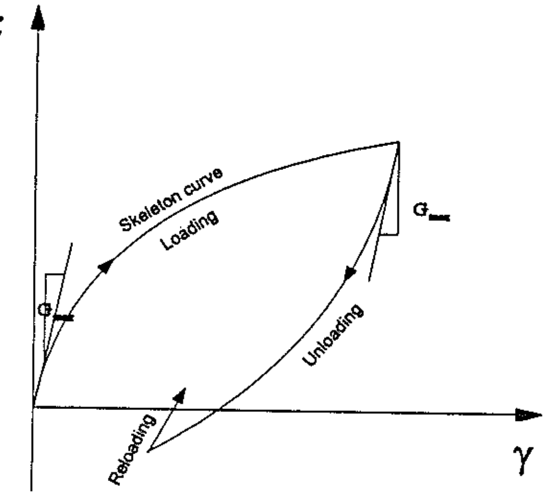


Figure 5.3 Non-linear stress/strain response of soils under cyclic loading (from Martin 1989).

• *CBR Test* The elastic modulus of a subgrade material is generally determined using the CBR test. In this test, the force required to push a 50 mm diameter plunger into a specimen of the subgrade soil is used to determine the CBR value of the material. This operation may be undertaken in situ if the designer is confident that the natural water content is appropriate, or in the laboratory. In the laboratory the soil is compacted in a steel mould and then soaked for at least four days. The CBR test result is expressed as a percentage of the CBR value of a specimen of high quality crushed rock. The CBR value can be empirically related to the elastic modulus.

The initial investigation of the relationship between the subgrade elastic modulus (E_{SG}) and CBR was carried out by Heukelom and Klomp (1962). These authors suggested the following relationship:

$$E_{SG} = k(CBR) \qquad \text{Equation 17}$$

where the constant *k* varied between 5 and 20. An average value for *k* of 10 was recommended.

The relationship adopted in the Transit New Zealand design manual (1989a) is as follows:

$$E_{SG} = 20\ (CBR)^{0.64} \quad \text{for CBR} =< 13 \qquad \text{Equation 18}$$

$$E_{SG} = 8\ (CBR) \quad \text{for CBR} > 13 \qquad \text{Equation 19}$$

where E_{SG} = subgrade elastic modulus (MPa)
CBR = California Bearing Ratio (%)

The AUSTROADS pavement design procedure (1992) uses a linear relationship between E_{SG} and CBR as follows. The AUSTROADS procedure also includes an anisotropy ratio of 2, i.e. the vertical subgrade elastic modulus is twice the horizontal elastic subgrade modulus:

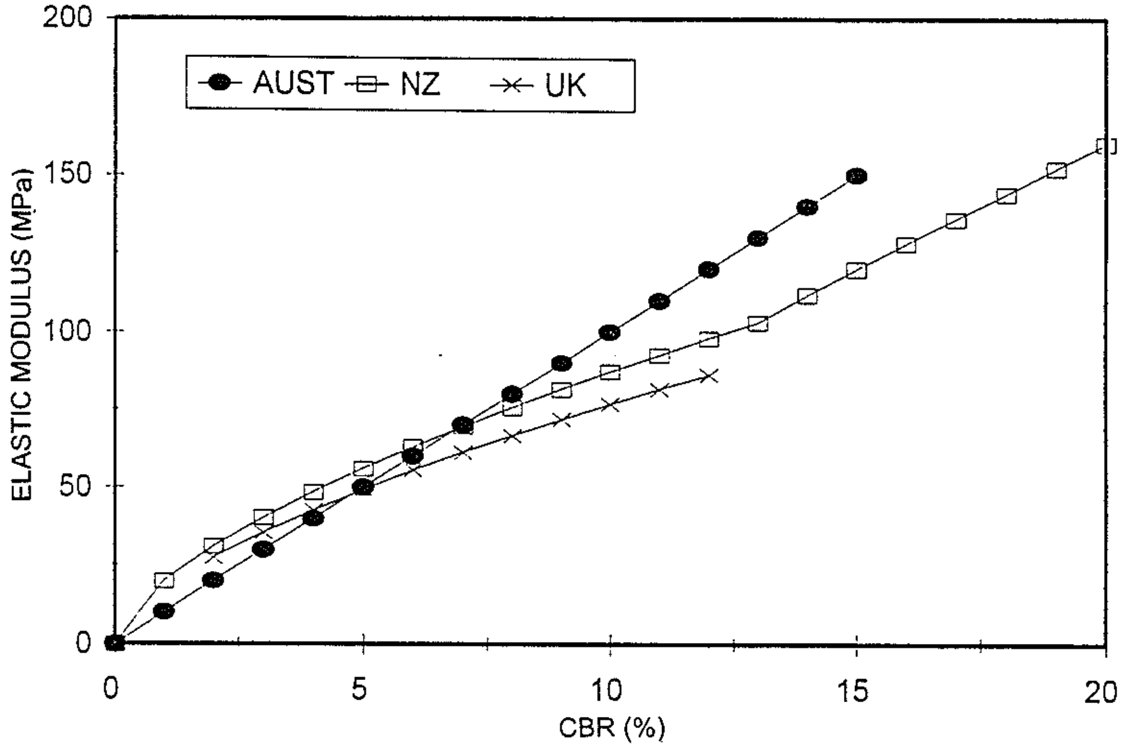
$$E_{SG} = 10\ (CBR) \quad E_{SG} <= 150\ \text{MPa} \qquad \text{Equation 20}$$

In the UK, the *TRRL LR1132* (Powell et al. 1984) pavement design procedure adopts a relationship between E_{SG} and CBR which is similar to the non-linear portion of the Transit design method relationship, i.e.:

$$E_{SG} = 17.6\ (CBR)^{0.64} \quad \text{for } 2 <= \text{CBR} =< 12 \qquad \text{Equation 21}$$

A plot of subgrade elastic modulus versus CBR is presented in Figure 5.4 to compare the various relationships described above. Figure 5.4 clearly shows that, of the three methods of subgrade characterisation presented, the Transit design method is the least conservative at low values of CBR, i.e. up to approximately 7%. At CBRs above 7% the Transit design method is intermediate between the Australian and UK methods. Another variation is with regard to subgrade elastic modulus limits, in that the Australian method has an upper limit of E_{SG} equal to 150 MPa, the UK method has a lower limit of CBR equal to 2% and an upper limit of CBR equal to 12%, and the Transit design method has no specified limits.

Figure 5.4 Relationships between elastic modulus for subgrades and CBR as used in New Zealand, Australia and the UK.



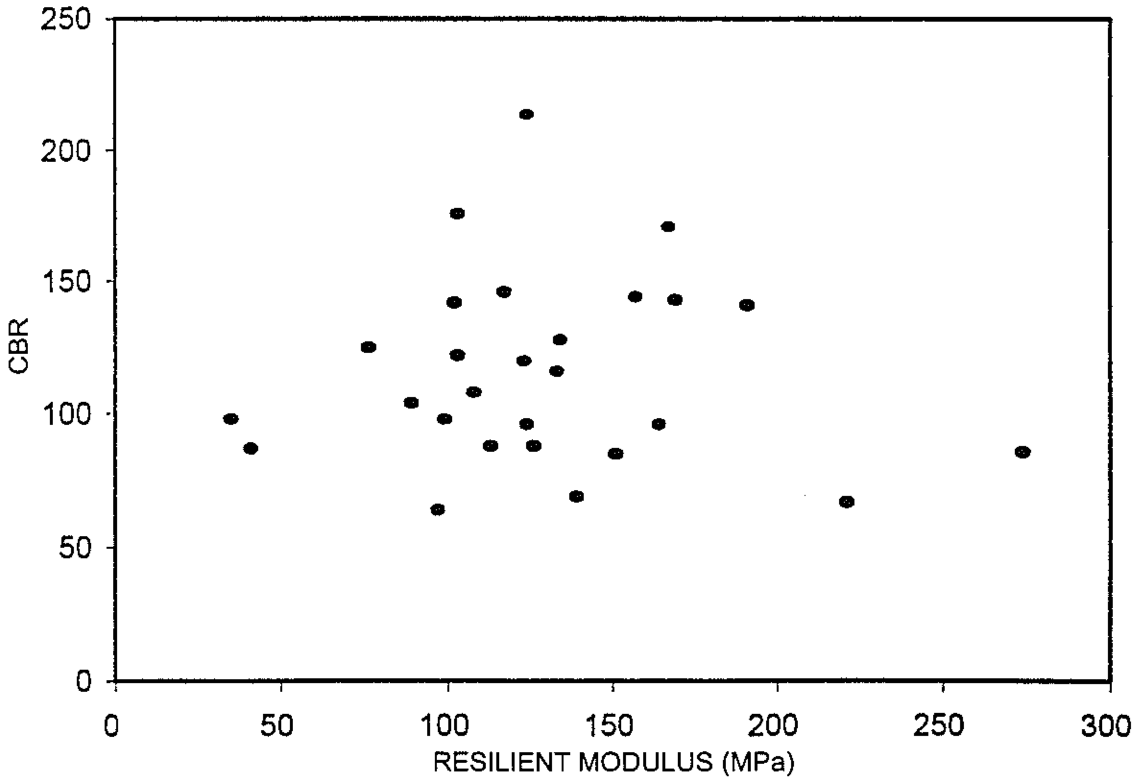


Figure 5.5 Plot of CBR versus resilient modulus for a wide range of subgrade soils (after Sweere 1989).

Criticism of CBR Test: The use of the CBR test has received criticism in the technical literature because it is not an elastic test. Very high stresses (infinite in theory) occur near the edges of the plunger with most of the deformation being plastic, even though the test is used to evaluate an elastic parameter. Containment of the specimen within a stiff steel mould also misrepresents the confinement conditions which would be experienced by a subgrade soil (Morgan 1972). The highly variable ratio of plastic to elastic deformation displayed by subgrade soils means that the CBR test is inappropriate for the evaluation of an elastic parameter such as subgrade elastic modulus (Sweere 1989).

A plot of resilient modulus versus CBR for subgrade soils as reported by Sweere is presented in Figure 5.5. The plot shows a considerable scatter of experimental data and confirms the poor relationship between these parameters. Similar findings have been reported by Brown et al. (1987). The use of the reported relationships between CBR and E_{SG} is therefore considered to be questionable particularly given the sensitivity of the current analytical design method to the subgrade CBR parameter.

Black (1962) proposed an empirical relationship between CBR and plasticity index (PI) for cohesive materials. This provides further confirmation of the fact that the CBR parameter is not suitable for the characterisation of a material's elastic properties.

Most roading authorities specify the use of the CBR test to characterise the subgrade modulus because of its simplicity and widespread acceptance. An exception is the AASHTO design method which leaves the designer to choose an appropriate test method, e.g. the dynamic triaxial test, the results of which are then used to determine the *soil support value (S)*. Several US states use the CBR test because the dynamic triaxial test is relatively expensive.

• The *dynamic triaxial test* provides good characterisation of the subgrade modulus because it can simulate the stress state experienced by an element of subgrade soil. In addition, the triaxial test apparatus accommodates the simple investigation of test parameters such as applied stress or strain amplitude, moisture content, confining pressure and drainage conditions. The ability to accurately control the magnitude of the applied axial stress or strain is very important since the modulus of subgrade soil is dependent on the prevailing stress/strain conditions (Sun et al. 1988, Pender et al. 1992).

One disadvantage of the triaxial apparatus is its inability to provide a continuous rotation of principal stress axes, as experienced by an element of pavement under a moving wheel load. The apparatus can only impart instantaneous 90 degree rotations of principal stress axes.

• Other *in situ testing methods* are frequently used to indirectly determine the subgrade CBR, e.g. Scala penetrometer, Dutch cone penetrometer, standard penetration test, Clegg hammer, etc. These methods introduce a further level of inaccuracy in terms of the empirical relationship with CBR. In addition, they are often used inappropriately, e.g. in situ dynamic tests can impose large dynamic pore pressures, and hence they should not be used in fine grained soils.

5.4.2 Basecourse and Sub-base Materials

5.4.2.1 Function of layers

The function of the basecourse layer is to dissipate the applied axle loads to such an extent that the underlying sub-base material is not over-stressed. Similarly, the function of the sub-base layer is to distribute the stresses imposed from the basecourse layer so that the subgrade material is not over-stressed. The sub-base also provides the following:

- a platform for the operation of construction vehicles; and
- a separation layer to protect the basecourse from contamination.

5.4.2.2 Modelling assumptions

In the design of unbound granular pavements, accurate modelling of the behaviour of both the basecourse and sub-base layers is very important if the design is to be realistic. The Transit design method treats the basecourse and sub-base layers as a single layer of homogeneous, isotropic, linear elastic material. These assumptions may over-simplify the actual situation to such a degree that they virtually negate the analysis.

• *Anisotropy* Aggregate layers are never completely homogeneous. They will inevitably contain local areas of variable compaction, moisture content, aggregate grading, etc. These variations should be minimised if a high standard of construction and supervision are achieved. A statistical characterisation of material properties may be adopted if variations are suspected.

The Transit design method assumes that the granular layers are isotropic, i.e. the elastic modulus is constant in all directions. The issue of anisotropy is not widely discussed in the literature, nor is it addressed by many roading authorities in their design methods. One authority which does consider anisotropic basecourse and sub-base conditions is AUSTROADS. The AUSTROADS pavement design method assumes that the vertical modulus of aggregate layers is twice the horizontal modulus owing to the compaction of aggregate in horizontal layers and the induction of preferred particle orientations. This assumption has been justified on the grounds of improved pavement deflection predictions in full scale pavement trials (AUSTROADS 1992).

• *Non-linear Response* Several investigators, e.g. Dehlen and Monismith (1970), Brown (1974), Boyce et al. (1976) and Thompson and Smith (1989) have studied the behaviour of unbound aggregates under repeated triaxial loading and found that the stress/strain response is non-linear, hence the resilient modulus is dependent on the prevailing state of stress, i.e.:

$$M_R = k1(\theta)^{k2}$$ Equation 22

where M_R = resilient modulus
$k1, k2$ = material constants
θ = sum of principal stresses

The non-linear resilient modulus relationship presented above is commonly referred to as the *k-θ relationship*. Brown and Pappin (1985) developed a finite element computer program *(SENOL)* which utilised a modified k-θ material model called the *contour model*. This was established from the results of repeated load triaxial tests using numerous stress path variations. Pavement analyses using both *SENOL* and a linear elastic computer program called *BISTRO* showed that the linear elastic approach tended to over-estimate the magnitude of pavement surface deflections.

• *Elastic Modulus* Figure 5.3 clearly shows that for soils at low states of stress, the elastic modulus of the material (represented by the slope of the tangent to the stress/strain curve) is relatively great. Conversely, at high states of stress the elastic modulus of the material is relatively low.

The situation for unbound aggregates is however reversed. For these materials the elastic modulus is relatively low at low states of stress and high at high states of stress. Since the state of stress varies continuously over the thickness of a pavement there is a non-uniform distribution of elastic moduli within each pavement layer.

Many authors consider however that the imposed stresses are sufficiently small so that a linear elastic material model is justified. The elastic modulus distribution is not only affected by traffic-induced stresses, it is also influenced by the confining stresses imparted by compaction and the weight of the pavement structure (Toan 1975).

The maximum elastic modulus achievable for a compacted granular layer is dependent on the modulus of the underlying layer. This is because the layer being compacted must receive adequate support from the underlying material for the compactive effort to be effective. The maximum modulus of an aggregate layer can be determined using the *modular ratio (k)*, i.e.:

$$E_2 = kE_3$$ Equation 23

where E_2 = basecourse elastic modulus ($E_{\text{unbound layers}}$ in Section 5.4.1)
E_3 = subgrade elastic modulus (E_{subgrade} in Section 5.4.1 of this report)

The modular ratio adopted for the Transit design method is that given by Dormon and Metcalf (1965) i.e.:

$$k = 0.2h^{0.45}$$ Equation 24

where h = thickness of the granular layer (mm)
and $2 < k < 4$

An alternative modular ratio relationship has been suggested by Edwards and Valkering (1970), i.e.:

$$k = 0.58h^{0.45} \qquad \text{Equation 25}$$

Doddihal and Pandey (1984) analysed a number of pavement structures using the finite element method and found that the Edwards and Valkering relationship for modular ratio is appropriate for well graded, well compacted limestone aggregate layers whereas the Dormon and Metcalf relationship is appropriate for poorly graded aggregate layers.

Claessen et al. (1977) reported that the modular ratio should fall within the range two to four. However the results of investigations reported by Kennedy (1985) indicate that k-values of up to six are possible, while Doddihal and Pandey (1984) found k-values greater than seven.

• *Sub-Layering in Design* Although the concept of a modular ratio is accepted, its generalised use implies that the analytical design method is insensitive to the nature of the basecourse and sub-base materials. This is accentuated when the basecourse and sub-base layers are combined as a single layer in a linear elastic analysis as in the Transit design method. By combining the layers in the model the method ignores the superior quality of the basecourse aggregate. It also disregards the benefit of having the compacted sub-base layer in place when the basecourse layer is placed, since the relatively stiff sub-base layer provides a superior compaction platform compared to the platform offered by the subgrade.

The AUSTROADS pavement design method allows the granular layers to be divided into 50 to 150 mm-thick sub-layers with a maximum modular ratio of two between adjacent sub-layers. The incorporation of aggregate sub-layers in the design is considered to be beneficial as it replicates the actual pavement structure more closely than a single layer model.

• *Pseudo-nonlinear Analysis* A pseudo-nonlinear pavement analysis which recognises the stress dependence of the granular layer's elastic modulus can be carried out using a pavement model in which the aggregate layers are divided into a large number of sub-layers. The method uses an iterative technique and linear elastic theory to produce a result which is significantly superior to the basic linear elastic approach. However, the non-linear stress/strain relationship of the materials must be known from triaxial or similar test results.

The pseudo-nonlinear method starts with the designer proposing a trial pavement cross-section which is divided into as many sub-layers as can be readily accommodated. Each sub-layer is assigned an elastic modulus value according to the judgement of the designer. The structure is then analysed using linear elastic theory with the resulting average confining stress being noted for each sub-layer. The appropriate modulus for each sub-layer can be calculated from the stress/strain relationship and compared with the initial modulus input by the designer. This process is iterated until the input moduli coincide with those calculated from the analysis (to an acceptable accuracy). The accuracy of the solution clearly increases with the number of sub-layers employed, but the computation time also increases significantly.

• *Finite Element Method* An extension to the iterative multi-layer analysis approach is the finite element method which not only subdivides the structure horizontally, but also vertically. Several investigators have described the advantages of the finite element method, although it remains a research tool because of its complexity.

5.5 Permeability of Granular Layers

5.5.1 Introduction

Water is an inherent part of the unbound granular pavement system. It is required to achieve the efficient placement and compaction of aggregate. Water can act as a lubricant between aggregate particles, reducing inter-particle friction and therefore decreasing aggregate stability as well as softening subgrade soils. Water may also interact with poor or marginal aggregates to mobilise deleterious secondary minerals. Therefore, in the ideal situation, water would be excluded from the upper one metre of the pavement structure after construction.

In a saturated pavement, stress is transferred to the water, effectively reducing the stability of the constituent materials. In reality water cannot be excluded totally, although the provision of adequate drainage systems and a high quality surface seal will minimise its adverse effects.

5.5.2 Aggregate Layer Permeability

Discussion in the literature concerns the suitability of aggregate layers to function as drainage courses. To drain successfully, a material must possess adequate permeability so that water can escape from the structure without a significant build-up of pressure. Aggregate layer permeability is influenced by several factors, i.e. particle size distribution, void ratio, composition, fabric and degree of saturation (Rutter 1986).

• *Pore Water Pressures* A fundamental characteristic of all soil (and aggregate) behaviour which is closely related to material permeability is the generation of pore water pressures. The concepts of pore water pressure and *effective stress* were discovered by Karl Terzarghi in the 1920s and marked the most significant advance in the history of geotechnical engineering. The effective stress is defined as the difference between the total stress existing at an element of soil (related to overburden pressure) and the pore water pressure at that point.

Pore water pressures are generated by the sharing of an applied load between the mineral (soil or aggregate) skeleton and the water in the spaces between soil particles.

The result of an increase in pore water pressure is a decrease in the stability, or effective strength of the material since the pore water carries a proportion of the applied load but itself has no shear strength. The pressurised pore water also tends to separate adjacent aggregate particles, thus reducing inter-particle friction and particle interlock.

• *Mechanical Analogy* Lambe and Whitman (1979) presented a mechanical analogy to explain the concepts of pore water pressure and effective stress (Figure 5.6). An element of saturated soil is placed in a cylinder, as shown in Figure 5.6(a).

The pore water and mineral phases are then separated, as shown in Figure 5.6(b), with the mineral phase represented by a spring. The spring supports a piston which has a weight corresponding to the weight of the material lying above the element of soil. The piston includes a valve which is initially closed.

If a load is instantaneously applied to the soil via the piston (Figure 5.6(c)), then that load is supported by the water and the spring. Since the water is virtually incompressible compared to the spring, the water carries most of the applied load and the water pressure increases.

Now the valve in the piston is opened slightly (Figure 5.6(d)) and allows some water to escape from the cylinder. The water pressure decreases and the piston moves down until an equilibrium position is achieved.

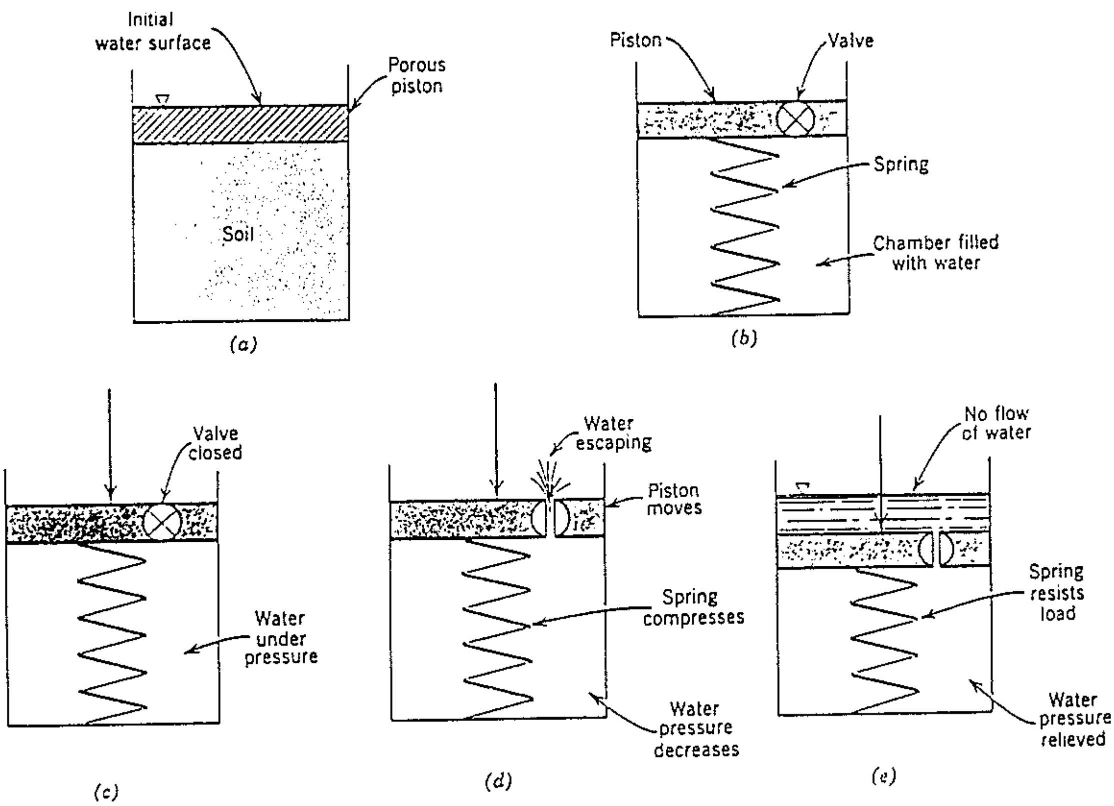
Under these circumstances the water pressure has reduced to hydrostatic level and all of the applied load is resisted by the spring (Figure 5.6(e)).

The reduction in volume of the material which results from dissipation of the pore water pressure is termed *consolidation*. The rate of pore pressure dissipation is dependent on the permeability of the soil (the magnitude of the valve opening in the mechanical analogy described above) and the length of the drainage path.

Pore water pressures in very permeable soils, e.g. course granular materials, may dissipate virtually instantaneously upon loading. Clays which have very low permeability may require tens of years to dissipate pore water pressures, especially if the drainage path is relatively long.

Bartley and Woodward (1981) calculated that pore pressure dissipation is achieved between successive vehicle passes if the aggregate permeability is greater than or equal to approximately $1.2x10^{-5}$ m/sec. Rutter (1986) carried out measurements of in situ aggregate permeability at a number of sites around Auckland and showed that a value of about 10^{-4} m/sec is typical. This suggests that significant pore pressures should not develop, at least not in the pavements considered in Rutter's study.

Figure 5.6 Mechanical analogy of load sharing in soils - see text for explanation of mechanism (from Lambe and Whitman 1979).



Aggregate permeability is significantly influenced by the *particle size distribution*. In particular, the size of the particles comprising the finest 10% of the aggregate (D_{10}) is a significant parameter. Bartley (1987) found that a D_{10} value of 0.425 mm was critical to aggregate stability. This parameter has been built in to the TNZ M/4 specification (1985) for basecourse aggregates.

The Transit New Zealand specifications stipulate that the upper region of the sub-base layer must comply with a permeability criterion ($\geq 10^{-4}$ m/s) to ensure that positive pore water pressures cannot develop in the basecourse layer. This is a contentious issue since the deliberate opening of an aggregate grading to promote permeability, results in a reduced density and an associated reduction in layer stability. It is also considered that most aggregates (of moderate hardness) compensate for open gradings by crushing at the contact points when loaded in a pavement until sufficient fines are generated to fill the voids. This fine material reduces the permeability of the layer making the effective preservation of a specific drainage zone impracticable.

6. REVIEW OF TECHNICAL LITERATURE FOR PAVEMENT MATERIALS

6.1 Attributes of Aggregates

Aggregates utilised in pavement basecourse and sub-base layers must possess a number of attributes to function successfully. These attributes can be divided into three main categories as follows:

- strength;
- permanence; and
- nature of fine fraction.

Numerous tests are available to quantify the attributes of an aggregate. The results of these tests can be compared with specified compliance parameters so that the suitability of a material source can be readily assessed. Details of relevant aggregate attributes are discussed in detail in Sections 6.1.1 - 6.1.3 of this report.

6.1.1 Strength

The strength attribute refers to the ability of an aggregate to withstand the repeated stresses imposed by axle loads. Strength is required to distribute surface loads sufficiently so that the underlying materials are not over-stressed. Aggregate strength is predominantly developed by the interlock of particles and inter-particle friction. To achieve these mechanisms, a number of important material characteristics must be considered, e.g. compaction, particle size distribution, particle shape and texture, and aggregate crushing strength.

6.1.1.1 Compaction

The unbound nature of the aggregate implies that a high degree of confinement is required for the material to be stable under loading. Confinement is achieved by compacting the layers to establish high horizontal stresses. These stresses develop further as the pavement matures under traffic loading.

Compaction arranges the particles into the closest possible packing regime so that further loading imposed by in-service traffic produces negligible permanent deformation. Compaction also densifies the aggregate to maximise the number of contact points with adjacent particles. This increases the stability of the material by promoting inter-particle friction and particle interlock. By providing increased particle contact areas the inter-particle stresses are reduced and particle breakdown is correspondingly reduced.

Compaction of loose cohesionless materials is achieved by the relocation of particles into a close packed arrangement by the provision of small magnitude cyclic shear strains. Small shear strains cause the voids in a loose aggregate structure to be filled by relocated particles while relatively large shear strains cause tight areas of aggregate to dilate and loosen. For each cycle of shear strain the densification achieved while the shear strain is small exceeds the loosening which occurs when the shear strain is relatively large. This behaviour is known as the *small strain densification bias* (Youd 1972). For this reason, the use of relatively light vibratory compaction plant may be preferred.

6.1.1.2 Particle size distribution

The density of a compacted aggregate is significantly influenced by its particle size distribution. A continuously graded aggregate contains a wide distribution of particle sizes allowing relatively small particles to be located in the voids between larger particles. Such an aggregate grading provides for ease of handling and has the least potential for segregation. A continuously graded aggregate is achieved by specifying a particle size distribution which conforms to the following equation (Bartley et al. 1988):

$$p = 100(d/D)^n \qquad \text{Equation 26}$$

where p = percentage by weight of aggregate passing a sieve aperture d
D = maximum particle size
n = constant (Talbot's exponent)
(n ranges between 0.35 and 0.50 for a dense grading, Bartley et al. 1988)

The particle size distribution specified in the TNZ M/4 (1985) specification conforms to Talbot's exponent (n) values of approximately 0.40 to 0.64, with a mean of 0.51, i.e. slightly higher than the range given by Bartley et al. (1988) for a dense grading.

The TNZ M/4 (1985) specification provides a relatively narrow envelope with a high mean value of n corresponding to a relatively open grading. In addition, M/4 requires that the grading curve is relatively smooth. A set of grading curve uniformity criteria is provided to ensure that the curve does not wander around excessively within the grading envelope.

An aggregate grading specification based on a statistical approach has been suggested by Siddiqui et al. (1986) to objectively achieve quality control on a rational basis. Their specification includes a price adjustment component to penalise the aggregate supplier if the material does not comply with the specified grading limits.

Effect of Grading Thom and Brown (1988) carried out a number of performance tests on aggregates of various gradings. They found that the dry density peaked at an n-value of 0.30 while the elastic modulus increased slightly with increasing n-values. The angle of internal friction increased significantly with increasing density with an optimum n-value of about 0.35. Specimens containing a very high percentage of fine particles showed sudden decreases in stiffness under repeated loading and they retained large quantities of water which could not be easily drained. The detrimental effect of excess fines was also reported by Marek (1977). Excess fines cause the larger particles to float amongst the fine particles, decreasing the degree of particle interlock. Thom and Brown (1988) confirmed that aggregate permeability is strongly related to grading with the open gradings clearly being more permeable than the dense gradings.

Open Grading Open-graded aggregates of moderate particle strength are considered to automatically adjust their grading to suit the prevailing stress conditions. If the inter-particle stresses are very high, some degree of attrition will occur that increases the inter-particle contact area and hence reduces the inter-particle stresses. This self-compensating behaviour is an important feature of aggregate performance and should be expected, particularly during construction when using heavy compaction plant. Adjustment of the specified particle size distribution should be considered if significant grading changes are anticipated as they may affect certain aspects of the material's behaviour, e.g. reduced permeability.

Compatibility of Grading The particle size distribution of an aggregate must be compatible with that of the materials on either side of it so that migration of particles either into or out of the layer does not occur. An effective aggregate filter may be required to separate the subgrade from the aggregate layers as intrusion of fine materials can influence both the stiffness and permeability of the material (Jorenby and Hicks 1986). The generally accepted grading guide for filters is that the D_{15} size of the filter should be between four times the D_{15} and four times the D_{85} sizes of the adjacent material. Alternatively, a geotextile separation material can be used.

Aggregate Density The importance of the compaction process is undermined in the TNZ B/2 (1987) specification because no criterion for aggregate density is provided. The document states that the nuclear densometer, Clegg impact hammer or Benkelman beam may be appropriate test equipment, but no target density is specified.

The specification states that the aggregate should be compacted until it reaches a *plateau* or *refusal* density. This approach is misleading since the plateau density is significantly influenced by the characteristics of the compacting plant and the particle size distribution of the aggregate.

Most other roading authorities specify a target dry density with respect to the maximum dry density obtained from either standard or modified laboratory compaction tests. This may also be expressed in terms of a maximum percentage of total voids, i.e. the higher the density, the lower the percentage of total voids. Examples of relevant density criteria have been presented in Section 4 of this report.

Strength Criterion An alternative approach to the specification of a standard density is to insist that each component of the pavement achieves a certain strength criterion as the construction progresses. The German practice is to carry out plate-bearing tests on the aggregate layers with compaction being continued until a specified minimum bearing resistance is achieved. This approach not only verifies that sufficient density has been achieved, but also checks the validity of many other assumptions that are made during the design of the pavement. The importance of the compaction component of construction is considered to justify the specification of some form of standard density or strength criterion, especially given that the cost of aggregate compaction is low relative to the total cost of pavement construction.

6.1.1.3 Particle shape and texture

The physical configuration of individual particles can have an influence on the performance of an aggregate. Particle configuration includes the particle size, shape, angularity and surface texture. Barksdale and Itani (1989) describe four categories of particle shape, viz. cubic, disc, blade and rod.

Elongated shapes for example may pass sieves and screens in either the long or short dimension and may distort gradings accordingly. They may also span coarse graded zones in a layer of aggregate to cause holes which can collapse suddenly, especially given that the elongated particle shape attracts relatively high bending stresses.

The influence of particle size on aggregate strength is unclear. Koerner (1970) reported that aggregate strength decreased with increasing particle size, while triaxial tests carried out by Marachi et al. (1972) showed that aggregate strength increased with increasing particle size. To complete this variation of results, Holtz and Gibbs (1956) reported that aggregate strength is not significantly influenced by particle size. This view is shared by Selig and Roner (1987).

Vallerga et al. (1956) tested both rounded and angular aggregates using a triaxial apparatus and found significantly higher angles of internal friction for the angular aggregate. They also reported that the surface texture of the particles influenced the strength characteristics with a rough texture producing a higher friction angle. These results are reasonable since angularity and surface roughness promote particle interlock and inter-particle friction respectively.

Selig and Roner (1987) showed that flaky aggregates demonstrate higher shear strengths in the triaxial apparatus, but that they are prone to breakage, and correspondingly display increased plastic strains and decreased stiffness. This was confirmed by Gur et al. (1967) who reported higher triaxial shear strength for flaky aggregates and also a potential for pavement rutting double that of unidimensional aggregates.

Shape Parameters Several parameters are used to characterise *particle shape*,

Flatness ratio (p) $= s/i$

Elongation ratio (q) $= i/g$

Shape factor (F) $= q/p$

where s = shortest dimension of particle
i = intermediate dimension of particle
g = greatest dimension of particle

In addition, the other shape-related parameters are:

Roundness (R)	Measure of curvature of particle edges and corners expressed as a ratio of the average curvature of the particle as a whole.
Angularity (A)	Description of wear of edges and corners.
Sphericity (Ψ)	Ratio of the surface area of a sphere of the same volume as the particle to the surface area of the particle (Barksdale and Itani 1989).
Flakiness ratio (f)	Percentage of particles with a least dimension of less than 0.6 times their average dimension (Lay 1984).

The particle shape parameters described above are predominantly influenced by the aggregate structure and method of formation. River gravels would be expected to have high roundness and sphericity, and low angularity and flakiness, related to their alluvial origin. Conversely, crushed rock will be very angular because of the presence of broken faces. The other shape parameters are influenced by the structure of the parent rock, e.g. laminar structures such as schist and shale contain preferred planes of cleavage and often result in flaky aggregate particles. Angular aggregates may be relatively difficult to work because they are less *mobile* than rounded aggregates. They are however more easily compacted to produce a stable particle arrangement having greater interlock and interparticle friction.

Most basecourse aggregate specifications specify some percentage of the particles that must have a certain minimum number of broken faces. The presence of broken faces not only guarantees adequate angularity, but also generally ensures a rough surface texture. The TNZ M/4 (1985) specifies a minimum of two broken faces on a minimum of 70% of the particles between the 37.5 mm and 4.75 mm sieves. This criterion is more stringent than most other aggregate specifications reviewed in this project.

Transit New Zealand specifications have no other limitations on particle shape. Some roading authorities use flakiness ratio, e.g. Queensland Department of Transport ($f \leq 35\%$) and UK Department of Transport ($f \leq 40\%$). No other particle shape-related parameters have been identified in this study.

6.1.1.4 Aggregate crushing resistance

Many roading authorities specify a minimum aggregate crushing resistance to ensure that the particles do not break down excessively under loading. The aggregate contact stresses associated with traffic loading are relatively low compared with the crushing strength of good quality rock. However some degree of particle attrition may be expected during construction, principally in the compaction process. Clearly, the production of fine particles is dependent on the strength of the parent rock, and hence aggregates produced from very hard rock sources may be deficient in fines.

The crushing resistance of an aggregate specimen is dependent on a number of factors, i.e.

- mineral composition and structure of the parent rock;
- degree of weathering;
- particle size distribution;
- density;
- angularity and surface texture of the aggregate particles; and
- water content.

The TNZ M/4 (1985) specification requires a crushing resistance of not less than 130 kN for basecourse aggregate. This condition is achieved if the sample has produced less than 10% of fines when so loaded that the peak load is reached in ten minutes.

The 10% fines test is commonly specified as a measure of crushing resistance, i.e. the load required to produce 10% fines from an aggregate sample. In New South Wales and Queensland the specification requires both a minimum wet aggregate strength and a maximum dry/wet strength ratio. A similar test called the *aggregate crushing value test* is described by Lay (1984). In this test an aggregate specimen is compacted into a cylinder and exposed to a compressive force of 400 kN applied at a constant rate of 670 N/s. When the load is removed, the additional percentage of fines passing the 2.36 mm sieve is the aggregate crushing test value.

6.1.2 Permanence

Aggregate permanence refers to the ability of the material to resist the adverse effects of certain loading, environmental and mineralogical factors that cause degradation and/or abrasion. In some respects these factors overlap with the strength criteria discussed in Section 6.1.1.4.

6.1.2.1 Degradation resistance

Degradation of rock particles by weathering is resisted by good quality basecourse and sub-base aggregates. The TNZ M/4 (1985) specification states that a basecourse aggregate should fall into the AA, AB, AC, BA, BB or CA categories when subjected to the *weathering resistance test*. The objective of this test is to determine the resistance of a sample of aggregate to degradation under conditions of wetting and drying, and heating and cooling. The outcome of the test is based on the percentage of aggregate retained on the 4.75 mm sieve and the result of a cleanness test.

The *cleanness test* determines the amount, fineness and character of the clay-like material in a sample of aggregate. The test uses a sedimentation process and assigns a cleanness value (CV) in the range 0 to 100 where high CV values correspond to a low presence of clay-like materials. The weathering resistance test produces a result called the *quality index* determined as outlined in Table 6.1.

Table 6.1 Definition of quality index parameter from the weathering and cleanness tests.

Cleanness Value	**Quality Index % Aggregate Retained on 4.75 mm Sieve**		
	96 - 100	**91 - 95**	**≤ 90**
91 - 100	AA	BA	CA
71 - 90	AB	BB	CB
≤ 70	AC	BC	CC

Another commonly specified degradation test is the *Washington test* which measures the quantity and activity of clay-sized particles produced by the attrition of aggregate particles when shaken under water (Lay 1984). The Washington test, which is included in the Queensland aggregate specification, is considered suitable only for igneous and metamorphic rocks. Fielding (1980) reported several problems inherent in the test, not least being the complexity and lack of understanding of the test mechanism.

The test is described as being a measure of the presence of hydrophilic clay minerals on, or close to the surface of, the aggregate particles. The test mechanism is reported to be dependent on a number of factors, e.g. mineralogy, grain size, mineral hardness, porosity, vesicularity and specimen preparation. Minty et al. (1980) reported that the Washington test has poor correlation with other degradation resistance tests.

The resistance of an aggregate to weathering may be evaluated by measuring the *disintegration* which occurs during immersion in various chemical solutions. One such method uses a 20% solution of sodium sulphate. This method has been investigated by Minty et al. (1980) who suggested that a good correlation exists between the sodium sulphate soundness test and the 10% fines test, although the test methodology has faults. Other researchers have concluded that the test is extremely variable and rate it very poorly. A variation of the sodium sulphate test uses a magnesium sulphate solution.

Woodside and Woodward (1989) suggested that no one individual test should be used to evaluate the durability of basalt aggregates. They reported that a *basalt durability index*, based on the porosity, the 10% fines and the methylene-blue absorption tests, is an appropriate procedure since poor performance in one or more of the tests will affect the overall prediction of in-service behaviour.

Sameshima and Black (1980) investigated the effects of temperature and pressure on the weathering resistance of greywacke aggregate samples from the Auckland area, New Zealand. They found that conditions of elevated temperature and pressure promoted the alteration of chlorite present in the aggregate to swelling clay minerals (smectite). The temperatures used in the investigation were however very high and are not expected to be encountered in a typical pavement situation.

6.1.2.2 Abrasion resistance

Abrasion resistance testing gives a measure of the hardness of an aggregate and indicates the likely performance of the aggregate during crushing, compaction and under traffic loads. The most common form of test is the *Los Angeles abrasion test* in which a dry sample of aggregate is rotated in a steel drum containing a number of steel balls. The percentage of fines produced during the test is designated as the Los Angeles value. As may be expected the Los Angeles test is used widely in the US although it is criticised by Lay (1984) for not providing a good representation of in-service conditions. The test is also used in South Australia and Victoria.

Minty et al. (1980) found that the Los Angeles test correlated well with the 10% fines test (dry) and hence suggested that aggregate specifications should include one test or the other, but not both. They also suggested that the applicability of the Los Angeles abrasion test may be limited given that the test specimen is dry, and that in practice the durability of many aggregates is significantly influenced by the presence of water.

Pintner et al. (1987) reported that the Los Angeles test is appropriate for the evaluation of aggregate degradation caused by handling as it correctly simulates the abrasion and impact stresses which occur during aggregate handling. They suggested that removal of the steel balls from the apparatus would be better still. They also found that the degradation of aggregates during handling is increased by high water contents.

The Los Angeles abrasion test is not specified in the TNZ M/4 (1985) specification, although it was in an early version of the TNZ M/6 specification for sealing chip. That early M/6 specification required a maximum Los Angeles value of 20% after 500 revolutions. A selection of US aggregate specifications reported by Barksdale (1989) show allowable Los Angeles values over the range ≤35% to ≤60%, i.e. significantly less stringent than the previous TNZ M/6 document. The current TNZ M/6 (1993b) specification does not have the Los Angeles abrasion test requirement.

Other abrasion test methods have been discussed by Senior and Rogers (1991). One, the *micro-deval test*, evolved from techniques and equipment developed in the grinding industry. It is similar to the Los Angeles test in that the aggregate is placed in a rotating mill with steel balls, but differs in that the sample is tested in a soaked condition.

The *aggregate impact test* measures the percentage of fines produced by the multiple impact of a falling hammer on a confined sample of aggregate. Senior and Rogers (1991) report that the test correlates well with the Los Angeles abrasion test and has the advantage of using inexpensive and portable equipment. The sample may also be tested in a partially or fully saturated state.

6.1.3 Nature of Fine Fraction

The ability to detect deleterious materials which may be present in the fine fraction of an aggregate or liberated as secondary minerals is very important. The presence of highly plastic clay minerals can significantly influence the performance of an aggregate. When wet they lubricate the surfaces of larger particles and reduce the frictional resistance. The presence of water may also interact with swelling clay minerals, resulting in large swelling forces which reduce inter-particle stresses, reduce frictional resistance and hence reduce the shear strength of the aggregate as a whole. The presence of clay-sized particles may not be detrimental if their plasticity is low, or if they are not present in excess quantities, or if the water content is low.

Brennan (1984) stated that, in the presence of plastic fines, a small quantity of water results in high cohesion whereas excess water results in the lubrication of particle surfaces. Bartley (1980) found that basecourse aggregates become unstable when the proportion of material passing the 425 μm sieve exceeds 10% or the water content of the fine fraction exceeds the plastic limit, i.e. when the fine fraction changes its state from being solid (or semi-solid) to plastic. Several test methods are available to detect the presence and nature of clay-sized materials, as detailed in Sections 6.1.3.1-6.1.3.1 of this report.

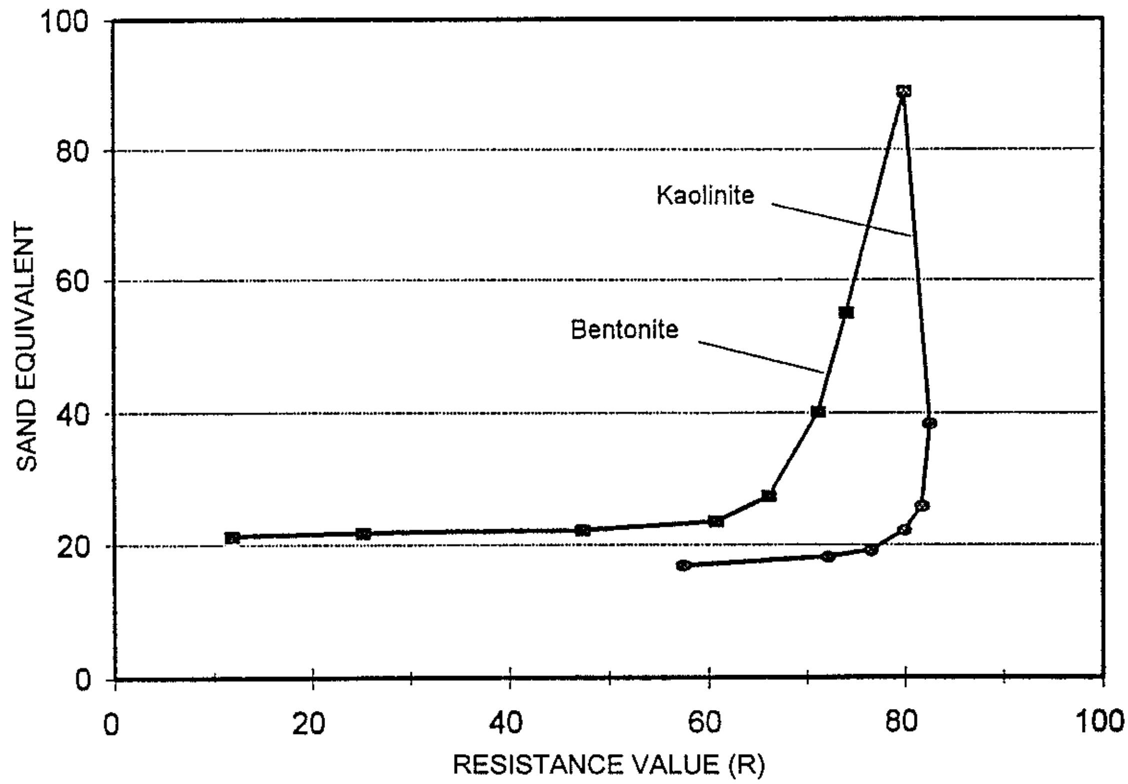
6.1.3.1 Sand Equivalent test

The TNZ M/4 (1985) specification stipulates the use of the *Sand Equivalent (SE) test* to evaluate the fine fraction of a basecourse aggregate. The SE test was developed by Hveem (1953) and uses a sedimentation process to determine the proportion of clay-sized particles relative to the larger sand-sized particles in a specimen of aggregate passing the 4.75 mm sieve. Hveem developed the formulation of the flocculating agent used in the test to accentuate the influence of high plasticity swelling clay minerals and to minimise the influence of less deleterious low plasticity clay minerals.

The SE test has received criticism in the literature, mainly from roading and quarrying practitioners (e.g. Gundersen 1979, Salt 1979, and Ferguson 1979) who state that many aggregates, which did not meet the TNZ M/4 (1985) requirement of SE greater than or equal to 40, performed perfectly adequately when constructed in pavements. Achieving the SE criterion of the specification appears to be almost a New Zealand-wide problem. On the other hand, a wide variation of minimum allowable SE values has been found in the literature, ranging from 22 in California and Georgia (US) to 55 in Victoria (Australia).

In the development of the SE test, Hveem (1953) used the stabilometer apparatus to evaluate the stability of aggregate specimens which had been dosed with montmorillonite and kaolinite. The results (Figure 6.1) show a separate response for each of the two clay minerals. This contradicts the intention of Hveem to compensate for the influence of the clay mineral type by using a specifically formulated flocculating agent. The plot in Figure 6.1 shows that the presence of non-plastic clay minerals (e.g. kaolinite) has no significant effect on the aggregate stability until the SE value drops below about 20. The plot also shows that the presence of plastic clay minerals (e.g. montmorillonite) does have a detrimental effect on aggregate stability, particularly when the SE value drops below about 30. Therefore if the SE test is to be adopted into a specification, the plasticity of the aggregate fines should first be established by some other means. This approach has been suggested previously by Van Barneveld (1983).

Figure 6.1 Plot of SE versus stabilometer resistance value (R) for crushed rock aggregates with added clay minerals (from Hveem 1953).



The dramatic loss of stability which occurs when the SE reduces to about 20, irrespective of the plasticity of the fines, is considered to correspond to the situation where the proportion of fine material becomes excessive and the large particles tend to float amongst the fine particles with a consequent loss of particle interlock. This is consistent with the findings of Marek (1977) discussed in Section 6.1.1.2 of this report. In this state the material would have a very low permeability and would be vulnerable to the generation of high pore water pressures.

Sameshima and Black (1979) reported that the SE test is primarily influenced by the particle size distribution within the fine fraction of the aggregate, rather than by the characteristics of the fines.

6.1.3.2 Clay Index test

Sameshima and Black (1979) developed a test method called the *Clay Index (CI) test* which exploits the affinity that high plasticity swelling clay minerals have for methylene-blue solution. The volume of methylene-blue solution absorbed by one gram of aggregate fines is termed the *clay index value*. The clay index value is assigned a grade, from which its soundness is determined, as outlined in Table 6.2. The distinction between sound and unsound CI grades was however made subjectively and may require verification by laboratory testing.

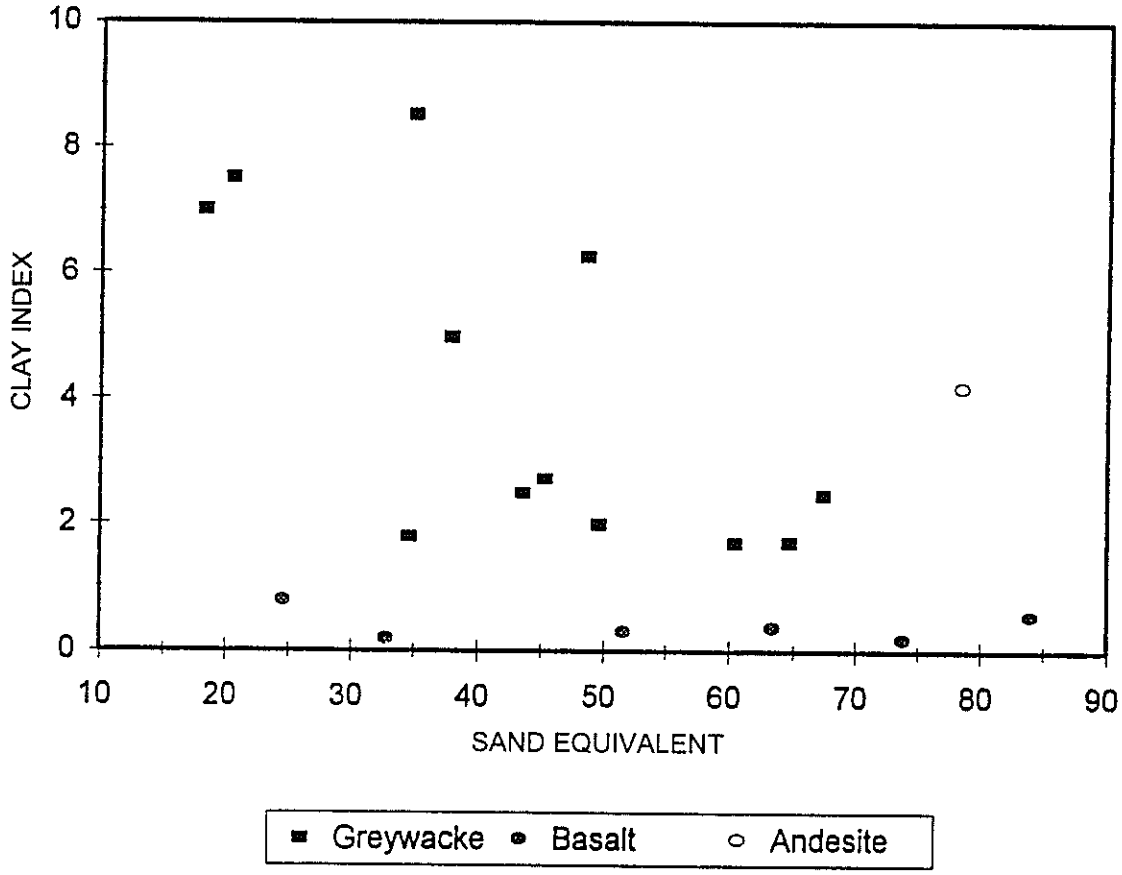
Table 6.2 Grade descriptions for the CI test (after Sameshima and Black 1979).

Clay Index	Grade	Description
$0.0 \leq CI < 1.0$	1	Sound
$1.0 \leq CI < 2.0$	2	Good
$2.0 \leq CI < 3.0$	3	Unsound
$3.0 \leq CI < 4.0$	4	Unsound
$4.0 \leq CI < 5.0$	5	Unsound
$5.0 \leq CI < 6.0$	6	Unsound

Sameshima and Black (1980) measured the CI and SE values for a number of aggregates specimens from the Auckland area and found no relationship between the two parameters, as indicated by the plot presented in Figure 6.2.

The CI test is more definitive than the SE test in terms of characterising the nature of the fine fraction. However it should be carried out in conjunction with another test method which determines the quantity of fine material in the aggregate. A standard particle size distribution determined by sieving and sedimentation would satisfy this requirement adequately.

Figure 6.2 Plot of SE versus CI for three aggregates from Auckland area, New Zealand (after Sameshima and Black 1980).



6.1.3.3 Plasticity Index

An alternative to the clay index parameter is the Plasticity Index (PI) which is derived from the *plastic limit and liquid limit tests* developed by Atterberg in the 1930s. The PI is defined as the difference between the liquid limit and the plastic limit and hence corresponds to the range of moisture content over which the fine fraction behaves plastically. The Atterberg limit tests have the disadvantages of being open to operator subjectivity and poor repeatability, particularly at the low values of PI normally associated with aggregates (i.e. zero to about 10%).

Clearly, if the aggregate displays no plasticity, the plastic and liquid limits will coincide and the PI is zero. Most aggregate specifications which include the PI (e.g. New South Wales, Victoria, South Australia, Western Australia, UK, and some states in the US), set a compliance criterion of PI being less than or equal to about 6%.

6.2 Aggregate Selection

Van Barneveld et al. (1984) proposed the use of the CI test in conjunction with the SE test as part of the aggregate selection process. They recommended the following selection criteria for basecourse aggregates in highly loaded pavements (i.e. with design traffic > $5x10^5$ EDA):

either (i) < 10% passing 425 μm sieve,
or (ii) SE > 40,
or (iii) SE > 20 and PI < 3,
or (iv) SE > 20 and PI < 6 and CI < 3.

The first two criteria are not consistent with these recommendations since they do not include a determination of fine fraction plasticity.

Another set of aggregate selection criteria was proposed by Bartley and Cornwell (1993). In this method, the aggregate selection process is based on a number of material parameters which affect the performance of the aggregate, e.g:

- particle size distribution,
- plasticity,
- permeability,
- proportion of fines, and
- drainage conditions.

The Bartley and Cornwell (1993) aggregate selection procedure is summarised in the flowchart shown in Figure 6.3. The flowchart assigns an appropriate end-use to a given aggregate, ranging from its use as a sub-base (lowest quality) to its use as a basecourse in a highly loaded pavement (highest quality).

6.3 Aggregate Production

Aggregates may be produced from a number of sources, e.g. river gravels or deposits of hard sedimentary or igneous rock. The crust of the earth provides practically an endless availability of rock, but the extent of all rock sources is restricted by boundaries related to material quality, land use and ownership, environmental concerns and economic viability (Bartley and Cornwell 1993). The rock source should be close to the ground surface and within reasonable proximity of the construction site to minimise the costs of overburden removal and aggregate transport (Blyth and de Freitas 1974). The best quarry sites are often found in mountainous country where active erosion processes minimise the overburden depth but access can be difficult.

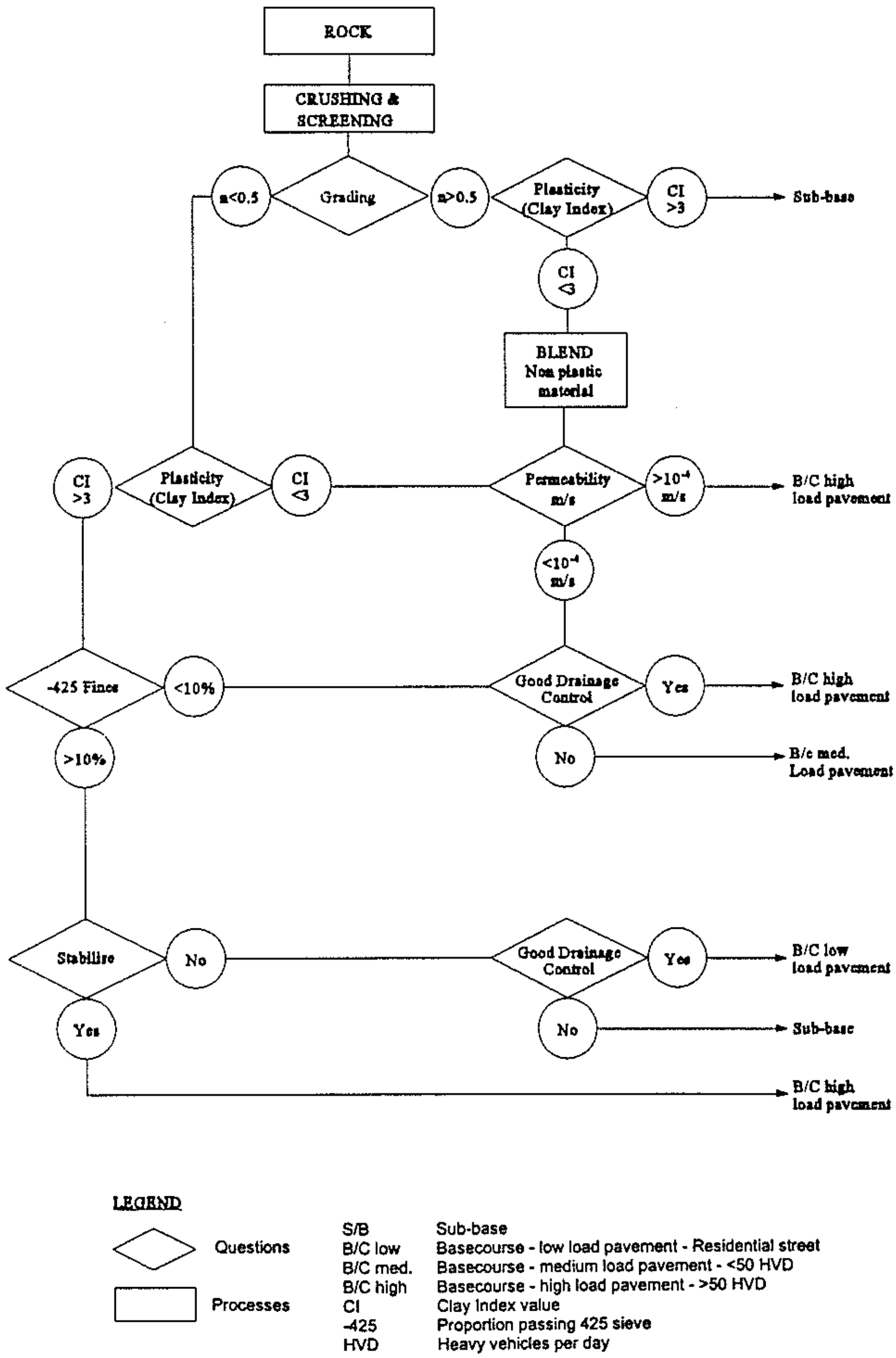


Figure 6.3 Aggregate selection chart (from Bartley and Cornwell 1993).

Quarry Sites Potential quarry sites must be thoroughly investigated for suitability by appropriately qualified and experienced personnel. Geologists should investigate the composition, quality and extent of the source, while financial planners evaluate the economic viability, and environmental consultants determine the environmental consequences of the extraction process and land restoration procedures.

The geological aspects of quarry site evaluation are initiated by the examination of geological maps followed by specific geological mapping of a potential site. This involves studying surface outcrops to formulate a preliminary model of the nature and extent of the underlying materials. The model is enhanced using a programme of drilling which generally entails up to 4.5 boreholes per one million tonnes of rock.

Boreholes may comprise diamond drilling with core recovery for physical testing or percussion drilling which is only suitable for the determination of the extent of the rock body and analysis of the fines or chips blown from the hole. One diamond-cored borehole is generally considered to be worth ten percussion boreholes. However the cost of one diamond-cored borehole is of the order of ten times that of one percussion borehole which also can be drilled faster.

Quality Assurance Once the geological and economic viability of a quarry site has been established, quality assurance procedures must be developed and documented. This should include strategies for determination of rock quality, face fines, grizzly fines, etc. In particular, the quality assurance strategies must include contingency procedures, i.e. appropriate courses of action which should be followed if the conditions encountered differ from those predicted by the initial investigations. They should also address operational factors such as the most suitable production equipment, the location of access roads, stockpiling methods and sites, maximum stockpile duration, bench locations, and compliance testing frequencies and locations.

A careful and consistent sampling technique must be used because the results of some tests may vary depending at which point they are sampled, e.g. samples may be taken from the quarry conveyor, the quarry stockpile, the delivery truck leaving the quarry, the truck arriving on-site, the stockpile on-site, the uncompacted aggregate layer, or the compacted aggregate layer. Wherever it is taken, the sample must be representative of the overall production, otherwise the test result is of little value.

Crusher System Most quarries producing aggregate for road construction have a two or three crusher system. Excavated rock is first passed through a grizzly to remove unwanted overburden or weathered material. It is then fed through two or three crushers with vibrating screens to produce the required product. Particle shape is predominantly determined in the final stages of crushing and is influenced by the type of crusher, the reduction ratio, the rate of feed, and the load on the crusher (Bartley and Cornwell 1993).

Stockpiling The aggregate should not be delivered to the road straight from the crusher, but should be carefully stockpiled at the quarry to minimise the effects of rock and production variations. Stockpiles also provide a buffer of available material to cope with variable demand. Conversely, aggregates should not be allowed to remain in a stockpile for such a long period of time that the quality of the product deteriorates from weathering processes.

Quarry stockpiles should have a prepared foundation to avoid contamination with underlying soils, and the material should be loaded by experienced loader operators to minimise segregation.

6.4 Alternative Materials

6.4.1 Regional Aggregates

The preservation of high quality aggregate sources can be fostered by the appropriate use of aggregates of lesser quality. This has been recognised by the development of the TNZ M/5 (1984-93) specifications. Potentially the TNZ M/4 (1985) specification covering the whole of New Zealand could be completely replaced.

Regional specifications would provide a more applicable set of aggregate specifications and would allay many of the current objections to the TNZ M/4 (1985) specification. This type of approach would parallel the system used in Australia where the aggregate specifications provide relevant compliance criteria for the different geological conditions encountered in each state.

Development of a nationwide set of TNZ M/5-type specifications would require a detailed investigation of material resources, aggregate properties, environmental conditions and pavement performance.

6.4.2 Waste Materials

In some countries the use of recycled building materials as a source of roading aggregate is actively promoted, particularly where the sources of natural stone have become depleted and the dumping of waste is restricted. Recycled materials such as crushed stone, mortar, bricks, asphalt, etc. as well as artificial materials such as steel mill slag are frequently used in these countries. Detrimental inclusions such as plastic, timber, glass, paper, etc. must be removed before the recycled materials can be used (Suss 1989).

The use of waste materials not only saves the cost of disposal but also eliminates part of the cost of traditional construction materials. It is also satisfying to the increasing environmental awareness of communities. An important consideration in the use of waste materials is the potential for chemical contamination of the existing ground by leachates. Hence, careful testing and evaluation of waste materials must be carried out before they are used as aggregates in a pavement.

The use of waste materials in New Zealand has not received significant attention to date (1995) because the supply of aggregates is relatively plentiful and suitable waste products are somewhat limited in supply. The use of waste products for pavement construction in New Zealand is however expected to escalate in the near future. The supplier of steel mill slag, operating at the site of BHP New Zealand Steel (South Auckland), processes and sells about 300,000 tonnes of this product per year.

7. REVIEW OF TECHNICAL LITERATURE FOR PAVEMENT CONSTRUCTION

7.1 Introduction

Construction of sealed unbound granular pavements involves predominantly the preparation of a suitable subgrade followed by the placement and compaction of aggregate sub-base and basecourse layers. The construction process should ensure that the aggregate layers perform in a manner which is consistent with that assumed by the designer. To achieve this, the construction materials and methods must be appropriate to produce pavement layers which are:

- of uniform density and shear strength;
- non-segregated;
- free from contamination by adjacent layer materials or other foreign matter; and
- as dry as practicable after construction.

The subgrade must have enough strength to support the loadings imposed by construction traffic and must be accurately shaped. A minimum practical subgrade CBR of 5% is recommended in the Transit New Zealand design method although Dunlop (1981) suggests a minimum value of 10%. If these values cannot be achieved, then stabilisation should be investigated. Alternatively, zones of poor subgrade material can be subexcavated and replaced with superior compacted fill. The strength of the subgrade must be such that the aggregate layers can be placed and compacted to a high standard. Drying the subgrade to improve the stiffness or using a geotextile to confine the overlying aggregate may be needed.

7.2 Segregation

Aggregate stockpiles are constructed so that a ready supply of material is available to the contractor on-site. However the process of stockpiling can be a major cause of the aggregate segregation and therefore should be carried out with great care. Stockpiles should be built up in layers with alternate trucks spreading from opposing directions in order to limit the effects of unloading. End tipping or dumping in separate piles should not be allowed. During loading out, the stockpile is excavated by cutting from the base to the top so that the layers are thoroughly mixed (Bartley and Cornwell 1993). Stockpiles should be kept moist to limit segregation during handling, to maintain the aggregate near optimum water content for compaction, and to ensure that dry, fine material does not blow away.

The placing of aggregate must also be carried out in such a way that will prevent segregation. The material should be placed in uniform layers to attain the finished pavement profile with minimal use of the grader. Laying granular layers by paver offers a number of advantages over the traditional method of aggregate spreading (Bartley and Cornwell 1993). The paving machine ensures that the aggregate layer is applied in a uniform thickness and produces an even surface after laying so that no further shaping is required. A vibrating screed on the paving machine provides a significant level of aggregate compaction during placement, therefore reducing the required rolling time and providing a superior construction surface. The method causes very little disturbance to the aggregate although some segregation may occur on either side of the paving machine. This segregated material can however be easily cut to waste.

7.3 Compaction

7.3.1 Compaction Mechanism

Compaction of loose unbound aggregates involves re-organisation of the aggregate particles into a closely packed state where small particles occupy the spaces between larger particles. The mechanism required to relocate aggregate particles from a loose state to a dense state is shear straining (Youd 1977). Shear strains are imposed on the material by both static and vibrating rollers, with the latter generally being the most effective and efficient. Youd states that the efficiency of the vibratory compaction operation is not influenced by the magnitude of the normal force, and in fact lower normal forces may result in a higher level of compaction. Hence the weight of the vibrating roller is a secondary factor in the compaction process providing the aggregate layer is relatively thin. This result was confirmed by Peploe (1991) who showed similar results from tests carried out in a simple shear apparatus. The reason for this behaviour is that high normal stresses tend to inhibit the movement of aggregate particles because of the resulting high inter-particle friction forces.

The compaction mechanism of vibratory rollers relies on the formation of a zone of material beneath the roller which is subjected to a state of free-fall caused by the vertical vibratory motion. It is important that the aggregate particles achieve a vertical acceleration equal to or greater than the acceleration of gravity. With the particles in the free-fall zone accelerating vertically at 1g they become effectively weightless and are free to relocate themselves into a more closely packed arrangement. Cohesive or cemented materials cannot achieve this state, and hence vibratory compaction is most effective for cohesionless materials.

Van der Merwe (1984) reports that low frequency, large amplitude vibrations are appropriate for deep lift compaction while high frequency, small amplitude vibrations are appropriate for shallow lift compaction. The amplitude of vibration influences the depth of the free-fall zone. Establishing a resonance condition within the aggregate layer is desirable, but determining the structure's natural frequency is difficult. The natural frequency will also be influenced by the changing density of the aggregate.

Static steel wheel rollers are an older form of compaction plant but are still widely used even though they are gradually being superseded by vibratory plant. Heavy static rollers may be preferred for very hard aggregates as they have the ability to crush the larger particles and therefore compensate for any deficiency of fines or for any variation in the particle size distribution. A heavy roller should not be used to compact a thin layer of aggregate placed over a weak subgrade because it may cause heaving and distortion of the subgrade.

Steel wheel rollers effectively apply a line load to the pavement and therefore have a predominantly two dimensional zone of influence. However rubber tyred rollers use a kneading action to compact the aggregate, i.e. shear strains are imparted to the aggregate by rotation of the principal stress axes. They have the advantage that they duplicate the compactive action of heavy traffic since they have a three dimensional zone of influence. The use of rubber tyred rollers with suitably high wheel loads for final compaction may cause a uniform increase in dry density across the width of the pavement and avoid the formation of ruts which frequently occurs when a new pavement is opened to traffic.

7.3.2 Compacted Dry Density

The *dry density* of a compacted soil is dependent on the water content of the material. The optimum water content can be observed as the water content corresponding to the maximum value of dry density on a compaction curve, i.e. a plot of compacted dry density versus material water content. Soils compacted dry of the optimum water content lack the water required for effective particle lubrication and hence relatively low values of dry density are achieved. Conversely, soils compacted wet of optimum water content contain excessive water which displaces a significant volume of soil particles and also results in relatively low values of dry density.

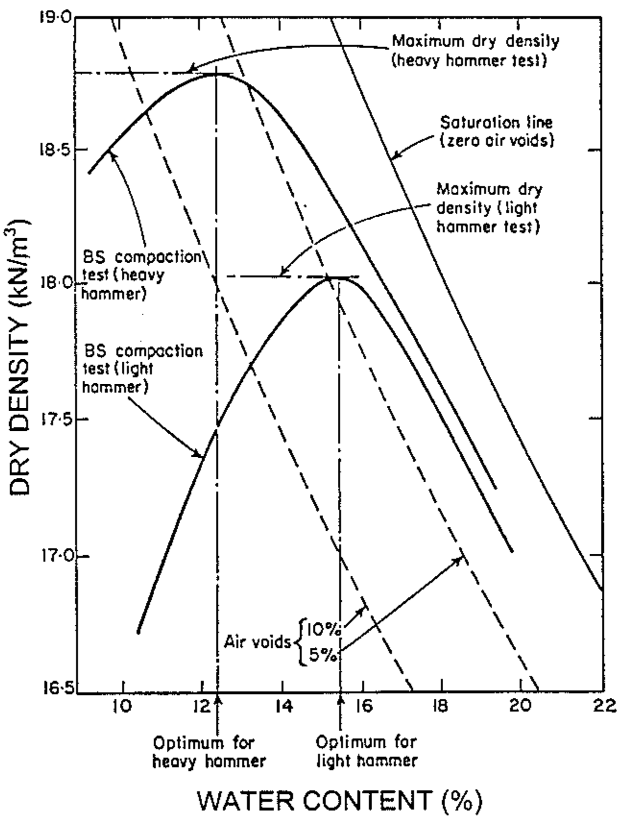
Cohesive soils have a pronounced optimum water content, whereas the relationship between water content and compacted dry density for non-cohesive aggregates may not be as strong. Aggregates generally show a flatter compaction curve with two optimum water contents, one when the material is dry and one when the material is almost saturated. The presence of water promotes the adhesion of fine particles to the coarse stone and limits segregation. Hence, to minimise segregation, aggregates should be compacted at or just below saturation. Initially, water should be added to the aggregate in the stockpile. Immediately after spreading, light compaction is desirable so that the water truck can move easily on the aggregate surface. Subsequently, rolling and watering should be carried out concurrently.

The dry density which can be achieved for a given soil (or aggregate) is also dependent on the amount of energy supplied by the compaction plant, often termed *compactive effort*. The maximum achievable dry density increases with increasing compactive effort. Note that compactive effort refers to the energy provided by the plant in a *single pass*. Successive roller passes will cause an increase in dry density, although the magnitude of the increase diminishes with each pass until the dry density effectively plateaus at a maximum value.

Once the maximum density is reached, further compaction cannot be achieved unless the compactive effort of the plant is increased. An increase in compactive effort results in a reduction of the optimum water content, and hence the selection of appropriate compaction plant is very important.

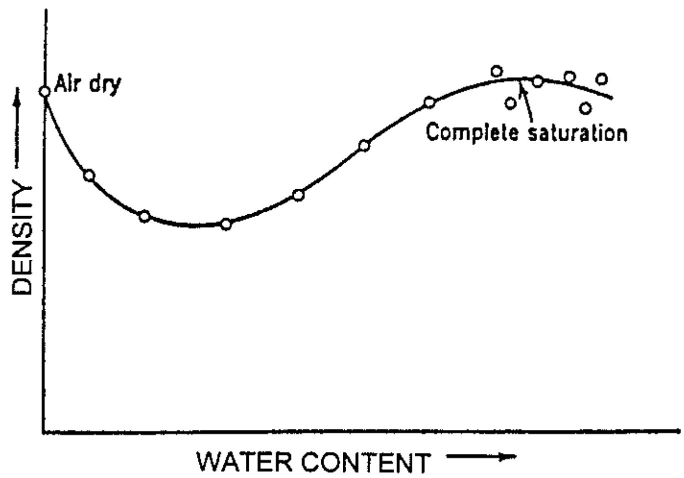
Figure 7.1(a) shows a typical set of compaction curves for a cohesive soil using various levels of compactive effort. Figure 7.1(b) shows a typical compaction curve for an aggregate.

Figure 7.1 Typical compaction curves for (a) cohesive soil (from Scott 1980) and (b) aggregate (from Lambe and Whitman 1979).



(a)

(b)



7.4 Acceptance Tests

7.4.1 Plateau Density

The current method of compaction acceptance is the maximum density approach (often termed *plateau density*) described in the TNZ B/2 (1987) specification. However rutting problems, experienced when new pavements were first opened to traffic, suggest that an alternative approach is necessary. One method used by several roading authorities, e.g. state authorities in Australia and the US, is to relate the field density to the maximum laboratory density. However, the relevance of the laboratory compaction test method is debatable because the specimen is contained in a steel mould of relatively small volume and the compaction technique and energy level can be quite different from that occurring on the site. Also, the particle size distribution of the laboratory sample may not be typical of the aggregate in the road.

7.4.2 Minimum In Situ Density

A preferred approach is to specify the minimum in situ density as a proportion of the solid density of the aggregate particles. This proportion is often taken to be about 80% to 90% of the solid density (i.e. 20% to 10% total voids). This relates the density entirely to the rock type used and is independent of any problems which may occur with the laboratory test, including that of the particle size distribution of the sample used. The approach is also independent of the compaction plant being used and the nature of the construction material.

Large, modern vibrating rollers are fitted with instruments which provide a measure of the density by monitoring the dynamic response of the material under the roller. While these instruments cannot be relied upon in absolute terms, they do establish the point at which further passes will have negligible effect on the density.

7.4.3 Plate-bearing Test

In Germany, the practice is to use the *plate-bearing test* to ensure that the design assumptions are satisfied at predetermined steps in the construction operation. A trial section is used initially so that the compactive effort and layer thicknesses can then be fine-tuned as the construction progresses. The advantage of this method is that the designer can be confident that the assumptions made in the design process are realised in the pavement. However, the plate-bearing test has some major disadvantages. It is a relatively time-consuming test, it requires a large reactive load, and a large number of tests are required to produce a statistically acceptable result.

7.4.4 Clegg Hammer Test

Another option is to use a *Clegg hammer test* (Garrick and Scholer 1985) even though the energy imparted by the device is relatively low and the zone of aggregate sampled may be small. It has the advantage that a large number of readings can be obtained quickly. The uniformity of compaction can then be assessed with little effort and negligible cost.

7.4.5 Deflection Tests

Deflection tests using the Benkelman beam or falling weight deflectometer (FWD) may be used. These methods utilise realistic pavement loading conditions and provide a direct measurement of the stiffness of the pavement structure. In theory, measurement of the shape of the surface deflection basin allows the modulus of each pavement layer to be back-calculated.

This procedure has been investigated by Toan (1975) who concluded that, for a simple two-layer pavement with an unbound granular base:

- the surface deflection measured 600 mm from the load is predominantly influenced by the stiffness of the subgrade, and
- the deflection at the loading point is influenced by the stiffness of both the subgrade and the unbound aggregate.

If the thickness of the aggregate layer is known, it is then a simple task to match the measured central deflection and the deflection at 600mm with the theoretical deflections calculated using a multi-layer elastic computer program. A number of simplifying assumptions have to be made in this procedure and hence the results should be viewed with care. Deflection tests also provide excellent data for determining the variability of a pavement's structural integrity both along its length and across its width.

7.4.6 Density on the Run

Seaman (1988) describes the use of a *density on the run* (DOR) technique for monitoring the compaction of aggregate layers. The DOR apparatus consists of a specially adapted nuclear density meter which is mounted on the compaction plant and provides the operator with density data as the compaction proceeds.

7.4.7 Statistical Approach

A statistical approach to compliance testing is appropriate for pavement construction because the materials and the testing methods can be somewhat variable. The Californian practice is to use a moving average over the duration of the construction operation (Barksdale 1989). It also uses a projected average where each prospective test result is assigned a value equal to the previous result. If the projected average is calculated to be outside the specified limit then appropriate action is taken in advance. This action may take the form of an amended material specification or construction procedure.

Many other US states use statistical approaches to material compliance testing, most of which impose cost penalties on the contractor (or supplier) if the material properties fall outside a set range. The severity of the penalty is generally dependent on the variability of the test results and the significance of the parameter under investigation.

7.4.8 Maximum Acceptable Surface Deflection

Many local authorities in New Zealand stipulate a *maximum acceptable surface deflection* for each of the various classes of pavement in their jurisdiction. The pavement is not accepted as complete until the contractor shows that satisfactory deflection values have been achieved. The Auckland City Council maximum Benkelman beam deflection values for different levels of design traffic are provided in Table 7.1. The data presented in Table 7.1 are appropriate for the pavement materials typically found in the Auckland region and therefore they may not be reliably used in other areas.

Table 7.1 Pavement deflection standards applied by Auckland City Council, New Zealand, using aggregates available in the Auckland region.

EDA Per Lane	Upper 95% Deflection (mm)
≤ 10,000	1.78
10,001 - 100,000	1.52
100,001 - 250,000	1.27
250,001 - 500,000	1.02
500,001 - 1,000,000	0.76
1,000,001 - 2,500,000	0.51

7.5 Specifications

Auff (1994) advocates that roading authorities are responsible for verifying the quality of pavement construction as they are the purchaser of a product on behalf of the users, i.e. the general community. The roading authority assures the quality of the product when it designs and specifies the pavement requirements while the contractor must assure that the quality of the construction processes adopted are appropriate.

7.5.1 End-Result Specifications

End-result specifications put the onus for the quality of the pavement onto the contractor. This is appropriate since the contractor, in carrying out the work, can control quality. Elliot (1991) states that end-result type specifications have the advantage that the contractor and the roading authority both work towards a common goal. Contractors have the opportunity to gain from end-result contracts because they can manage their staff and equipment in the most efficient way without being directed by their client. The client can also benefit by the economies which are able to be achieved by a resourceful contractor. In this way, competent and well managed contractors prosper and incompetent contractors suffer.

This type of specification thus has advantages compared to the traditional method type specification in which the relationship between the authority and the contractor often becomes adversarial. The roading authority inspectors are often perceived to be trying to catch out the contractor who may in turn be trying to avoid carrying out instructions.

7.5.2 Quality Assurance

Quality assurance is an important component of end-result contracts. A programme of material and construction tests is specified which the contractor must implement. The contractor is required to monitor the results and ensure that they satisfy prescribed criteria. The results are also supplied to the client who can then be assured that it is receiving the product that it specified. If unsatisfactory results are recorded, appropriate action can be taken to change construction procedures to ensure that the completed pavement meets the specified compliance criteria.

There must be sufficient incentive for all people involved to ensure that quality is achieved (Elliot 1991). The implications of constructing a non-complying pavement may be sufficient motivation for most contractors to take quality assurance very seriously. However quality assurance may be promoted by other positive means. For example a monetary bonus for high standards of quality may be very effective. It may also have the effect of instilling worker pride which is far greater than the dollar value of the bonus.

The quality assurance clauses of a highway construction specification issued by the South Australia Department of Road Transport have been reviewed. The section on quality system requirements includes the following main clauses:

- Tenderers must submit a copy of their current Quality Manual.

- Selected tenderers must submit their Quality Plan which includes:
  - statement of quality management policy;
  - management structure; and
  - responsibilities of key personnel.

- Register of procedures for:
  - earthworks;
  - placing and compaction of aggregate;
  - application of bituminous materials;
  - placing and finishing of concrete;
  - installation of pipes and culverts;

- methodology for quality assurance in terms of:
  - purchasing;
  - materials, i.e. mix design, delivery and testing;
  - traffic management; and,
  - pavement marking.

- Tenderers must appoint a Quality Management Representative.

- All construction activities must be assigned a code which can be cross-referenced to the contractor's programme.

- All materials must be identifiable and traceable with records of batch quantities, time, compliance test details, and location of placement.

- Material compliance tests must be carried out by an appropriately accredited laboratory and test results reported in a standard format.

- The client may undertake an audit of the contractor's systems, procedures or products. These audits require advance warning periods of seven days, two days and no notice respectively.

8. REVIEW OF TECHNICAL LITERATURE FOR ROAD TYPE

8.1 Pavement Serviceability

The Transit pavement design method recognises four categories of pavement, designated Group 1 to Group 4, where Group 1 corresponds to the highest serviceability requirement and Group 4 corresponds to the lowest. Each road group has a target level of serviceability both at the time of construction and at the end of the pavement's design life.

Group 1 and 2 roads comprise major highways, motorways and arterials, while Group 3 and 4 roads comprise all other minor routes (Dunlop et al. 1983). The Transit New Zealand *Pavement Design Manual* (TNZ 1989a) contains charts for two pavement standards, *premium* for pavement groups 1 and 2, and *lower grade* for pavement groups 3 and 4. Interpolation between the two charts is allowed for intermediate situations.

The serviceability of a pavement may be determined using either a quantitative or a qualitative approach. The quantitative approach is used in New Zealand and involves the use of the AUSTROADS roughometer. This device, which is mounted on a vehicle, counts the number of standard bumps per kilometer. A similar device called a *bump integrator,* developed by TRRL, integrates the deflection of a vehicle's suspension system as it travels over a measured distance to obtain an indication of pavement roughness. Qualitative methods such as the *present serviceability index* (PSI) and the *riding comfort index* (RCI) are frequently used in the US and Canada respectively. These methods involve a panel of assessors selected from the general public who traverse the pavement in a motor car and rate the quality of ride. The rating is on a scale from zero to five (PSI) or zero to ten (RCI), where zero is very poor and five (or ten) is very good (Carey and Irick 1960, Nick and Janoff 1983, Moore et al. 1987, RTAC 1977).

8.2 Estimation of Traffic

The provision of these two standards of design (premium and lower grade) is generally considered adequate since under the current (1995) analytical approach the terminal condition of a pavement is related to the expected number of standard axle loads over its life. The design traffic value is at best an estimation. Hence the designer may be justified in adjusting the design traffic to obtain a slightly more or less conservative structure depending upon the given situation. The AUSTROADS Design Guide (1992) formalises this design traffic adjustment process, allowing the designer to achieve a preferred terminal serviceability level.

The design traffic loading is an important input parameter to the pavement design method used in New Zealand. In the design of major highways and motorways, the expected traffic loading and rate of growth is relatively predictable given the significant proportion of heavy commercial vehicles which use these roads. The prediction of traffic loading for minor urban streets is more difficult because of the lack of data. For this reason, TRRL Road Note 29 (TRRL 1970) gives a table of typical heavy commercial traffic volumes for urban type roads, as outlined in Table 8.1.

Table 8.1 Typical volumes of heavy commercial vehicles (HCV) on urban type roads (after TRRL 1970).

Type of Road	**Estimated HCV Per Direction Per Day**
Culs-de-sac, minor residential roads	10
Through roads + regular bus routes	75
Major through roads + bus route	175
Main through roads & main shopping centres with goods carrying vehicles	350

To put these into the New Zealand context, Table 8.2 outlines a similar set of pavement classifications in terms of loading for urban pavements, as specified by Manukau City Council.

Table 8.2 Traffic loading classifications specified by Manukau City Council, New Zealand.

Type of Road	**Estimated HCV Per Day**
Culs-de-sac, roads on small estates	0 - 15
Minor through roads on housing estates, no public service vehicles	15 - 45
Through roads on housing estates, up to 50 public service vehicles per day	45 - 150
Main roads on housing estates, bus routes, general traffic circulation	150 - 450
Streets in main shopping centres of large developments, large numbers of public service vehicles, main traffic through estate	450 - 1500

9. REVIEW OF TECHNICAL LITERATURE FOR ENVIRONMENTAL CONDITIONS

The performance of a pavement is strongly influenced by the environmental conditions prevalent at the site. The most important environmental factor is the presence of water in the pavement.

9.1 Water

Water may result from stormwater inflow from the pavement surface or shoulder, a high ground-water table, lateral seepage, springs or capillary rise. Every effort must be made to exclude water from the upper one metre of the pavement structure as it may have several deleterious effects such as:

- Promote the mobilisation of deleterious secondary minerals from the unbound aggregate if these materials are present. Such minerals are prone to swelling and hence to separate and lubricate adjacent aggregate particles.
- Lubricate the faces of the aggregate particles and reduce inter-particle friction.
- Stimulate the weathering process of aggregates of marginal quality.
- Soften fine grained subgrade soils and induce pumping of material into the aggregate layers.
- Provide a medium by which fine grained materials can be transported from areas of relatively high hydrostatic pressure to areas of low pressure.
- May generate positive pore water pressures if the pavement is saturated and the permeability of the materials is very low.

9.1.1 Pore Water Pressures

Reports are conflicting as to the susceptibility of aggregate layers to pore water pressures generated by axle loads. Bartley and Woodward (1981) reported that the permeability of basecourse materials measured in their investigation was sufficiently high that pore pressures would dissipate very quickly and not accumulate with successive traffic load applications.

Laboratory investigations carried out by Toan (1975) showed that pore pressures could develop in saturated samples of aggregate. These are irrelevant if pavements have adequate drainage outlets and the surface seal is well maintained. Surface cracks should be sealed promptly, both on the carriageway surface and the shoulder. Side, interceptor and subsoil drains should be installed to keep the pavement structure free of water. To ensure that the size of these drains is adequate, rainfall duration and frequency data should be considered in the design. The drains should be connected to an overall collection system at regular intervals to minimise drainage path lengths. The topography of the adjacent land is a major factor which influences the type of drains to be installed and their most effective locations.

Situations may develop which result in the pavement being inundated for a significant period of time. But even under these conditions positive pore pressures are unlikely to develop if the saturation ratio is less than about 85% (Lambe and Whitman 1979).

Negative pore water pressures can be induced in the pavement by the surface tension of the capillary water if the water table is well below the pavement structure and the material pore sizes are conducive to capillary rise. This increases the effective stress and hence promotes pavement stability. Capillary rise is only possible if the pores are continuous.

9.1.2 Drainage

The use of the adjacent land may also be important, e.g. vegetation close by may influence the moisture regime. Adequate crossfall and accurate shape control on the road surface will ensure quick stormwater run-off and minimise ponding.

A situation which must be avoided is the so-called *bathtub effect*, reported by Cedergren (1974). In this situation the pavement has been constructed in a trough of low permeability soil and any water which enters the pavement has no opportunity to escape. The aggregate layers may therefore be in a continuously saturated condition. A saturated aggregate layer will have a reduced load spreading ability so that stresses on the underlying materials will be increased.

The provision of adequate drainage may permit aggregates of lesser quality to function well, since many of the deleterious properties are only realised in the presence of excess water. The Transit New Zealand pavement design method (1989a) stipulates that a relatively high permeability layer is constructed in the sub-base immediately below the basecourse layer.

A similar stipulation is made by many of the roading authorities in the US. Baldwin (1987) reported that 24 states incorporated a free-draining layer at some elevation in the pavement structure. The position of the layer ranged from immediately above the subgrade to immediately below the pavement surface. The latter position would appear to be difficult to justify given that an open aggregate grading compromises layer stability, particularly at the level of stress associated with a high elevation in the pavement.

Although exclusion of water from the pavement is required during service, at construction time the aggregate should be saturated to facilitate compaction and minimise segregation. It is important that the compacted aggregate is allowed to dry out as much as possible before the pavement surface is sealed (Bartley and Cornwell 1993).

9.2 Temperature

Temperature may affect a sealed unbound granular pavement in one of two ways.

- At high temperatures the stability of the surface seal will decrease as the viscosity of the binder decreases. A suitable binder and application rate must be selected for the prevailing climate to ensure that the sealing chip is securely held without flushing or stripping.
- At very cold temperatures frost may form in the aggregate layers. Water which freezes in the aggregate's pore spaces will increase in volume and reduce the strength of the material. The initiation of freezing draws further water from the subgrade and the frozen front advances. Once the frost thaws the excess water in the upper levels of the pavement further decreases the stability of the aggregate and may mobilise deleterious secondary minerals. The frost-thaw cycle also promotes weathering of the aggregate.

Conflicting philosophies concern the treatment of frost-susceptible pavements. One approach is to ensure that the permeability of the frost-susceptible region is high enough so that water can drain away quickly and therefore no water is present to freeze. Sayward (1979) reports that soils which are susceptible to frost heave generally contain greater than 3% of particles finer than 20 μm. These materials should be excluded from pavement construction in areas where ground freezing is common. Alternatively, a coarse layer of aggregate should be provided in the sub-base. This may be effective in breaking the hydraulic conductivity between the subgrade and the basecourse.

Another approach is to stabilise the aggregate with a binding agent such as bitumen, lime or cement so that it possesses sufficient tensile strength to resist the expansive tendencies of the frozen pore water. Further discussion of this approach is beyond the scope of the present research.

In the UK a frost-susceptibility test has been devised, but the reliability of the method has been criticised in the literature. Curtis (1989) reports that the reproducibility of the test is extremely poor and it is not deserving of a British Standard endorsement.

9.3 Axle Loading

The relationship between the axle load and the damaging effect it imparts on a pavement has been discussed in Section 5.2.4. However, the nature of the contact between the vehicle tyre and the pavement surface is relevant since it is one of the environmental factors which impacts on the pavement.

9.3.1 Wheel load

Most pavement design methods (including that of Transit New Zealand) assume that the wheel load is transferred as a uniform vertical stress from the vehicle tyre to the pavement surface via a circular contact area. Although this assumption may be reasonable for the purposes of pavement design, in reality the tyre contact area is rarely circular, nor is the stress uniform. Both factors are influenced by the construction of the tyre, the stiffness of the tyre rubber, the tyre inflation pressure, the axle load and the tyre tread pattern. Lister and Jones (1967) found that underloaded tyres have approximately circular contact areas while fully loaded tyres produce an approximately elliptical contact area with straight sides and a length to breadth ratio of about 1.4. Overloaded tyres have a highly elongated contact area with a length to breadth ratio of up to 2.

The results of other research reported by Lister and Jones (1967) show that under full tyre loads the vertical contact stress is approximately uniform while both underloaded and overloaded tyres cause non-uniform vertical stresses. Underloaded tyres attract maximum stresses at the centre of the contact area while overloaded tyres attract maximum stresses at the sides of the contact area, beneath the tyre walls. Lister and Jones (1967) concluded that under typical loading conditions the assumption of a circular contact area and a uniform vertical stress distribution is reasonable. However, when tyres are severely overloaded the stress conditions are markedly non-uniform.

A study by Sebaaly and Tabatabaee (1989) showed that the contact stress between a truck tyre and the pavement surface is non-uniform, although the uniformity increases with increasing axle load. They found contact stresses of up to 175% of the tyre inflation pressure. The assumed EDA contact pressure of 580 kPa used in the Transit New Zealand design charts (1989a) is relatively low, given that truck tyre inflation pressures are typically in the range of 550 to 800 kPa. In New South Wales and Victoria the equivalent standard axle tyre contact pressure has been increased to 700 kPa to recognise the tendency for increased tyre inflation pressures.

9.3.2 Tyre Type and Inflation Pressure

The construction of the tyre has also been found to influence the loading. Wide-based single tyres produce the greatest compressive stresses while radial-ply dual tyres provide the least. Bias-ply dual tyres are intermediate between the wide-based single tyres and the radial-ply tyres.

An investigation by Huhtala et al. (1989) rated tyres in terms of their aggression towards damaging the pavement. Wide-based single tyres were found to be more aggressive than dual tyres even when the inflation pressures in the two dual tyres were significantly different. They also found that tyre aggression increases with tyre pressure and that there is no optimum tyre pressure. The maximum contact pressure was at the centre of the contact area for truck tyres, but beneath the tyre walls for passenger car tyres.

The aggression of wide-based single tyres is recognised in the New Zealand heavy vehicle axle loading regulations. The maximum load allowed on a wide-based single-tyred axle is set at 7.2 t compared with 8.2 t for dual-tyred axles and 6.0 t for standard single-tyred axles.

9.3.3 Dynamic Loading

The current analytical design methods do not take dynamic loads or surface shear stresses into account. Dynamic loads are caused when axles bounce on uneven pavement surfaces causing some parts of the pavement to be more heavily loaded (and other parts less heavily loaded) than assumed in design. Lister and Jones (1967) reported that dynamic axle loads are influenced by the quality of the riding surface, the speed of the vehicle and the damping characteristics of the suspension system. The latter is itself influenced by the load carried by the axles. The most favourable suspension type is the air bag if it is properly maintained, followed by the four-leaf unit, and the walking beam suspension is the least favourable.

Shear stresses occur at the pavement surface when vehicles perform manoeuvres such as turning, accelerating or braking. However few references to the effects of surface shearing have been found in the recent technical literature. Surface shear stresses may be expected to cause damage to the surface seal and to disrupt the upper region of the basecourse layer. Barber (1962) found that the shear stresses caused by tyre traction under acceleration and cornering are small, although the stresses caused by heavy braking may be quite significant.

Bonse and Kuhn (1959) reported that inward facing radial shear stresses occur under pneumatic tyres caused by tensile stresses in the rubber caused in turn by the inflation pressure. They found shear stresses of more than 275 kPa under truck tyres at only moderate levels of acceleration.

10. STRATEGIES FOR THE FUTURE

10.1 Introduction

The literature search has highlighted a number of aspects of the current Transit New Zealand approach to the design and construction of sealed unbound granular pavements which are in need of modification or revision.

The Transit method of design is based on the concept that a pavement deteriorates with time, i.e. a fatigue mechanism implying *failure*. In fact the pavement does not fail in the accepted normal engineering sense of failure. Rather the roughness of the surface of the pavement develops to the stage when it reduces the serviceability below some pre-determined level. The design approach focuses on the elastic response of the structural layers of the pavement under a standard wheel load and uses an empirical association between the vertical strain at the top of the subgrade and the number of load applications required to produce the terminal level of serviceability.

Few of the components that make up an unbound granular pavement deteriorate with time. This issue is not addressed by current pavement design methods.

The review has shown that Transit New Zealand is not the only roading authority to subscribe to the *mechanistic* approach to design. This is not surprising since the technology associated with unbound granular pavements has advanced only slowly over the last 40 years. Many of the advances have been based on the overseas work on deep asphaltic pavements.

A new technology needs to be developed that is based on sound geotechnical principles. Such an approach has not been possible in the past because it involves complex three dimensional analysis. However the development of the shakedown theory, and the availability of powerful computers to carry out the wide array of calculations required, provide some confidence that the theory can be developed into a practical design tool in the near future.

In the meantime a number of actions could be put in place to improve current practice. They are outlined in Section 10.2 of this report.

10.2 Pavement Design Strategies

10.2.1 Pavement Design Using Shakedown Theory

Shakedown theory is considered to provide the best possible characterisation of pavement behaviour. A two-dimensional version of the theory has been proposed in the literature but this is about to be superseded by a three-dimensional model currently being devised by Professor Ian Collins at the University of Auckland (pers.comm. 1995).

The shakedown approach to design is based on the fact that, under cyclic loading conditions, pavement materials possess a threshold stress, below which their long-term behaviour is purely elastic.

By designing pavements with component layers exposed to stresses below these threshold values the structural performance is independent of the number of load applications, i.e. the pavement effectively has an infinite life.

Clearly, surface maintenance would be required but expensive rehabilitation in the form of structural overlays would not be needed.

The adoption of a shakedown approach to pavement design is expected to provide significant economic benefits by eliminating the requirements for structural overlays and also conserving aggregate resources.

10.2.2 Layering of Unbound Courses

Although the current Transit method for designing unbound aggregate pavements TNZ 1989a) uses multi-layer elastic theory to determine the vertical compressive strain occurring at the top of the subgrade, the analysis treats the unbound granular layers as a single layer. The elastic modulus of the layer is determined using a modular ratio relationship. The design method should analyse the basecourse and sub-base layers as two separate layers to reflect the relative quality of the two kinds of materials and the layered construction of the pavement. This analysis will generally result in lower strains being calculated at the top of the subgrade and hence will reduce the required aggregate thickness. This concept could be extended to include multiple sub-layering of the aggregate courses.

10.3 Material Characterisation Strategies

10.3.1 Alternative Subgrade Support Tests

The elastic properties of the subgrade must be accurately established because the performance of the overlying pavement structure is significantly influenced by the modulus of the subgrade. Although the CBR test is widely used throughout the world it is not well suited for evaluating the elastic properties of pavement materials. The result is significantly influenced by the plastic behaviour of the test specimen which makes the correlation between CBR and elastic modulus poor. An alternative method of evaluation, such as dynamic triaxial testing or the use of a modified Clegg hammer type device, is considered to be more appropriate.

10.3.2 Anisotropic Behaviour of Aggregates

The AUSTROADS pavement design model (1992) includes an allowance for anisotropic aggregate behaviour as a result of the back-analysis of pavement deflected shapes. The model treats the aggregate as being twice as stiff in the vertical direction as it is in the two horizontal directions. This approach has merit considering the layered construction of a pavement and the greater degree of vertical confinement afforded to the aggregate compared to horizontal confinement.

10.3.3 Sand Equivalent (SE) Specification

The TNZ M/4 (1985) specification requires that an SE value of 40 must be achieved for premium basecourse aggregate. Many aggregate producers find this requirement difficult to achieve. The literature suggests that an SE of 40 is too conservative and a value of approximately 25 would be reasonable.

It is recommended that the TNZ M/4 (1985) specification should be revised with the SE test requirements replaced by, or at least combined with a test reflecting the plasticity of fines, e.g. the clay index test.

10.3.4 Regional Aggregate Specifications

Transit New Zealand has initiated this approach by producing TNZ M/5 specifications for aggregates available in different regions of New Zealand. The number of TNZ M/5-type specifications is expected to grow as experience with regional aggregates develops. This approach would be beneficial to follow so that the TNZ M/4 (1985) specification is replaced with a TNZ M/5 specification for each region of New Zealand.

This approach would first require identification of suitable regional boundaries. Detailed information regarding available materials, their properties and performance would then need to be collated and assembled into the new specifications. This approach is followed in Australia where specifications are provided for the different geological conditions encountered in each state.

10.3.5 Waste Products for Sub-Base Construction

To conserve resources of high quality aggregate, the testing of waste and marginal products such as steel mill slag, crushed concrete and regional aggregates should be encouraged. This may take the form of monitored test strips or laboratory investigations using equipment such as the dynamic triaxial or simple shear apparatus.

10.4 Design Traffic Characterisation Strategies

10.4.1 Load Equivalence Exponent

The current Transit pavement design approach (TNZ 1989a) conforms with that of most international roading authorities in the use of the fourth power law to relate the damaging effect of axle groups to the axle loading. The literature suggests that a load equivalence exponent less than four may be appropriate for unbound aggregate pavements, as this reflects a relatively low sensitivity to traffic loading for this type of pavement. Under the present design procedure, the adoption of a lower load equivalence exponent would result in reduced aggregate layer thicknesses, and the associated economies, for very heavily trafficked pavements only. Further research based on pavement structures used in New Zealand would be required before this strategy could be implemented.

10.4.2 EDA Tyre Contact Pressure

The Transit design model currently incorporates a tyre contact pressure of 580 kPa. The tyre pressure should be increased to reflect inflation pressures of modern truck tyres which typically fall into the range 550 to 800 kPa. This approach has already been taken by some Australian states. The effect of tyre type may also need to be considered.

10.4.3 Use of Air Bag Suspension Units

This review has shown that air bag units are the least damaging type of heavy vehicle suspension. The use of these units should be encouraged, possibly by offering a reduced road user charge.

10.4.4 Dynamic Loading in Pavement Design

In addition to raising the tyre contact pressure in the EDA model because of increased tyre inflation pressures, an allowance for dynamic loading caused by an uneven pavement surface may be required. Research shows that dynamic loadings may be up to 150% of static loadings under some circumstances. Research is under way in New Zealand on this topic and hence any significant design methodology changes should be reserved until these projects have been completed.

10.5 Construction Strategies

10.5.1 Plateau Density

The dependence on compaction plant configuration inherent in the plateau density approach of the TNZ B/2 (1987) specification is a significant weakness. This approach should be replaced with a compaction compliance criterion based on a maximum void percentage, minimum percentage of solid density, or a minimum percentage of laboratory determined dry density. Of these three alternatives, the maximum void percentage criterion is preferred. Rejection of the plateau density approach should minimise the potential for the formation of wheel track ruts in newly constructed pavements.

10.5.2 Paver Laid Aggregates

The use of paving machines to lay aggregate courses should be encouraged because they have superior accuracy for the material placement, compared with the traditional methods of spreading and grading. The increased cost of paver laying is offset by the reduced compaction time and the quality of the ride on the completed pavement with the associated minimising of dynamic loads.

10.5.3 Specification for Aggregate Stockpiling

The construction of aggregate stockpiles should be formalised in a specification to educate contractors about the correct method of forming and loading out from stockpiles. To minimise material segregation, stockpiles should be formed by spreading in layers in alternating directions. The material should then be loaded out by excavating through the layers from the bottom to the top of the stockpile. Indiscriminate end-dumping should not be allowed.

10.5.4 End-Result Construction Specifications

End-result type contracts produce significant benefits for both resourceful contractors and roading authorities. The responsibility for construction and material quality is placed on the contractor, which is appropriate since the contractor is the one who is directly involved.

The client can be assured that an acceptable level of quality is being met by reviewing appropriate compliance tests at regular construction milestones. The revised tests would also allow any assumptions made in the design to be verified or rejected. If unexpected conditions are encountered during construction, the design can be altered before further construction commitments are made.

10.5.5 Statistical Compliance Testing Specifications

A compliance testing specification based on a statistical approach has the advantage that the level of quality provided by the contractor is quantified. It can also be used as a basis to reward quality of construction based on the variability of the test results and can be used as a predictive tool to anticipate situations where quality may become compromised.

The Californian approach (Caltrans 1990) to statistical compliance testing appears to have considerable merit. In this method, allowable compliance criteria are set at the start of the project and a running average of test results is kept. At the completion of each test, the running average is calculated to ensure that the value is acceptable. A prediction of the successive running average is also made by assuming that the next test will provide the same result. If the anticipated running average falls outside the acceptable range, steps for rectification are formulated before the level of quality becomes unacceptable.

11. BENEFIT/COST RANKING OF RESEARCH FINDINGS

11.1 Introduction

In this chapter the relevant research findings identified in Chapter 10 are ranked in priority of benefit/cost. Benefit/cost analyses are presented for all of the items which have quantifiable benefits and costs. All costs are in NZ$ as at 1995.

11.2 Benefit/Cost Analyses of Pavement Design Strategies

11.2.1 Base Pavement Design on Shakedown Theory

Two components to the benefit/cost analysis are inherent in the development of a pavement design method based on shakedown theory. One concerns the design of new pavements and the other concerns the rehabilitation of existing pavements.

- *Design of New Pavements* The shakedown approach to design results in pavements which effectively possess an infinite life, provided that the applied stresses do not exceed the design stresses. Although a premium cost may be incurred for the construction of these pavements, the benefits are infinite since there is no requirement for future structural rehabilitation. The only post-construction costs are those associated with the maintenance of a waterproof and skid-resistant surface.

The development of the theory and establishment of the methodology on a nation-wide basis would be a one-off cost. The benefit/cost ratio for the development of a shakedown approach to pavement design is therefore infinite.

However, if a practical pavement life of 100 years is assumed, then the benefit of adopting the shakedown philosophy is that the cost of rehabilitation will be avoided. Bartley Consultants (1995) shows that this cost amounts to approximately $M6 when discounted back to the present day (1995).

As a shaken-down pavement remains relatively smooth for all time there is also a considerable saving in vehicle operating costs (VOC). Bartley Consultants (1995) shows that VOCs amount to $M78.

- *Rehabilitation of Existing Pavements* Adopting a shakedown approach to the rehabilitation of existing pavements also provides significant benefits. The key factor in this analysis is to recognise that, in a rutted pavement, the most stable materials are generally those beneath the wheeltracks (provided shear failure has not occurred). These are materials that have densified under repeated axle loadings and consequently developed the residual horizontal stresses in accordance with the shakedown theory.

The method of rehabilitation of rutted pavements should therefore not disturb the residual horizontal stresses wherever possible. This can be achieved by filling the ruts with a thin layer of asphaltic concrete to correct the surface shape. The conventional rehabilitation method of constructing a thick aggregate overlay is both expensive and inappropriate. The thick overlay is itself vulnerable to rutting when the material beneath the wheeltracks densifies, hence the overlaying cycle is perpetuated.

Bartley Consultants (1995) showed that the benefit associated with pavement rehabilitation using a thin layer of asphaltic concrete in preference to a granular overlay was about $M55. This figure is considered to be conservative as it does not include the savings associated with the reduced interference to traffic when a thin asphaltic concrete layer is used.

The total benefit is the sum of the values presented above, i.e.

$M6 + $M78 + $M55 = $M139

The cost portion of the analysis comes from the one-off expense of developing the theory and implementing the methodology on a nationwide basis. This is estimated to be approximately $M1.4, comprised as follows:

• Development of the shakedown theory and laboratory testing	300,000
• CAPTIF trials (three pavements)	750,000
• Implementation	200,000
• Contingency	150,000
Total	$1,400,000

The benefit/cost ratio is therefore:

139,000,000 / 1,400,000 = 99

Note that the real benefit/cost ratio is greater than 99 since the benefits have been considered only over a finite period of time.

11.2.2 Allow for Layering of Unbound Courses

In the current pavement design method, the base and sub-base layers are assumed to be a single combined layer. However, pavements are not constructed in one layer, nor are the materials in the successive layers exactly the same. To treat them as such in the design methodology is therefore inappropriate.

A multi-layer elastic computer program has been used to investigate the effect of considering the base and sub-base layers as separate layers. For typical pavements the results show that an increase of approximately 10% in the design EDA can be achieved. While this may appear to be significant, it makes very little impact on the pavement layer thicknesses required.

A significant benefit is achieved however when relatively thick unbound pavements, i.e. pavements at least 350 mm thick are analysed. In these pavements at least three construction layers are placed, e.g. two 150 mm layers of sub-base and one 150 mm layer of basecourse.

A three layer analysis, as a preliminary to further work, indicates that for typical deep pavements the aggregate layer thickness can be decreased by approximately 7.4% for the same number of load repetitions. Assuming that 25% of the length of newly constructed pavements fall into the three layer category (i.e. at least 350 mm thick), an annual saving of the order of $260,000 can be achieved, based on the 1992 Transit New Zealand road construction statistics (TNZ 1992).

Considering the savings over a 25 year period, the total present value of constructing pavements is approximately $M2.6.

On the cost side of the equation the one-off expense is performing the multi-layer elastic analyses and amending the design charts.

This cost is estimated to be approximately $60,000.

The benefit/cost ratio is therefore:

$$2{,}600{,}000 / 60{,}000 = 43$$

This is considered to be a conservative result since the cost part of the analysis involves a one-off expense.

11.3 Benefit/Cost Analyses of Material Characterisation Strategies

11.3.1 Evaluate Alternative Subgrade Support Tests

The CBR test is considered to be inappropriate for the accurate characterisation of a pavement subgrade. The apparatus subjects the test specimen to significant plastic deformations and yet the results are used to determine an elastic parameter. Although the CBR test is widely used, this does not justify its shortcomings.

The dynamic triaxial test, considered to be a preferable alternative, subjects the test specimen to stress conditions approximately equivalent to those experienced beneath a pavement. The disadvantage is one of cost since the dynamic triaxial test is approximately five times more expensive than the CBR test. The test would therefore only be justified for use on high cost projects where the quality of material test data is paramount.

The benefits of using the dynamic triaxial test over the CBR test are not possible to quantify.

11.3.2 Allow for Anisotropic Behaviour of Aggregates

The effect of introducing an allowance for anisotropic aggregate behaviour has not been quantified for this review as Bartley Consultants do not have the computer software necessary to undertake the analysis. Without undertaking at least a preliminary analysis, it is not possible predict whether allowing for anisotropy results in thinner or thicker pavements.

Therefore the adoption of anisotropic parameters is recommended to be worthy of further investigation, and the assignment of a benefit/cost ratio is inappropriate at the present time. It may be feasible to simply follow the AUSTROADS approach (1992) and adopt an anisotropy factor of 2.0 and adjust the design charts accordingly.

The cost of carrying out the multi-layer elastic analyses to include anisotropy is estimated to be \$30,000, plus the cost of the software which is approximately \$2,000.

11.3.3 Relax Sand Equivalent (SE) Specification

A number of aggregate producers throughout New Zealand have difficulty complying with the TNZ M/4 (1985) specification requiring an SE value of 40 for premium basecourse aggregates.

The SE test should be either replaced completely or the acceptance criterion reduced to a conservative maximum value of 30. This relaxation would have the effect of lowering the cost of basecourse aggregate by approximately 15% in many parts of New Zealand.

Assuming that a 15% saving is achieved on 20% of the basecourse aggregate supplied, the present value of the total saving over 25 years is estimated to be \$M3.7.

On the cost side of the equation is the one-off cost of rewriting the specification and implementation, which is estimated at \$40,000.

The benefit/cost ratio is therefore:

$$3{,}700{,}000 / 40{,}000 \quad = 93$$

This is considered to be a conservative result since the cost part of the analysis involves a one-off expense.

11.3.4 Replace M/4 (1985) Specification With Regional Specifications

Replacing the TNZ M/4 (1985) specification for basecourse aggregates with a number of regional M/5-type specifications is considered to be desirable, although the implementation would be costly and the benefits are impossible to quantify. A benefit/cost ratio is therefore inappropriate.

11.3.5 Investigate Use of Waste Products in Sub-Base Construction

In many countries, waste materials such as steel mill slag, crushed concrete and bricks are used in the construction of pavement layers. This use results in conservation of dwindling resources of natural aggregate and reduces the over-use of waste disposal sites. In New Zealand, steel mill slag provides an opportunity to achieve economic sub-base construction using waste material from the BHP New Zealand Steel site at Glenbrook, in Franklin County, South Auckland. Franklin District Council have constructed pavements using slag sub-base layers, but no performance monitoring information is available at this time.

A benefit/cost analysis has been carried out assuming that a saving of approximately 40% can be achieved on the delivered cost of steel mill slag compared with quarried aggregate. The analysis has only considered Manukau, Franklin and Papakura areas to be appropriate, being close to source. For other areas, transportation costs will become significant.

Transit New Zealand (1992) road statistics indicate that for these three areas, using slag for sub-base construction would provide a saving of approximately $80,000 per year provided the material characteristics are appropriate. This corresponds to savings with a present value of $800,000 over a 25 year period.

The cost part of the equation comprises the expenses involved in evaluating the material characteristics both by laboratory testing and the monitoring of existing pavements incorporating a slag sub-base, along with the analysis of such pavements using multi-layer elastic theory. It is estimated that the cost of carrying out these tasks would be of the order of $40,000.

The benefit/cost ratio is therefore:

$$800{,}000 \,/\, 40{,}000 = 20$$

This is considered to be a conservative result since the cost part of the analysis involves a one-off expense. The three areas considered in this investigation are subject to rapid roading development and further savings could be achieved by the use of slag in subdivisional and other roads not included in Transit New Zealand subsidies.

11.4 Benefit/Cost of Design Traffic Characterisation Strategies

11.4.1 Reduce Load Equivalence Exponent

The literature suggests that a load equivalence exponent of 3.0 may be more appropriate for unbound aggregate pavements than the widely used value of 4.0. This lower value reflects a lower sensitivity of this type of pavement to the spectrum of imposed axle loads. By inspection of the EDA versus axle load relationship, it is clear that, for a dual wheel axle, decreasing the load equivalence exponent results in higher values of EDA for axle loads less than 8.2 t and lower values of EDA for axle loads in excess of 8.2 t. The discrepancy between the EDA calculation for exponent values of 3.0 and 4.0 increases as the axle load increases. Changing the exponent is considered to have very little impact at the present time (1995), although if the axle load limits were to be raised a significant reduction in pavement thickness could be achieved by reducing the load equivalence factor.

The costs associated with this item would include the performance of further research using pavement materials and loadings typical to New Zealand conditions.

11.4.2 Increase EDA Tyre Contact Pressure

Preliminary analyses show that increasing the EDA tyre contact pressure to 700 kPa, which is the usual pressure in modern truck tyres, makes very little difference to the stresses and strains in the pavement. Adopting this condition would therefore have little or no economic benefit but would portray axle loading in a more accurate manner. Similarly for tyre type, the effect is not considered to be significant. The benefit is therefore intangible and a benefit/cost ratio is inappropriate.

11.4.3 Encourage Use of Air Bag Suspension Units

Air bag units are the least damaging type of heavy vehicle suspension and their use should be encouraged to minimise the dynamic loadings imposed on New Zealand's pavements. It is not possible to analyse the use of air bag suspension units in terms of a benefit/cost ratio because quantifiable data are insufficient. However, research into the behaviour of heavy vehicle suspension units is currently under way in New Zealand and it may provide sufficient information for this purpose at a later date.

11.4.4 Allow for Dynamic Loading in Pavement Design

The literature suggests that dynamic loading caused by an uneven pavement surface may result in axle loads of up to 150% of the static axle load. This should be allowed for in the design methodology although the expected result is impossible to predict in terms of a benefit/cost analysis because of a lack of quantifiable information. The research mentioned in Section 11.4.3 may provide further data, but at 1995 calculation of a benefit/cost value is not possible.

11.5 Benefit/Cost of Construction Strategies

11.5.1 Remove Plateau Density Concept From TNZ B/2 Specification

The plateau density concept is considered to be inappropriate for the specification of aggregate layer construction. A specification criterion which is independent of the compaction plant should be adopted, e.g. maximum total aggregate voids, minimum percentage of solid density or minimum percentage of a laboratory-determined dry density. Analysis of this item in terms of a benefit/cost ratio is inappropriate since the benefits and costs are impossible to quantify at the time of this review (1995).

11.5.2 Encourage Use of Paver Laid Aggregates

The construction of pavement layers using a paving machine provides many benefits over the conventional method of spreading and grading. The construction time is significantly reduced, as is the compaction requirement. The resulting ride quality is superior and the control on the layer thickness is improved.

Actual construction cost savings are difficult to ascertain since contractors are reluctant to detail their costs. Major savings to road users can be attributed to reduced delays during overlay works. However the magnitude of the saving is dependent on the type of road and the traffic using it.

Conversely, plant charge rates are higher since specific paving plant is required to carry out this work, and the wear and tear on the plant is increased. The difficulty of obtaining comparative cost information and the uncertainty of road user savings makes this item impossible to analyse with respect to a benefit/cost ratio, but it is considered that the benefits would exceed the costs by a significant margin.

11.5.3 Produce Specification For Aggregate Stockpiling

The benefits gained from a stockpiling specification are impossible to quantify, although improved stockpiling techniques will have the effect of reducing segregation and contamination of aggregates. The cost of this item comprises the expenses associated with researching, writing and implementing the specification.

11.5.4 Develop End-Result Construction Specifications

End-result specifications provide the resourceful contractor with sufficient freedom to carry out the pavement construction as efficiently as possible. Appropriate incentives ensure that the construction is of high quality and the completion is timely. A benefit/cost analysis is however inappropriate for this item since the benefits and costs could not be quantified at the time of this review.

11.5.5 Introduce Statistical Compliance Testing Specifications

The use of a statistical approach to compliance testing ensures that the characteristics of the pavement materials maintain adequate consistency over the area of the pavement and throughout the period of construction. A benefit/cost analysis is inappropriate for this item since the benefits and costs could not be quantified at the time of this review.

11.6 Summary of Benefit/Cost Analyses

Table 11.1 summarises the results obtained from the benefit/cost analyses made in this study. The results are ranked in order of priority. The items labelled N/A have benefits which are either intangible or impossible to quantify and their ranking has been based on a qualitative assessment.

Table 11.1 Summary of benefit/cost analyses.

ITEM	B/C RATIO
Base design on shakedown theory	99
Relax Sand Equivalent specification	93
Allow for layering of unbound courses	43
Investigate use of waste products for sub-base construction	20
Encourage use of paver laid aggregates	N/A
Remove plateau density concept from specification B/2	N/A
Develop end-result construction specifications	N/A
Allow for dynamic loading in pavement design	N/A
Evaluate alternative subgrade support tests	N/A
Produce specification for aggregate stockpiling	N/A
Encourage use of air bag suspension units	N/A
Allow for anisotropic behaviour of aggregates	N/A
Introduce statistical compliance testing specifications	N/A
Reduce load equivalence exponent	N/A
Increase EDA tyre contact pressure	N/A
Replace M/4 specification with regional specifications	N/A

The costs that are presented in Section 11.2-11.5 for further investigation and implementation of the research findings are indicative only.

12. RECOMMENDATIONS

Four top-ranked research strategies could be employed to modify or revise the current Transit New Zealand approach to the design and construction of sealed unbound granular pavements.

12.1 Development of Shakedown Theory

Development of shakedown theory for pavement design is considered to have a top priority for implementation because it has the potential to characterise pavement performance in a superior manner to the current mechanistic pavement design procedures. This will result in optimised pavement designs with significant reductions in life cycle costs.

If the shakedown approach to pavement design is to be developed for use by the New Zealand roading industry, many of the research findings discussed in Chapter 11, and summarised in Table 11.1, would not be applicable. The issues would either be addressed in the development of the theory or they would become irrelevant.

Shakedown theory would provide an improved characterisation of stress distribution within the pavement, and hence the issues of layering and modular ratio relationships would be accounted for.

Shakedown theory would simplify characterisation of pavement loading to the concept of a single design loading, hence the issue of equivalency exponents disappears, while other loading issues such as tyre contact pressure and the dynamic response of the vehicle axle would be incorporated in the development of the design methodology.

As well, development of shakedown methodology would address material characterisation issues such as anisotropy, density compliance specifications and appropriate materials testing procedures.

12.2 Relaxation of the Sand Equivalent Requirement

Relaxation of the SE requirement from the current value of 40 to a value of about 30 may be appropriate. This would result in a reduced aggregate cost as it would be easier to meet than the current specification which is difficult for many producers to meet.

12.3 Sub-layering of Aggregate Layers

Sub-layering of aggregate layers should be included in the Transit design method. The current design method combines the basecourse and the sub-base layers into one layer for design. Such design for only one layer resembles neither the construction sequence, nor the difference in quality of the materials used in basecourse and sub-base layers. With such a change in approach, reduced pavement thicknesses may be achieved by the superior pavement characterisation provided by sub-layering the granular layers.

12.4 Use of Waste Products for Sub-base Construction

In the future, the use of waste and marginal materials, such as steel mill slag, for sub-base construction will become more important as other resources diminish. Steel mill slag is currently being trialled in the South Auckland - Waikato regions, and its use should be encouraged.

APPENDIX
REVIEW OF ADDITIONAL LITERATURE

REVIEW OF ADDITIONAL LITERATURE

A1. Introduction

Since completion of the initial draft of this report (1995), four additional documents have been produced by, or for, Transit New Zealand which have relevance to this project. They are:

- Transit New Zealand Specification for Basecourse Aggregates, TNZ M/4:July 1995 (TNZ 1995a).
- Transit New Zealand Specification for Construction of Unbound Granular Pavement Layers, TNZ B/2:1995 (Draft 4/95) (TNZ 1995b).
- AUSTROADS Pavement Design, A Guide to the Structural Design of Road Pavements. New Zealand Supplement (TNZ 1995c).
- Design Guide for Assessing Freeze-Thaw Effects in Pavements (draft report PR3-0065/1) (WCS 1995a, b).

This appendix summarises the main points relevant to granular unbound pavements that are contained in these documents. Any factors or strategies which are considered to be potentially of benefit to the design or construction of unbound granular pavements are identified and discussed. In addition, any factors that were identified in the research described in this report and that have since been incorporated in these later documents are identified.

A2. Transit New Zealand M/4:1995 Specification

The TNZ M/4 *Specification for Basecourse Aggregates* (TNZ 1995a) includes a number of variations from the previous version (TNZ 1985), e.g.

- a distinction is made between an aggregate's source properties and its production properties;
- the material source is subject to a petrographic examination;
- a minimum CBR is specified;
- compliance testing frequencies are provided;
- the Sand Equivalent requirement is altered; and,
- the earlier TNZ M/5 specifications (TNZ 1993a) for regional aggregates have been merged into the M/4 document.

A2.1 Source Property Tests

Source property tests comprise crushing resistance, weathering quality index and CBR. These tests must be carried out at a frequency of at least one test per 10,000 m^3 of material. The compliance criteria for the crushing resistance and weathering quality index tests are unchanged from the 1985 M/4 document. The CBR test is a new requirement and a compliance criterion of 80% has been adopted.

A petrographic examination of the source material is required at least every four years, or biennially if the source material exhibits significant changes over a two year period. The petrographic examination must follow the guidelines given in *ASTM C295-Standard Practice for Petrographic Examination of Aggregate for Concrete* (ASTM 1990).

A2.2 Production Properties

Production properties comprise Sand Equivalent value, clay index, plasticity index, broken face content and particle size distribution. The compliance criteria for the broken face content and particle size distribution are unchanged from the 1985 M/4 document.

The Sand Equivalent value requirement has been relaxed from the previously stipulated value 40, to allowing three options comprising the sand equivalent value, the clay index and the plasticity index.

The 1995 version of TNZ M/4 states that, if the sand equivalent value is less than 40, the contractor can carry out clay index or plasticity index tests as well. Then, as long as a clay index value of 3 or less, or a plasticity index of 5 or less, is achieved, the aggregate is in compliance with the specification. If these additional tests confirm that the clay index or the plasticity index is sufficiently low, then the fine fraction of the aggregate is confirmed because of this clause to be of low plasticity even though the sand equivalent value may be relatively low. This clause is consistent with the recommendation made in Chapter 12 of this report regarding relaxation of the sand equivalent requirement.

While the authors of this report are in general agreement with the basis of the TNZ M/4 (1995) specification, a review of the source and production testing requirements may be worthy of consideration. The clay index test is considered to be more indicative of aggregate source properties than production properties. Therefore the clay index test should be included in the source test classification. Further, specimens of powdered rock should be used for the test.

A2.3 Compliance Criteria for Regional Aggregates

The other difference between the 1985 and 1995 versions of the M/4 specification is that the latter includes compliance criteria for regional aggregates. These materials were previously included in a collection of TNZ specifications entitled TNZ M/5 *Regional Basecourse Aggregates* (TNZ 1993a). The creation of separate specifications for aggregates available in different regions gave those aggregates an unwarranted reputation of inferiority. By combining them as part of the M/4 specification should dispel this attitude to some degree. The regional aggregates should be regarded as having to meet certain conditions rather than being significantly inferior to the higher specification M/4 materials. This clause is consistent with the recommendation made in Chapters 10 and 11 of this report to merge the M/4 and M/5 specifications into a single document.

A2.4 CBR Requirement

Another issue of contention in the TNZ M/4 (1995) specification is the inclusion of the CBR requirement. The reason for including this test is assumed to be to substantiate the elastic modulus value used in the pavement design. However, the CBR test is widely recognised as not being a particularly reliable indicator of elastic modulus. A large scale triaxial test would provide accurate resilient modulus values and be more useful to the pavement designer. Notwithstanding this point, the CBR compliance criterion of 80% is considered to be relatively low. If the material satisfies the other criteria in the M/4 specification, the CBR result should always be well in excess of 80%. It is noted however that the CBR requirement has been removed from most of the regional basecourse specifications included in the TNZ M/4 (1995) document.

An alternative method of substantiating the elastic modulus of the basecourse layer is to prepare a trial section of unbound granular pavement and carry out back-calculation analysis of the deflected shape, as determined using a falling weight deflectometer (FWD) or a Benkelman beam.

A3. Draft Transit New Zealand B/2:1995 Specification

The TNZ B/2 Draft *Specification for Construction of Unbound Granular Pavement Layers* (TNZ 1995b) includes a number of variations from the previous (TNZ B/2:1987) version. For example:

- construction tolerances have been adjusted;
- aggregate layer thickness tolerances have been changed;
- the exposure to road traffic of layers under construction is allowed;
- the previous plateau density approach has been replaced with target dry density criteria;
- the requirements for running course materials have changed slightly.

A3.1 Construction Tolerances

For unconstrained layers the upper limit on the constructed width tolerance has been increased from +50 mm to +100 mm. The lower limit on the width tolerance is unchanged at -20 mm.

The upper limit on the vertical tolerance for basecourse layers has been increased from +15 mm to +40 mm for pavements without a concrete channel. For pavements including a concrete channel the upper limit on the vertical tolerance ranges linearly from +5 mm at the lip of the channel to +40 mm at the centreline of the road.

Although only a limited number of construction specifications have been reviewed in the research described in this report, the range of vertical tolerances from +5 mm to +40 mm appears to relatively tight at the low end and relatively generous at the high end.

A3.2 Aggregate Thickness Tolerances

The allowable (average) thickness of aggregate layers has been tightened so that the minimum uncompacted layer thickness is 2.5 times the maximum particle size and the maximum uncompacted layer thickness is 200 mm. Previously the minimum thickness was 2.0 times the maximum particle size and the maximum thickness was 250 mm.

A3.3 Exposure to Road Traffic

The 1995 version of TNZ B/2 gives the engineer the option to allow normal traffic to use a pavement layer under construction. This trafficking will assist in the compaction process, especially if the traffic lanes are moved progressively across the full width of the pavement to obtain a uniform compaction effect. The specification suggests that narrow traffic lanes are adopted to help maintain relatively low traffic speed. The mixture of light and heavy vehicles, and the kneading action that they produce, should contribute significantly to the compaction of aggregate layers. This should also help to alleviate the problem of rutting in newly constructed pavements that sometimes occurs.

In addition, the draft TNZ B/2 specification now requires that sealing shall not be commenced until the basecourse layer has been trafficked by a minimum of 1500 vehicles. However it is important that the basecourse surface is not loosened or otherwise damaged by this trafficking.

A3.4 Target Dry Density Criteria

A frequent criticism of the 1987 TNZ B/2 specification is its reliance on a *plateau density* approach to compaction. The plateau density is the threshold of compaction at which extra rolling results in only a negligible increase in density. The deficiency of this approach is that the plateau density is dependent on the compactive effort provided by the particular compaction plant being used and the particle size distribution of the aggregate. The 1995 draft TNZ B/2 specification however replaces the plateau density approach with target dry density values. The target values for basecourse and sub-base layers are expressed as a percentage of maximum dry density, as shown in Table A1. The change away from the plateau density approach is consistent with the recommendation made in Chapters 10 and 11 of this report.

Table A1 Dry density compliance criteria for basecourse and sub-base layers from the draft TNZ B/2 specification (TNZ 1995b).

Value	**Percentage of Maximum Dry Density**	
	Sub-base Pavement Layer	**Basecourse Pavement Layer**
Mean	≥ 95	≥ 98
Minimum	≥ 92	≥ 95

The target dry density values in Table A1 are expressed with respect to the maximum dry density obtained in the laboratory using the New Zealand vibrating hammer compaction test (NZS 4402:1986 Test 4.1.3 (SANZ 1986)). This test uses a compactive effort approximately equivalent to the heavy (or modified) compaction test. The densities specified in the draft TNZ B/2 specification are generally similar to those in other specifications presented in Section 4 of this report.

The comparison of dry densities measured in the field with dry densities measured in the laboratory is a point of contention, because of difficulties involved in preparing laboratory specimens incorporating relatively large particle sizes in reasonably small steel moulds. Therefore the field and laboratory measurements can be inconsistent. A compliance criterion that determines the percentage of total voids is considered to be more appropriate, e.g. approximately 10 to 15% total voids is a better measure of the level of compaction.

The nuclear density meter (NDM) is recommended as an appropriate tool for validating the density of aggregate courses while final consolidation and/or stability after traffic use can be evaluated using the NDM, Clegg Impact Soil Tester, the Loadman Portable Falling Weight Deflectometer (FWD), or the Falling Weight Deflectometer. The Loadman apparatus is relatively new to New Zealand and, although it has received positive reviews (Whaley 1994, Fleming and Rogers 1995), it will require some time to become known and accepted into general use.

A3.5 Requirements for Running Course Materials

The 1995 draft TNZ B/2 specification includes a number of minor changes to the requirements for running course materials. A significant addition is the requirement that the water content of the material must be less than 70% of the optimum water content before a seal coat can be applied. This change is considered to be consistent with good construction practice, although it may be difficult to achieve at some locations in New Zealand because of climatic conditions.

A4. New Zealand Supplement to AUSTROADS Pavement Design Guide

In July 1995, Transit New Zealand adopted the AUSTROADS (1992) document *Pavement Design, A Guide to the Structural Design of Road Pavements* for use in New Zealand. The change was inspired by Transit New Zealand's recent incorporation into the AUSTROADS organisation and the desire for a consistency of approach between the New Zealand and Australian roading fraternities. Adoption of the AUSTROADS pavement design procedures has realised a number of the recommendations made in Chapters 10 and 11 of this report. They are:

- allowance for layering of unbound courses;
- support for the use of alternative subgrade support test methods;
- inclusion of anisotropic behaviour of aggregate layers (and the subgrade); and,
- increasing the equivalent design axle contact pressure to a more realistic value.

A supplement to the AUSTROADS Guide has been produced (TNZ 1995c) to translate a number of the design, construction and materials issues from the Australian to the New Zealand context. These changes are generally of a minor nature, although the New Zealand Supplement is expected to evolve relatively quickly as New Zealand's roading design and construction practitioners gain experience in using the AUSTROADS procedures.

The main clauses of the New Zealand Supplement (TNZ 1995c) are as follows:

- ***Introduction*** Justification of the adoption of the AUSTROADS Guide is presented along with a brief description of the philosophy behind the procedures.

- ***Construction and Maintenance Considerations*** In this section the designer is reminded of the requirements of New Zealand's Resource Management Act (1991), i.e. to promote the sustainable management of natural and physical resources. The Act also requires the developer to consult with all interested parties and to obtain resource consents for virtually all activities which affect waterways or involve earthworks.

  Boxed construction is described as being inappropriate for use in New Zealand and the engineer is required to undertake suitable materials and construction monitoring tests to ensure that the assumptions made during design can be substantiated.

- ***Environment*** This section simply states that freeze-thaw conditions do occur in some locations in New Zealand and the engineer must ensure that suitable materials and construction procedures are adopted in such circumstances.

- ***Subgrade Evaluation*** This section rejects the use of the modulus of subgrade reaction (k) and indicates that the ratio of CBR_{OMC}* to CBR_{SOAKED} may be significantly higher than the values presented in the AUSTROADS Guide. The supplement states that soaked CBR values are appropriate whenever the groundwater may rise to a level within one metre of the top of the subgrade, or the pavement could be subject to inundation by flooding.

  The section also presents the appropriate test references for dynamic cone, in situ CBR and laboratory CBR tests that are used in New Zealand.

- ***Pavement Materials*** This section states that presumptive values of elastic modulus for stabilised subgrade soils are not included in the AUSTROADS Guide. Although a research project is under way addressing this issue, no data are currently available and the engineer should obtain the appropriate data from laboratory testing. The failure criterion for stabilised soils is to be taken as Line 1 in Figure 6.1 of the AUSTROADS Guide, i.e. the same as that for cemented materials with an elastic modulus value of 2000 MPa. This is a conservative approximation, and the results of current research should provide an improved design criterion.

  A reference is made to the requirements of the TNZ M/4 *Specification for Crushed Basecourse Aggregate* (TNZ 1985). The appropriate elastic modulus values for New Zealand aggregates need to be established.

- ***Design Traffic*** This section simply points out that there is a lack of appropriate traffic data for use New Zealand. A research project is currently (1996) under way to address this issue.

- ***Design of New Flexible Pavements*** This section states that the critical response of a cemented soil is the same as that for a cemented granular material, i.e. the tensile strain is at the bottom of the layer. The reader is also referred to the requirements of the TNZ M/4 (1985) specification with regard to basecourse material compliance criteria.

- ***Overlay Design*** This section encourages the engineer to utilise the FWD to determine the response of an existing pavement structure to the application of a load at the surface. However, a cautionary note states that a correlation from the FWD data to equivalent Benkelman Beam data may be required to use the overlay design charts in the AUSTROADS Guide.

  The AUSTROADS Guide includes a description of the seven NAASRA Road Functional Classes whereas Transit New Zealand do not have road classes for pavement design (except lower grade and premium). The supplement simply states that references to road classes should be ignored except where the engineer

* OMC optimum moisture content

is required to decide on an appropriate terminal roughness or f-value (for overlay design).

An observation is made regarding the build-up of successive chipseals on some pavements resulting in a significant thickness of material with a single particle size and an excess of bituminous binder. Such a layer generally behaves as a very poor asphaltic concrete and should be removed or stabilised appropriately before an overlay is applied.

- ***Appendixes*** The appendixes to the New Zealand supplement contain appropriate New Zealand data for pavement life multipliers and weighted mean annual pavement temperatures. A guide to pavement condition investigation and assessment is also presented.

A5. Design Guide for Assessing Freeze-Thaw Effects in Pavements

A design guide for assessing freeze-thaw effects in pavements is currently being developed by researchers in New Zealand. Although the guide is not yet available, the project has produced draft reports describing a laboratory-based frost test investigation, and a review of literature and field practices. The main conclusions of these reports are described below.

The first draft report, entitled *Freeze-Thaw Behaviour of Basecourse Materials: TNZ/CL Frost Test Results and Identification Method* (Works Consultancy Services Ltd 1995a) presents and discusses the results of tests on basecourse aggregates carried out using the TNZ/CL Frost Test Cabinet. The apparatus accommodates aggregate samples in CBR moulds and can subject the materials to both freezing and thawing conditions.

Ten test samples were used in the investigation, five of which exhibited good freeze-thaw performance in the field, four performed poorly and one was marginal. The authors reported that a heave displacement of 14 mm or less after four days freezing in the TNZ/CL apparatus indicated that the basecourse aggregate was frost resistant.

Material classification tests were also carried out and the following criteria were reported as being appropriate for frost-resistant basecourse aggregate:

- particle size distribution satisfying the TNZ M/4 specification for basecourse aggregates;
- sand equivalent value exceeding 50; and
- fines (<75 μm) content less than 5%.

The second draft report, entitled *Freeze-Thaw Behaviour of Basecourse Materials Phase 2 Stage 1 Report on Review of Literature Since 1985 and Field Practices* (Works Consultancy Services Ltd 1995b) discusses the technical literature available on the topic of basecourse behaviour under freeze-thaw conditions. This report incorporates the conclusions of the first report, but also concludes that most of the overseas research is not directly applicable to the New Zealand situation. However, three areas of the literature have been identified as worthy of further investigation, i.e.

- theoretical models to predict frost heave and frost penetration;
- empirical methods to determine frost penetration and monitor pavement performance during thaw; and
- stabilising agents to improve frost resistance of basecourse.

The report takes the approach that basecourse aggregate should have a relatively open grading so that water can drain out of a layer that is prone to freezing temperatures. It is claimed that the relatively high permeability of the TNZ M/4 grading is responsible for this material being frost resistant. The accuracy of this claim is questionable because the permeability of a layer of compacted M/4 aggregate cannot be guaranteed to provide free drainage. The report does not acknowledge that a low permeability material may be more suitable than a high permeability material for frost resistance. However, results are cited from a TRRL trial which showed that adding a fine grained material (bentonite) to gravel improved frost-heave resistance.

In separate research, Rutter (1986) investigated basecourse permeability values and found that, in a well compacted basecourse layer, the permeability could be less than 10^{-6} m/s. The argument is that a low permeability layer will not allow water to enter the pavement and therefore the volume of water available for freezing is minimised.

A6. Conclusions

A6.1 Recommendations Addressed by Additional Documents

In this extension to the original literature review discussed in the body of this report, four recent documents have been reviewed with respect to the design, construction and materials required for unbound granular pavements. Three of the documents have been (or are in the process of being) officially adopted by Transit New Zealand. In addition, Transit New Zealand has recently (1995) adopted the AUSTROADS *Pavement Design Guide* for use in New Zealand. These documents address a number of the issues raised in the body of this report, as shown in Table A2.

Table A2 Summary of issues identified in this literature review and subsequently addressed by documents recently adopted by Transit New Zealand.

Issue	**Document**	**Outcome**
Relaxation of Sand Equivalent value criterion.	TNZ M/4	If SE < 40 the aggregate is tested for CI or PI. A CI≤3 or PI≤5 → aggregate is in compliance with M/4.
Incorporate TNZ M/5 specifications for regional aggregates into TNZ M/4.	TNZ M/4	Tables of compliance criteria for regional aggregates included in the TNZ M/4 document.
Removal of the plateau density approach to aggregate compaction.	TNZ B/2	Target dry densities for sub-base and basecourse are given with respect to %age of maximum dry density.
Sublayering of unbound courses in design.	AR Guide	Sublayering of unbound courses is a component of the AUSTROADS pavement design procedure.
Support for the use of alternative subgrade support test methods.	AR Guide	The AUSTROADS Guide allows the engineer to use test methods other than the CBR to determine subgrade support.
Allow for anisotropic material properties.	AR Guide	The AUSTROADS Guide recommends that anisotropic material parameters are used for subgrade and unbound pavement layers.
Increase EDA contact pressure to a more realistic value.	AR Guide	The AUSTROADS Guide allows the engineer to use an appropriate tyre contact pressure in the range 550-800 kPa.

Notes: TNZ M/4 Transit New Zealand M/4 Specification for Basecourse Aggregates (TNZ 1995a)
TNZ B/2 Transit New Zealand B/2 (Draft) Specification for Construction of Unbound Granular Pavement Courses (TNZ 1995b)
AR Guide Pavement Design, A Guide to the Structural Design of Road Pavements (AUSTROADS 1992); New Zealand Supplement (TNZ 1995c)

A6.2 Further Recommendations to be Addressed

The updates to the Transit New Zealand M/4 and B/2 specifications are considered to be beneficial, although consideration should be given to the following amendments or additions:

TNZ M/4:1995 (TNZ 1995a)

- Placing the clay index test in the material source test category rather than in the production test category.
- Removal of the CBR requirement, and replacement with at least a recommendation to use the resilient modulus triaxial test.

TNZ B/2 Draft:1995 (TNZ 1995b)

- Replacement of the dry density compaction criteria for unbound aggregates with criteria based on the percentage of total voids.

New Zealand Supplement to AUSTROADS Pavement Design Guide (TNZ 1995c)
The New Zealand Supplement to the AUSTROADS Pavement Design Guide provides a New Zealand perspective to the AUSTROADS design procedures. It requires additional information regarding the performance criterion for stabilised subgrade materials and appropriate traffic loading data for New Zealand roads. Research into these issues is understood to be under way and the findings will be available for inclusion in a future update of the supplement.

Freeze-thaw Projects
A proposed report describing guidelines for assessing freeze-thaw effects in pavements is not available at the time of writing this review. However, the findings of two preliminary reports on the topic have not produced definitive conclusions.

Issues Not Yet Addressed
Comparing the list of issues identified in the body of this report (Table 8.1) with those adopted in the recently published documents (Table A2) indicates that 9 of the original 16 issues remain unaddressed. These are as follows:

- base pavement design on shakedown theory;
- investigate use of waste products for sub-base construction;
- reduce load equivalence exponent;
- encourage use of air bag suspension units;
- allow for dynamic loading in pavement design;
- encourage use of paver laid aggregates;
- produce specification for aggregate stockpiling;
- develop end-result construction specifications;
- introduce statistical compliance testing specifications.

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