Origins of AUSTROADS design procedures for granular pavements

G.W. Jameson
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ABSTRACT: In Australia the thickness design of asphalt and granular overlays for flexible pavements has, for over 30 years, been based on maximum rebound deflection measured with a Benkelman Beam under a Standard Axle wheel load. Recently, however, mechanistically-based overlay methods have been developed to complement the new pavement design procedures and it has been found that the thicknesses of overlays determined using these mechanistic procedures can differ substantially from those determined using the existing procedures.

To assist in understanding the reasons for these differences, the origins of several cornerstones of the AUSTROADS pavement design and overlay design procedures were examined.

The design deflection criteria developed during the 1960s were derived from the unbound granular thickness chart and from field measurements of the dependence of maximum deflection on subgrade CBR and granular thickness. The granular thickness chart was developed from the 1940's Californian State Highway Department CBR design method. Using the granular thickness chart and field data a single design deflection criteria was adopted for all pavement types despite the observation that deflection depended on granular thickness and subgrade CBR. This significantly effects the credibility of the design deflection criteria.

The current mechanistically-based overlay design procedures are based in part on the subgrade strain criterion used for the design of new flexible pavements. This criteria was derived from an analysis of the granular thickness chart. A re-analysis of the data indicated that its use can now be extended to lightly trafficked roads.

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Origins of AUSTROADS design procedures for granular pavements

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Origins of AUSTROADS design procedures for granular pavements

Executive Summary

Over the last 30 years measurements of pavement deflection caused by a Standard Axle load have been used by Australian Road Agencies to evaluate the structural adequacy of existing flexible pavements. Originally, the maximum rebound deflection measured with a Benkelman Beam was compared to the design deflection for the expected traffic. During the 1980s, the curvature function \((D_o - D_{200})\) was adopted to provide a more reliable estimate of asphalt fatigue life, with the maximum rebound deflection retained to assess permanent deformation. These elements formed the basis of the AUSTROADS (1992) overlay design procedures.

Following the development of the mechanistically-based design procedures for new pavements, concerns were expressed that the outputs from the overlay design procedures recommended in AUSTROADS (1992) were not consistent with the mechanistically-based design procedures for new pavements. Consequently, two interim mechanistically-based overlay design procedures were issued in 1994: the General Mechanistic Procedure (GMP) and the AUSTROADS Simplified Mechanistic Overlay (ASMOL) procedure. The validity of the ASMOL procedure was subsequently examined under an AUSTROADS research project (Jameson, Moffatt and Armstrong 1995) and it was concluded that the overlay thicknesses derived using the mechanistic procedures could be substantially greater than those calculated using the AUSTROADS (1992) procedures.

It was considered that the cause of the increased overlay thickness requirements may be related to the criteria used to control permanent deformation. In terms of assessing permanent deformation over the design period, the AUSTROADS (1992) overlay design procedure is based on limiting maximum deflection under Benkelman Beam loading to a design deflection level. The interim mechanistic overlay procedures, on the other hand, are based on conventional mechanistic design principles and use the AUSTROADS subgrade strain criterion to estimate the allowable load repetitions.

Given the increased cost of the overlays that would result from the adoption of these new mechanistically-based procedures, it was clear that the origins of the AUSTROADS (1992) and interim overlay design procedures needed to be investigated as these were not well understood. The findings of this review are as follows.

Unbound Granular Thickness Chart

The design chart for granular pavements with thin bituminous surfacing, currently Fig. 8.4 of AUSTROADS (1992) evolved from the 1940s Californian State Highway design thickness curves. These curves were refined and extended by the U.K. Road Research Laboratory. With minor modifications these design curves have been widely
used by Australian State Road Authorities for over 30 years. There has been no research undertaken to verify these design curves. A limited survey of experienced engineers indicated that pavements designed using these chart have about a 90% chance of exceeding the design traffic loading.

**Subgrade Strain Criterion**

The subgrade strain criterion was derived from the above granular thickness chart. Pavement modelling, undertaken as part of this project indicates that its use can now be extended to lightly-trafficked roads.

As the criterion is based on the granular thickness chart, and since this chart is conservative, the criterion underestimates average allowable traffic loading. This may lead to overestimates of average overlay thicknesses when the criterion is used in the interim AUSTROADS mechanistically-based overlay procedures. The appropriate strain criterion for use in overlay design needs to be reconsidered when design reliability procedures are developed for overlay design.

Amendment to the criterion is, however, required to express life in terms of Standard Axles rather than Equivalent Standard Axles.

**Design Deflections**

The AUSTROADS (1992) design deflections for traffic loading less than $10^6$ ESAs was derived from research undertaken by Scala in the early 1960s. These design deflections were derived from field measurements of the dependence of Benkelman Beam deflections on granular thickness and subgrade CBR and the relationship between pavement composition and design traffic from the granular thickness chart. The Benkelman Beam data used to derive the relationship was not the mean deflection data for a given granular thickness and subgrade CBR. As the low deflection data (<0.8 mm) was excluded from the analysis, the design deflection criterion relates to the maximum expected deflection for a given design traffic loading rather than the mean deflection. Consequently, this criterion is applicable to pavements with lower strength (CBR < 8) subgrades. There is some doubt about the appropriateness of the 1992 AUSTROADS design deflections for assessing the overlay requirements of pavements with subgrade CBR's exceeding 8. For such pavements there are large differences between the overlay thicknesses determined using the AUSTROADS (1992) and mechanistically-based AUSTROADS (1994) overlay design procedures.

For traffic loadings exceeding $10^6$ ESAs the origins of the AUSTROADS (1992) design deflections are less clear. From the limited information available it seems that they were estimated from Californian design deflections for asphalt-surfaced pavements. There appears to be no substantial published data to support the use of these design deflections for traffic loadings exceeding $10^6$ ESAs. They have, however, been used in Australia for overlay design for over 20 years without any obvious problems.

**Recommendation**

After the ASMOL procedures have been extended, it is recommended that the differences in overlay thicknesses of the existing overlay design procedures and ASMOL should be re-examined to enable a recommendation as to an appropriate overlay design procedure for adoption by AUSTROADS. In this evaluation and recommendation, cognisance should be taken of the origins of the two overlay procedures.
1. Introduction

Over the last 30 years measurements of pavement deflection caused by a Standard Axle load have been used by Australian Road Agencies to evaluate the structural adequacy of existing flexible pavements. Originally, the structural adequacy was assessed by comparing the maximum rebound deflection measured with a Benkelman Beam with the design deflection for the expected traffic. During the 1980s, the curvature function \( D_0 - D_{200} \) was adopted to provide a more reliable estimate of asphalt fatigue life, with the maximum rebound deflection retained to assess permanent deformation. These elements formed the basis of the AUSTROADS (1992) overlay design procedures.

Following the development of the mechanistically-based design procedures for new pavements, concerns were expressed that overlay thicknesses derived from the overlay design procedures recommended in AUSTROADS (1992) were not consistent with the mechanistically based design procedures for new pavements. Consequently, two interim mechanistically-based overlay design procedures were developed and issued in 1994: the General Mechanistic Procedure (GMP) and the AUSTROADS Simplified Mechanistic Overlay (ASMOL) procedure. The validity of the ASMOL procedure was subsequently examined under an AUSTROADS research project (Jameson, Moffatt and Armstrong 1995) and it was concluded that the overlay thicknesses derived using the mechanistic procedures could be substantially greater than those calculated using the AUSTROADS (1992) procedures.

It was considered that one of the reasons for the increased overlay thickness requirements may be related to the criteria used to control permanent deformation. In terms of assessing permanent deformation over the design period, the AUSTROADS (1992) overlay design procedure is based on limiting maximum deflection under Benkelman Beam loading to a design deflection level. The interim mechanistic overlay procedures, on the other hand, are based on conventional mechanistic design principles and use the AUSTROADS subgrade strain criterion to estimate the allowable load repetitions.

Given the increased cost of the overlays that would result from the adoption of these new mechanistically-based procedures, it was apparent that the origins of the AUSTROADS (1992) and interim overlay design procedures needed to be investigated and documented. As the subgrade strain criterion was derived from the unbound granular thickness chart (Fig. 8.4 of AUSTROADS (1992)), it was also necessary to review the origins of this chart.

This report reviews the origins of:

- the unbound granular thickness chart (Fig. 8.4 of AUSTROADS (1992)),
- the subgrade strain criterion, and
- the design deflection criterion (Fig. 10.3 of AUSTROADS (1992)).

2. Unbound Granular Thickness Chart

The AUSTROADS (1992) thickness design chart for granular pavements with thin bituminous surfacings is shown in Fig. 1. The origins of this chart can be traced back to the Californian State Highways Department CBR method of pavement design (Porter 1942). From 1928-1942, the Department examined the quality and thicknesses of base, subbase and subgrade materials under both failed and sound sections of flexible pavements throughout the California highway system. From these data, curves were formulated for determining the total depth of
construction (base, subbase and imported fill) required to carry the anticipated traffic. The resulting design curves are given in Fig. 2.

![Design Traffic Chart](image)

**Fig. 1 - AUSTROADS design chart for granular pavements with thin bituminous surfacings (Fig. 8.4 of AUSTROADS 1992)**

![CBR Chart](image)

**Fig. 2 - California State Highway Department 1940's CBR method thickness design curve (RRL 1952; Porter 1942)**

In 1945 the Victorian Country Roads Board (CRB) proposed a tentative thickness design chart (Fig. 3) which seems to have been based on the Californian design curves (Gawith and Perrin...
1962). This design procedure was an improvement on the 1942 Californian procedure in that it quantified the traffic, provided factors which allowed for transverse distribution of traffic, and a factor to correct thickness for rainfall. This method was refined further when the CRB issued Technical Bulletin 4 in 1949. This was used by the CRB until Technical Bulletin 21 was issued in 1960, as discussed below.

![Diagram](image-url)

**Fig. 3 - 1945 Victorian Country Roads Board tentative thickness design curves**  
*(Gawith and Perrin 1962)*

In the late 1940s the U.K. Road Research Laboratory (RRL) compared the total pavement thicknesses required by the Californian CBR method with actual thicknesses of roads of various condition (Davis 1949). The results are shown in *Fig. 4*. Data was examined from seven sites where at least part of the road was distressed due to deformation of the subgrade. In making the comparison with the Californian curves it was considered that:

- the Californian design curve for a maximum wheel load of 7,000 lb was equivalent to less than 50 commercial vehicles per day (light traffic),
- the Californian design curve for a maximum wheel load of 9,000 lb was equivalent to medium traffic of 50 to 300 commercial vehicles per day (medium traffic), and
- the Californian design curve for a maximum wheel load of 12,000 lb was equivalent to more than 300 commercial vehicles per day (heavy traffic).

Davis concluded that:

"Evidence of the validity of the design curve is provided by the fact that all "critical condition" points lie close to the 45° line, all the 'definite failure' points lie below the line and all the "no failure" points lie above the line. The number of points in this figure (*Fig. 4*) is hardly sufficient to provide conclusive evidence and further investigations of this type are desirable."
Several years later MacLean (1954) reported that the design curves A-F in Fig. 5 were being considered for use by the RRL. According to MacLean:

"The form of these curves is based on a consideration of the results of full-scale road experiments carried out by the Laboratory and of information supplied by county road authorities who have applied the Californian bearing ratio method of design in normal road construction. Six curves A, B, C, D, E and F are shown relating the thickness of construction to the Californian bearing ratio of the sub-soil for roads carrying six different intensities of traffic. This classification of traffic into 6 groups is based on the number of vehicles using a road per day having a loaded weight exceeding 3 tonnes.

Curve A is a new curve which has been proposed for roads carrying 0-15 vehicles per day weighing more than 3 tons. It refers to cul-de-sacs on housing estates and to isolated roads built in connection with limited private housing development.

Curve B is another new curve for roads carrying 15-45 vehicles per day weighing more than 3 tons. It refers to minor through roads on housing estates which carry a fair amount of traffic during the period of house construction but which carry no heavy lorry traffic or public service vehicles subsequently.

Curve C for roads carrying 45-150 vehicles per day weighing more than 3 tons, has been in use for many years. It applies to lightly trafficked county roads and to roads on housing estates carrying up to 50 public service vehicles per day together with a fair number of tradesman's vehicles.

Curve D has also been in use for some time and is for roads carrying 150-450 vehicles per day weighing more than 3 tons. It refers to county roads carrying a medium intensity of traffic and to main roads in urban areas where form 50-150 public service per day are operating."
Curve E, for roads carrying 450-1500 vehicles per day weighing more than 3 tons, has also been in use for many years. It refers to principal shopping streets in large towns and to main county roads.

Curve F is another new curve for heavily-trafficked truck roads carrying 1500-4500 vehicles per day weighing more than 3 tons. The need for this curve has become apparent as the result of investigations of structurally weak sections of roads and of full-scale experiments on truck roads.

![Diagram of CBR design curves for different classes of roads](Fig. 5 - Proposed CBR design curves for different classes of roads (MacLean 1954 and 1959))

It should be noted that curves C, D and E were very similar to Californian CBR design curves (Fig. 2). Note that the traffic loadings associated with these three curves differ from those initially adopted by Davis (1949). In 1959 MacLean reported that curve G was being used for new roads "which may carry traffic in excess of 4,500 commercial vehicles per day."

In 1955 the RRL published Road Note 20 "Construction of Housing-Estate Roads using Granular Bases and Subbases Materials". For such roads design curves A-E were proposed.

As stated by Leigh and Croney (1972), the Fig. 5 design curves:

"...provided a means for estimating the total thickness of construction necessary for various traffic and foundation conditions, but gave no guidance on the relative thicknesses of surfacing, base and subbase"

Accordingly, a series of full-scale experiments of in-service roads was conducted to examine the performance of roads with variations in materials and layer thicknesses. By 1960 sufficient data was available to issue preliminary design standards and these were contained in RRL's Road Note 29 "A Guide to the Structural Design of Pavements for New Roads". This document superseded Road Note 20 for roads with a traffic loading of more than 150
commercial vehicles per day. In 1965 the second edition of Road Note 29 was extended to lightly-trafficked roads and the use in Britain of Road Note 20 presumably ceased.

In 1960 the CRB adopted (CBR 1960) the 1959 RRL design curves (Fig. 5). These curves were revised in 1969 (CBR 1969) to provide higher minimum pavement thicknesses (Fig. 6). It was also specified that the curves were only applicable for pavements in rural areas, that is granular pavements with a sprayed seal surface.

```
<table>
<thead>
<tr>
<th>CALIFORNIA BEARING RATIO - PER. CENT.</th>
</tr>
</thead>
<tbody>
<tr>
<td>DEPTH OF CONSTRUCTION - INCHES</td>
</tr>
<tr>
<td>---------------------------------------</td>
</tr>
<tr>
<td>2 3 4 5 6 7 8 9 10 15 20 30 40 50 60 70 80 90 100 150</td>
</tr>
<tr>
<td>---------------------------------------</td>
</tr>
<tr>
<td>TRAFFIC CLASSIFICATION</td>
</tr>
<tr>
<td>CURVE NUMBER OF VEHICLES PER DAY</td>
</tr>
<tr>
<td>EXCEEDING 3 TONS LOADED WEIGHT</td>
</tr>
<tr>
<td>A 0 15</td>
</tr>
<tr>
<td>B 15 45</td>
</tr>
<tr>
<td>C 45 150</td>
</tr>
<tr>
<td>D 150 450</td>
</tr>
<tr>
<td>E 450 1500</td>
</tr>
<tr>
<td>F 1500 4500</td>
</tr>
<tr>
<td>G ABOVE 4500</td>
</tr>
<tr>
<td>S UNSEALLED SHOULDERS</td>
</tr>
</tbody>
</table>
```

Fig. 6 - 1969 Country Roads Board pavement thickness design curves for roads in rural areas (adapted from MacLean 1959)

At that time traffic loadings were expressed in terms of:

"the average number of commercial vehicles (CV) exceeding three tons in weight (approximately this means vehicles with dual tyres on one or more axles) which the road is expected to carry in 24 hours at some time towards the end of its life, e.g. in about 20 years time. This is the total traffic in both directions of a two lane pavement or on both carriageways of a divided highway." (CRB 1969).

Using this design commercial vehicle volume, the road was considered to be in one of seven RRL traffic categories as indicated in Table 1.
Table 1

Traffic Loading Characterisation

<table>
<thead>
<tr>
<th>Traffic Category</th>
<th>Two-Way CV/day</th>
<th>Two-Way CV/20 years</th>
<th>One-Way ESAs/20 years</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0-15</td>
<td>$7 \times 10^4$</td>
<td>$3 \times 10^4$</td>
</tr>
<tr>
<td>B</td>
<td>15-45</td>
<td>$2 \times 10^5$</td>
<td>$1 \times 10^5$</td>
</tr>
<tr>
<td>C</td>
<td>45-150</td>
<td>$7 \times 10^5$</td>
<td>$3 \times 10^5$</td>
</tr>
<tr>
<td>D</td>
<td>150-450</td>
<td>$2 \times 10^6$</td>
<td>$1 \times 10^6$</td>
</tr>
<tr>
<td>E</td>
<td>450-1500</td>
<td>$7 \times 10^6$</td>
<td>$3 \times 10^6$</td>
</tr>
<tr>
<td>F</td>
<td>1500-4500</td>
<td>$2 \times 10^7$</td>
<td>$1 \times 10^7$</td>
</tr>
<tr>
<td>G</td>
<td>&gt;4500</td>
<td>$7 \times 10^7$</td>
<td>$3 \times 10^7$</td>
</tr>
</tbody>
</table>

When the granular thickness chart was adopted by NAASRA (Fig. 2.2 of NAASRA 1979), the characterisation of traffic loading was converted from traffic categories to cumulative equivalent standard axles (ESAs) over the design period. Based on Black's (1977) explanatory notes it seems the following conversion procedure was adopted:

- The mid-range values of two-way CVs towards the end of the design period were divided by 1.5 to derive the two-way CVs on opening (the factor of 1.5 is equivalent to a compound growth rate of 3% over 10-15 years; this factor appears to have obtained from CRB Technical Bulletin 21).

- Assuming a growth rate of 3%, the two-way CVs over 20 years were determined.

- The traffic was equally divided between the two directions to estimate the one-way CVs over 20 years.

- One commercial vehicle equalled one Equivalent Standard Axle.

The cumulative ESA values so determined are given in Table 1.

It should be noted that the minimum pavement thicknesses of the NAASRA granular thickness chart (Fig. 7) are greater than those derived from the RRL (1959) curves and different from the CRB (1969) values. These minimum thicknesses only influence the thickness adopted for the unusual cases where a design subgrade CBR exceeds 15. The AUSTROADS 1992 granular thickness chart (Fig. 1) is similar to the NAASRA, except for changes to the minimum pavement thickness and the design traffic range.

A limited survey (Potter et al 1996) of experienced engineers on design reliability has indicated that pavements designed with the AUSTROADS 1992 granular thickness chart have a low probability of premature distress. There was a wide scatter of responses in the survey with an average response being that pavements designed in accordance with the chart had about a 90% probability of exceeding the design traffic. As discussed below, this design conservatism has implications for the appropriate subgrade strain criterion for use in pavement design and overlay design.
3. Subgrade Strain Criterion

A principal distress mechanism of flexible pavements is permanent deformation or rutting
induced by traffic loading. In the AUSTROADS (1992) procedures for the design of new
flexible pavements, the allowable traffic loading to a terminal severity of rutting is assessed
using the vertical compressive strain on the top of subgrade. The concept of using the subgrade
strain to estimate permanent deformation was originally developed by Dorman and Metcalf
(1965) in their analysis of the AASHO road test.

The AUSTROADS subgrade strain criterion is:

\[ N = \left[ \frac{8511}{\mu \varepsilon} \right]^{7.14} \]  

where: \( N \) is the allowable number of strain repetitions before an unacceptable level of rutting,
\( \mu \varepsilon \) is the vertical compressive strain (microstrain).

This relationship was derived (Youdale 1984) from back-analyses of 25 pavements selected
from the NAASRA granular thickness chart (Fig. 7). For each pavement the linear elastic
model CIRCL Y was used to calculate the compressive strains at the top of subgrade between
the dual wheels of a Standard Axle load. The following procedures were used in the modelling:

- The base thickness was made equal to 150 mm except where the total pavement thickness
  was less than 200 mm, in which case the base thickness was reduced to 100 mm. The base
  vertical modulus used was 350 MPa.
Sub-base layers were subdivided such that layer thicknesses did not exceed 150 mm and the ratio of the moduli of any two layers was less than 2.

The Poisson's ratio of all the granular pavement layers was 0.35.

The subgrade modulus (MPa) was taken as 10 times the subgrade CBR and the Poisson's ratio was assumed to be 0.45.

Bases, subbases and subgrades were considered as cross-anisotropic, with the vertical modulus being twice the horizontal modulus. This anisotropy was regarded (Potter and Donald 1985) as a device to compensate for the absence of a lateral stress dependent mechanism for elastic modulus.

The Standard Axle loading consisted two 110 mm radii circular loads separated by 330 mm centre to centre, with a tyre pressure of 550 kPa.

The calculated subgrade strains were plotted (Fig. 8) against the design traffic loadings for each granular thickness and subgrade CBR determined from the NAASRA granular thickness chart.

The results for pavements with a CBR of 20 were not consistent with the other results. This was not considered to be of great importance (Youdale 1984) because the correlation between CBR and modulus is questionable at high CBR values. In addition, it was suggested that subgrades with high CBRs would generally have low plasticity and hence would not tend to deform plastically.

The calculated subgrade strains were plotted (Fig. 8) against the design traffic loadings for each granular thickness and subgrade CBR determined from the NAASRA granular thickness chart.

For the above reasons a linear regression analysis was carried out by Youdale on the results excluding the pavements with subgrade CBR of 20 and the following relationship was obtained:
\[ \log \mu e = 3.93 - 0.14 \log N_{ESA} \] (2)

where: \( N_{ESA} \) is the allowable number of Equivalent Standard Axles of loading before an unacceptable level of rutting, and

\( \mu e \) is the vertical compressive strain under a standard axle (microstrain).

By rearranging the equation 2, the AUSTROADS subgrade strain relationship (eqn (1)) was obtained.

The above regression was obtained using \( \log N_{ESA} \) as the dependent variable and \( \log \mu e \) as the independent variable. Such a regression is appropriate when subgrade strain is to be predicted from the allowable repetitions of loading. However, in practice the AUSTROADS subgrade strain criterion is also used to determine allowable load repetitions from the calculated subgrade strain. In this case a more appropriate form of linear regression analysis is one where \( \log \mu e \) is the dependent variable and \( \log N_{ESA} \) is the independent variable. Conducting this analysis on the original data yielded the following relationship:

\[ N_{ESA} = \left[ \frac{11290}{\mu e} \right]^{6.2} \] (3)

However, given that only one relationship should be used regardless of which parameter is being predicted, the most appropriate relationship would be the following relationship which bisects eqns (2) and (3):

\[ N_{ESA} = \left[ \frac{9657}{\mu e} \right]^{6.68} \] (4)

The procedures used by Youdale for calculating the subgrade strains differ in the following ways from those subsequently adopted in the AUSTROADS (1992) Pavement Design Guide:

- The manner in which the moduli for the granular layers was estimated. In the AUSTROADS Guide the total thickness of granular material is subdivided such that layer thicknesses are in the range 50-150 mm and the ratio of the moduli of adjacent layers is less than two. As discussed above, Youdale used a different procedure.
- The subgrade strains between the dual wheel loads were calculated by Youdale rather than the maxima of the strains between and under the wheels.
- The tyre pressure used by Youdale was 550 kPa. It is common practice now to use a pressure of 700 kPa.

As part of this project the subgrade strains were recalculated using the current AUSTROADS procedures for a wide range of pavements selected from:

- the AUSTROADS granular thickness chart (Fig. 8.4 of AUSTROADS (1992)) for traffic loading greater than \( 10^5 \) ESAs and
- the granular thickness chart proposed for lightly-trafficked roads based on a 90% confidence level (AUSTROADS 1996), for traffic loadings between \( 10^3 \) and \( 10^5 \) ESAs. The 90% confidence level chart was selected rather than the 80% and 95% confidence level charts because the granular thicknesses were similar to those of the AUSTROADS Fig. 8.4 at a traffic loading of \( 10^5 \) ESAs.
The data are plotted in Fig 9. It can be seen that the data for a subgrade CBR of 20 are no longer anomalous.

\[ N_{ESA} = \left( \frac{8,012}{\mu e} \right)^{7.39} \]  

(5)

As can be seen from Fig. 9, eqn (5) fits the data only slightly better than the AUSTROADS relationship (eqn (1)). Consequently, no change to the existing AUSTROADS (1992) relationship is proposed on the basis of this analysis, except that now it is also applicable to lightly-trafficked roads.

Another important point is that all of the above relationships are in terms of Equivalent Standard Axles of loading rather than Standard Axles of loading. When traffic loading is expressed in terms of ESAs, road wear of an individual axle load is proportional to the fourth power of the load magnitude. However, eqn (1) indicates that permanent deformation damage varies with 7.14 power of strain. Using the 7.14 power rather than the fourth power results in a greater number of allowable Standard Axles of loading for a particular strain level. Assuming the distribution of axle loads is in accordance with Table I-1 Rural of the AUSTROADS Pavement Design Guide the number of Standard Axles \( N_{SA} \) is 1.89 times the number of ESAs. Using this factor in eqn (1) results in the following relationship:

\[ N_{SA} = \left( \frac{9,300}{\mu e} \right)^{7.14} \]  

(6)
As discussed in Section 2, the design traffic loadings in ESAs, determined from the granular thickness chart, are conservative. Consequently, the mean design traffic loading for a given subgrade strain exceeds that indicated by eqn (6).

4. Design Deflections

4.1 Origins of AUSTROADS Design Deflections

The AUSTROADS (1992) overlay design procedures include a design deflection curve (Curve 1, Fig. 10) for use in controlling "the rate of permanent deformation in the pavement and subgrade and may be used for all pavements regardless of surfacing types". Contrary to the mechanistic approach to overlay design (AUSTROADS 1994), Curve 1 applies to all pavement types; different curves are not provided for varying subgrade strengths or pavement thicknesses. Consequently, it was of interest to trace the origins of the AUSTROADS design deflection criterion.

![Design Deflections Diagram](image)

**Fig. 10 - AUSTROADS design deflection criteria (Fig. 10.3 of AUSTROADS 1992)**

AUSTROADS Curve 1 is the same as NAASRA (1979) Curve 1 (Fig. 11), except that the AUSTROADS Curve 1 is only applicable to pavements with unbound bases with thin asphalt surfacings. The other principal difference is that the AUSTROADS Curve 1 is applicable to traffic loadings up to $10^8$ ESAs, whereas the upper limit of NAASRA Curve 1 is $3 \times 10^7$ ESAs.

Note that in adopting the NAASRA Curve 1 for all pavements types, it seems that the NAASRA Working Group revising the 1979 NAASRA Guide reasoned that NAASRA Curves 2, 3 and 4 were provided to control fatigue of asphalt and cemented materials. Consequently, the Group adopted NAASRA Curve 1 to control rutting for all pavement types. NAASRA Curve 4 was retained to inhibit cracking in cemented bases and a new curvature function was adopted to control asphalt fatigue.
According to Black (1977) the 1979 NAASRA design deflection curves:

"were adopted from the information available and appropriate to Australian conditions. In particular Curve B (Curve 2) was derived from the deflections levels adopted by the 40th NAASRA (1969) for unbound base to be surfaced with bituminous concrete, and Curves A, C and D (Curves 1, 3 and 4) were derived from experience in CRB VIC and DMR NSW."

The basis of the 1979 NAASRA Curve 1 seems to be the design deflections agreed at the 40th meeting of NAASRA in 1969. These design deflections are given in Table 2 and Fig. 12. Note that the 1969 NAASRA design deflections assumed that pavements have sprayed seal surfaces for traffic up to 450 commercial vehicles per day and asphalt surfaces for higher traffic volumes.

The minutes of the 40th NAASRA meeting give some background to the origin of the 1969 NAASRA design deflections:

"The values for tolerable deflections of pavement are based on:

(a) Consideration of recommended values of tolerable deflections for economic performance of pavements reported from extensive deflections surveys in Europe and Northern America.

(b) Limited surveys conducted by ARRB in Victoria and Tasmania correlated with later performance.

(c) Experience of State Road Authorities with deflection surveys."

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Table 2
Early Design Deflections for Granular Pavements with a Sprayed Seal Surfacing

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<td>Allowable Deflection (mm)</td>
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* Design deflections for asphalt surfaced pavements.

Fig. 12 - Comparison of design deflections

The design deflections adopted for the sprayed seal pavements appear to be largely based on research conducted by Scala (1965). As discussed below, Scala conducted field measurements of 200 Victorian pavements with sprayed seal surfaces. Using the high deflection data (>0.8 mm), and taking account of the findings of the AASHO Road Test, Scala proposed allowable deflections which were identical to those later adopted by NAASRA in 1969 (see Table 2) for traffic loadings up to $1 \times 10^6$ ESAs. These are the highest expected deflections over all subgrade CBRs and granular thicknesses, as discussed in Section 4.2.

The origins of the 1969 NAASRA design deflections for higher traffic loadings (asphalt surfaced pavements) are less clear. The design deflection for $3 \times 10^6$ ESAs is the same as that recommended by Scala (1965). It is noted that all three design deflections are similar to the Californian design deflection for a granular pavement with a 50 mm thick asphalt surfacing.
(Zube and Forsyth 1966). These Californian design deflections were based on the performance of heavily-trafficked (about $10^7$ ESAs) roads extrapolated to other loadings using the slope of a laboratory asphalt fatigue line. It seems, then, that these Californian design deflections were related to asphalt fatigue rather than to rutting.

In 1969 the CRB adopted separate design deflections for sprayed seal surfaced and asphalt surfaced pavements (Currie 1969). The sprayed seal surfaced values for traffic loadings exceeding $10^6$ ESAs seem to have been estimated by adding 0.25 mm (0.01 inch) to the 1969 NAASRA asphalt surfaced values. This adjustment factor was considered to be somewhat conservative based on CRB experience but in line with specifications used in the United States (Currie 1969). For traffic loadings of $10^6$ ESAs and less, the CRB (1969) design deflections were slightly less than the NAASRA (1969) values.

In 1975, the CRB issued Technical Bulletin 29 "Pavement Deflection Testing Using the Benkelman Beam". As seen in Table 2, these design deflections for sprayed seal surfaced pavements were similar to the CRB (1969) values except for some rounding off in the metrication process.

In summary, for traffic loading below about $10^6$ ESAs the current AUSTROADS design deflections appear to have been derived from research conducted by Scala taking account of the findings of the AASHO Road Test. Above $10^6$ ESAs, the design deflections may have been based on the Californian design deflections for 50 mm thick asphalt pavements adjusted to estimated equivalent sprayed seal values.

### 4.2 Design Deflection Dependence on Granular Thickness and Subgrade CBR

As already mentioned, the AUSTROADS design deflection curve is applicable to all pavements irrespective of granular thickness or subgrade CBR. The concept of a single design deflection curve for all pavements is at variance with the mechanistic approach to overlay design (AUSTROADS 1994). Research conducted by Scala, on which the AUSTROADS design deflections are in part based, is discussed below in relation to this issue. In addition, the relevant findings of TRRL research are reviewed.

**Scala's Findings**

In the early 1960s, Scala derived a method of new pavement design based on deflections to complement the CBR approach to pavement design. As it is impossible to measure deflections on a proposed pavement, Scala's deflection method involved comparing the predicted deflection for the proposed pavement with the design deflection. Based on field measurements of about 200 Victorian pavements with sprayed seal surfaces, Scala (1965) developed relationships to predict Benkelman Beam deflections from subgrade strength (CBR) and granular thickness. Scala determined two such relationships:

- the line of best fit, and
- the line estimating the maximum expected deflections for any combination of granular thickness and subgrade CBR.

As stated by Scala:

"The line of best fit will give a relation to be expected for the average structural condition of the pavement material and subgrade. If the pavement materials are not compacted as well as the average condition in Victoria, which is thought to be higher than normal, then the actual deflections will be higher than expected from the (line of best fit) prediction. Further, the CBR
of a subgrade is a very variable quantity. In this investigation, the variation is both longitudinally and vertically (with depth). Neglecting any longitudinal variation the choice of the correct CBR rating for a subgrade which varies with depth is difficult; this variation must be reflected in total deflection.

To allow for these conditions, or to cover any risk of failure it is probably preferable to fit the envelop or the line giving the maximum expected deflection for any combination of CBR and granular thickness.

**Mean Deflection Relationship**

The following line of best fit was obtained by Scala using multiple regression analysis on all the data:

\[
\log(d) = -(0.88 + 0.0165t) - 0.5 \log(CBR)
\]

where

- \(d\) = Benkelman Beam maximum deflection (inches),
- \(t\) = thickness of granular material (inches), and
- \(CBR\) = subgrade CBR

This relationship, which is plotted in metric units in Fig. 13, gives the expected or mean deflection for a given granular thickness and subgrade CBR.

![Mean Deflection Relationship Graph](image)

**Fig. 13 - Measured dependence of mean Benkelman Beam deflection on subgrade CBR for various granular thicknesses (Scala 1965)**

"High" Deflection Relationship

As stated above Scala also developed a relationship to predict the maximum expected deflection for any combinations of subgrade CBR and granular thickness. Pavements with low deflections (< 0.8 mm) were excluded from this analysis. This resulted in less than half the original 200 pavements available for analysis. In order to increase the sample size use was made of the results from an additional 60 sites from another investigation in which residential streets were tested. The following equation was obtained from regression analysis of the combined data set:
\[
\log(d) = -0.34(1+0.1t) - 0.7 \log(CBR) \tag{8}
\]

where \(d\) = Benkelman Beam maximum deflection (inches),
\(t\) = thickness of granular material (inches), and
\(CBR\) = subgrade CBR

This relationship, which is plotted in metric units in Fig. 14, enabled the designer of a new pavement to determine the granular thickness required in order that, when the pavement was constructed,

"the structure is 95% certain, after testing by the normal beam procedure, to have a deflection less than the specified standard of deflection." (Scala 1965)

Note that for a given design deflection and subgrade CBR, higher thicknesses of granular materials were required by the "high" deflection relationship than by the mean deflection relationship. As such the use of the "high" deflection relationship was considered to be a conservative approach to the design of new pavements.

![Fig. 14 - Measured dependence of "high" Benkelman Beam deflection on subgrade CBR for various granular thicknesses (Scala 1965)](image)

**Design Deflection**

Scala proposed deflection criterion using:

- the "high" Benkelman Beam deflection dependence on granular thickness and subgrade CBR (eqn. (8)), and
- the relationship between design traffic loading for a given subgrade CBR and granular thickness, obtained from the 1959 RRL granular thickness chart (Fig. 5) and a conversion between traffic category and design ESAs (see Table 1).

The resulting relationships between "high" deflection and design traffic loading are given in Fig. 15 for various thicknesses of granular material and Fig. 16 for various subgrade CBRs. Using this data and in view of the AASHO road test findings, Scala proposed the design deflections given in Table 2. As mentioned above, for traffic loading less than \(10^6\) ESAs
Scala's design deflections formed the basis of the 1992 AUSTROADS design deflections. These AUSTROADS (1992) design deflections are also shown in Fig. 15 and Fig. 16. It is apparent that the AUSTROADS design deflections are the maximum expected deflections for a given design traffic over all subgrade CBRs and granular thicknesses.

It should be noted that Scala used the "high" deflection relationship rather than the mean deflection relationship. When the mean deflection relationship (eqn (7)) is used to derive design deflection curves, the relationship between deflection and design traffic loading is very different, as illustrated in Fig. 17. In this case, there is a much stronger dependence of design deflection on granular thickness and the design deflections are substantially lower than those adopted by AUSTROADS for all granular thicknesses. This suggests that the AUSTROADS (1992) design deflection curve overestimates the average allowable design traffic loadings. However, it should be noted that these overestimates of average loading in part offset the conservatism of the relationship between deflection and allowable loadings resulting from the use of the conservative granular thickness chart (Fig. 1) in their deviation.

Fig. 15 - Relationship between "high" Benkelman Beam deflection and design traffic loading for various granular thicknesses
The above discussion suggests that Scala's data, on which the AUSTROADS design deflection criterion is based, does not support the use of a single design deflection curve for all granular thicknesses and subgrade CBRs. The AUSTROADS design deflections are the maximum expected deflections over all granular thicknesses and subgrade CBRs.

TRRL Research Findings

The NAASRA design deflection criterion (Curve 1 NAASRA 1979) was also re-examined by the NAASRA Working Group Revising the NAASRA Interim Guide to Pavement Thickness Design. Using the mechanistic design procedures, Youdale (1984) derived a similar dependence on design deflection on granular thickness to that observed in the derivation of
ASMOL, the interim AUSTROADS Simplified Method of Overlay design. As there was no dependence on granular thickness in NAASRA Curve 1 design deflection criterion used at that time (the same as AUSTROADS Curve 1), the Working Group (Anderson 1984) reviewed some of the work of Lister (1973) of TRRL to assess whether the thickness dependence was supported by British performance data.

Lister used data obtained in the Alconbury Hill experiment to investigate the relationship between deflection, structural pavement parameters and performance. Test sections with rolled asphalt bases, granular subbases and relatively uniform subgrades provided data which enabled the influence of asphalt base and sand subbase on deflection to be estimated. *Fig. 18* was derived by Lister after correcting the deflections at individual points to take account of differences in subgrade CBR from the mean value of 4.5%.

The results indicate that the same design deflection curve can be applied to a range of sand subbase thicknesses (125-500 mm). However, this is not necessarily in disagreement with either the observed thickness dependence in mechanistically-based overlay design procedures or the above analysis of Scala’s data. Lister’s results only apply to one subgrade strength (CBR = 4.5). As seen from *Fig. 16*, Scala’s data indicates a single design curve can be used for a single subgrade strength. Consequently, contrary to the conclusions of the NAASRA Working Group, Lister’s data does not support the use of a single design deflection curve for all granular thicknesses and subgrade CBRs.

---

*Fig. 18* - Relationship between deflection, critical life and thickness for pavements with rolled asphalt bases at Alconbury Hill (Fig. 31 of Lister 1973)
4.3 Comparison with ASMOL

It was of interest to compare Scala's data with that obtained by ASMOL, the interim AUSTROADS Simplified Method of Overlay design.

Shown in Fig. 19 is the ASMOL relationship between Benkelman Beam deflection and design traffic loading. There is a strong dependence of design deflection on granular thickness, more consistent with the observed thickness dependence of Scala's mean deflection relationship (Fig. 17) than his "high" deflection relationship (Fig. 15). The fact that the relationship between "high" deflection and design traffic loading is substantially less dependent on granular thickness and subgrade CBR may be related to the limited range of subgrade CBR and granular thicknesses used in the "deflection limited" data set. Unfortunately this cannot be verified as the data used by Scala was not published.

![ASMOL relationship between Benkelman Beam deflection and design traffic loading](image)

**Fig. 19 - ASMOL relationship between Benkelman Beam deflection and mean design traffic loading for unbound granular pavements**

5. Summary

The origins of several cornerstones of the AUSTROADS pavement design and overlay procedures have been reviewed. The findings of this review are as follows:

Unbound Granular Thickness Chart

The design chart for granular pavements with thin bituminous surfacing, currently Fig. 8.4 of AUSTROADS (1992) evolved from the 1940s Californian State Highway design thickness curves. These curves were refined and extended by the U.K. Road Research Laboratory. With minor modifications these design curves have been widely used by Australian State Road Authorities for over 30 years. There has been no research undertaken to verify these design curves. A limited survey of experienced engineers indicated that pavements designed using these chart have about a 90% chance of exceeding the design traffic loading.
Subgrade Strain Criterion

The AUSTROADS subgrade strain criterion is used to control permanent deformation in the mechanistic design of new pavements and in the mechanistic overlay procedures.

The subgrade strain criterion was derived from the above granular thickness chart. Pavement modelling undertaken as part of this project indicates that its use can now be extended to lightly trafficked roads.

As the criterion is based on the granular thickness chart, and since this chart is conservative, the criterion underestimates average allowable traffic loading. This may lead to overestimates of average overlay thicknesses when the criterion is used in the interim AUSTROADS mechanistically-based overlay procedures. The appropriate strain criterion for use in overlay design needs to be reconsidered when design reliability procedures are developed for overlay design.

Amendment to the criterion is required to express design traffic in terms of Standard Axles rather than Equivalent Standard Axles.

Design Deflections

The AUSTROADS (1992) design deflections for traffic loading less than $10^6$ ESAs was derived from research undertaken by Scala in the early 1960s. These design deflections were based on field measurements of the dependence of Benkelman Beam deflections on granular thickness and subgrade CBR and the relationship between pavement composition and design traffic loading from the granular thickness chart. The Benkelman Beam data used to derive the relationship was not the mean deflection data for a given granular thickness and subgrade CBR. As the low deflection data (<0.8 mm) was excluded from the analysis, the design deflection criterion relates to the maximum expected deflection for a given design traffic loading rather than the mean deflection. Consequently, this criterion is applicable to pavements with lower strength (CBR < 8) subgrades. There are substantial doubts about the appropriateness of the 1992 AUSTROADS design deflections for assessing the overlay requirements of pavements with subgrade CBR's exceeding 8. For such pavements there are large differences between the overlay thicknesses determined using the AUSTROADS (1992) and mechanistically-based AUSTROADS (1994) overlay design procedures.

For traffic loadings exceeding $10^6$ ESAs the origins of the AUSTROADS (1992) design deflections are less clear. From the limited information available it seems that they were estimated from Californian design deflections for asphalt-surfaced pavements. There appears to be no substantial published data to support the use of these design deflections for traffic loadings exceeding $10^6$ ESAs. They have, however, been used in Australia for overlay design for over 20 years without any obvious problems.

In terms of the implications of this to an appropriate overlay design procedure for adoption by AUSTROADS, such a decision is made more difficult by the lack of performance data in support of either the mechanistically-based overlay procedures or the AUSTROADS (1992) design deflection criterion. Research on the performance of pavements is needed to clarify the issue.
6. Recommendations

Due to the findings of a previous project on the validity of ASMOL for adoption by AUSTROADS, ASMOL is currently being extended to cover a wider range of pavements configurations.

After the ASMOL procedures have been extended, it is recommended that the differences in overlay thicknesses of the existing overlay design procedures and ASMOL should be re-examined to enable a recommendation as to an appropriate overlay design procedure for adoption by AUSTROADS. In this evaluation and recommendation, cognisance should be taken of the origins of the two overlay procedures.

Additional research on pavement performance would assist in making this recommendation.

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