RECENT DEVELOPMENTS IN AUSTRALIA IN CONCRETE PAVEMENT THICKNESS DESIGN

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Abstract:

Over the last few years, Australian engineers have continued to review and refine the Austroads Rigid Pavement thickness design model to increase the design reliability and make the design process more efficient to allow more pavement configurations to be analysed and assess for life cycle costing.

Australia adopted the PCA thickness design method in 1992 with the addition of a new axle group making the total of four axle groups to analyse for fatigue and erosion distress. Similar to other International pavement engineers, Australia has recognised the shortcomings of the PCA method and had made small adjustments in the thickness design approach to ensure the design allowed for the construction practice and materials in Australia.

The PCA thickness design method with its tables and nomographs made it a cumbersome task to design a pavement with four or six axle groups of data. All the PCA charts and tables have been converted to algorithms in metric units. The development of these algorithms is outlined in the paper.

1 INTRODUCTION

Over the last few years, Australian engineers have continued to increase the design reliability and efficiency of the Austroads rigid pavement thickness design model to allow more pavement configurations to be analysed and assessed for life cycle costing to select the best option for the project. Australia adopted the PCA thickness design method (1) for rigid pavements in the 1992 Austroads Pavement Structural Design Guide (2). Like other international pavement engineers, many Australian engineers have recognised the shortcomings of the PCA method and adjusted the thickness design approach to ensure current Australian construction practices and materials are taken into consideration.

Austroads is currently revising the 1992 version of the Pavement Design Guide (2) with a view to publishing the new edition in late 2001. In updating the chapter on concrete pavements, Australian engineers have had the opportunity to incorporate recent research studies on the performance of existing heavy-duty concrete pavements.

Figure 1 lists the subtle differences in pavement terminology between Australian and American practices. One other recent difference is that the PCA refers to a Commercial Vehicle Axle Group (CVAG) and Australia refers to this as the Heavy Vehicle Axle Group (HVAG) to closely represent “heavy” vehicles (ie over 3 tonnes) used in the analysis.

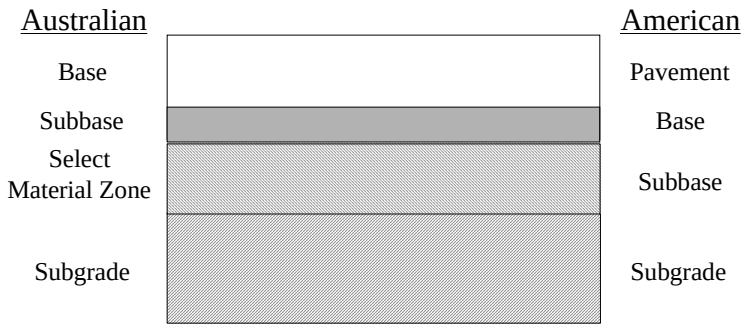

Figure 1 Description of Australian and American terminology to describe pavement configurations.

This paper looks at design developments in Australia over the last four years in terms of:

- ☐ Identifying shortcomings in the 1992 Austroads design approach.
- ☐ A move from using charts and tables to computer computations.
- ☐ Handling construction tolerances

2 DESIGN EFFICIENCY

The original work of the PCA design method (3,4) was expressed as an algorithm, as shown in Equations 1 and 2, where incremental axle loads from various axle load groups, the modulus of rupture of concrete, and equivalent stresses; were the basic inputs used to establish the allowable repetitions of axle loading to avoid concrete cracking from fatigue. These algorithms were finally presented as a series of nomographs and tables of equivalent stress and erosion factors, in the 1984 PCA Thickness Design Method (1). Whilst this method was satisfactory before the use of desktop computers, the use of charts and tables resulted in a slow and tedious design procedure for many engineers.

$$\text{Log}_{10}\, N = (0.9718 - S_r) / 0.0828 \qquad \text{(Equation 1)}$$

Where N = Number of allowable axle load repetitions
S_r = Stress ratio factor

$$S_r = \left(\frac{S_e}{M_r}\right) \text{x} \left(\frac{P}{18}\right)^{0.94} \qquad \text{(Equation 2)}$$

Where M_r = modulus of rupture of the concrete base slab
P = (single) axle load (kips)
S_e = equivalent stress

In 1996 the author published (5) a series of algorithms converted from kips to SI units for the determination of the allowable load repetitions from fatigue and erosion distress mechanisms. These algorithms still relied on inputs from various tables which slowed the computation process. In 1998, ARRB Transport Research (6) generated one algorithm to describe the

equivalent stress and erosion factors for the four common axle groups for various pavement configurations. This algorithm allows for the full thickness design process to operate more efficiently plus with modern computers the cumulative distress calculations can be easily made. These algorithms are listed in Appendix A and elements of their origin are detailed in the following sections.

The benefit of using algorithms was to allow designers to write computer programs in Visual Basic to quickly examine various pavement configurations to minimise construction and life cycle costs. The use of algorithms to establish the concrete base thickness for various pavement configurations is therefore an efficient design process.

Many pavement engineers in Australia have a desire to produce an all encompassing model to analyse concrete pavements for both wheel and thermal loading, vehicle speed and composition, and for vertical bending and lateral stresses. But before engineers embark on this challenge, other issues need to be addressed in the thickness design model, such as:

- Axle group configurations for modern heavy vehicles.
- Making the design model assumptions agree with appropriate performance-based specification limits.
- Refinement of the input to allow for the effects of temperature curling.
- Effects of settlement or movement of support.

Only the first two issues are discussed in this paper.

3 TRAFFIC LOADING

3.1 Axle group configurations

The 1984 PCA thickness design approach considered three axle groups[1] and work by CTL (7) in 1990 expanded this to four axle groups (see Figure 2). Over the last ten years, road authorities in Australia have expanded the number of high-speed weigh-in-motion (WIM) sites to over 170 (8). The data collected from these sites improves the traffic loading prediction and provides a more confident final design thickness. The types and mass of various axle groups at these sites are recorded and later analysed to form a traffic distribution suitable for design. Table 2 shows a typical urban distribution with six axle group configurations as described in Figure 3.

[1] The initial three axle groups were SD, TAD and TRD.

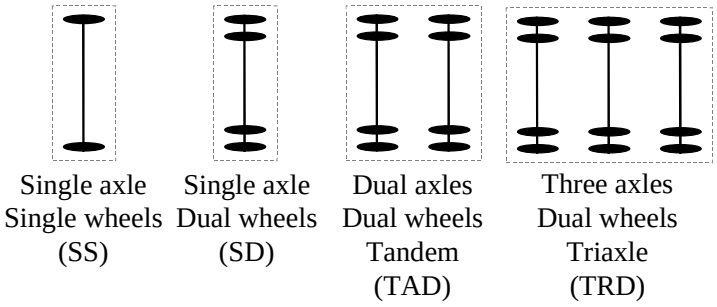

Figure 2 Plan of the four most common commercial vehicle axle groups in Australia.

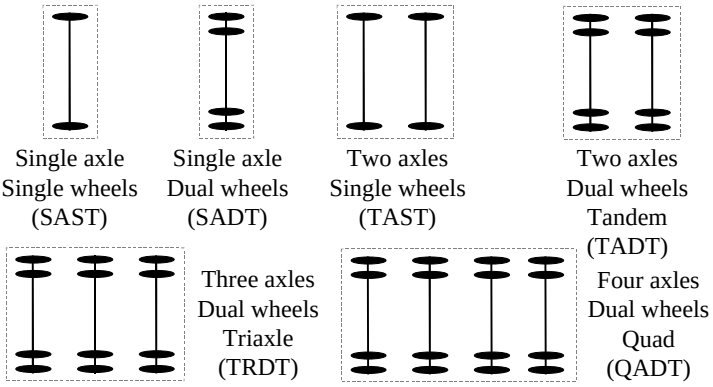

Figure 3 Plan of the six heavy vehicle axle groups in Australia.

Two new axle groups, as shown in Figure 2, are the Two Axle Single Tyre (TAST) or twin steer axle group and the quadruple axle group (QUADs), and are being introduced in the revised Austroads Design Guide. Some information is required in the new guide for designers to consider the impact on the thickness design procedure. In the past, engineers who had traffic composition data followed this approach:

- *convert TAST axle* groups to SAST axle groups on the basis that a TAST is equivalent to two SAST, each with half the load, and
- convert QADT axle *groups to TADT axle groups on the basis that a QADT is equivalent to two TADT, each with half the load.*

It is emphasised that although these two axle groups do not represent a high proportion of the total axle group, they do produce distress in concrete pavements. As a results of these new axle groups and a lack of research funds, a conservative approach using the data from the existing four axle groups has been used.

Table 1 lists three approaches considered in the design thickness sensitivity analysis. More detailed information on Option 3 can be deduced from the equations and tables listed in Appendix A.

Table 1 Three approaches considered by engineers for the two new axle groups.

Option	Proposed adaptation to design model
1	Convert TAST to two SADT, and a QADT equates to two TADTs.
2	Equate TAST as TADT with 4 tyres and QUAD as TRDT with 16 tyres.
3	Equate TAST as SADT for fatigue and TADT for erosion distress modes Equate QADT as TRDT for fatigue allowing 16 tyres and 12 tyres for erosion distress mode

Option 1 was too simplistic an approach and misleading, and an analysis of Option 2 found that 3% of the TAST axle group within the traffic distribution comprised over 70% of the fatigue damage. This observation of a very dominant axle group causing major distress is inconsistent with other axle groups. Option 3 was considered less conservative than Option 2 and should any unusual thin slabs be derived from the approach, a table outlining minimum slab thickness for various levels of design traffic and concrete pavement type, would provide a safe approach.

3.2 Traffic distribution

The desire for road transport to be efficient has led to many new configurations of heavy vehicles. Figure 3 show examples of new transport vehicles in Australia common to both the urban and rural road network. Australia also has long road trains that thunder along rural roads on predominantly flexible pavements (see Figure 3).

The current legal limits for the basic six axle groups are listed in Table 3 together with recorded levels of axle overloads from various WIM sites. A typical axle group traffic distribution from a WIM site is listed in Table 2 along with recorded overloaded vehicles. It is these overloaded vehicles which cause most damage to concrete pavements and require careful attention in the design model.

Table 2 Typical Australian urban traffic axle load distribution.
NOTE: The last row refers to the proportion of axle groups in the total traffic composition.

Axle group loads (kN)	Axle Groups (%)					
	SAST	**SADT**	**TAST**	**TADT**	**TRDT**	**QUAD**
10	0.1	1.8	0	0.2	0	0
20	3.9	5.4	0.2	0.4	0.1	0.5
30	8.8	18.9	0.3	0.7	0.4	0.7
40	11.8	20.4	0.7	2	1.6	0.9
50	29.9	16.6	2.6	3.8	3.2	1.8
60	35.7	12.4	8.8	6	4.4	2.3
70	8.8	9.4	15.6	6.5	4.6	2.3
80	0.8	6.8	16.3	6.1	3.9	2.6
90	0.1	4.6	15.1	5.6	4.3	3.4
100	0	2.3	13.6	5.7	3.5	4.3
110	0	0.9	11.3	6.3	3	4.8
120	0	0.3	7.3	6.8	3.7	3.4
130	0	0.1	4.1	7.2	3.5	2
140	0	0.1	2	8.5	3.3	1.9
150	0	0	1	10.2	4.5	1.8
160	0	0	0.5	9.8	4.8	2
170	0	0	0.3	7	5.1	2.4
180	0	0	0.2	3.9	8	3.6
190	0	0	0.1	1.8	9.1	4.7
200	0	0	0.1	0.7	8.5	5.8
210	0	0	0	0.3	8.8	8.7
220	0	0	0	0.2	5.4	8.8
230	0	0	0	0.1	2.9	7.7
240	0	0	0	0.1	1.8	5.7
250	0	0	0	0	0.7	4.3
260	0	0	0	0	0.4	2.9
270	0	0	0	0	0.2	2.7
280	0	0	0	0	0.1	1.6
290	0	0	0	0	0.1	1.2
300	0	0	0	0	0.1	1.5
310	0	0	0	0	0	1.0
320	0	0	0	0	0	1.0
330	0	0	0	0	0	0.5
340	0	0	0	0	0	0.2
350	0	0	0	0	0	0.3
360	0	0	0	0	0	0.2
370	0	0	0	0	0	0.2
380	0	0	0	0	0	0.1
390	0	0	0	0	0	0
400	0	0	0	0	0	0.2
Group distribution	**35.5%**	**10.3%**	**0.7%**	**31.1%**	**22.4%**	**< 0.1%**



Figure 3 A view of a B-Double (above) and a road train commonly used in Northern Australia.

Table 3 Legal axle limits in Australia and overloads recorded from WIM data for various axle groups.

Axle Group	Legal Limit (t)	Overloads recorded (t)	Comments
SAST	6	12	
SADT	9 (10)	19	Buses are permitted to carry 10t with approved air suspension
TAST	10 (11)	20	11t for load sharing suspension
TADT	16.5	34	
TRDT	20	36	
QUAD	20	40	

3.3 Super single tyres

Australian engineers have no firm data on the use of super-single tyres[2], however, it is known that only a small proportion of the heavy vehicle fleet in Australia is configured with super-single tyres (refer to Figure 4). Whilst it is well known that these tyres are more damaging on flexible pavements than two narrow tyres, the use of super-singles is currently limited as they are not cost effective under the proposed new mass limit regulations.

Work by Ioannides (8) in the 1990s showed that super-singles increased edge loading by about 10% compared to dual tyres. Given the very small number of vehicles using super-singles and that many of our heavy duty concrete roads are designed with erosion distress as the dominant failure mode, the current approach in Australia is to consider that any super-single configuration is equivalent to two narrow tyres in the slab thickness analysis.

3.4 Loading rate and repetition

In 1995, research work by the RTA[3] in Australia uncovered a possible discrepancy in the PCA thickness design approach related to the number of allowable repetitions to failure for fatigue distress from tandem and triaxle groups. Upon reviewing a publication by Tayabji, Ball and Okamoto (9), it appeared that the PCA design method had not reduced the allowable repetitions from fatigue by 2 and 3 times for tandem and triaxle loads respectively, due to the peak stress loading curves as shown in Figure 5. Tayabji (10) confirmed this discrepancy in 1995 but

[2] Super single tyres is where the tread section width exceeds 375 mm.
[3] RTA – Roads & Traffic Authority, NSW

Packard (11) was not convinced that the reduction in allowable repetitions was necessary due to the high strain rate of tyre loading, and claiming that these peaks were already taken into consideration in the algorithms.



Figure 4 View of dual tyre configuration (left) and super-single tyre.

The RTA (12) uses the following approach[4] for the fatigue distress mode calculations:

Tandems: For a given load, the expected repetitions are given by:

Design traffic in heavy vehicle axle groups **x** Proportion of axle groups represented by tandems **x** Proportion of tandems at that load level **x** 2

Triaxles: For a given load, the expected repetitions are given by:

Design traffic in heavy vehicle axle groups **x** Proportion of axle groups represented by triaxles **x** Proportion of triaxles at that load level **x** 3

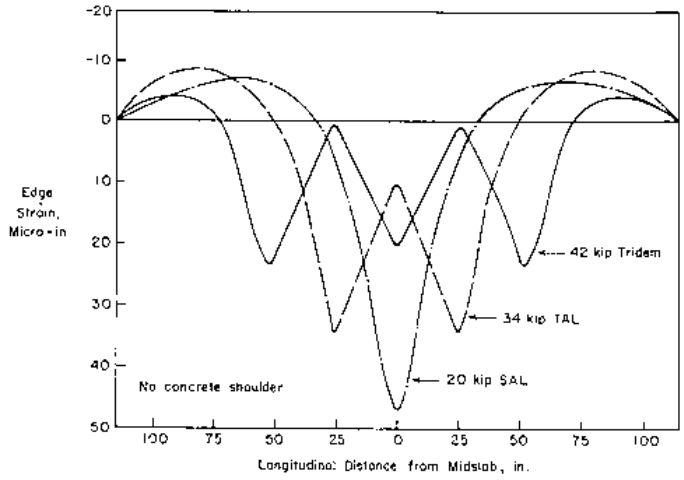

Figure 5 Calculated edge strain profiles showing multiple stress peaks from tandems (TAL) and triaxles (Tridem). SAL refers to single axle load. (9)

[4] These calculations have not been adopted by Austroads in the existing and proposed 2001 Guide.

3.5 Maximum axle load

One of the challenges with the use of algorithms is to provide appropriate limits to the input values so that calculations are not carried out beyond the design envelope and its assumptions. In the PCA design method, the maximum input load for the fatigue and erosion distress analysis is 260 kN and 520 kN for single axle and tandem axle load groups respectively. These upper limits to the nomographs equate to a maximum 65 kN load per tyre after allowance for a load safety factor. This tyre load limit may represent a 10 tonne single axle load with single tyres with a LSF of 1.3, not an unreasonable axle load configuration given the number of overloaded heavy vehicles using the Australian road network.

There appears to be no clear rationale for the axle load input limits presented in the PCA nomographs (1). Upon examination of the behaviour of the fatigue distress algorithms beyond the 65 kN limit (refer to Figure 6), the curves do not appear to "misbehave". More work is required to assess the traffic load limits in the design model and the revised Austroads design guide therefore expresses caution at tyre loads exceeding 65 kN.

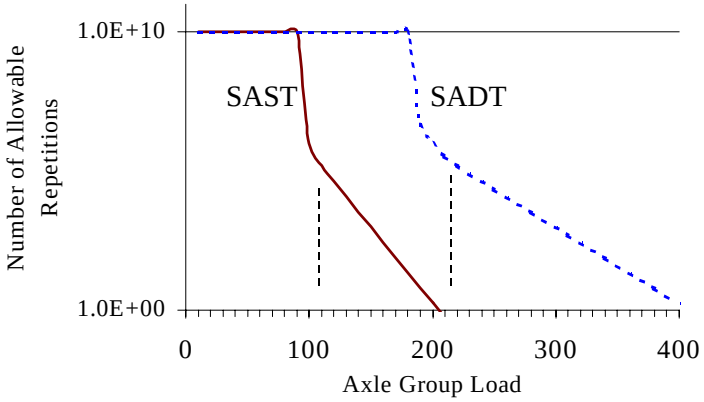

Figure 6 The allowable repetitions to failure for fatigue from SAST and SADT axle group loads.
The two vertical dashed lines represent the 65 kN limit for each axle group for a LSF=1.2.

4 DESIGN MODEL VERSUS CONSTRUCTION

4.1 Concrete properties

Insitu concrete strength in the pavement will vary depending on variations in materials (aggregates, cement etc), mixing, age of concrete and compactive effort. Test methods and the preparation of samples also leads to variations. As noted by Rollings (17) "concrete strength is not an absolute, intrinsic property, but instead numerical results of tests on any given concrete". Even if one considers the desired concrete properties in relative terms, there needs to be a suitable reference from the design model to the specification and the desired performance.

The PCA thickness design model allows for the average concrete flexural strength with a coefficient of variation of 15%. In the 1980s in the USA, this represented a fair-to-good quality

control. In Australia the standard deviation and coefficient of variation on several concrete properties from major road projects over the last ten years is summarised in Table 4. It may be concluded that appropriate Australian tolerances in the specifications along with financial deductions are producing construction variations within the design model assumptions.

Table 4 Typical properties of concrete manufactured for various plain jointed, heavily trafficked, concrete roads in NSW (13).

Concrete properties	Specified target	Standard Deviation	Coefficient of Variation
Compressive strength	Min. 35 MPa	1.5 - 3.6	3.0 - 7.3
Flexural strength	Min. 4.5 MPa	0.26 – 0.42	5.5 – 7.1
Cylinder unit mass	Not specified	10 - 18	0.44 – 0.70
Insitu concrete unit mass	Not specified	14 - 21	0.61 – 0.90

In the 1992 Austroads Guide, the 28-day average strength, as noted in the PCA approach, was replaced with the characteristic strength (i.e. 95 percentile of strength in any sample). This approach allowed pre-mix companies to supply concrete for roads with their known mix designs and was consistent with concrete specifications for buildings. The ratio between the characteristic and average strength is 1.18 representing a significant shift in the design input and is therefore appropriate for the mean strength as intended in the assumptions in the original model.

The PCA approach to specifying flexural strength is no different to the Austroads model. However, the 28-day compressive strength is used as the main performance criterion due to the difficultly of transporting flexural beams long distances to regional laboratories, insufficient numbers of moulds in regional centres, and the variability in flexural beam results for identical mixes. The Australian experience with flexural beam testing is well documented by Hodgkinson (14). As the relationship between compressive and flexural strength is verified during the trial mixing and assessment[5], no further discussion on the merits of flexural versus compressive strength testing is contained in this paper.

Most Australian road specifications require the contractor to meet the minimum compressive strength and unit mass, expressed as a percentage of the target value. Below these values, financial deductions are incurred and possibly rejection of the concrete. The contractor is required under a QA contract to evaluate the strength by crushing cylinders under standard curing conditions for each designated lot, up to 50 m^3 of concrete paving. This is expressed diagrammatically in Figure 7.

It is well recognised that concrete gains strength with age and finally plateaus at a maximum value. Figure 8 shows the relationship used in the design model such that at 90 days the strength is 10% higher than the 28-day strength, and at 5 years it is 22% higher than the 28-day value. Packard and Tayabji (4) note that the incremental strength gain yields equivalent strength factors of 1.235 and 1.278 for 20-year and 40-year design periods respectively.

[5] This is a common approach for main road specifications in Australia.

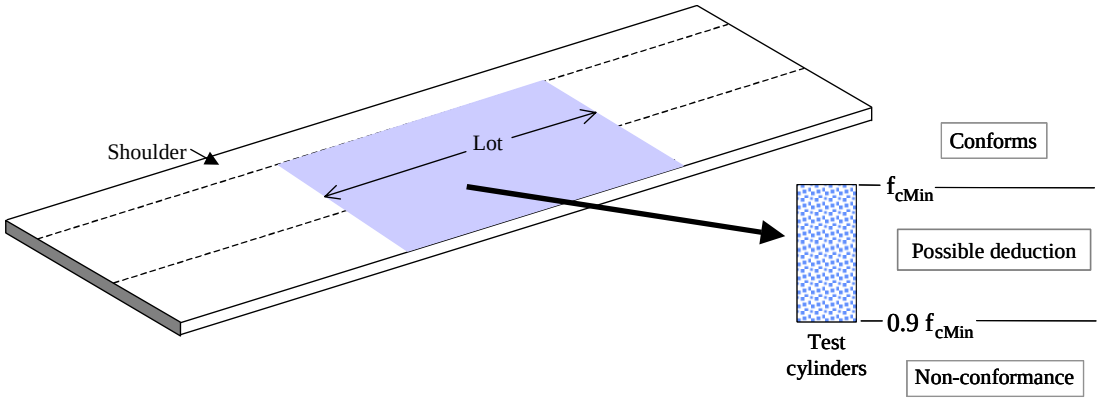

Figure 7 The RTA specification requires the contractor to target a minimum concrete compressive strength.

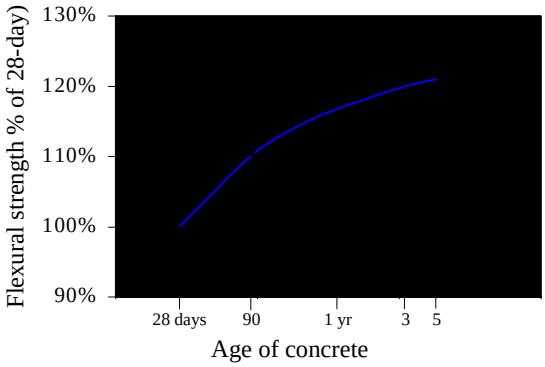

Figure 8 Concrete strength gain assumed in the PCA design model (1).

This approach to the design model is valid as traffic loading occurs gradually and if the pavement is designed for 40 years, 90 days typically represents 0.6% of the traffic life. However, many Australian pavement engineers are concerned that concrete strength growth is not entirely compatible with the assumptions in the model due to the fineness of current Australian produced cement (about 330 to 360 μm) versus cement produced in the 1930s to 1960s which had a fineness of 240 to 280 μm. This is attributed to the fact that most Australian cement is produced for the domestic and high-rise building construction markets and pre-mix concrete requires high early strength gain and limited shrinkage which leads to little strength gain after 28-days.

The introduction of supplementary cementitious materials such as fly ash, improves strength growth beyond 28-days and the DMR (15) subjects contractors to a 10% increase in the target flexural 28-day strength if less than the specified fly ash and slag combinations are used[6].

Figure 9 highlights the benefit of developing strength with age beyond 28-days as it provides a significant safety margin over the first few years of traffic. It is believed that this safety margin provides for some tolerance to the overloaded axles on our roads and climatic induced stresses beyond the design envelope.

[6] Minimum percentages of fly ash (FA), and GGBFS & FA is 15% and 30%, respectively.

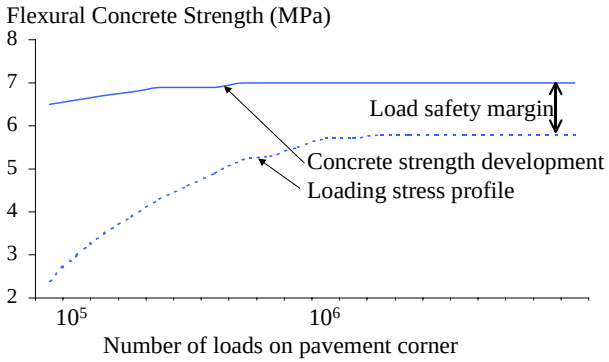

Figure 9 Concrete strength gain and loading with increasing repetitions (16).

In summary, the above discussion on specified concrete properties and their impact on the design model assumptions highlights the following challenges in Australia:

- ☐ to specify average versus characteristic strength
- ☐ flexural versus tensile versus compressive strength testing and conformance
- ☐ concrete strength growth over years rather than months

These issues are now being taken into consideration in the Austroads design algorithms used to calculate the allowable repetitions due to fatigue. The equation below for the determination of the stress ratio shows the original and final adjustment to the algorithm, where the 0.944 coefficient to the characteristic 28-day flexural concrete strength (f'_{cf}), was derived from the adjustment of characteristic to average strength (1.18) and the adopted age adjustment strength factor of 1.25 for 40 year design period.

$$S_r = \frac{S_e}{f'_{cf}} \left[\frac{P.L_{SF}}{4.45F_1} \right]^{0.94} \rightarrow \frac{S_e}{0.944 f'_{cf}} \left[\frac{P.L_{SF}}{4.45F_1} \right]^{0.94}$$

4.2 Fatigue or erosion distress dominant mode

With the increasing traffic volumes in Australia, there are fewer opportunities to close trafficked lanes for repairs. In terms of risk management, it is therefore preferable to design a pavement material such that it does not fail in fatigue leading to full depth cracking. This suggests that the dominant distress mode for rigid pavement configurations should be erosion of the subbase or subgrade.

In Australia, there is almost universal acceptance of the use of lean mix concrete subbases (LMS) with a minimum 28-day compressive strength of 5 MPa. Also, recent accelerated load testing in Yarra (19) confirmed that heavily bound or LMS are required to prevent erosion of the subbase and subgrade. If LMS cannot erode by virtue of its strength and durability, the thickness design model prediction is inconsistent with observations in the field. Therefore what is the mode of distress of this pavement configuration and is it possible that the pavement life exceeds the predicted erosion life in the model? Road authorities should allocate research funds

to address the field performance of undowelled plain concrete pavements to get a better understanding of this discrepancy between model prediction and field performance.

Current specifications place stringent requirements on the contractor to ensure the subbase debonds from the base layer to allow the base to warp and curl with minimal restraint. But should the debonding action not occur, a composite subbase and base pavement material is likely to have significant serviceability problems due to cracking from significant horizontal restraint.

5 FUTURE RESEARCH

One of the challenges in Australia is for road authorities to fund further research into what many senior managers see as a mature design process. A usual remark is "Have we not designed and built these concrete roads since the 1920s?"

There is no doubt that further research on our pavements is required to fully understand the combined traffic and climatic loading using Australian road making materials and practices. The recent accelerated loading research work at Yarra on two long test pads (19), provides some evidence that the combined effects on traffic loading and diurnal ambient temperatures places high stress on the concrete and contraction joints. It was concluded that climatic loads substantially reduce the design traffic life of our major concrete roads (19).

In the mid-1990s at a time when desktop computers just "broke" the 100 MHz clock speed work commenced in Australia on finite element modelling (FEM) of concrete pavements. The quarter size model was a major shortcoming in the use of FEM, but the ability to model the separation of the subbase and base layers, along with a contraction joint element provided strong evidence that this approach could be used to resolve many of the issues noted in this paper.

The five main areas recognised for further study are:

- to carry out field studies of the performance of undowelled contraction joints due to diurnal and annual thermal movements,
- temperature loading and deflections of slabs with skewed transverse joints,
- allowances for loading and concrete strength due to age,
- better design models to allow for subgrade settlement or movement (20),
- the calibration of the 2002 AASHTO design model to Australian conditions, and
- the generation of larger size finite element models to accept full axle load configurations

I believe that once engineers have a greater understanding of the performance of concrete roads due to climatic and traffic loading, their ability to assess increases in legal axle limits with greater confidence will encourage funding for future road research.

6 CONCLUSIONS

The proposed Austroads rigid pavement design approach for the 2001 edition of the guide will result in better efficiency at the thickness design stage, assisting to reduce the effects of human error, and overcome some challenges of the design intent and the delivery of the concrete at site. The main feature of the Guide on rigid pavement design will be the use of algorithms to determine base thickness using a computer program or spreadsheet.

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APPENDIX A

The determination of the allowable axle repetitions may be determined from the following equations:

(a) Fatigue distress mode - Allowable load repetitions (N_f) for a given axle load is:

$$\log (N_f) = \left[\frac{0.9719 - S_r}{0.0828} \right] \quad \text{when } S_r > 0.55 \qquad \text{(Equation A1)}$$

$$N_f = \left[\frac{4.258}{S_r - 0.4325} \right]^{3.268} \quad \text{when } 0.45 \le S_r \le 0.55 \qquad \text{(Equation A2)}$$

where $S_r = \dfrac{S_e}{0.944 f'_{cf}} \left[\dfrac{P.L_{SF}}{4.45 F_1} \right]^{0.94}$

S_e = equivalent stress (MPa)
f'_{cf} = design characteristic flexural strength at 28 days (MPa)
P = axle group load (kN)
L_{SF} = load safety factor
F_1 = load adjustment for fatigue due to axle group
= 9 for SAST and 18 for SADT axle groups
= 18 for TAST and 36 for TADT axle groups
= 54 for TRDT and 72 for QADT axle groups

N_f is infinite or commonly referred as unlimited when S_r is less than 0.45.

The equivalent stress (S_e) is determined from equation A3 using the coefficients a to j in Table A1. The coefficients for TAST and QADT axle groups are not listed in this Table as the data is not available from the PCA Design Method (1) adopted for Austroads. However, it is suggested that a conservative

approach for the fatigue distress analysis is to use the SAST and TRDT coefficients for TAST and QADT axle groups respectively.

$$S_e \text{ or } F_3 = a + b/T + c.ln(E_f) + d/T^2 + e.[ln(E_f)]^2 + f. ln(E_f)/T + g/T^3 + h.[ln(E_f)]^3 + i.[ln(E_f)]^2/T + j. ln(E_f)/T^2 \quad \text{(Equation A3)}$$

Where a, b, c, d, e, f, g, h, i, j are coefficients in Tables 9.3 to 9.5.
T = base thickness (mm)
E_f = effective design CBR

(b) Erosion distress mode - Allowable load repetitions (N_e) for a given axle load is:

$$\log (F_2 N_e) = 14.52 - 6.77 \left[\left(\frac{PL_{SF}}{4.45F_1} \right)^2 \frac{10^{F_3}}{41.35} - 9.0 \right]^{0.103} \quad \text{(Equation A4)}$$

where P and L_{SF} are similar to previous definitions.
F_2 = adjustment for slab edge effects (0.06 for no shoulder & 0.94 for shoulder)
F_3 = erosion factor
F_4 = load adjustment for erosion due to axle group
= 9 for SAST and 18 for SADT axle groups
= 18 for TAST and 36 for TADT axle groups
= 54 for TRDT and QADT axle groups

The erosion factor (F_3) is determined from equation A3 using coefficients a to j not listed in this paper but can be obtained from work by Moffat (6).

There are no limits set for the axle load input and load safety factors to Equations A2 and A4. However, designers should be cautious when the values of ($4.5PL_{SF}/F_1$) or ($4.5PL_{SF}/F_4$) exceed 65 kN.

Table A1 Coefficients for prediction of equivalent stresses.

Coefficient	No Shoulders				Shoulders			
	Axle Group Type				Axle Group Type			
	SAST	SADT	TADT	TRDT	SAST	SADT	TADT	TRDT
a	0.118	0.560	0.219	0.089	-0.051	0.330	0.088	-0.145
b	125.4	184.4	399.6	336.4	26.0	206.5	301.5	258.6
c	-0.2396	-0.6663	-0.3742	-0.1340	0.0899	-0.4684	-0.1846	0.0080
d	26969	44405	-38	-10007	35774	28661	4418	1408
e	0.0896	0.2254	0.1680	0.0830	-0.0376	0.1650	0.0939	0.0312
f	0.19	19.75	-71.09	-83.14	14.57	2.82	-59.93	-61.25
g	-352174	-942585	681381	1215750	-861548	-686510	280297	488079
h	-0.0104	-0.0248	-0.0218	-0.0120	0.0031	-0.0186	-0.0128	-0.0058
i	-1.2536	-4.6657	3.6501	5.2724	1.3098	-1.9606	4.1791	4.7428
j	-1709	-4082	2003	4400	-4009	-2717	1768	2564

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