Improvement of the performance of hotmix asphalt surfacings in New Zealand

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An important note for the reader

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Abbreviations and acronyms

AADT	annual average daily traffic
AC	asphaltic concrete
CAPTIF	Canterbury Accelerated Pavement Testing Indoor Facility
FEM	finite element analysis
HCV	heavy commercial vehicle
HiTAC	Highways Industries Technical Advisory Committee
HMA	hotmix asphalt
NCAT	National Center for Asphalt Technology
NPV	net present value
NZTA	NZ Transport Agency
OGPA	open graded porous asphalt
SMA	stone mastic asphalt
TNZ	Transit New Zealand
vpd	vehicles per day
v/l/d	vehicles per lane per day
MPD	mean profile depth (texture) in mm
PMB	polymer-modified binder
RAMM	Road Assessment and Maintenance Management system
RCA	road controlling authority
RP	Route Position (in metres)
RS	Route Station (location reference)
SH	state highway (number)
TRL	Transport Research Laboratory Ltd (Crowthorne, England)
VMA	Voids in mineral aggregate

Contents

Executive summary

This research project had the objective of identifying areas where changes could be made in the use of thin layers of asphalt so that improvements in performance could be obtained. The project was not designed to investigate quality issues, but was to concentrate on materials and selection. The project was initiated because Transit New Zealand (now the NZ Transport Agency) had found that the costs of resurfacing using asphalt had escalated and the lives being achieved appeared to be short.

This research in this report, which was undertaken between 2007 and 2012, investigated the following areas:

- analysis of currently achieved lives
- whole-of-life costs
- fatigue of thin surfacings
- shear strength
- durability of open graded porous asphalt (OGPA)
- acceptance schemes.

The research has led to the following conclusions and recommendations.

Current lives

The current lives being obtained on thin surfacings on the state highway system are based on 2008 data and reflect the widespread use of OGPA on the motorways. The relatively high percentage failure of surfacings within the first four years prompted this investigation, and the RAMM data does not provide information to assist in identifying the reasons.

Premature failures are often blamed on factors such as:

- treatment selection
- construction quality
- material problems.

However there is no database that records the reasons or gives any information that can be used to prevent the reoccurrence.

Overall, cracking and low skid resistance stand out as major motivations for resurfacing.

ACs and OGPAs last, on average, for 82.4% and 86.7% respectively of their standard default lives; however, there is a wide range of lifetimes for both surface types.

The average lives are similar to those reported in the US and Australia, but significantly shorter than those reported in Britain. The higher British lives could be associated with their analysis being on surfacings on structural asphalt pavements, whereas New Zealand thin surfacings are often over a granular basecourse.

In the early 2000s, changes to OGPA in Auckland through the use of polymers appear to have increased the life of surfacings, and this initiative will not have been reflected in the statistics.

It is recommended that premature failures should be investigated more fully and a system devised to alert the NZTA's National Office of the occurrence of premature failure, and an appropriate level of investigation should be performed to identify the reasons for it.

Whole-of-life costs

With the cost of thin asphalt layers being four to six times more expensive than a chipseal, and the lives not significantly greater, then the justification of using hotmix asphalt cannot be made in terms of cost in areas where both chipseals and asphalt would perform.

Where the traffic stress is such that a chipseal would fail very early in its life then asphalt is the obvious choice.

The main benefit of hotmix asphalt is associated with factors such as ride, rolling resistance and noise, as well as the aesthetic and functional benefits associated with its use in pedestrian areas and CBDs.

It is recommended that the cost of noise detailed in the NZTA EEM should be evaluated to ensure it is reflecting current user expectations.

Fatigue

The analysis demonstrated that fatigue cracking from the top can occur in thin (less than 40mm) layers. The estimation of the level of strain is very dependent on the tyre footprint that is used in the analysis, as well as the pavement structure.

The level of strain on the surface of a thin layer is approximately the same as the level that is estimated at the bottom of an asphalt layer 40–60mm thick. This applies for the FEM analysis using a CAPTIF tyre, as well as for the Circly analysis. This implies that the fatigue relationships in Austroads could be used for layers less than 40mm by estimating the maximum strain at the surface or at the bottom of the asphalt layer.

At this stage of knowledge it is recommended that the Circly-calculated results for strains at the top of the pavement, where they are higher than those at the bottom of the asphalt layer, be used as the 'critical' strains for fatigue in the pavement design.

It is recommended that the deflection relationship given in the Austroads 2011 *Guide to pavement technology part 5: pavement evaluation and treatment design* be used for layers thinner than 40mm by taking the curvature function at 40mm to represent thinner layers.

Shear strength

The strength of asphalt is a function of:

- the aggregate grading
- aggregate particle properties eg % crushed, shape
- binder properties
- degree of compaction
- volume of voids.

New Zealand asphalt mixes are typically composed of crushed coarse and fine aggregate. The literature indicates that the binder properties can play a major role in determining the shear strength and can be more influential than changing the grading from fine to coarse. The changes introduced through the Austroads mix design methods have resulted in a denser grading and a higher level of energy in compaction of the mix design. It is considered that the changes in the compaction energy, rather than the changes in the aggregate grading, may have more effect on the shear performance of the asphalt produced under the Austroads guide.

The use of the Servopac method of compaction at 120 cycles results in lower air voids in a mix than 75-blow Marshall compaction, and thus mixes designed with this method will have lower binder contents than if Marshall compaction had been used.

We recommend the following:

- The relationship between the plateau density of asphalt that is achieved under different traffic conditions, and the density obtained in the Servopac method of compaction, needs to be quantified.
- The benefits of changing to the US Superpave method design, which is continuing to be developed and fine-tuned, should be investigated.
- The effect of changes in binder grade and type on rut resistance, and the effect this could have on fatigue resistance, needs to be quantified and included in the NZTA asphalt guides.
- The use of the Asphalt Mixture Performance Tester in the New Zealand context should be investigated.

Acceptance schemes

The physical works contract form and the specification requirements can act as an incentive to improve the performance of asphalt. The use of warranties, percent within limits combined with proportional payments, and certified products all have a place in a matrix of methods that can be used, depending on the project.

For thin layers of asphalt, the use of a certification scheme such as HAPAS could encourage the use of proprietary surfacing systems that British experience suggests will result in enhanced lives.

The use of a proportional payment system requires that there is an agreed relationship between a performance or test measure and its effect on life for the scheme to be accepted by industry.

We recommend the following:

- An approval scheme based on the HAPAS model should be developed and introduced.
- A performance-related specification for asphalt should be developed.
- More emphasis should be placed on 5–10-year performance warranties, initially as trials.
- A statistical approach to acceptance and payment for thin asphalt surfacings should be developed.

Abstract

This research project had the objective of identifying areas where changes could be made in the use of thin layers of asphalt so that improvements in performance could be obtained. The project was not designed to investigate quality issues, but was to concentrate on materials and selection. The project was initiated because Transit New Zealand (now the NZ Transport Agency) had found that costs of resurfacing using asphalt had escalated and the lives being achieved appeared to be short.

This research in this report, which was undertaken between 2007 and 2012, investigated the following areas:

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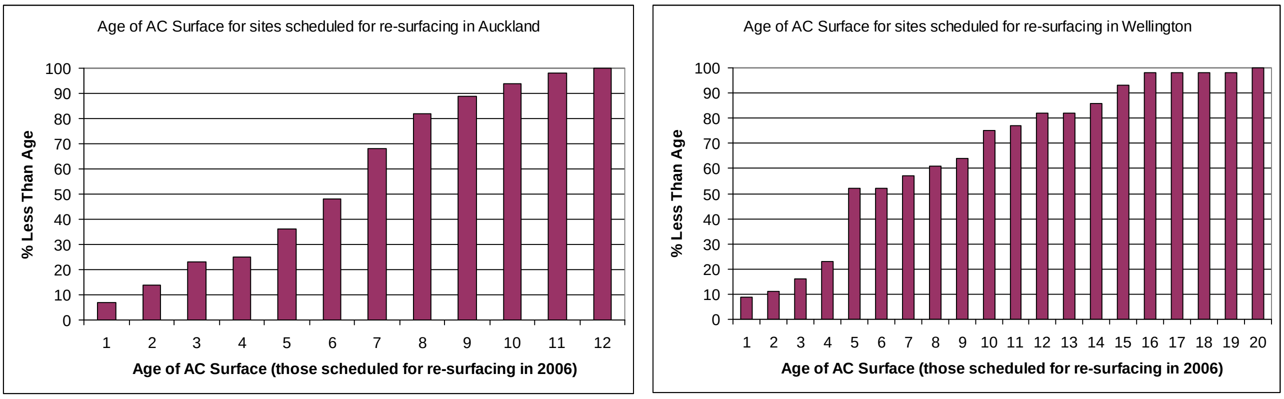
- shear strength
- durability of open graded porous asphalt (OGPA)
- acceptance schemes.

1 Introduction

1.1 Purpose of the research

In 2006, Transit New Zealand's (TNZ, now the NZ Transport Agency – NZTA) annual requests for funding identified a significant increase in the cost of asphalt surfacings. A survey of TNZ regions revealed that some of the cost increases were a result of cost escalations (labour and oil price increases) and shorter lives of asphalt surfacings than expected. A quick analysis from TNZ's RAMM[1] database showed that in 2006, 25% of new asphalt surfaces were on existing asphalt surfaces aged four years old or less (see figure 1.1).

Figure 1.1 Distribution of asphalt surfacings' lives in Auckland and Wellington



There is evidence from other regions of shorter than expected lives and premature failure within two years. The reasons cited for early failure include:

- poor construction/quality control
- poor choice of thin asphalt treatment through a lack of understanding and guidance
- not strengthening the underlying pavement
- lack of experience in the region in building specialty asphalt mixes
- the life expected was too high (eg figure1.1 above shows there is a 70% chance in Auckland that the asphalt surfacing will not reach seven years of age)
- lack of knowledge of the expected life of a range of asphalt surface treatments (as well as the full range of asphalt treatments available) to allow comparisons of whole-of-life costs.

An asphalt forum highlighting the above issues was held with industry on 30 November 2005, for the purpose of determining what research was required to increase the life of our asphalt surfacings and reduce whole-of-life costs. The industry group agreed with the above issues and suggested additional issues, such as:

- the need for a closer look at the asphalt life statistics, to improve confidence in the data, to determine failure modes, and to track the costs of asphalt surfacings

1 Road Assessment and Maintenance Management system.

- assessing how New Zealand's asphalt surfacing lives compare with the rest of the world
- the effect of the underlying pavement strength on asphalt surfacing life, and how this factor should be incorporated into design
- the need for a design guide for asphalt surfacing in New Zealand
- the need for more information on treatment types and their performance
- how the benefits of adding polymers to the asphalt binder could be quantified.

The research for this report, which was undertaken between 2007 and 2012, investigated the following areas:

- analysis of currently achieved lives
- whole-of-life costs
- fatigue of thin surfacings
- shear strength
- the durability of open graded porous asphalt (OGPA)
- acceptance schemes.

The issue of quality control was also raised as a priority, but this was already being addressed by Roading New Zealand in their development of a certified asphalt plant scheme.

2 Currently achieved asphalt life.

In New Zealand the RAMM asset management system includes a table of expected life for road surfacings. This covers chipseals, slurries and thin asphalt surfacings. The default lives are shown in table 2.1.

The life for a thin asphalt surfacing is a function of the traffic volume. Figure 2.1 is a graphical view of the RAMM default values together with the life expected for a thicker structural asphalt layer. It can be seen that OGPA has the lowest expected life (six years), and a thin asphalt surfacing has an expected life under low traffic of 16 years. These default values are considered representative of local authority roads and state highways. Road controlling authorities can and do modify these values to reflect the experience in their area.

The RAMM expected lives are not the same as a design life. In the Austroads *Guide to pavement technology part 2: pavement structural design* (Jameson 2008), which is used in New Zealand, the design life is that which the project has a 95% probability of achieving. The RAMM expected lives are closer to an average life, and if a 95% probability was used then the lives would be significantly shorter.

Figure 2.1 RAMM default lives for asphalt surfacings

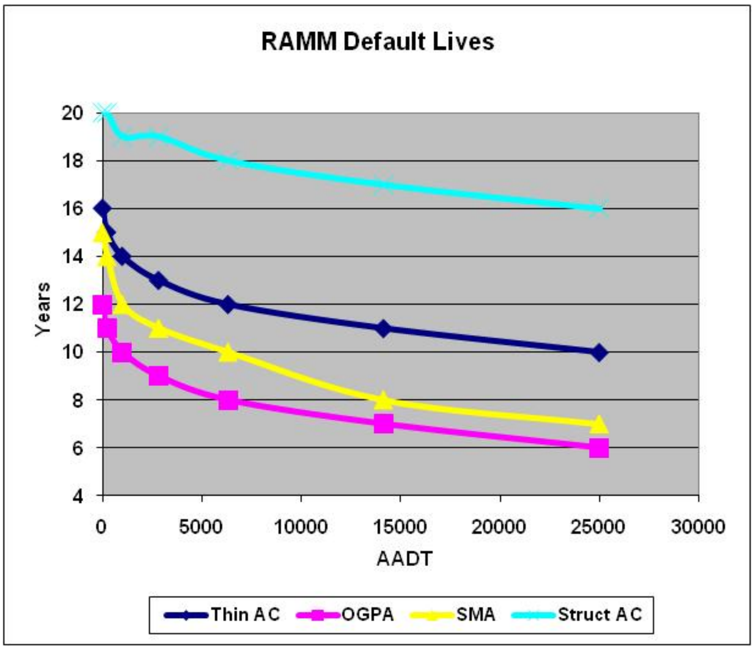


Table 2.1 Default surfacing lives from RAMM

Surfacing type	Use 1 (<100vpd[a]) years	Use 2 (100–500vpd) years	Use 3 (500–2000vpd) years	Use 4 (2000–4000vpd) years	Use 5 (4000–10,000vpd) years	Use 6 (10,000–20,000vpd) years	Use 7 (>20,000vpd) years
Reseals							
Grade 6	6	5	4	3	2	1	1
Grade 5	8	7	6	5	4	3	2
Grade 4	12	10	8	7	6	5	4
Grade 3	14	12	10	9	8	7	6
Grade 2	16	14	12	11	10	9	8
Grade 4/6	14	12	10	9	8	6	4
Grade 3/5	16	14	12	11	10	8	6
Grade 2/4	18	16	14	13	12	10	9
Slurry seal	8	7	6	5	4	3	2
Thin AC[b]	16	15	14	13	12	11	10
OGPA[c]	12	11	10	9	8	7	6
SMA[d]	15	14	12	11	10	8	7

a) Vehicles per day.

b) Asphaltic concrete.

c) OGPA.

d) Stone mastic asphalt.

2.1 State highway asphalt surfacing lives

As part of this research, an analysis was performed to assess the lifetimes being achieved by thin asphalt surfacings on state highways.

The methodology built on research that was carried out for TNZ in 2005, using the RAMM database for chipseals on state highways (Ball and Patrick 2005). That work examined levels of chipseal distress observed in the 2002/3 surveys for sites that had been resealed during 2003/4. Significant variations of average seal life and reasons for resealing were observed between state highway regions, and a significant number of unexpectedly low seal lives occurred in some instances. This methodology was applied to asphaltic surfacings, as described below.

2.1.1 Source of data

Treatment length data for more heavily populated regions (Auckland, Hamilton, Wellington, Christchurch and Dunedin) was downloaded from the TNZ RAMM databases for the 2003/4 and 2004/5 years, and transformed into Microsoft EXCEL files. Treatment lengths for the two years were matched using Microsoft ACCESS. Resurfaced thin asphalt areas were then located by looking for changes in the year of surfacing. Details of thin asphalt surface types and numbers are shown in table 2.2.

Table 2.2 Thin asphalt surface types (RAMM state highway database)

Surface type	No. of surfaces (RAMM 2003/4)	No. replaced (RAMM 2004/5)	Percent replaced
AC	346	19	5.5
OGPA	456	52	11.4
SMA	86	4	4.7
SLRY[a]	48	6	12.5
Totals	950	81	8.5

a) Slurry seal

2.1.2 Condition of surfaces

The relatively small number of sites that had been resurfaced allowed a detailed examination of all of them. Surface type, location, age, traffic levels, percentage life achieved, environment and surface condition are shown in the tables following.

The last column in each of these tables lists types of distress found on the 2003/4 surfaces that conceivably could have led to the decision to resurface. The distress types are defined in RAMM and are used in an algorithm to indicate to the engineer the sections that should be examined in more detail, as the level of distress indicates that treatment may be required. Use of capital letters indicates that the distress type has reached a critical level; lower-case letters indicate that the distress type is present to a degree which, although not yet critical, might encourage early resurfacing as a preventative action.

Table 2.3 Thin AC sites resurfaced

SH	RS	RP start m	RP end m	SH region	Original surface date	Chip size mm	Surface depth mm	Resurfacing date	Age years	RAMM default life years	% Life achieved	Resurfacing material	ADT	% Heavies	Ramp?	Urban/rural	RAMM-deduced reasons for resurfacing
01N	0386/09.31-I	9307	9623	2	12/05/99	14	30	27/05/05	6.0	12	50.4	SMA	7920	7		R	CRACKING SKID txtr300 mpd
01N	0386/09.31-D	9307	9623	2	12/05/99	14	30	27/05/05	6.0	12	50.4	SMA	7920	7		R	CRACKING SKID txtr300 mpd
01N	0414/00.72-X414-R3-OFF	500	612	2	1/06/95	5	10	7/03/04	8.8	6	146.1	SMA	13443	5	OFF	R	SKID
01N	0431/12.94-X444-R4-ON	0	232	2	28/02/99	14	30	22/01/04	4.9	11	44.5	SMA	12616	4	ON	R	SKID TXTR300 mpd
20A	0000/00.26-I	2012	2115	2	23/03/00	14	35	25/05/04	4.2	11	37.9	PSV70+	17566	6		U	skid TXTR300 mpd
20A	0000/02.12-I	3075	3160	2	9/03/98	10	25	8/06/04	6.3	11	56.8	PSV70+	19770	6		R	skid TXTR300 MPD
01N	0552	1447	1518	3	12/05/98	19	50	19/10/04	6.4	6	107.3	SMA	21400	11		U	CRACKING skid potholes
01N	0680	14400	14661	3	24/03/97	10	30	28/02/05	7.9	8	99.2	VF	5100	20		R	CRACKING
01N	0680	14661	14668	3	24/03/98	10	30	5/04/05	7.0	8	87.9	Rack	5100	20		R	CRACKING MPD
01N	0763	10705	10803	3	16/12/99	10	30	14/12/04	5.0	9	55.5	SMA	3500	15		R	CRACKING skid
01N	0763	12805	13100	3	4/03/98	10	30	17/01/05	6.9	9	76.4	2Coat	3500	15		R	CRACKING skid mpd
01N	0763	13100	13170	3	4/03/98	10	30	17/01/05	6.9	9	76.4	2Coat	3500	15		R	CRACKING
01N	0763	13170	13386	3	4/03/98	10	30	17/01/05	6.9	9	76.4	2Coat	3500	15		R	skid CRACKING

SH	RS	RP start m	RP end m	SH region	Original surface date	Chip size mm	Surface depth mm	Resurfacing date	Age years	RAMM default life years	% Life achieved	Resurfacing material	ADT	% Heavies	Ramp?	Urban/rural	RAMM-deduced reasons for resurfacing
01N	0777	13	69	3	9/05/96	10	25	27/03/01	4.9	9	54.2	VF	3500	15		R	CRACKING potholes rutting
01N	0777	5700	5907	3	18/12/98	10	30	20/12/04	6.0	9	66.7	SMA	3500	15		R	skid CRACKING potholes
01S	0501/04.48-I	4910	4938	11	18/03/97	10	25	8/12/03	6.7	7	96.1	AC	6410	6		U	SKID TXTR300 MPD
73A	0000/01.20-D	2114	2139	11	1/10/88	10	25	18/11/95	7.1	9	79.2	AC	12668	7		U	SKID TXTR300 MPD
01S	0704-D	200	420	13	25/12/84	8	25	1/12/04	19.9	9	221.5	AC	11752	4		U	CRACKING SKID TXTR300 mpd
01S	0872/06.04	12700	12760	14	20/05/03	30		11/01/05	1.6			Text	3826	11		R	TXTR300 mpd
01N	0261/02.40-I	2399	2758	1	24/03/98	14	30	7/08/04	6.37	8	79.7	AC	11880	10		U	CRACKING SKID TXTR300 mpd
01N	0261/02.40-I	2758	3149	1	24/03/98	14	30	7/08/04	6.37	8	79.7	AC	11880	10		U	CRACKING skid
01N	0261/02.40-I	3149	3372	1	25/03/98	14	30	7/08/04	6.37	8	79.6	AC	11890	10		U	CRACKING
01N	0261/02.40-I	3149	3372	1	24/03/98	14	30	7/08/04	6.37	8	79.7	AC	11880	10		U	CRACKING skid
01N	0261/02.40-D	2608	2958	1	25/03/98	14	30	7/08/04	6.37	8	79.6	AC	11890	10		U	CRACKING txtr300
01N	0261/02.40-D	2958	3149	1	25/03/98	14	30	7/08/04	6.37	8	79.6	AC	11890	10		U	
01N	0261/03.37	3372	3648	1	25/03/98	14	30	7/08/04	6.37	7	91.0	AC	25335	10		U	CRACKING SKID TXTR300 mpd

SH	RS	RP start m	RP end m	SH region	Original surface date	Chip size mm	Surface depth mm	Resurfacing date	Age years	RAMM default life years	% Life achieved	Resurfacing material	ADT	% Heavies	Ramp?	Urban/rural	RAMM-deduced reasons for resurfacing
01N	0266	2485	2955	1	9/06/98	14	25	26/07/04	6.13	7	87.6	AC	14543	10		R	CRACKING skid potholes txtr300
01N	0266	2955	3395	1	9/06/98	14	25	26/07/04	6.13	7	87.6	AC	14543	10		R	CRACKING skid txtr300
01N	0292	201	1246	1	2/06/98	14	25	3/11/04	6.42	8	80.3	2Coat	9471	10		R	CRACKING skid potholes mpd
01N	0373	10060	10707	2	1/05/00	14	25	12/10/04	4.45	7	63.6	SMA	14666	6		R	SCABBING skid
01N	0373	11945	12514	2	17/04/00	14	25	6/10/04	4.47	7	63.9	SMA	14666	7		R	SCABBING skid POTHOLES
01N	0386	0	445	2	30/04/98	10	25	19/10/04	6.47	7	92.5	SMA	19704	4		R	scabbing skid mpd
01N	0386	445	1155	2	15/06/00	14	25	19/10/04	4.35	7	62.1	SMA	19704	4		R	SCABBING SKID
01N	0386/06.19	7539	7661	2	17/05/99	14	30	25/12/99	0.61	6	10.1	AC	23851	7		U	TXTR300 mpd rutting
01N	0398/15.45-X414-R4-ON	110	352	2	9/03/95	10	25	7/03/04	9.00	9	100.0	OGPA	9080	5	ON	R	
01N	0414/06.61-X420-R3-ON	165	823	2	14/12/94	10	20	8/03/04	9.23	8	115.4	SMA	17680	4	ON	R	skid
01N	0414-D	5845	7805	2	5/03/95	10	25	1/01/02	6.83	8	85.4	OGPA	60240	4		R	
01N	0414-D	8283	8543	2	1/09/00	20	25	7/11/04	4.18	6	69.7	OGPA	79150	4		R	
01N	0414-D	8767	9020	2	25/12/97	10	25	7/11/04	6.87	6	114.5	OGPA	81304	4		R	

SH	RS	RP start m	RP end m	SH region	Original surface date	Chip size mm	Surface depth mm	Resurfacing date	Age years	RAMM default life years	% Life achieved	Resurfacing material	ADT	% Heavies	Ramp?	Urban/rural	RAMM-deduced reasons for resurfacing
01N	0431/12.94-X444-R4-ON	0	232	2	11/02/98	10	25	22/01/04	5.94	7	84.9	SMA	12724	5	OFF	R	SKID txtr300 mpd
01N	0461/01.31-I	2274	3040	2	19/11/97	10	25	7/04/05	7.38	7	105.4	OGPA	19150	5		R	
01N	0461/01.31-I	3040	3400	2	19/11/97	10	25	7/04/05	7.38	7	105.4	OGPA	19150	5		R	skid
01N	0461/01.31-I	3400	3700	2	19/11/97	10	25	7/04/05	7.38	7	105.4	OGPA	19150	5		R	CRACKING skid RUTTING
01N	0461/01.31-I	3700	4000	2	19/11/97	10	25	7/04/05	7.38	7	105.4	OGPA	19150	5		R	shoving skid RUTTING
01N	0461/15.67-X478-R1-OFF	0	300	2	5/04/00	14	25	13/02/05	4.86	8	60.7	SMA	6268	5	OFF	R	skid
01N	0461/15.67-X478-R1-OFF	300	493	2	5/04/00	14	25	14/02/05	4.86	8	60.8	SMA	6268	5	OFF	R	txtr300
01N	0461/15.48-X478-R3-ON	0	232	2	24/02/99	20	35	16/01/05	5.89	8	73.7	OGPA	6285	12	ON	R	
01N	0461/15.48-X478-R3-ON	0	232	2	24/11/97	10	25	16/01/05	7.15	10	71.5	OGPA	1597	6	OFF	R	SKID
01N	0461/15.48-X478-R3-ON	232	849	2	26/03/98	20	35	15/01/05	6.81	8	85.1	SMA	6285	12	ON	R	SKID
20A	0000-I	0	260	2	25/12/96	10	20	18/11/03	6.90	7	98.5	OGPA	16060	6		R	

SH	RS	RP start m	RP end m	SH region	Original surface date	Chip size mm	Surface depth mm	Resurfacing date	Age years	RAMM default life years	% Life achieved	Resurfacing material	ADT	% Heavies	Ramp?	Urban/rural	RAMM-deduced reasons for resurfacing
20A	0000-D	0	157	2	25/12/96	10	20	1/12/03	6.93	7	99.0	OGPA	13340	6		R	
20A	0000/00.26-I	1000	2012	2	25/12/01	10	25	20/11/03	1.90	7	27.2	OGPA	17566	6		R	
20A	0000/02.10-D	2265	3150	2	27/10/93	13	20	30/04/04	10.51	8	131.4	OGPA	19770	6		R	skid
01N	0543	6511	6581	3	28/04/99	13	30	20/10/04	5.48	6	91.4	SMA	24000	11		U	skid TXTR300 mpd
01N	0543	6581	6654	3	28/04/99	13	30	20/10/04	5.48	6	91.4	SMA	24000	11		U	skid mpd
01N	0552	2638	2822	3	8/05/96	13	30	12/10/04	8.43	6	140.5	SMA	21400	11		U	CRACKING skid potholes txtr300 mpd
01N	0744	7732	7859	3	25/12/89	10	25	30/11/04	14.93	8	186.7	Text	4800	15		U	skid
01N	0967	13946	14038	8	2/05/95	20	50	11/05/05	10.03	10	100.3	AC	14044	10		U	CRACKING SKID TXTR300 mpd
01N	0995	5255	5715	9	1/03/97	10	25	27/02/05	7.99	10	79.9	AC	16736	6		U	CRACKING SKID mpd
01N	1012	4300	4600	9	1/01/96	10	25	14/03/05	9.20	10	92.0	AC	20934	6		U	skid mpd
01N	1050/03.19-X1053-R1-OFF	0	250	9	12/12/93	10	25	13/02/05	11.17	8	139.7	AC	9057	2	OFF	R	rutting
01S	0028	0	297	10	14/11/96	10	25	1/12/04	8.05	8	100.6	OGPA	10914	11		U	skid CRACKING mpd
01S	0430	130	150	11	6/03/94	10	35	19/10/03	9.62	9	106.9	AC	41413	10		U	SKID TXTR300 mpd RUTTING

SH	RS	RP start m	RP end m	SH region	Original surface date	Chip size mm	Surface depth mm	Resurfacing date	Age years	RAMM default life years	% Life achieved	Resurfacing material	ADT	% Heavies	Ramp?	Urban/rural	RAMM-deduced reasons for resurfacing
73A	0000-I	889	1115	11	25/12/95	10	25	19/02/98	2.15	7	30.8	OGPA	16460	9		U	potholes SKID TXTR300 mpd
01S	0583/07.12-I	7120	7960	13	26/04/01	10	25	16/11/01	0.56	8	7.0	OGPA	8224	14		U	SKID mpd
01S	0709-I	134	320	13	29/03/92	10	30	13/11/04	12.63	8	157.8	AC	9901	11		U	CRACKING
01S	0712-I	300	820	13	19/04/96	10	30	17/11/04	8.58	8	107.3	AC	15147	6		R	CRACKING skid
01S	0920-I	0	650	14	12/04/99	14	30	4/03/05	5.89	9	65.5	BBM	6928	4		U	skid txtr300
01S	0920-I	0	650	14	12/04/99	14	30	4/03/05	5.89	9	65.5	BBM	7207	4		U	CRACKING SKID txtr300
01S	0920-D	0	650	14	12/04/99	14	30	4/03/05	5.89	9	65.5	BBM	6928	4		U	skid txtr300
01S	0920-D	0	650	14	12/04/99	14	30	4/03/05	5.89	9	65.5	BBM	7207	4		U	CRACKING SKID txtr300

Table 2.4 OGPA sites resurfaced

SH	RS	RP start m	RP end m	SH region	Original surface date	Chip size mm	Surface depth mm	Resurfacing date	Age years	Expected slurry life years	% Life achieved	Resurfacing material	ADT	% Heavies	Urban/rural	RAMM-deduced reasons for resurfacing
01N	0319	14140	14500	1	10/04/00	3	10	1/04/04	3.98	7	56.8	2Coat	8651	10	R	SKID txtr300 mpd
01N	0319	14500	14600	1	10/04/00	3	10	1/04/04	3.98	7	56.8	2Coat	8651	10	R	
01N	0967	13796	13946	8	14/02/98	3	5	11/05/05	7.24	10	72.4	AC	14044	10	U	CRACKING SKID TXTR300 MPD
01S	0583/07.12-I	7960	8360	13	28/02/98	6	6	1/03/04	6.00	4	150.1	2Coat	6489	14	U	cracking SKID TXTR300 mpd
06A	0000	5970	6420	13	20/04/96	6	6	9/12/04	8.64	4	216.0	BBM	17443	6	U	CRACKING SKIDTXTR300mpd
06A	0000	6420	6889	13	20/03/97	6	6	6/05/05	8.13	10	81.3	BBM	17443	6	U	CRACKING SKIDTXTR300mpd

Table 2.5 Slurry seal sites resurfaced

SH	RS	RP start m	RP end m	SH region	Original surface date	Chip size mm	Surface depth mm	Resurfacing date	Age years	Expected SMA life years	% Life achieved	Resurfacing material	ADT	% Heavies	Urban/rural	RAMM-deduced reasons for resurfacing
01N	0386/09.31-I	9307	9623	2	6/03/03	11	35	27/05/05	2.23	8	27.8	SMA	7920	7	R	CRACKING skid
01N	0386/09.31-D	9307	9623	2	6/03/03	11	35	27/05/05	2.23	8	27.8	SMA	7920	7	R	CRACKING skid
01N	0414-D	8543	8647	2	25/03/03	11	35	7/11/04	1.62	7	23.2	OGPA	79150	4	R	
01N	0552	589	660	3	10/11/03	14	35	20/10/04	0.94	7	13.5	SMA	34000	11	U	CRACKING SKID TXTR300 mpd

Table 2.6 Numbers of different surface distress types for different asphalt surface types

Surface type	Total surfaces	CRACKING	cracking	Total cracking	SHOVING	shoving	Total shoving	SCABBING	scabbing	Total scabbing	POTHOLES	potholes	Total potholes	RUTTING	rutting	Total rutting	SKID	skid	Total skid	TXTR300	txtr300	Total txtr300	MPD	mpd	Total mpd
AC	18	12	0	12	0	0	0	0	0	0	0	3	3	0	1	1	7	7	14	7	2	9	4	7	11
OGPA	52	18	0	18	0	1	1	3	1	4	1	4	5	3	2	5	13	23	36	7	10	17	0	16	16
SLRY	6	3	1	4	0	0	0	0	0	0	0	0	0	0	0	0	5	0	5	4	1	5	1	4	5
SMA	4	3	0	3	0	0	0	0	0	0	0	0	0	0	0	0	1	2	3	1	0	1	0	1	1
All	80	36	1	37	0	1	1	3	1	4	1	7	8	3	3	6	26	32	58	19	13	32	5	28	33

Table 2.7 Percentages of different surface distress types for different asphalt surface types

Surface type	CRACKING	cracking	Total cracking	SHOVING	shoving	Total shoving	SCABBING	scabbing	Total scabbing	POTHOLES	potholes	Total potholes	RUTTING	rutting	Total rutting	SKID	skid	Total skid	TXTR300	txtr300	Total txtr300	MPD	mpd	Total mpd
AC	66.7	0	66.7	0	0	0	0	0	0	0	16.7	16.7	0.0	5.6	5.6	38.9	38.9	77.8	38.9	11.1	50.0	22.2	38.9	61.1
OGPA	34.6	0	34.6	0	1.9	1.9	5.8	1.9	7.7	1.9	7.7	9.6	5.8	3.8	9.6	25.0	44.2	69.2	13.5	19.2	32.7	0	30.8	30.8
SLRY	50.0	16.7	66.7	0	0	0	0	0	0	0	0	0	0	0	0	83.3	0	83.3	66.7	16.7	83.3	16.7	66.7	83.3
SMA	75.0	0	75.0	0	0	0	0	0	0	0	0	0	0	0	0	25.0	50.0	75.0	25.0	0	25.0	0	25.0	25.0
All	45.0	1.3	46.3	0	1.3	1.3	3.8	1.3	5.0	1.3	8.8	10.0	3.8	3.8	7.5	32.5	40.0	72.5	23.8	16.3	40.0	6.3	35	41.3

2.1.3 Surface statistics

Inspection of the resurfacing dates in tables 2.3–2.5 reveals that not all resurfacing was carried out in the 2004–5 resurfacing season. Evidently some resurfacing data had been entered more than a year after the resurfacing took place. Given the late entering of earlier data, it is likely that not all of the 2004–5 year data had been entered. Accordingly, the assumption was made in the following analyses that the resurfacing data dating from before the 2004–5 season compensated for missing 2004–5 data that had not yet been entered.

The age (attained life) was compared to the default expected life for that particular traffic volume to produce the life ratio E (equation 2.1). If a surfacing was replaced when it was aged at exactly its expected default life (ie attained life = default seal life), it would have a life ratio of E = 1.

$$E = \frac{C}{S} \qquad \text{(Equation 2.1)}$$

Where:

E = life ratio

C = attained life

S = default seal life (age in years expected from the chipseal according to table 2.1.

Table 2.8 lists the basic statistics for two types of thin asphalts that had been resurfaced (due to the low number of slurry seals the results were not statistically meaningful).

Table 2.8 Basic statistics for the ratio attained life/default surface life

Surface type	No. of surfaces	Mean life ratio	Life ratio std deviation	Median life ratio	Minimum life ratio	Maximum life ratio	Lower quartile	Upper quartile
AC	18	0.824	0.437	0.764	0.379	2.215	0.542	0.961
OGPA	52	0.867	0.320	0.852	0.070	1.867	0.676	1.030

ACs and OGPAs last, on the average, for 82.4% and 86.7% respectively of their standard default lives; however, there is a wide range of lifetimes for both surface types, as reflected in the size of the standard deviations.

The nature of this variation can be better appreciated by cumulative plots of the percentage of newly resurfaced asphalts against the fraction of the default life, as shown in figures 2.2 and 2.3. In particular, these show that for the ACs and OGPAs there is a range of lifetimes over which probability of failure is fairly constant (0.4–1.1 times the default life for ACs, 0.6–1.1 times default life for the OGPAs). Rates of failure outside this lifetime range (approximately the first 10% and last 10% of seal lives) are lower. This behaviour is different than that found for reseals and second-coat seals (Ball and Patrick 2005), for which the rate of seal failure was almost constant, from 0.1–1.5 times the default life.

Figure 2.2 Surface lives achieved by thin ACs

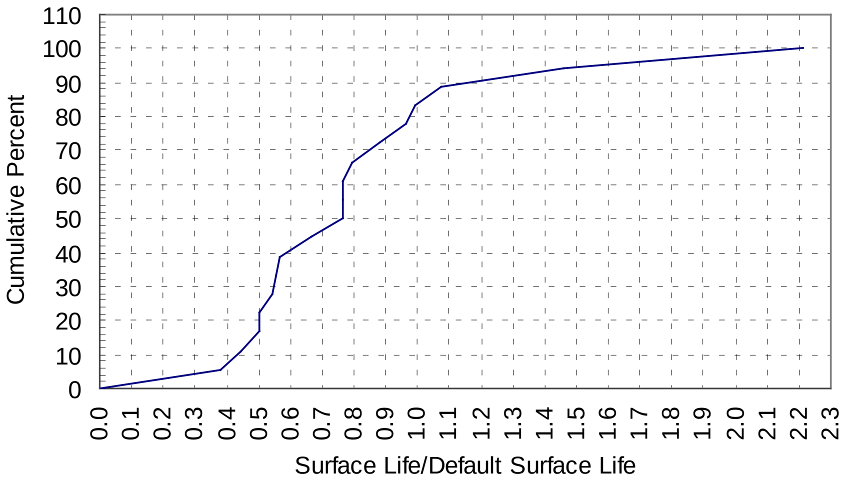
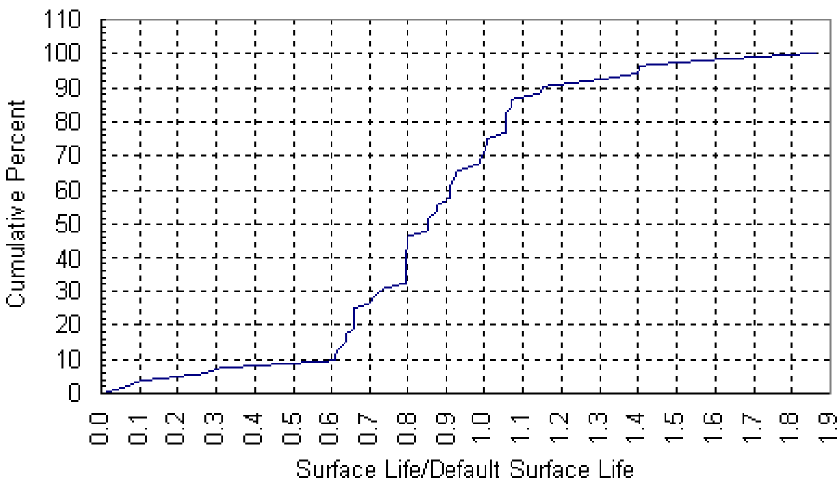


Figure 2.3 Surface lives achieved by OGPAs



In general, relative to default surface lifetimes, it can be seen that thin asphalts are giving a slightly better performance than state highway chipseals. An alternative view would be to say that the thin asphalt default lifetimes give a slightly better indication of actual lives being achieved than do chipseal default lifetimes.

While there are a few cases where no reason for resurfacing could be deduced from the RAMM surface condition data, in the great majority of cases more than one reason was obtained (the last column in tables 2.3–2.6). This is in strong contrast to the earlier findings for chipseals (Ball and Patrick 2005), where frequently a reason for resealing could not be found. For surfaces that had been resurfaced, table 2.7 shows the percentages of each surface type that had given levels of surface distress. For example, 66.7% of resurfaced ACs had cracking at 3% of the wheel-track length or greater, as did 34.6 % of resurfaced OGPAs.

For ACs, the most common distress type was cracking (66.7%), followed by low skid resistance (38.9%) and low texture (38.9%). The total of surfaces with skid resistance below the investigatory level (IL) (77.8%) suggests that pre-emptive action to stop surfaces falling below the skid resistance threshold limit may have also been considered in the decision to resurface. A low texture depth (MPD) did not necessarily coincide with a low skid resistance.

For OGPAs, cracking was much less prominent than for ACs (34.6% as against 66.7%), but was still the most common type of distress. Low skid resistance (25% below the threshold level (TL) but 60.2% below the investigatory level) may, in fact, have been a more common reason for resurfacing.

Overall, cracking and low skid resistance stood out as major motivations for resurfacing.

Reasons for chipsealing over thin asphalt surfacing were not always given, but prominent among those that were given were waterproofing and noise reduction.

2.2 Local authorities

A comprehensive study of the performance of pavement surfacings in Auckland City was performed in 2002 by Greenwood, in order to determine actions that could be taken to improve the life that was being attained.

This analysis investigated the lives achieved of thin asphalt surfacings in Auckland city and developed the average lives shown in figure 2.4. It can be seen that at higher traffic volumes the lives are less than the RAMM default lives, but at lower traffic volumes greater lives are obtained.

Christchurch and Wellington City Councils' RAMM data has also been analysed (see figures 2.5 and 2.6). In using the database to obtain the lives achieved of surfacings in the state highway and Auckland City analysis, the life was obtained by comparing two years of data. One year was used to determine the locations of new surfacings, and the previous-year data was used to determine the old surfacing and its construction date. Without the two years of data, only the distribution of lives achieved for the current surfacings could be obtained.

Figure 2.4 Asphalt lives in Auckland, from Greenwood (2002)

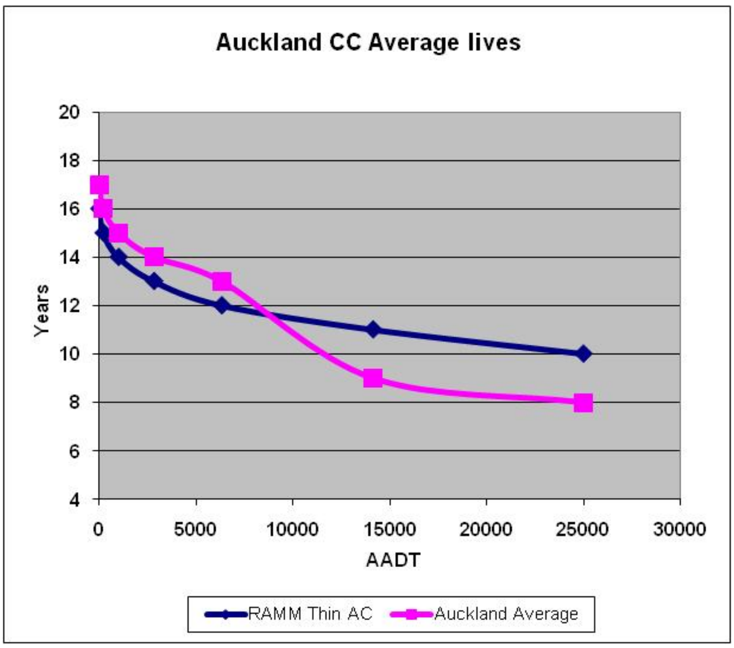


Figure 2.5 Asphalt lives in Wellington

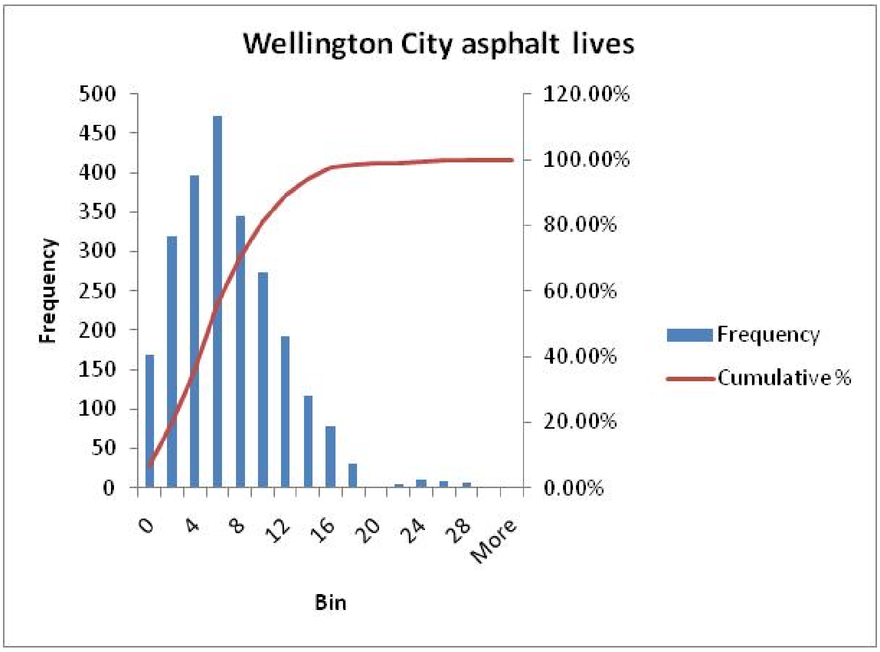
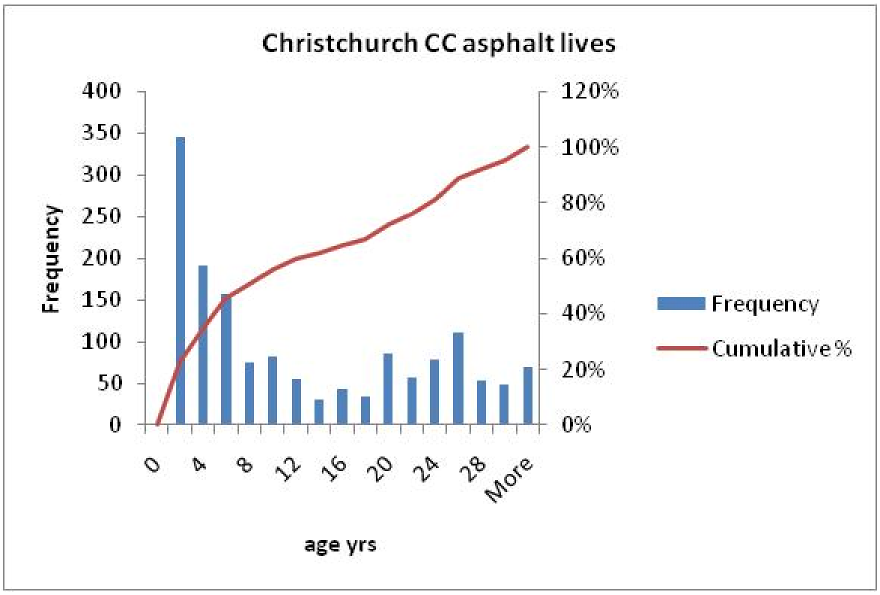


Figure 2.6 Asphalt lives in Christchurch



2.3 International

Austroads' *Guide to pavement technology part 3: pavement surfacings* (Rebbechi and Sharp 2009) gives a range of life of thin asphalt surfaces of 8–20 years, depending on traffic, stress and climate.

Nicholls (2004) from Transport Research Laboratory (TRL) in Britain estimated that the average service life of thin surfacings is between 13 and 14 years. These estimates were made on a range of surfacings on structurally sound pavements.

The types of surfacings that were assessed were:

- paver-laid surface dressings (PLSD)
- thin asphalt concrete (TAC)
- thin stone mastic asphalt (TSMA)
- multiple surface dressing (MSD).

Based on an expert panel's assessment of surfacings in service and the results of trials, Nichols proposed a 'conservative' estimate of life as given in table 2.9.

Table 2.9 Estimation of thin surfacing life (in years) (from Nichols 2004)

	PLSD		TAC		TSMA		MSD
Panel mark	14mm	10mm	14mm	10mm	14mm	10mm	
Moderate	6	5	7	7	8	8	3
Acceptable	12	9	12	13	15	>20	5
Suspect	18	12	16	19	>20	>20	8

Most of the surfacings assessed by Nichols were propriety treatments. These treatments have to go through a formal assessment before being used on British trunk roads. The formal system known as HAPAS (Highway Authorities Product Approval Scheme) assesses the supplier's claims with regard to skid resistance macrotexture and durability.

Details of the materials were not given in the report. However the report noted that the thin asphalt typically contained polymer-modified bitumen, and the TSMA contained unmodified bitumen. The thin asphalt surfacing also tended to have texture depths greater than 1mm. These mixes would not be similar to a New Zealand asphalt complying with NZTA M/10 (2005), as both a 14mm and 10mm mix would have a texture depth of less than 0.7mm.

A thin SMA's expected life of 15 years, or even more than 20 years, using unmodified bitumen is significantly more than that presumed in New Zealand. Figure 12 suggests that the RAMM default life for an SMA would be 10 years.

Newcombe (2009), for the US National Asphalt Paving Association, reviewed the performance of thin asphalt overlays by surveying surfacings in some US states and conducting a literature review. This data is summarised here in table 2.10.

The mixes used in the US were dense-graded asphalt and were designed to the Superpave requirements, although Ohio was still using the Marshall mix design methodology.

Table 2.10 Performance of thin overlay (based on Newcombe 2009)

Location	Traffic	Existing pavement	Expected life (yrs)
Ohio	High & low	Asphalt	16
	Low	Composite	11
	High	Composite	7
Nth Carolina		Concrete	6–10
Ontario, Canada	High	Asphalt	8
Illinois	Low	Asphalt	7–10
New York		Asphalt	5–8
Indiana	Low	Asphalt	9–11
Georgia	Low	Asphalt	10
Austria	Low or high	Asphalt	≥10
	High	Concrete	≥8

In table 2.10 a composite pavement refers to a concrete pavement that has had an overlay of asphalt. Thus the surfacing is asphalt over the existing asphalt.

2.4 Discussion

The range of expected lives given in table 2.10 for a dense-graded asphalt range from a low of 5 years in New York to a high of 16 years in Ohio. These compare with the 10–12 years for medium to high traffic volumes in the RAMM database. However, the analysis of life currently being achieved on the US highways suggests that on average, dense-graded asphalt is achieving approximately 80% of the RAMM default life. This means the life for medium to high traffic is 8–9.5 years. These lives are similar to those being achieved in Auckland, as shown earlier in figure 2.4.

The data tends to suggest that the current performance of dense-graded asphalt in New Zealand is similar to that being achieved in the US. Austroads (Rebbechi and Sharp 2009) suggests a minimum of 8 years and is in the same range as expected for the higher traffic volumes in New Zealand.

The lives reported by Nichols (2004) of 12 to more than 20 years for thin asphalt and thin SMA are significantly greater than those being achieved in New Zealand. It must be noted that the Nichols data applies to trunk roads that would be surfacing on structural asphalt, where fatigue cracking would be expected to be minimal. The New Zealand data will include thin asphalt surfacings on a granular base, which would be expected to suffer fatigue cracking.

3 Whole-of-life costs of thin asphalt surfaces

3.1 Construction costs

Typically, a roading engineer will decide on the surfacing based on the lowest cost. This will normally be a chipseal unless there is high traffic stress, consent requirements, noise limitations or a policy reason not to.

The cost of the different surfacings depends on a wide range of factors, especially the quantities of materials required and the transport distances involved. Table 3.1 gives an approximate ranking of costs based on construction cost data from Auckland City and Hawke's Bay. The default surfacing is a chipseal that is the 'typical' chipseal – for the urban area this is taken as a grade 4 and has a cost of '1' seal; for the rural areas it is a grade3/5 seal. The ratios are relative for the 'urban' or 'rural' settings and do not imply that the cost of grade 4 in an urban area is the same as grade 3/5 in a rural area. This is highlighted as a cost of '1' treatment for the urban or rural environment. In high-demand areas, polymer modification of the binder (PMB) is often used. Where a PMB is used then the ratio should be increased by 2. That is, a PA 10 OGPA in a rural environment will be approximately 6.5 times more expensive than a grade 3/5 chipseal.

Table 3.1 Typical ratio of surfacing costs for urban and rural surfacings

Comparison of costs			
	Surfacing aggregate size	**Urban**	**Rural**
Chipseals	6	0.6	0.5
	5	1	0.9
	4	1	0.7
	3		
	2	N/A	
	4/6	1.1	0.8
	3/5	1.3	1
	2/4	1.6	1.2
Asphalt	10	4.5	N/A
	14	5.5	4
	20	6.5	5
OGPA	PA 10	6	4.5
	PA 10HS	6	4.5
SMA	10	8	6
	14	10	7.5

If the pavement structure is such that the asphalt type surfaces would deflect too much, leading to premature cracking, then the pavement structure would need to be strengthened. The cost of using a structural (rigid) layer of asphaltic concrete at 100mm thick would increase the cost by a factor of at least 20.

For example, a straight urban road carrying a traffic volume of 15,000vpd could be surfaced for say \$4/m^2 with a grade 4 chipseal. If the pavement deflections were high and a lower-noise surfacing was required, then the cheapest option would be a cape seal, as this could withstand the flexing.

If this was not acceptable then an asphalt mix could be used. However the pavement would need to be strengthened and therefore the cost could be 20 times greater; ie \$80/m^2. For a 1-kilometre section 10 metres wide, then the cost would be \$40,000 for a chipseal and \$800,000 for the asphalt.

This example illustrates that the use of a hotmix asphalt surfacing can be extremely expensive if the existing underlying pavement structure has not been designed for an asphalt-type surfacing.

3.2 Whole-of-life costs

The whole-of-life agency costs associated with the choice of treatment is a function of the discount rate (currently 8%) and the life achieved. When comparing treatments, the higher-cost treatment needs to last longer in order to be economically justified.

The typical lives of road surfacings were given earlier in table 2.1. It can be seen that the typical life of a thin asphalt surfacing in higher-traffic areas is 10–12 years and a grade 4 chipseal is 4–6 years. Although the asphalt life is double that of the chipseal, the cost from table 3.1 for a 10mm asphalt is approximately 4.5 times greater. This significant extra cost makes chipseal the preferred economic option when net present value (NPV) of the construction and maintenance costs are considered. It has been assumed that the maintenance costs are similar.

The above discussion considered the agency costs and not the intangibles. A more rigorous whole-of-life cost, as outlined in the (2010) NZTA *Economic evaluation manual* (EEM), considers the following:

- construction costs
- construction-vehicle delay costs
- pavement maintenance costs
- vehicle operating costs
- environmental costs.

An analysis of chipseal and hotmix asphalt road surfaces commonly employed in two road network areas in New Zealand was undertaken by extending the previously developed software called ‘TNZ Surfacing Selection Algorithm’. The applicability was improved so the software could be used to analyse several options with different resurfacing at different ages within the life of the pavement. The resulting economic analysis facilitated the output, including benefit–cost ratio (BCR) and incremental benefit–cost ratio (IBCR).

Residents and motorists often prefer an asphalt surfacing rather than a chipseal, mainly because it is quieter, but other significant factors are loose chips and bleeding. The smooth asphalt surface is also favoured by pedestrians and cyclists. The choice of a more expensive asphalt surface over a chipseal can be a road controlling authority (RCA) policy decision, but at higher traffic volumes it can also be justified on whole-of-life costs.

The ‘surfacing selection algorithm’ was originally developed in previous research (McLarin 1996). This algorithm calculates the whole-of-life costs based on the Land Transport NZ (now NZTA) EEM (2010) and takes into account factors such as fuel consumption, tyre wear, pavement deflection and noise in calculating both direct and indirect costs of various pavement surfacings. It has since been enhanced to include maintenance costs and compare the whole-of-life costs, using a combination of surfacings over a user-defined analysis period. The model has the ability to perform an incremental benefit–cost analysis based on the 'annual cost method', thereby enabling the optimum surface to be selected as a function of traffic composition and volume.

A comparison of the construction costs with the maintenance and intangible costs needs to be performed over a set analysis period. In order to ensure the analysis is robust, the following procedures set out in the NZTA EEM (2010) need to be followed.

3.2.1 Construction costs

The costs of materials depend upon their geographic location and the transport distance. These are input by the user. For example, if the choice of the surface is SMA, then the cost is dependent on the transport distance between the plant and the construction site. If the transport distance is too far, then the option may not be viable.

3.2.2 Vehicle delay costs

An algorithm that estimates the vehicle delay costs associated with construction has been developed and included in the software. It assumes that traffic will be controlled at 30kph for 48 hours after a seal is constructed, but this will not occur with a hotmix surface. It is possible to choose to construct the surface at night, in order to minimise vehicle delay costs.

3.2.3 Pavement maintenance costs

Pavement maintenance costs need to be included in any total cost comparison. Data from state highways is available and has been included in the algorithm.

3.2.4 Vehicle operating costs (VOC)

The EEM (2010) notes that the vehicle operating costs depend on the following:

1. base running costs by speed and gradient
2. road roughness costs
3. road surface texture costs
4. pavement elastic deflection costs
5. congestion costs
6. bottleneck costs
7. speed change cycle costs.

In a comparison of different surfaces, it is assumed that the geometry is constant and thus in normal operation, items 1, 5, 6 and 7 are constant.

A pavement will deteriorate with age. The road roughness will depend on the strength of the pavement, and the road roughness progression associated with a user variable 'structural number' is used in the calculation. This is recognised as an approximation only, and more research into the rate of pavement deterioration is needed. The decrease in roughness when the surface is surfaced with hotmix is estimated but there is no change in roughness when the surface is sealed.

The pavement elastic deflection cost, which affects heavy-vehicle operating costs, is also estimated from the structural number.

3.2.5 Environmental costs

The software uses the EEM valuation for the calculation of the cost of noise. The manual has the costs as $190 per household per dB per year, or $65 per person per dB per year. The figures are based on an average value for an urban property of $150,000. The EEM notes that:

> *A reasonable figure for New Zealand is suggested as being 1.2% of the value of properties affected per dB of noise increase. This figure should be applied in all areas, since there is no reason to suppose that noise is less annoying to those in areas with low house prices. So, a constant cost should be applied to all areas.*

At the time of this research, the current average house price was $400,000, and we have used this average price in our analysis.

Other costs (eg from vehicle pollution) are not included.

3.3 Software outline

The software and associated algorithms include the following:

1. Assumed data input – contains the current values of the main variables used in the calculations, such as discount rate, material cost data and road usage categories.
2. A surfacing strategy is input over the analysis period.
3. Road surface cost – material, labour and life expectancy data – are combined to calculate the annual cost per lane-metre of road.
4. Variation of macrotexture with time – the weighted average macrotexture over the life of the resurface is calculated.
5. Calculation of user fuel costs – the present value of fuel expenditure over the life of the surface is calculated from the known variation of average fuel consumption with average macrotexture. A contribution from road roughness for this period is also included.
6. Calculation of user tyre costs – similar to the procedure used for fuel costs.
7. Calculation of road roughness dependent user costs – the present value of fuel, tyre and repairs/depreciation expenditure over the evaluation period is calculated from predicted average values of road roughness for this period.
8. Calculation of delay costs at resurfacing works – delay costs are calculated from an analysis of a two-lane, 1km resurfacing operation. Resurfacing can be performed either in the daytime or at night, with a subsequent difference in road user costs.
9. Calculation of noise costs – costs of vehicle exterior and interior noise are derived from a relationship between noise levels, vehicle speeds and surface macrotexture. Hedonic pricing based on property value changes is used for the exterior noise costing, while the cost of noise suppression is used for the interior noise costing.
10. Maintenance costs of the various surfacings are derived from NZTA or RCA data.
11. Calculations of total costs – the cost from parts 3–10 are combined.

12 Benefit–cost analysis results – the benefit and costs for the analysis are calculated from the total costs and the resurface construction costs.

3.4 Analysis

The software was used to assess two different network areas: major city streets (referred to as 'City') and a rural road state highway in a 50kph area near housing (referred to as 'Rural'). The material and construction costs were obtained from the local network consultant and are given in table 3.2. The choice of treatments is constrained by the locality and speed environment. For example, OGPA is not used in the City due to its low shear strength and low resistance to oil from buses, and SMA is not currently available in the rural state highway area. Also a mix 10 would not be used on a state highway (even in a town), as the macrotexture would not comply with NZTA requirements. In both areas, two-coat seals are used rather than single coats, and thus the comparison is between a chipseal and a mix option, and the selected surface options are not the same for each network. The surface options analysed in this example are tabulated in table 3.3 for a traffic volume range of 50–50,000AADT. The life of each surface was taken from the RAMM default values.

The economic analyses were done for a range of traffic volumes – ie 50, 250, 1000, 3000, 6000, 15,000, 20,000AADT – in order to define which option is the best for the road that has different levels of traffic.

The calculations are based on a 1km resurfacing section of road, with resurfacing works carried out during the day-time only.

Table 3.2 Cost of pavement surfacing construction

		Material and construction costs $/m^2	
Code	**Treatment**	**City**	**Rural**
Ms Mx 10	Mix 10 with membrane seal	13.05	N/A
Tack Mx 10	Mix 10 with tack coat	11.88	N/A
MEM SMA	SMA + membrane	31.84	N/A
Mm OGPA	OGPA + membrane seal	N/A	13
Tc OGPA	OGPA + tack coat	N/A	10.75
TWO 2/4	Gd 2/4 chipseal	N/A	4.5
TWO 3/5	Gd3/5 chipseal	3.9	4.1
TWO 4/6	Gd 4/6 chipseal	3.39	N/A
Gr 2	Gd 2 chipseal	N/A	3.71

The combination of surfacing treatment and timing were developed for each traffic volume using the default lives from RAMM. An example for 6000v/l/d (vehicles per lane per day) is given in table 3.3.

Table 3.3 Surfacing options for City and Rural for a traffic volume of 6000v/l/d[a]

Year	City			Rural	
	Opt A1	Opt A2	Opt A3	Opt N1	Opt N2
1	TW 03/5	Ms Mx 10	MemSMA	TW02/4	MmOGPA
2					
3					
4					
5					
6					
7					
8					
9					
10	TW04/6				TcOGPA
11					
12				TW03/5	
13			TacSMA		
14		TackMx10			
15					
16					
17	TW 03/5				
18					
19					TcOGPA
20					
21				TW02/4	
22					
23					
24					
25			TacSMA		

a) See table 3.2 on previous page for codes.

Table 3.4 gives the results of the analysis with noise costs both included and excluded. For the City area, when noise is not considered the use of a10mm hotmix asphalt does not obtain a BC of over 2 until the traffic volume is 15,000vpd. However, the high cost of noise swamps the analysis to such an extent that it can be justified for traffic volumes as low as 50v/l/d.

On the rural highway through a small town, the low-noise option is OGPA and this can also be justified for the low traffic volumes. However, when noise is not considered then the traffic volume needs to be around 20,000vpd before the advantages of the lower rolling resistance of OGPA and its ability to smooth a pavement at each treatment becomes such that its use can be justified.

Table 3.4 Analysis of surface options for the city (IBCR[a] and BCR[b])

Traffic (v/l/d)	50		250		500		1000		3000		6000		15,000		20,000	
Description	**IBCR**	**BCR**	**IBCR**	**BCR**	**IBCR**	**BCR**	**IBCR**	**BCR**	**IBCR**	**BCR**	**IBCR**	**BCR**	**IBCR**	**BCR**	**IBCR**	**BCR**
Noise cost considered																
Opt-1: TWO3	0	0	0	0	0	0		0		0		0		0		0
Opt-2: MsMx10		5.5		4.5		4.2		3.9		3.6		3.9		5.6		13
Opt-3: MemSMA		0.4		0.3		0.3		0.2		0.2		0.2		0.4		1.6
Noise cost ignored																
Opt-1: TWO3	0	0	0	0		0		0		0		0		0		0
Opt-2: MsMx10		0		0.1		0.2		0.3		0.8		1.3		3.1		10
Opt-3: MemSMA		0		0		0		0		0.1		0.1		0.2		1.5

a) Incremental benefit-cost ratio.

b) Benefit-cost ratio.

Table 3.5 Comparison of BCR for the rural highway

Traffic (v/l/d)	50	250	500	1000	3000	6000	15,000	20,000
Noise cost considered								
Opt-1: TWO2/.04								
Opt-2: MmOGPA	4.2	3.6	3.4	3.2	2.9	3.1	3.1	9
Noise cost ignored								
Opt-1: TWO2/.04								
Opt-2: MmOGPA	0	0.1	0.1	0.2	0.4	0.6	0.7	4.3

3.4.1 Sensitivity to noise costs

The noise costs are a considerable influence on the preferred option of road surface. The NZTA (2010) EEM derives the cost based on a percentage of the median house price, suggesting 1.2% of the median house price per dBA of noise change. Table 3.6 illustrates the effect on the benefit-cost analysis of a change in house value for the rural example, where the do-nothing option is a two-coat seal and the comparison is with OGPA. The change in house value from $150k to $400K changed the traffic volume for a benefit-cost of 2 to go from 15,000vpd to under 50. The up and down trend in the BCR is associated with the change in surfacing default life for the change in traffic volumes that occurs in step changes in the analysis.

Table 3.6 Effect on BCR of median house price on noise costs for the rural example

	BCR		
Traffic (v/l/d)	**No noise**	**$150k value**	**$400k value**
50	0.0	1.6	4.2
250	0.1	1.4	3.6
500	0.1	1.4	3.4
1000	0.2	1.3	3.2
3000	0.4	1.4	2.9
6000	0.6	1.7	3.1
15,000	0.7	1.9	3.1
20,000	4.3	7.7	9.0

For the examples examined in this report, the road roughness was assumed as 60NASSRA. At this level, repair and maintenance costs for vehicles are taken as zero in the EEM. Therefore the costs included were:

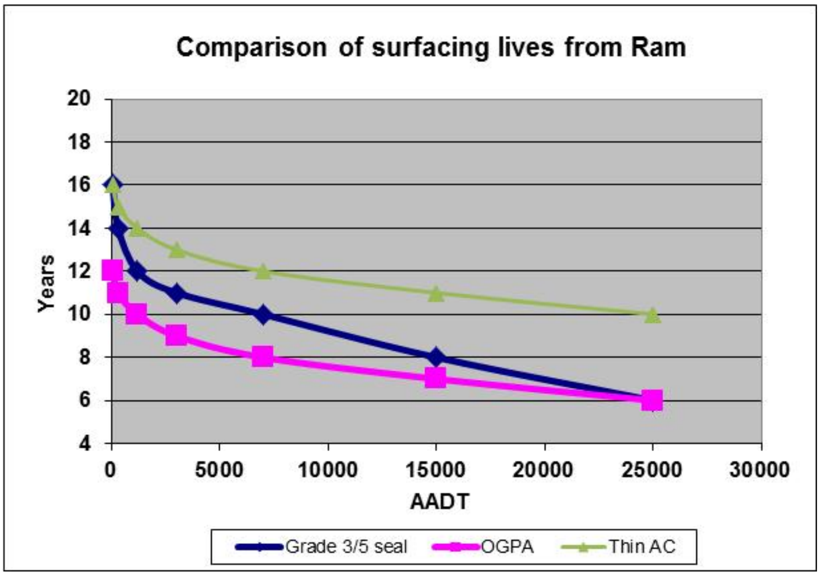
1 fuel
2 tyres
3 traffic delay during resurfacing
4 noise
5 surfacing and maintenance costs.

Table 3.7 gives the breakdown in the costs of excluding noise for the OGPA example on the rural road, in terms of the NPV over a 25-year period, in dollars per metre of road. It can be seen that the VOC increase exponentially with traffic volume increase, but the increase in surfacing and maintenance costs is less dramatic. In comparing different treatments at higher traffic volumes over the 25-year period, the tyre and fuel cost difference between a chipseal and an asphalt is not as influential as the difference that is associated with traffic delay. At higher traffic volumes, the expected life of a chipseal drops off more markedly than for an OGPA or thin asphalt, as shown in figure 3.1. Thus the increased cost of asphalt surfacings is offset by their longer lives at high traffic volumes.

Table 3.7 NPV cost in $/m over a 25-year analysis period for OGPA on the rural road

Traffic (v/l/d)	Surface + maintenance	Fuel	Tyres	Delay
50	6.99	1.8	0.24	0.002
250	7.35	9.0	1.20	0.009
500	7.52	18.0	2.38	0.018
1000	7.72	36.0	4.71	0.038
3000	8.06	107.8	13.95	0.133
6000	8.31	215.3	27.67	0.345
15,000	8.68	537.6	68.39	0.996
20,000	8.81	716.5	90.87	2.547

Figure 3.1 Comparison of RAMM default lives for a chipseal and two asphalt surfacings



3.5 Conclusion

The two examples of analysis of chipseal and hotmix asphalt road surfaces commonly employed in New Zealand showed that the cost of noise can swamp the analysis if the EEM value of 1.2% of house values is used. Using typical construction costs, the extra cost of asphalt over chipseals cannot be justified on maintenance costs, traffic delay and VOC unless the volume is greater than about 15–25,000vpd.

4 Fatigue of thin asphalt layers

A fundamental assumption was that thin layers of hotmix would fail in fatigue through the classic fatigue mechanism associated with the tensile strain at the bottom of the hotmix layer.

When examining the literature it became apparent that the classical theories state that thin layers are not in tension bur rather in compression under the tyre. This is why Austroads (Jameson and Shackleton 2011) does not have deflection criteria for chipseals and in the manual, the deflection criteria stop at 40mm. it is assumed that layers less than 40mm will not crack in classic fatigue (see figure 4.1 below).

However the results of the lives achieved by surfacings in section 2 showed that cracking was the predominant failure mode, which suggests that the classic theory may not be taking into account other factors.

Figure 4.1 Example of Austroads (Jameson and Shackleton 2011) deflection criteria

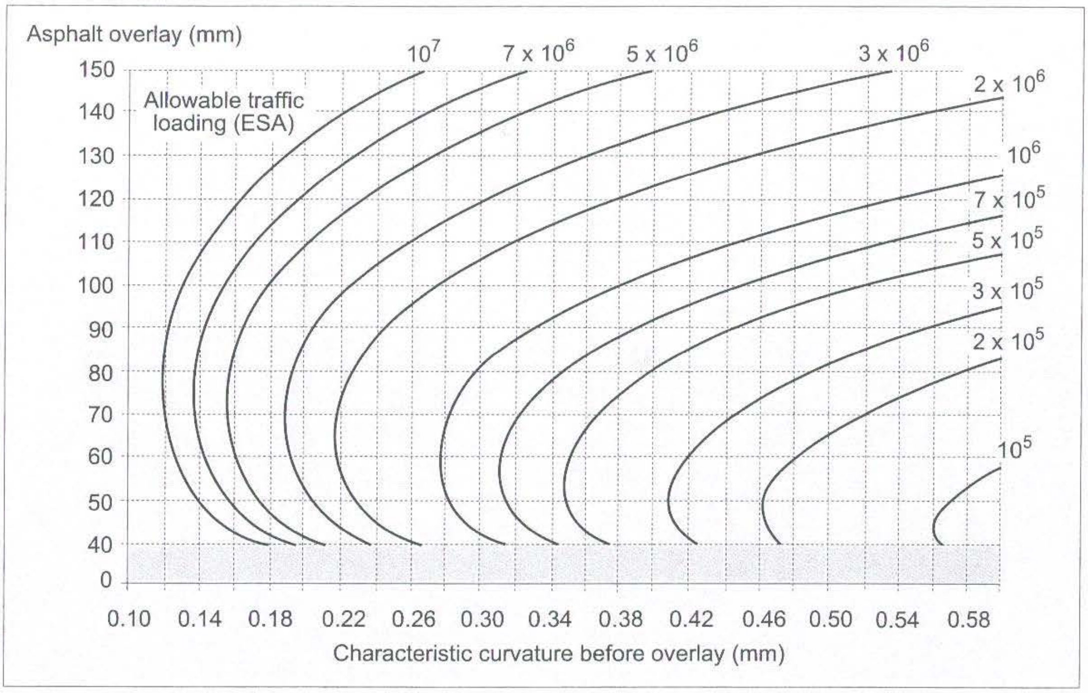


Figure A6.2.2 Asphalt overlay fatigue lives WMAPTs 20–25°C

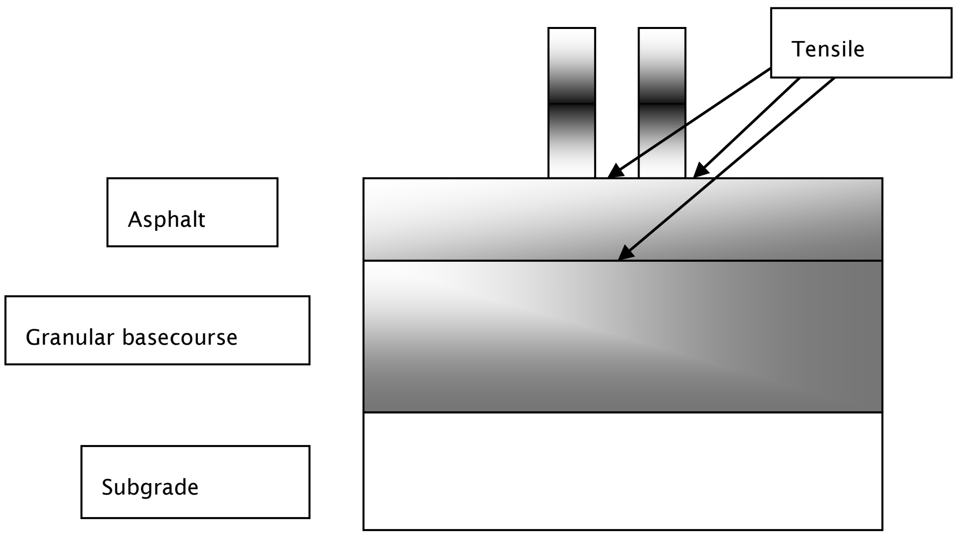
An analysis was performed to determine the position and magnitude of the tensile strains developed under vehicle loading, by using the stress distribution under real tyres and finite element analysis.

This research was aimed at investigating the effect of asphalt thickness on two different pavements under dual tyre load. The tyre imprint and the vertical, transverse and longitudinal stresses values were obtained from the data published in the literature. Strains were determined at the locations shown in figure 4.2

Table 4.1 Pavement configuration

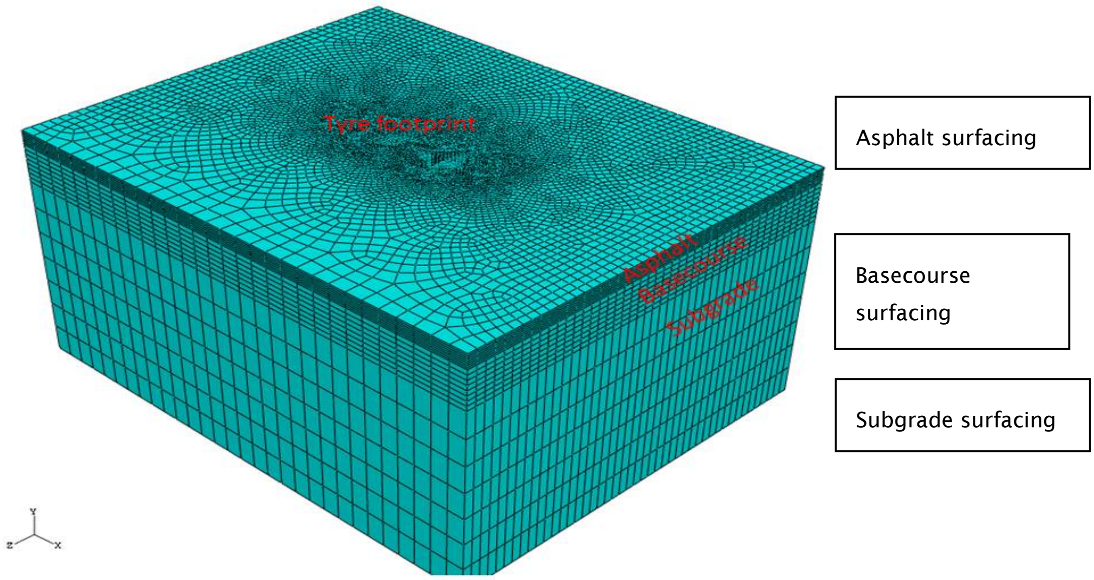
Asphalt thickness (mm)	Pavement 1		Pavement 2	
	Basecourse thickness (mm)	Subgrade mod MPa	Basecourse thickness (mm)	Subgrade mod MPa
20	220	150	400	50
40	220	150	400	50
60	220	150	400	50
80	220	150	400	50

Figure 4.2 Sample model and location of strain determination



The general-purpose FE software ABAQUS (Standard v6.7) was used to set up a forward finite element model in three dimensions, as illustrated in figure 4.3. To reduce the computational effort, use was made of the symmetry of the geometry, and loading only a quarter of the model.

Figure 4.3 Finite element models



4.1 Material properties

In this work Uzan's equation

$$E_1 = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3}$$

(Equation 4.1)

was used for the modelling of the stress-dependent granular basecourse. Uzan's model takes into account both the confining and wheel-load deviator stresses and thus deals very well with the modulus changes in an unbound aggregate base layer. The material constants used for modelling the basecourse were taken from repeated-load triaxial tests performed on Canterbury M/4 (2006) basecourse and are considered typical of a premium basecourse. The values used were:

K1=1800

K2=0.75

K3=-0.35.

In addition the material was modelled as isotropic. This is similar to values used by Arnold et al (2008).

The asphalt was modelled as an isotropic linear elastic material with a modulus of 3750MPa. The subgrade was also modelled as an isotropic.

The modelling did not consider the anisotropic behaviour of the basecourse and subgrade as included in the Austroads design methodology.

4.2 Tyre loading

As the strains that develop near the surface of a pavement are known to be influenced by the tyre configuration, two tyres were modelled. The first, referred to as the Austroads tyre, was an 11R22.5 tyre, which is the most common tyre used in Australia. The stress distribution under this tyre was measured in South Africa and the derivation of an imprint in 8mm squares was performed by Gonzalez (2009).

The second loading was taken from the results of research at CAPTIF and is referred to as the CAPTIF tyre. The measurement of the stress was performed by Douglas (2009) and the derivation of the loading pattern was performed by Werkmeister and Gribble (2009). The stress distribution on the two tyres on the CAPTIF tests appeared to be different but no attempt was made to normalise the data. The CAPTIF tyres could therefore be regarded as typical, while the Austroads are more the ideal.

The third loading pattern that was investigated was circular loading as described in Austroads (Jameson 2008). This loading was used to compare the results of a Circly analysis with the more sophisticated methods.

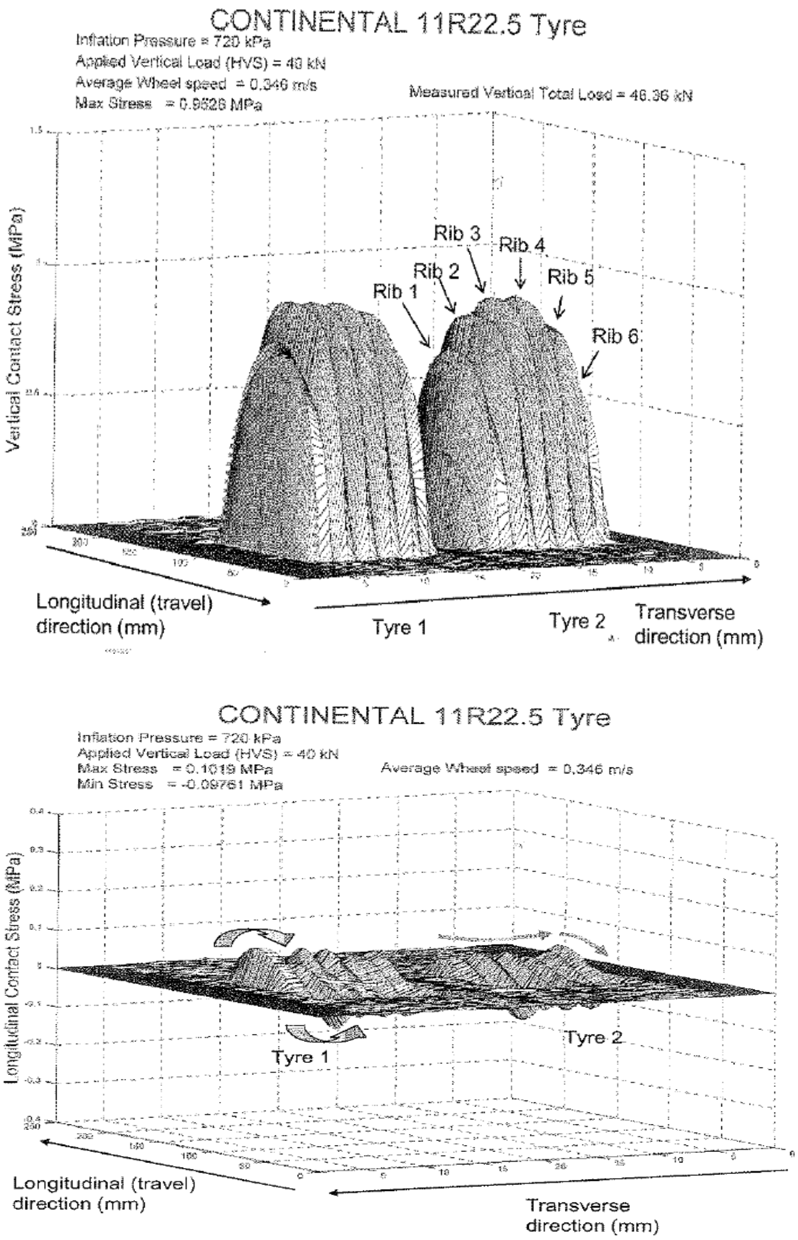
All the loadings were based on a 20kN wheel load.

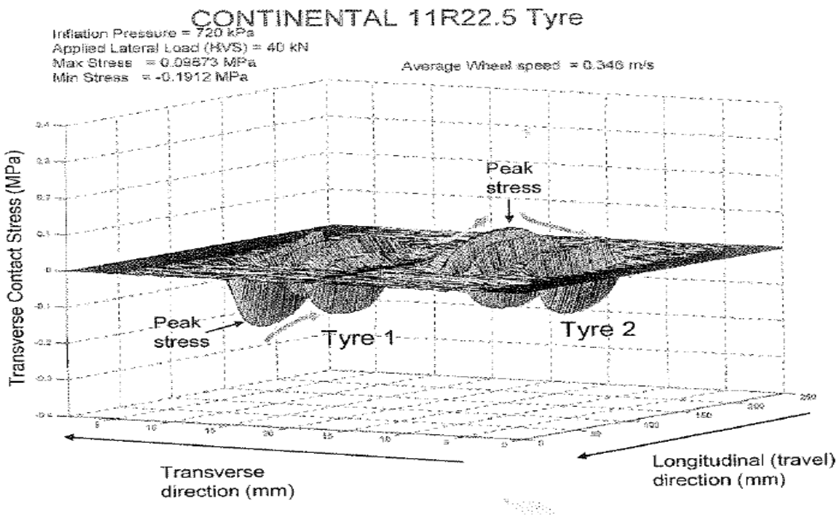
4.2.1 Austroads tyre

Figure 4.4 is a picture of the Austroads tyre and figure 4.5 gives a pictorial view of the vertical horizontal and transverse stresses

Figure 4.4 Austroads tyres

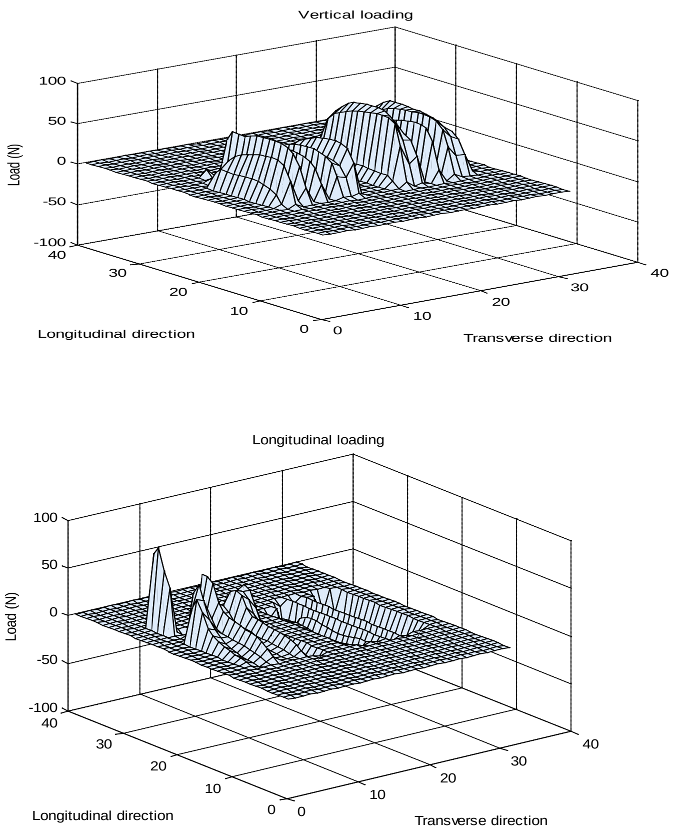
Figure 4.5 The tyre imprint and the loads (vertical, transverse and longitudinal) for the Austroads tyre

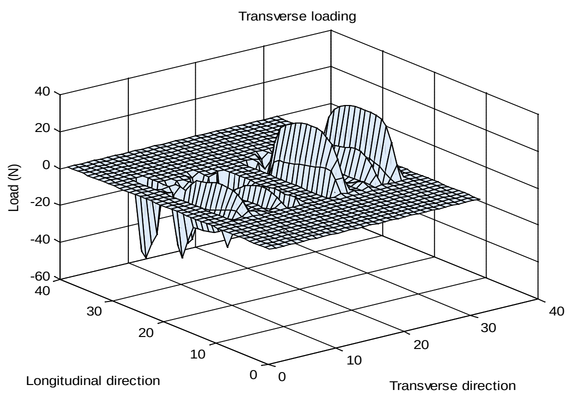




4.2.2 CAPTIF tyres

Figure 4.6 The tyre imprint and the loads (vertical, transverse and longitudinal) for the CAPTIF tyre





The stress distribution for the CAPTIF tyre was very non-uniform in that the stress on the right-hand tyre was higher than on the left-hand one. The data was taken from a report by Werkmeister and Gribble (2009), who commented that they did not know the reason for the difference, but that the wheels were those driven on the machine and the higher-loaded tyre was the inner wheel.

4.3 Results

The results for the first pavement configuration with 220mm basecourse are presented in this section.

Figure 4.7 shows the horizontal tensile strain at the base and top of asphalt against asphalt thickness. As asphalt thickness was increased, horizontal strain at the bottom of the layer increased from a negative value (compression) to positive values (tension). A peak strain level was reached at approximately 60mm thickness. This is in agreement with the New Zealand practice of considering that the asphalt thickness in the 50–80mm range is the pessimism thickness for fatigue performance. At thicknesses greater than 80mm, the structural support given by the asphalt results in a significant decrease in the tensile strain.

Figure 4.7 Strain at surface and base of asphalt for the Austroads tyre imprint

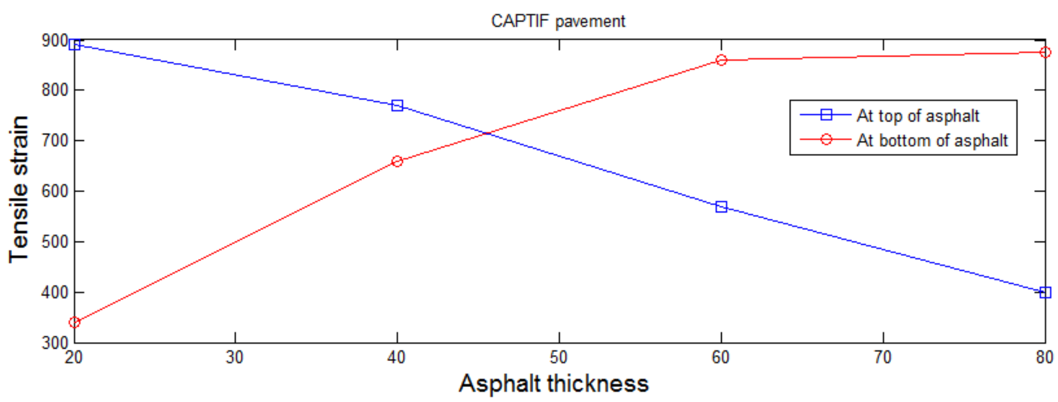


Figure 4.8 gives similar results for the CAPTIF tyre, which showed a similar trend to the Austroads tyre, although the values of strain were significantly higher. The non-uniform loading of the CAPTIF tyres contributed to these very high strains. The results are in agreement with the analysis of Werkmeister and Gribble (2009), who noted that the strains distribution under the CAPTIF wheels resulted in high vertical

strains near the pavement surface, compared with a uniformly loaded area. The results of their finite element analysis (FEM) are shown in figure 4.9

Figure 4.8 Strain at surface and base of asphalt for the CAPTIF tyre imprint

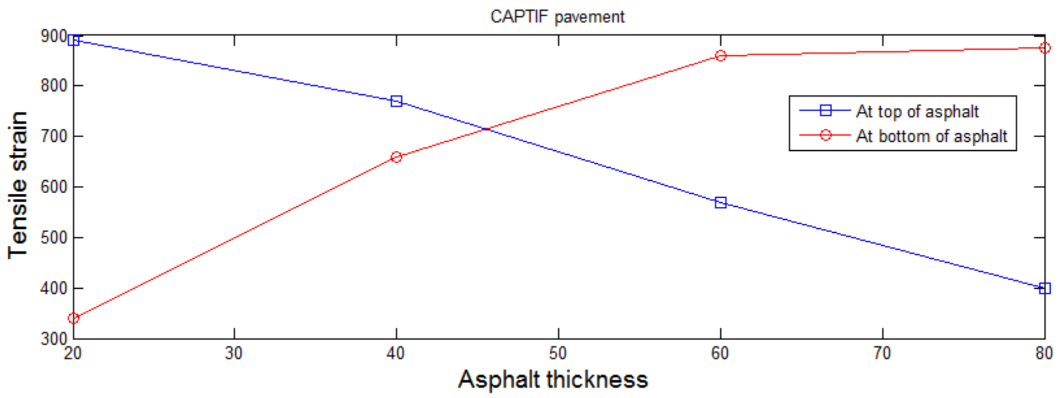
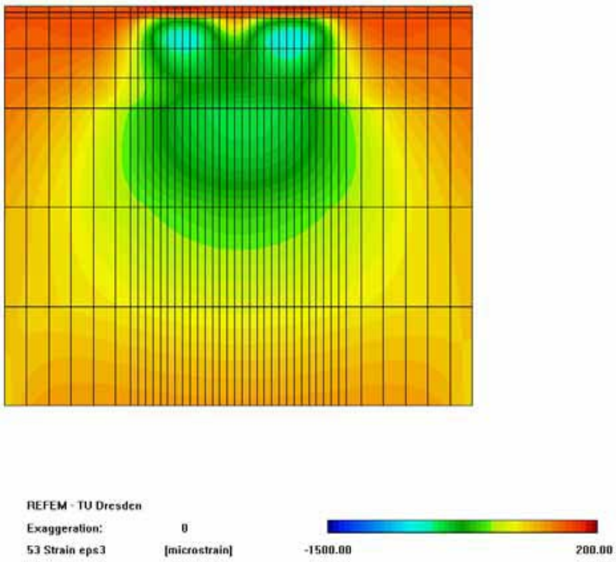
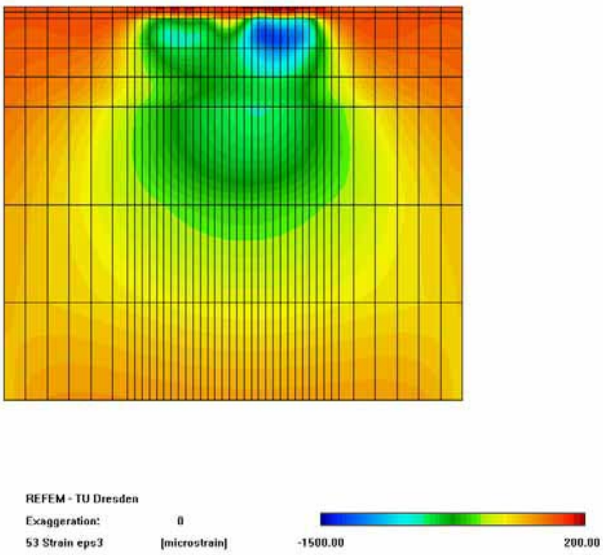


Figure 4.9 Vertical elastic pavement strains under a 40kN dual tyre load with tyre inflation pressure of 690kPa (Werkmeister and Gribble 2009)

Uniform vertical contact pressure



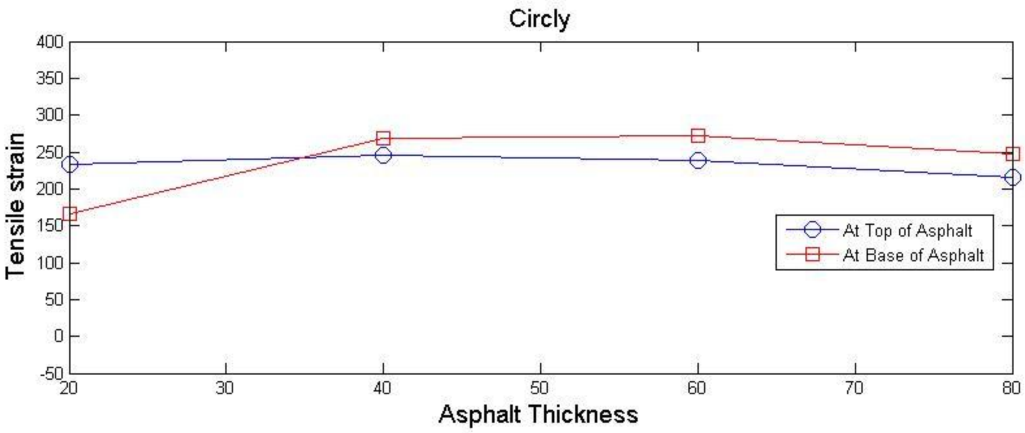
Modelled vertical, transverse, and longitudinal contact stresses



Generally, the maximum tensile strain is found at the bottom of the asphalt layer when the asphalt thickness is larger than 45mm, but for thinner layers, the maximum strains are at the top. The results confirm the Austroads (Jameson and Shackleton 2011) assumption that in thin layers, the strain at the bottom of a thin asphalt layer tends towards compression. This clearly shows that the thin asphalt layer leads to top-down cracking and thick asphalt layer leads to bottom-up cracking.

The Circly analysis also tended to support the analysis – ie in thin layers the maximum tensile strain was at the top. Figure 4.10 illustrates the results.

Figure 4.10 Strain at surface and base of asphalt for the Circly analysis



The Circly analysis underestimated the tensile strain at the surface when the thickness of the asphalt was very small. The maximum tensile strain was found at the bottom of the asphalt layer when the asphalt thickness was larger than 35mm. This value was lower than the FEM-based calculation with the real tyre imprint. Also, the difference between the tensile strain at the bottom and at the surface was very small when compared with the results for the Austroads and CAPTIF tyres when the asphalt thickness was more than 40mm.

Table 4.2 summarises the results of pavement 1 and includes a measure of pavement curvature D_0-D_{200} (the pavement deflection between the tyres and that 20mm in the longitudinal direction. The effect of the uneven loading at CAPTIF is evident from the maximum deflection results. The D_0-D_{200} results are not the same as those used in the Austroads pavement rehabilitation manual, which are appropriate for FWD measurements.

Table 4.2 Tensile strains and deflection results for pavement 1

	11R22.5 tyre				CAPTIF tyre				Circly			
Asphalt thickness (mm)	Top	Bottom	D0 (mm)	D200 (mm)	Top	Bottom	D0 (mm)	D200 (mm)	Top	Bottom	D0 (mm)	D200 (mm)
20	390	-5	0.48	0.24	890	340	1.62	1.07	234	166	.63	.37
40	275	225	0.51	0.29	770	660	1.6	1.05	246	268	.57	.36
60	115	250	0.48	0.31	570	860	1.51	1.02	239	272	.50	.34
80	20	235	0.43	0.3	400	875	1.39	0.99	215	248	.44	.32

Table 4.3 Tensile strains and deflection results for Pavement 2

Asphalt thickness (mm)	11R22.5 tyre				Circly			
	Top	Bottom	D0 (mm)	D200 (mm)	Top	Bottom	D0 (mm)	D200 (mm)
20	440	210	0.89	0.62	135	165	.92	.77
40	180	220	0.88	0.55	134	268	.88	.75
60	70	320	0.83	0.62	61	282	.83	.72
80	-30	265	0.74	0.58	48	263	.78	.70

The results of the FEM showed that the maximum tensile strain in thin layers of asphalt occurred at the top and could be significantly greater than the strains generated at the bottom of the thicker layers. The level of the strains generated were dependent on the tyre properties and pressures and these influences decreased with the depth of the asphalt.

There was not a consistent pattern in the results and there was a significant difference in the strain and the deflection estimated for the Austroads tyre and the CAPTIF loading. This illustrates the significant difference that can occur between theory and practice, and the dangers of relying on theoretical analysis in pavement design. This statement can be further supported by test results from CAPTIF, where thin layers of OGPA were found not to crack under accelerated loading (Alabaster 2006). This result does not match field experience or the expectations from the analysis reported here.

In the determination of the fatigue parameters to be used in design, Austroads (Jameson 2008) used linear elastic theory coupled with an assumption of a uniformly loaded circular tyre footprint. The strain levels thus calculated were compared with a laboratory fatigue test and a shift factor was developed, based on agreed levels of risk. There is therefore no straightforward way to go from strains calculated from FEM modelling and derive an appropriate shift factor for use in pavement design. The analysis performed in this current project does, however, show that Circly analysis gives the same trends as the FEM, in that the maximum strains appear on the surface for thin layers.

The implication for pavement design is that strains at the surface of thin layers of asphalt can be the critical area. The assumption that thin layers of asphalt do not need to have a deflection criteria therefore requires further investigation. The strains generated will be influenced by the tyre properties and the pavement deflection characteristics. For pavement design it is therefore recommended that the Circly-calculated results for strains at the top of the pavement, where they are higher than those at the bottom of the asphalt layer, be used as the ‘critical’ strains for fatigue in the pavement design.

5 Shear strength

The perceived poor shear performance in Auckland of asphalt manufactured to the 1975 version of the TNZ M/10 specification prompted the industry to explore and then introduce the Austroads mix design methodology (Ribbechi 2007). Since the mid-1960s, New Zealand had used the Marshall method of mix design and the dense grading envelopes used in the US, with considerable success. It was thought that the increase in tyre pressures and heavy-vehicle traffic may have resulted in the Marshall mix design methodology being inappropriate for modern traffic.

In the US it was recognised in the 1980s that the use of empirical mix design methods such as the Marshall method could be improved through a more rational approach. In the 1990s the US embarked on the SHRP program, with the objective of improving the performance of pavements through the development of a more fundamental range of tests. The design methodology is referred to as Superpave.

The Austroads mix design methodology is similar to that developed in the US in the 1990s. The main emphasis in the design factors have been described by Oliver (2008) as follows:

> *The balance in designing the most appropriate mix is to balance the shear strength with the fatigue resistance and durability of the material. Choosing the most appropriate mix will result in the most cost-effective overall performance of the pavement. The traditional mixes used in New Zealand and specified in the M/10 specification were introduced for use on the motorway system – they were thus geared towards having high fatigue resistance but with adequate stability. On pavements with fast moving traffic stability issues are not as prominent compared to slow moving heavy traffic in urban speed and such as intersections and motorway ramps.*

An example of the different drivers for mixes in different areas, and methods for achieving these, is shown in table 5.1, which is from Austroads *Guide to pavement technology part 4B – asphalt* (Ribbechi 2007).

Table 5.1 Asphalt requirements for different uses

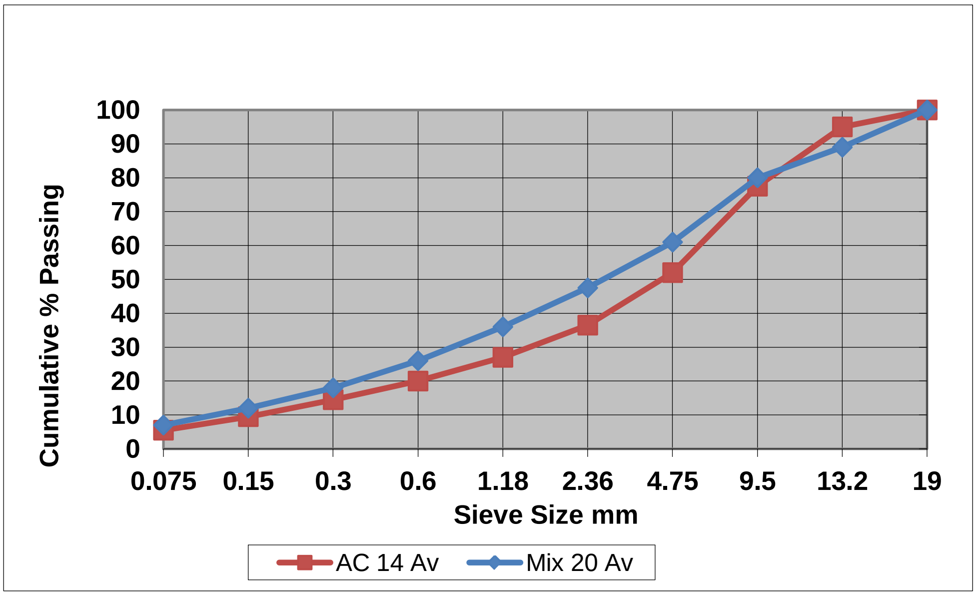
Residential mix	Highway mix
High density, low air voids High binder content Impermeable	Stable – resists deformation Binder content – suitable for cohesion and fatigue resistance
Main aim – workability	Main aim – stability
↓	↓
Finer grading Gap-type grading High binder content Smooth/round fines Soft grade of binder	Coarse grading Angular/crushed fines Rough-textured aggregates Hard grade of binder Modified binder

The table illustrates the direction taken by the industry in New Zealand to improve the rutting resistance of New Zealand mixes through using coarse gradings, ensuring the presence of crushed fines, and using harder grades of binders. In New Zealand main centres where rutting is a problem, most asphalt has been reduced from crushed quarried aggregate and thus the coarse and fine aggregate is angular. The changes introduced by the Austroads mixes have been mainly associated with the grading and the use of the gyratory compactor rather than the Marshall compactor.

These changes are similar to those proposed and adopted in the US through the SHRP program.

A comparison of the mean aggregate gradings for a traditional NZTA M/10 (2005) Mix 20 and an Austroads AC 14 is shown in figure 5.1. It can be seen that the proportion of fine aggregate (that passing the 4.75mm sieve) has been reduced from 61% to 52%. The grading of the AC 14 is close to the maximum density curve for this size material and thus the available space to include the bitumen is reduced.

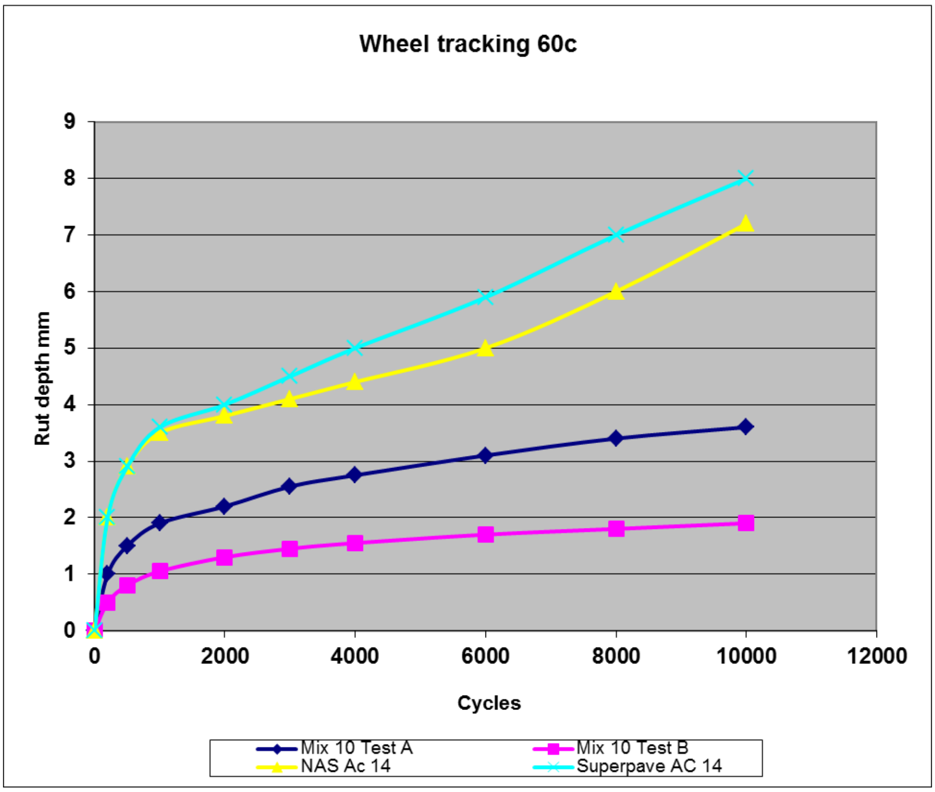
Figure 5.1 Comparison of Austroads and traditional NZTA mix grading



In New Zealand and Australia the current method of assessing the resistance of asphalt is through the use of the wheel-tracking test (Austroads 2006a and 2006b). This test uses a loaded wheel that is run across a specimen of the mix that has been compacted to 7% air voids, and the deformation with the number of loading cycles is measured.

Figure 5.2 shows a comparison of three mixes at 60C. The AC 14 and Superpave mix are coarser graded, and use a different aggregate source than the Mix 10. These mixes also have a larger maximum particle size than the Mix 10, which complies with the NZTA M/10: 2005 specification. In contrast to the assumption that the high coarse-aggregate mixes are more rut resistant, the Mix 10 is performing significantly better than the other mixes.

Figure 5.2 Comparison of three New Zealand mix wheel-tracking results



The results of the wheel-tracking tests have also been validated by trials in the US. Figure 5.3 is taken from test track trials at the National Centre for Asphalt Technology (NCAT), where the coarser mixes did not perform as well as the finer-graded materials (Choubane et al 2006).

Figure 5.3 Test track results from NCAT (Choubane et al 2006)

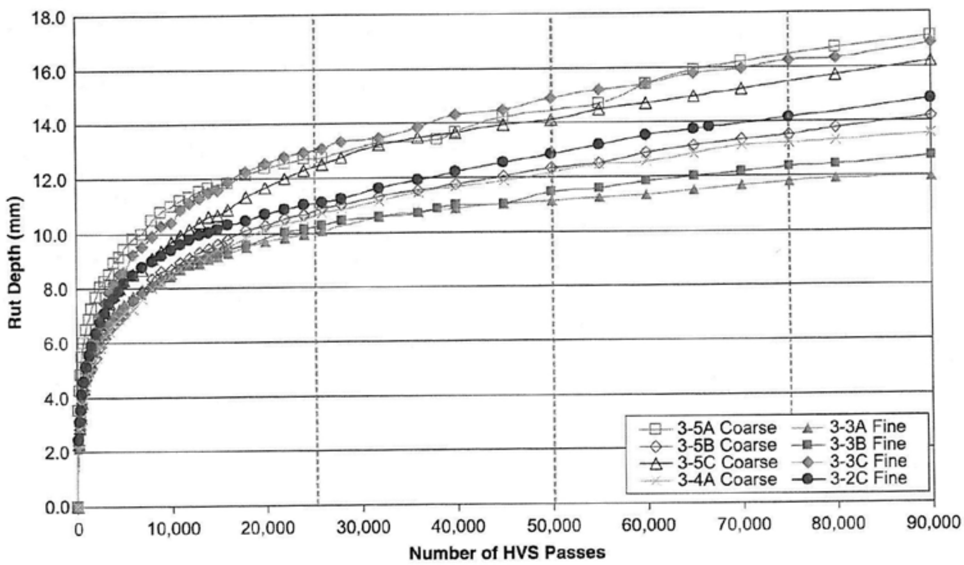


FIGURE 6 Comparison of rutting performance of coarse- and fine-graded mixtures in Experiment 3.

Choubane et al (ibid) reached the following conclusions:

- *When fine-graded mixes were compared to coarse-graded mixes they were equally resistant to rutting, less likely to be permeable, quieter, similar in friction, possibly easier to compact and higher in optimum asphalt content.*
- *The factor that most affected rutting of HMA*[2] *was the AC grade. The modified asphalt reduced the rutting by over 50% when compared to unmodified asphalt.*
- *SMA mixes had more rutting than Superpave mixes but neither had significant rutting.*

Kandhal and Cooley (2002) also compared a range of gradations on both the coarse and fine sides of the Superpave grading limits, using different aggregates and the same binder. Their conclusions were as follows:

> *Statistical analyses of the test data obtained by the three performance tests indicate no significant difference in rutting resistance of coarse- and fine-graded Superpave mixtures. It has been recommended that mix designs not be limited to designing mixes on the coarse or fine side of the restricted zone.*

These test results do not necessarily prove that the coarser-grade mixes that approach the maximum density curve are less rut resistant than the finer gradings, but do highlight that aggregate grading is only one factor.

If the rutting resistance is taken as a shear strength problem, then the classic Mohr Columb approach suggests that the strength is a function of the aggregate interlock and the cohesion imparted by the binder, as given in the following equation:

$$\text{Shear strength} = c + p \times \tan\theta \qquad \text{(Equation 5.1)}$$

Where:

C = cohesion in MPa

P = vertical stress

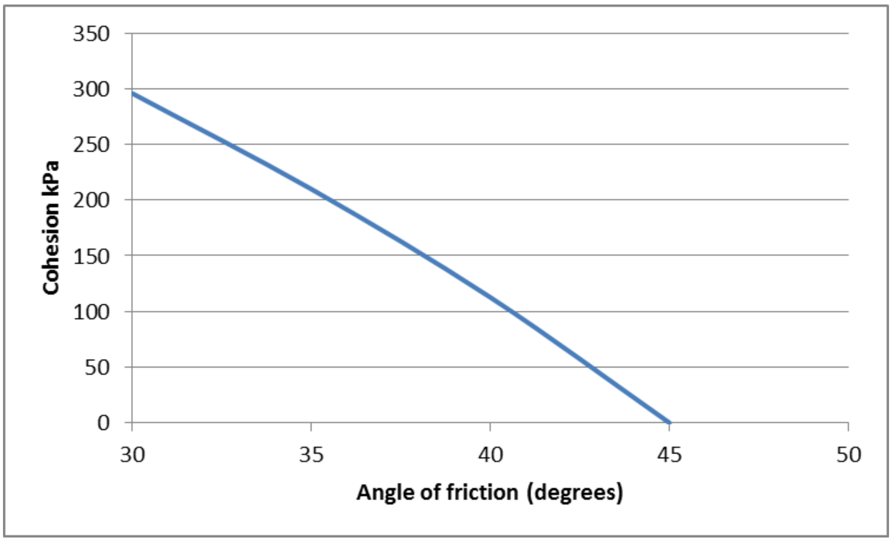
θ = angle of friction.

To obtain the shear parameters, a triaxial test is normally performed at a relatively slow strain rate. This approach was common in the early 1950s and was also employed by Fwa et al in 2004.

With a normal stress of 700kPa (typical of a heavy-vehicle load), the combination of cohesion and angle of friction that is required to prevent shear failure is shown in figure 5.4

2 Hotmix asphalt.

Figure 5.4 Combination of angle of friction and cohesion to resist a vertical load of 700kPa



The figure appears intuitively correct in that it is known that a crushed basecourse typically used under a thin chipseal layer on New Zealand roads can resist shear failure. This type of material will have an angle of friction of greater than 45 degrees and minimal cohesion – ie it would fall on the right of the line in the figure.

The triaxial approach gives a simple approach to shear failure but it does not duplicate that repeated loading under a wheel, and it also assumes that the layer is thick enough to generate a classic shear failure. It does, however, help to explain that at higher pavement temperatures and with slow traffic, where the binder viscosity is low the cohesion of the mix will drop and thus the shear strength will also fall. At lower temperatures and higher traffic speeds, the cohesion and thus the shear strength will rapidly increase.

A discussion on the shear performance of asphalt also has to address the effect of the air voids of the mix. The traditional approach, based on the work of McLeod in the late 1950s, has been to aim for the air voids in the asphalt in the pavement being within the range of 3–5%. These values came from the realisation that when the air voids in asphalt are above approximately 6%, the mix becomes permeable to air and binder hardening can occur. Oliver (1992) showed that this assumption was still valid. McLeod also considered that when the air voids dropped below about 2%, shear strength was compromised. The consensus today appears to be that the air voids should not drop below approximately 3%.

The low air voids can also be considered from a saturation point of view used in granular basecourse construction. If a mix is considered to have a voids in mineral aggregate of 14% and air voids of 2%, then the degree of saturation is 100*(1–2/14) = 86%. In the NZTA specification for basecourse *B2 Specification for construction of unbound granular pavements* (2005), the maximum degree of saturation before sealing is 80% and the recommendation is that it should be closer to 65%. For an asphalt mix with a 'voids in mineral aggregate (VMA) of 14%, this is equivalent to approximately 5% air voids.

In the specification of the compaction requirements for granular basecourse, the target density is obtained by compacting the material at optimum water content under a vibratory hammer. The validation of this test to duplicate the density that the material will obtain under traffic was performed by Patrick and Alabaster (1998).

For asphalt mix design, the same approach is used – ie samples of mix are compacted under a specified energy that is meant to duplicate the density that the mix will achieve under traffic. Thus under light traffic a lower design compaction energy is used compared with that for a heavily trafficked road. The traditional approach in New Zealand has been to design the mix using 75-blow Marshall compaction. This level has been recommended by the Asphalt Institute in the US for heavy traffic and thus is appropriate for New Zealand motorways. In Australia, 50-blow compaction has been adopted as the default compaction level, which would be appropriate for the medium traffic level.

The confirmation of the 75-blow compaction level was the subject of the first CAPTIF pavement testing by Paterson in his PhD thesis in 1971. In his test track studies, he examined the density to which asphalt compacted under heavy traffic and at higher pavement temperatures (obtained under 75-blow Marshall compaction). Paterson concluded:

> *The stable state densities obtained on the Test Track under high contact pressure and high temperature conditions were generally close to Marshall predictions.*

In a 1991 NCAT report, Brown researched the density that was obtained under traffic in the US. His report states:

> *Eighteen different pavements were sampled from six states. Thirteen of the pavements were experiencing premature rutting and five of the pavements were performing satisfactorily.*
>
> *Construction history including mix design data, quality control and/or quality assurance data, traffic data and laboratory data of the physical properties of the pavement cores were analysed from each site. The results show that in-place air void contents below 3% greatly increase the probability of premature rutting and the in-place unit weights of the pavements after traffic usually exceed the mix design unit weight resulting in low air voids and hence premature rutting.*

He also concluded:

- *Compaction utilizing 75 blows per side with the manual Marshall hammer gives sufficient design density and void content for up to 6 million ESALs.*
- *Construction quality control documentation is not adequate on many paving projects.*
- *Samples of asphalt mixtures from the mixing plant should be compacted in the laboratory during construction to verify that the air voids are within an acceptable range. If the air voids are not within an acceptable range adjustments to the mix should be made.*
- *Most of the pavements evaluated in this study utilized a 50-blow Marshall mix design. Mixtures to be exposed to high traffic volumes should utilize a 75-blow Marshall mix design to insure adequate voids throughout the life of the pavement.*

Tyre pressures and loads have increased since the 1970s, so the 75-blow compaction level may not now duplicate the density obtained under traffic. A comparison of the density of the mix sample taken during construction with that in the field was performed for this research project in Wellington. The Mix 10 mixes had been laid under lower traffic volumes and had not compacted down to near the 75-blow Marshall values. The Mix 15 mixes, on the other hand, were approaching the 75-blow values, with only one site below this level.

Table 5.2 Air voids in NZTA Mix 10 mixes in Wellington

Street	Months since laying	Daily traffic per lane	% heavy	Air voids in w.track	Air voids out of w.track	Marshall air voids of mix
MIX 10						
Site 1	33	453	2.3	6.1	9.7	3.3
Site 2	28	N/A	N/A	5.0	10.2	3.9
Site 3	41	145	2.0	10.6	11.9	4.0
Site 4	29	N/A	N/A	6.6	9.2	3.9
Site 5	29	N/A	N/A	10.6	15.7	3.9
Average				**7.8**	**11.4**	**3.8**

Table 5.3 Air voids in NZTA Mix 15 Mix in Wellington

Street	Months since laying	Daily traffic per lane	% heavy	Air voids in w.track	Air voids out of w.track	Marshall air voids of mix
Site 6	14	2036	N/A	5.2	7.3	2.7
Site 7	27	7235	4.5	4.4	9.3	3.0
Site 8	41	1850	4.5	6.8	11.2	2.1
Site 9	41	1600	1.5	7.8	10.1	3.6
Site10	29	3970	2.7	3.9	14.8	2.2
Site 11	29	4880	2.7	3.4	5.0	2.2
Site 12	27	876	11.8	3.5	13.2	2.4
Site 13	28	N/A	N/A	8.8	13.2	3.5
Site 14	28	N/A	N/A	5.8	6.6	3.5
Site 15	40	7350	16.2	**2.3**	9.6	**2.8**
Site 16	29	6339	14.4	4.5	10.5	2.6
Site 17	28	8530	N/A	4.9	10.6	2.6
Site 18	28	7455	8.6	3.4	8.6	3.0
Average				**5.0**	**10.0**	**2.8**

The introduction of gyratory compaction rather than Marshall compaction raises the issue of whether this form of compaction and the number of gyrations used in design accurately models the behaviour under traffic. The Austroads method specifies three compaction levels of 50, 80 and 120 cycles, and recommends these levels for light, medium and heavy traffic. For each level of compaction the design air voids are 4.0%. Austroads chose these levels of compaction using their Gyropac compactor to approximate the 30-, 50- and 75-blow Marshall compaction.

For this research, a number of Wellington asphalt mixes were compacted both with 75-blow Marshall and with Servopac, and the number of cycles required to obtain the equivalent Marshall air voids was determined from an analysis of the Servopac height-recording system. The results for four different mix types are given in table 5.4.

Table 5.4 Comparison of number of Servopac cycles required to obtain equivalent air voids as Marshall compaction

Mix	n	Mean number of cycles	Std. dev.	Marshall blows
6	6	38	9.1	50
10	14	57	6.2	75
15	12	55	6.6	75
20	8	69	7.4	75

It can be seen from the table that the Servopac cycles required to obtain equivalent Marshall voids were significantly lower than the 80 and 120 cycles that Austroads recommends for 50- and 75-blow Marshall.

Pidwerbesky (2002) also compared the difference in air voids obtained from 75-blow Marshall with 120 Servopac cycles. Table 5.5 gives the results for nine different New Zealand mixes. It can be seen that the Servopac gave consistently lower air voids.

Table 5.5 Comparison of Marshall and Servopac compaction (Pidwerbesky 2002)

Mix ID	1	2	3	4	5	6	7	8	9
Marshall	1.9	4.3	3.8	1.6	3	2	3.5	3.5	2.8
Servopac	0.5	2.1	1.8	0.3	0.3	0.7	1.8	1	1.1
Difference	1.4	2.2	2	1.3	2.7	1.3	1.7	2.5	1.7

Austroads research (Oliver 2008) also tends to suggest that the number of gyrations recommended for the Gyropac should not be used directly with the Servopac. In table 5.6, the compaction levels obtained in the Wellington mix study is compared to results from Oliver's 2008 data.

Table 5.6 Comparison of Gyropac and Servopac cycles required to obtain equivalent air voids as Marshall compaction

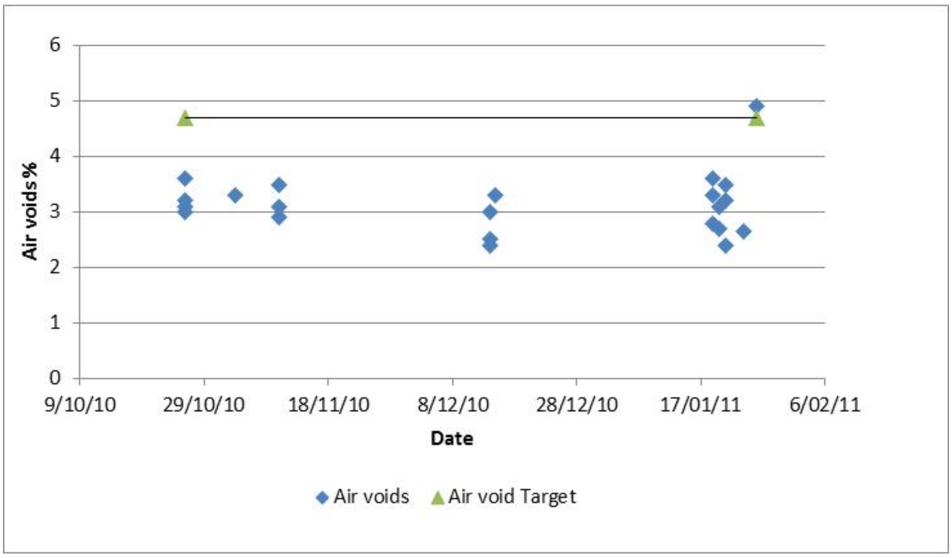
Marshall blows	Gyropac cycles	Servopac cycles	
		Oliver (2008)	Wellington
35	50	28–41	
50	80	41–68	38
75	120	57–105	57–69

Both sets of results indicate that more research is required to determine the number of Servopac cycles that should be used in the design of New Zealand mixes.

Table 5.3 shows that the Marshall air voids for the Mix 15 were consistently below 3.0% in the Wellington mixes. These mixes all complied with the NZTA specification for bitumen content and aggregate grading and had Marshall design air voids of 3.4%. This demonstrates that the laboratory design cannot be directly transferred to the field and that the air voids and hence degree of saturation can be very high in the plant-mixed material.

Figure 5.5 shows the variation in air voids, over time, of mix produced in a plant. Except for one test, the air voids were consistently between 2.3 and 3.8%. However the target 'design' voids were 4.7%. This high design level was chosen to minimise the risk of the pavement compacting to under 3% air voids. Although the bitumen content and grading complied with the NZTA specification, the mix produced has the potential to compact to an unstable state.

Figure 5.5 Air voids variation from plant production



A balance needs to be achieved between a mix's durability in terms of fatigue resistance, impermeability, minimising oxidation and it having sufficient shear strength to resist the traffic load – this means that both the design and production of the mix needs to be closely controlled. It is suggested that with the emphasis in the older M/10 on obtaining fatigue-resistant mixes, that the minimum bitumen contents in that specification meant that design air voids were close to 3.0% rather than the upper limit of 4.0%. With no control in the specification on the air voids being produced from the plant, many of the mixes would have been produced with 75-blow Marshall air voids below 3.0%. In higher-traffic areas, the mixes would then compact down to this 3.0% level and instability would occur.

Although the wheel-tracking test can be used as an indication of the shear strength of an asphalt mix, it cannot be easily used as a routine quality-control test and it is suggested that a simpler test, such as the Superpave Simple Performances Tester, should be investigated for adoption in New Zealand. The main recommendation is, however, to control the air voids from plant-produced samples though compacting samples of plant-produced mix and adjusting the proportions to ensure the 'design' air voids do not drop too low.

There is also an urgent need to determine the relationship between traffic compaction levels and the number of Servopac cycles to be used. With the coarser aggregate grading and high-compaction Servopac energies, there is the danger that the mixes will be lower than desired in binder and thus durability will be affected. The coarser grading makes the mix harder to compact and thus there is a danger that the mixes will be permeable to air and rapid oxidation could occur. In the US, some states are reviewing the compaction levels to promote higher VMA and binder content levels, as they consider that the Superpave mixes are lacking durability – this example should be followed in New Zealand.

6 The durability of open graded porous asphalt

6.1 Methodology

The durability of open graded porous asphalt (OGPA) was investigated through the use of the test developed by Herrington et al (2005) in a Transfund (now NZTA) project and the Austroads (Alderson 2008) Asphalt Particle Loss test (commonly termed the Cantabro Test. These tests were used to determine the comparative rate of hardening and abrasion resistance of OGPA.

The test conditions adopted were:

- conditioning of loose mix as a 20–30mm layer in a tray at 125°C for two hours
- ageing of compacted specimens at 80°C and 2069kPa air.

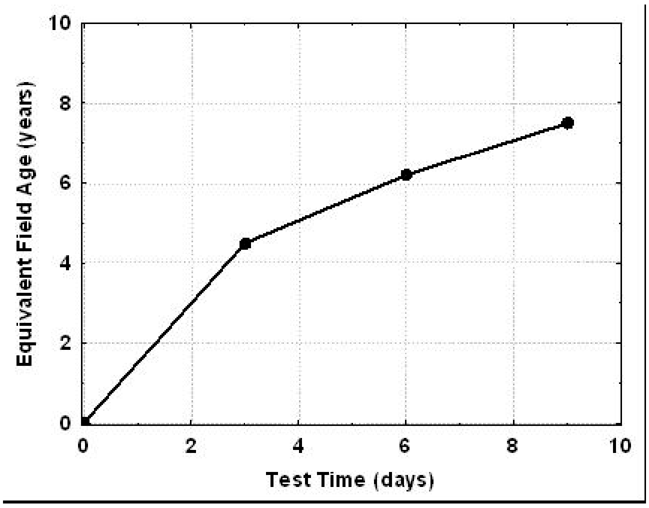
Binder drainage under these conditions was negligible.

The pressure vessel (see figure 6.1) was cylindrical (110mm i.d. x 200mm high), with a flat lid, and capable of holding two specimens. The specimens were loaded into the vessel on a simple rack constructed from perforated galvanised plate and wire. To reduce the risk of slumping, a strip of heavy brown paper was taped around the sides of the blocks. The vessel was held at the test temperature for several hours before the specimens were introduced.

Figure 6.1 Pressure ageing vessel

In the development of the test, samples of OGPA with a range of ages were recovered from the field, the binder extracted, and the recovered binders' viscosity measured. These were compared with bitumen aged in the vessel, and the relationship between the field ageing and test time was developed (see figure 6.2).

Figure 6.2 The relationship between the test and field ageing (Herrington et al 2005)



For this research, samples of OGPA that complied with the requirements of the NZTA P/11 (2007) PA10 specification were prepared. The mix design air voids were 20% with an 80/100 grade bitumen.

Specimens were prepared at optimum binder content, determined by the Austroads (Alderson 2008) Asphalt Drain-off Test and at 1% below optimum, and 1% above optimum. These samples were then aged until at equivalent field ages of 4.5 and 9 years. After the samples were aged they were tested to determine the resilient modulus, and then Cantabro tested to determine the abrasion resistance of each of the mixes.

The Cantabro test was performed at 25°C in accordance with the Austroads method. The test takes Marshall-compacted specimens and subjects them to 300 cycles in a Los Angeles test drum without the steel balls. The percentage loss in mass is then determined.

The research by Herrington et al (2005) recommended that the loss with zero ageing should be less than 15%, and less than 30% after ageing equivalent to 4.5 years (three days in the durability vessel).

Resilient modulus tests were performed using the indirect tensile test configuration and in accordance with AS2891.13.1 at 25°C, with a 0.1sec rise time and 0.3sec delay between pulses.

6.2 Results

Resilient modulus testing was not possible for the unaged samples, as the mix had a very low resilient modulus and the samples deformed under loading. The above mixes had a significantly lower modulus (about half) than the optimum mixes, even after simulating aging for eight years. The above mixes showed improvement in the abrasion loss testing, with 10% less abrasion loss for both the unaged and simulated eight-year mixes.

The results of the Cantabro and modulus tests are shown in figures 6.3 and 6.4.

Figure 6.3 Cantabro test results

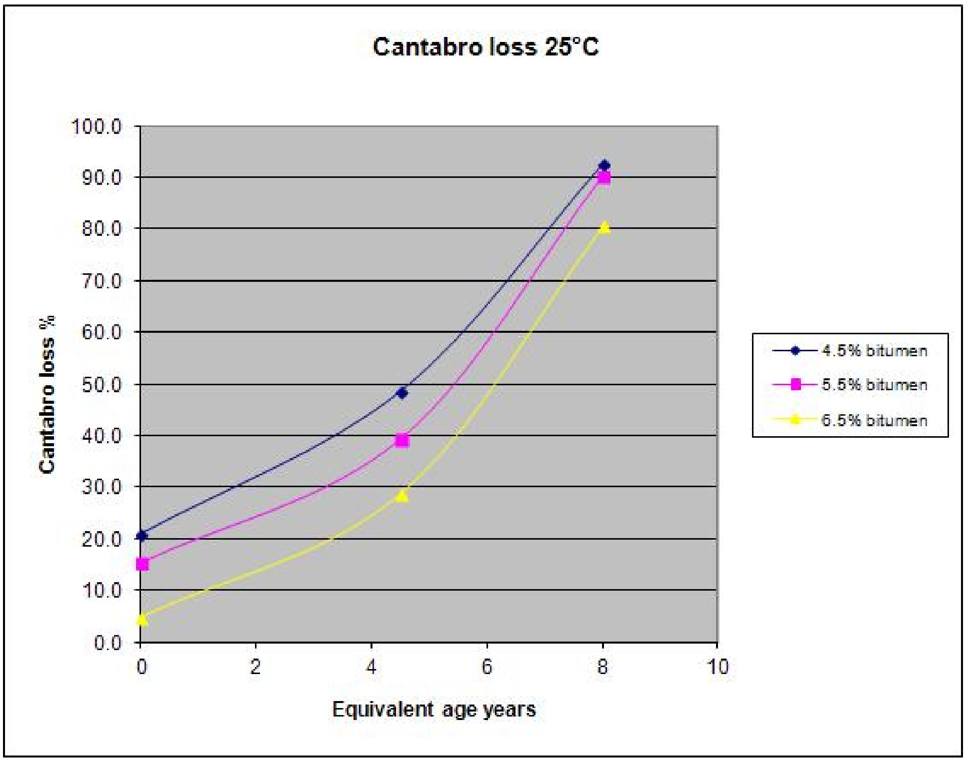
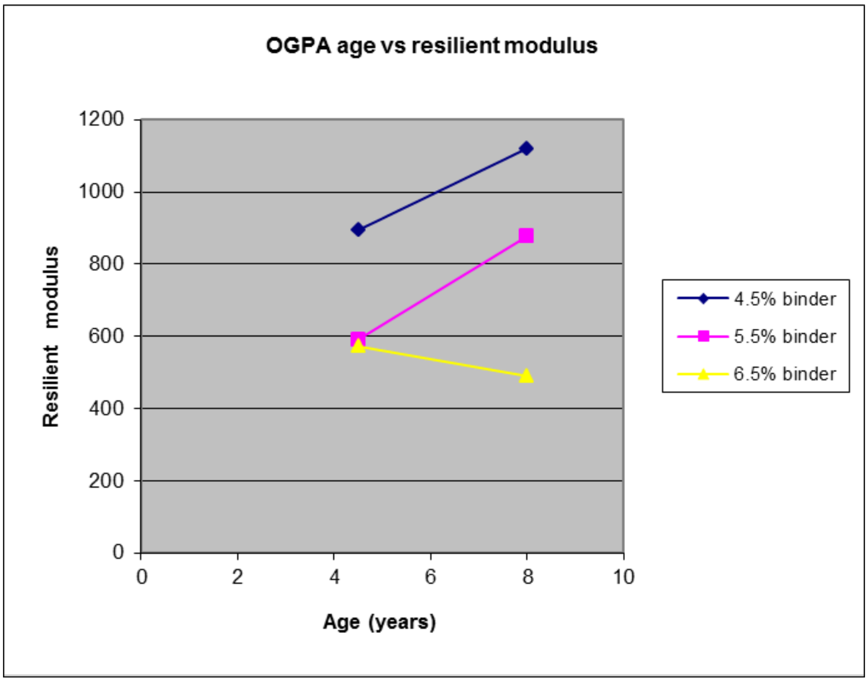


Figure 6.4 Resilient modulus test results



6.3 Effect of polymers

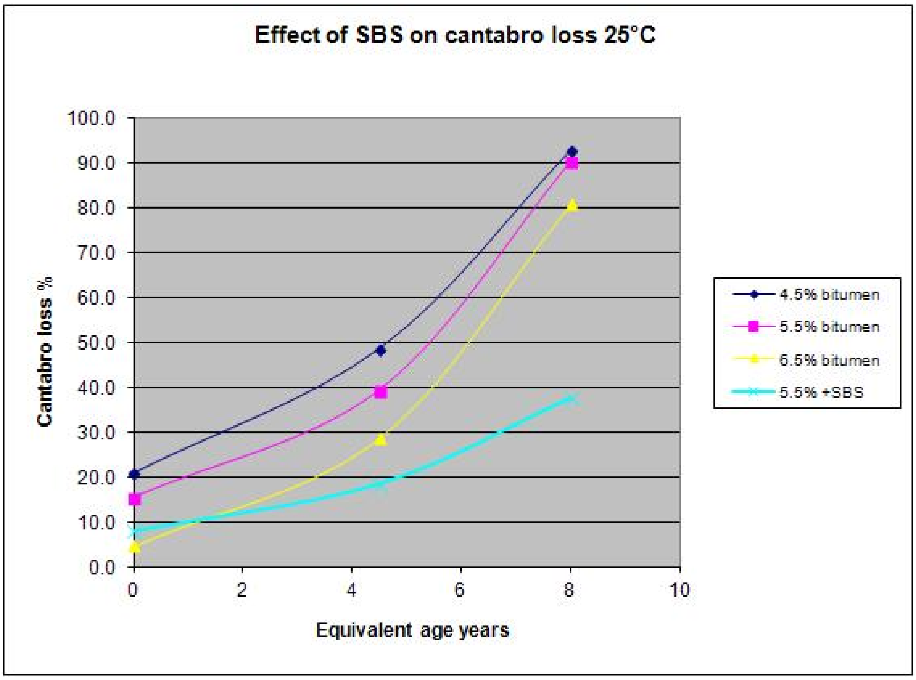
In the late 1990s TNZ instigated the use of polymers in Auckland after concerns regarding the low lives they were getting for OGPA on the Auckland motorway. The other two major uses of OGPA, namely Wellington and Christchurch, continued to use unmodified 80/100 bitumen.

The OGPA durability test discussed earlier provided a tool to assess whether the polymer was having a beneficial effect. The alternative method of specification would have been to develop detailed specifications for the polymer-modified binder (PMB). It was considered that the durability test had the potential to determine the properties of the mixture and thus was a more direct measure of the polymers' effectiveness. PMB manufacturers do not divulge the proportion of polymer, its type, or the use of any other additives in their products, so in this research a 'standard' product from Downers was used.

The PMB was mixed into the same aggregate as used in the initial test at a binder content of 5.5% and the samples were aged.

The results of the Cantabro test compared with the controls are shown in figure 6.5.

Figure 6.5 Comparison of Cantabro loss, with and without a PMB



Further testing of the effect of polymers was performed by taking samples of OGPA currently being produced in Auckland, Wellington and Christchurch. The samples were subjected to the standard ageing condition of 72 hours (equivalent to 4.5 years in the field). The results are given in table 6.1, together with data from production used in developing Herrington et al's 2005 test.

Table 6.1 Summary of results from plant-produced OGPA

Material	Air voids %	Wt. loss initial %	Wt. loss after oxidation %	Mass loss ratio oxidised/initial
ChCh 80/100	18	5.9	12.2	2.1
Wgtn 80/100	22.1	14.9	26.2	1.8
Auck PMB A	26.3	14	21.3	1.5
Auck PMB B	24.1	5.7	16	2.8
Auck PMB C	24.9	5.5	9.4	1.7
Auck PMB D	25.4	5.8	14.1	2.4
Auck PMB E	23.9	2.5	4.4	1.8
Auck PMB F	22.5	1.9	3.8	2.0
Wgtn 80/100 OGPA14[a]	19.2	7.3	13.7	1.9
Wgtn 80/100 OGPAHS14[a]	15.4	9	13.3	1.5

a) Based on Herrington et al (2005).

6.4 Discussion

In the design of OGPA there is a balance required between durability and porosity. The more binder that is included, the better the durability but the lower the porosity. This has always been recognised in the New Zealand specifications and the TNZ *P/11: 2007 Specification for open graded porous asphalt* specifies that the mixing temperature of OGPA with straight bitumen should be such that the binder is at a viscosity of 1Pas. For an 80/100 bitumen, this is equivalent to approximately 125°C. This low mixing temperature is used to ensure that at the higher binder contents levels, drain-down of binder is minimal. The specification also includes the use of a drain-down test, to ensure that the maximum binder content is selected but ensuring that the binder does not run off the mix.

In this research, the mixing temperatures used for OGPA with straight bitumen could not be lowered significantly from that currently specified, as aggregate drying would not occur. Higher binder contents may be able to be accommodated through the use of fibres or polymers. Figure 6.5 earlier illustrated that the increase in binder content has a significant effect on life, in terms of the Cantabro life. The Cantabro test is designed to model the fretting and loss of aggregate that is often associated with the end of life of OGPA. It does not model cracking associated with fatigue, which can occur where the basecourse for the OGPA is unbound granular.

The minimum temperature at which a PMB can be mixed with an aggregate depends on the formulation of the binder. A PMB often has non-Newtonian properties in the mixing temperature range, which means that the mixing temperature cannot be specified in terms of a simple viscosity number. The P/11 (2007) specification recognises this and thus at present requires that the PMB binder content is the same as a straight-run bitumen.

The test results shown in figure 6.6 indicate that the addition of polymer can result in a significant decrease in the Cantabro loss after accelerated ageing, and thus should significantly increase the durability of OGPA. However, table 6.1 earlier showed no significant difference in the performance of the Auckland OGPAs (containing PMB) compared with the Wellington and Christchurch materials (not containing PMB). The introduction of PMB into the Auckland OGPA resulted in a reported increase in life of at least two years, which would make their lives similar to those in Wellington and Christchurch. The durability test may have reflected that the non-modified Auckland OGPAs may have performed poorly. Further research is required to determine whether the addition of PMB was the main reason for the improved performance in Auckland, or if there were other factors that also contributed to the improvement.

An increased binder content can also result in an increase in life. However, the increase will result in a lower air voids content, which will negate some of the advantages of the material. The effect of increased binder content on the void content of the OGPA can be estimated by assuming that the aggregate structure formed by the aggregate grading is constant and that the volume of voids decreases in direct proportion to the binder content. If the VMA is assumed at 30%, then with an aggregate specific gravity (sg) of 2.65, the air voids would change with an increase in binder content as shown in table 6.2, where it can be seen that a change in binder content of 0.5% by mass affects the air voids by 1%.

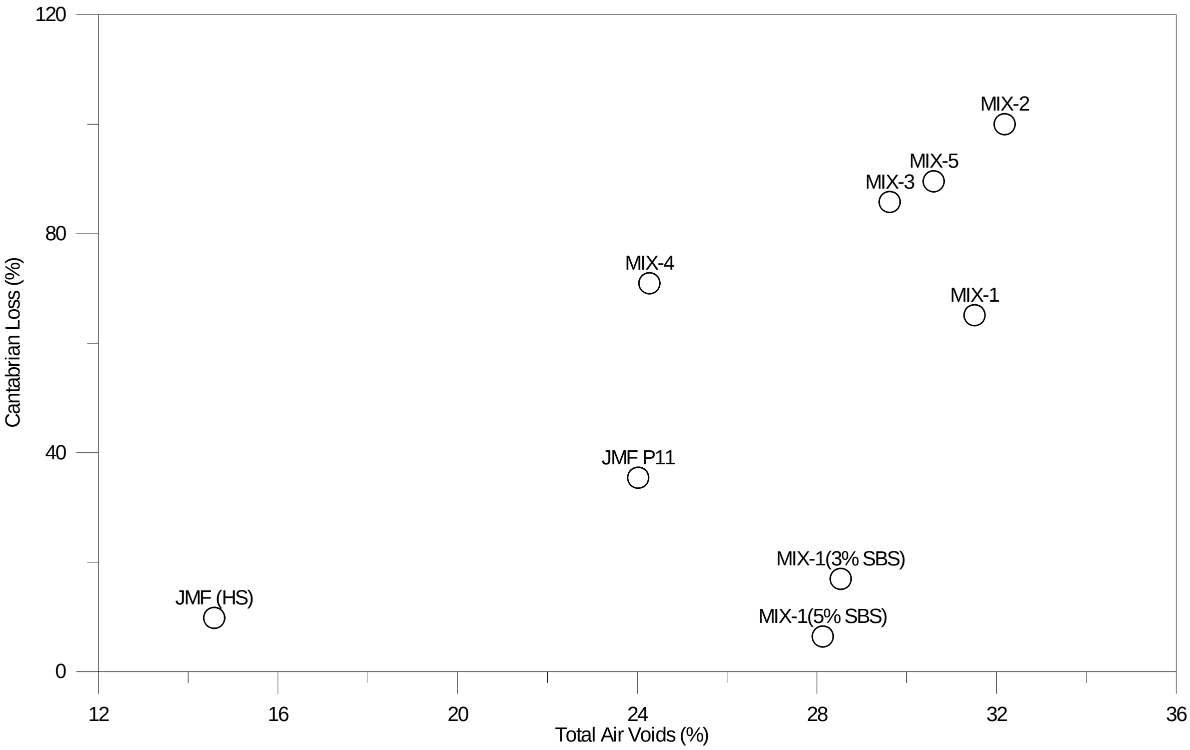
Table 6.2 Estimated change in air voids with an increase in binder content

% binder by mass	Vol bit. %	Air voids %
5.0	9.6	20.4
5.5	10.6	19.4
6.0	11.6	18.4
6.5	12.6	17.4

A typical New Zealand OGPA has a binder content of 5.5% and air voids of 20%. To obtain these air voids with a binder content of 6.5%, the aggregate structure would need to be opened up. The opening up of the grading could decrease the abrasion resistance, but the addition of polymer could more than balance this effect.

The effect of void content on Cantabro tests was explored in research by Jamieson and Patrick (2001) in trials of high air voids OGPA. The Cantabro loss at 18°C for a range of air voids is shown in figure 6.6. For the 28% air void mixes, the use of a PMB brought the Cantabro loss down to under 20%. (Note: the test temperature of 18°C results in higher losses than the now standard temperature of 25°C).

Figure 6.6 Cantabro test results from Jamieson and Patrick (2001)



The addition of polymer and an increase in total binder content should be trialled to confirm the practical limits that should be applied on a project. Further research is also required to determine how this could be specified under New Zealand's current specifications with the contractor pricing in terms of square metres. The lower the polymer and binder content that can be used, the lower the cost. The specifications therefore at present are not geared to optimising the durability of OGPA.

7 Acceptance schemes

A literature review was undertaken regarding the state-of-the-art of acceptance schemes and the use of bonus payments. Methods used to determine the quality of the mix and the use of warranty periods or maintenance periods were also researched.

Local and international asphalt acceptance schemes, including warranty schemes and performance-based pay factors, were reviewed – including methods used to determine the quality of the mix, the on-road performance of the mix, and the use of warranty periods and maintenance periods.

The methods used for asphalt acceptance schemes are summarised in table 7.1.

Table 7.1 Summary of acceptance schemes

Acceptance scheme name	Description	Notes
Percent within limits (PWL)	A sample is taken. The sample average and standard deviation are used to estimate the population. Payment is based on the percentage of material within the specification limits.	• Easy to understand • Encourages contractors to produce a lower standard deviation – then they can work closer to the specification limits • Does not address material just outside specification limits
Certification schemes	'Product approval schemes' are set up with the objective of developing national approval arrangements for innovative products, materials and systems for use in highways and related areas. This removes the need for individual authorities to carry out their own assessments and tests.	eg HAPAS
Warranties	A type of performance-based contract that guarantees the integrity of a product and assigns responsibility for the repair or replacement of defects to the contractor.	Divided into two types: • workmanship and materials warranty • performance warranty
Performance-based pay factors	Based on computing the differential present worth of the future rehabilitation required two different types of model: • a performance model for determining the effect of construction quality on expected pavement performance • a cost model for translating these effects into rehabilitation dollars.	In the ideal world, this would be what payment for asphalt would be based on.

- In the New Zealand experience, percent within limits (PWL) is essentially the system under which all testing and reporting is undertaken.
- Certification schemes are used in New Zealand for products such as road-marking paint and edge-marker posts, but there are likely to be high costs and a long time frame required to develop an asphalt certification scheme for New Zealand that would be acceptable and affordable for both clients and contractors.

- Warranties are widely used in New Zealand. These are known locally as maintenance periods or defects liability periods. Usually Workmanship and Materials warranties are used. Performance warranty asphalt contracts have been let in the Auckland area for a number of years now. However, changes and improvements could be made to current New Zealand specifications for performance assessment of asphalt. These changes would result in specifications where success depends more on knowledge and technology, with the contractor assuming full responsibility for pavement performance during the warranty period and, in effect, the contractor guaranteeing that the pavement will perform at the desired quality level. This is best achieved through empowerment or ownership of the project by the crew, and a working atmosphere of teamwork between road owner and contractor to produce a good product: a long-lasting pavement.
- Performance-based pay factors would, in the ideal world, be the best and most scientific basis on which to make payment for asphalt. However, in the New Zealand environment there would be a number of obstacles to overcome, including data collection, performance modelling and industry acceptance, before performance-based pay factors could be used in the ideal way. Nevertheless, New Zealand has a history of cooperation within the industry, and performance-based pay factors could eventually become the norm.

The desirability of a performance-based surfacing specification for high-performance asphalt pavements can best be expressed by the following extract from Weston et al (2001):

> *Previous work, funded by the sponsors, demonstrated that well-constructed flexible pavements, built above a threshold strength, will remain structurally sound for longer than their design lives provided that deterioration is treated in a timely manner. In these heavily trafficked, long-life roads, deterioration will normally take the form of cracks developing at the surface or deformation occurring in the surface course and the layer immediately beneath. Maintenance treatments will therefore be principally concerned with replacing the surfacing. The overall objective of the research described in this report is to improve the performance of asphalt surfacing under the more demanding conditions that are likely to be encountered in the future.*

Performance-based specification offers great potential to both the industry and the customer. It will encourage better mix design, give the industry more freedom to produce more commercially attractive designs, and the customer assurance that the pavement will not deteriorate prematurely.

7.1 Description of each acceptance scheme

7.1.1 PWL and quality assurance

Simply explained, PWL works in the following way:

- A sample of material is taken.
- The sample average and standard deviation are used to estimate the population.
- Payment is based on the percentage of material within the specification limits.

Hughes et al (2003) explain that PWL is a quality measure that combines the sample average and standard deviation into an estimate of the population – sometimes called percentage conforming. PWL's complement is 'percent defective' (PD).

The advantages of PWL (or PD) are as follows:

- They can provide an incentive for contractors to improve their variability and provide a more uniform product.
- There is software available that can easily calculate and balance agency risks (β) with contractor risks (α).
- It is compatible with AASHTO standard recommended acceptance plans.

In terms of quality assurance, Burati et al (2003) state that relating quality and performance to payment is the most desirable form of payment relationship because the relationship supports and defends the decision. Negative payment adjustments are typically viewed with scepticism by the contracting industry. When the payment system can be shown to be related to quality, and preferably to performance, it is seen as more credible than when it is established arbitrarily.

The use of incentives is not viewed positively by all agencies. Some think that use of an incentive is paying extra for what is typical quality. Therefore it is important to try and ensure that that the acceptable quality level (AQL) is properly established so that the incentives are applied only for exceptional quality.

Types of payment schedules are outlined in Burati et al (2003). The earliest payment schedules were usually stepped schedules based on PWL as shown in table 1 and figure 1.

Table 7.2 Typical stepped payment schedule based on PWL

Estimated PWL	Payment factor, %
95.0–100.0	102
85.0–94.9	100
50.0–84.9	90
0.0–49.9	70

More recently, continuous (equation-type) payment schedules have been used, such as shown in equation 7.1 and figure 7.1 below.

$$PF = 55 + 0.6\ PWL \qquad \text{(Equation 7.1)}$$

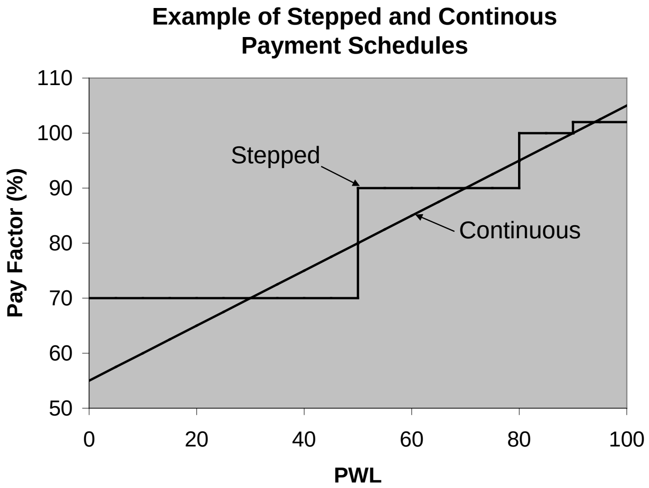
Where:

PF = payment factor as a percent of contract price

PWL = estimated percent within limits.

The advantage associated with the continuous form is that when the true quality of the work happens to lie close to a boundary in a stepped payment schedule, the quality estimate obtained from the sample may fall on either side of the boundary, due primarily to chance. Depending upon which side of the boundary the estimate falls, there may be a substantial difference in payment level, which may lead to disputes over measurement precision, round-off rules, and so forth. This potential problem can be completely avoided with continuous-payment schedules, which provide a smooth progression of payment as the quality measure varies.

Figure 7.1 Example of stepped and continuous payment schedules



The PWL concept is one of small sample sizes and the distribution used is the non-central t distribution. This distribution converges with the normal distribution when n approaches infinity. Material outside the limits may not be strictly defective, but of lesser quality than the material within the limits.

Other advantages of PWL are as follows:

- It provides the best combined estimate of population parameters.
- It is a variability-sensitive quality measure, which gives an advantage to contractors who are better able to control their variability.
- The lower the sample standard deviation (s), the closer to the specification limit the contractor can work and still produce acceptable quality.
- Risks can be calculated and balanced between the contractor and the agency.

Disadvantages of PWL are as follows:

- If the sample average is outside a specification limit, a higher standard deviation will produce a higher PWL.
- The PWL concept assumes a clear and meaningful specification (ie that the job mix formula (JMF) is right), which is often not available.
- It does not adequately address the impact (eg shortened life) of material that is outside the specification limits and may not be strictly defective, but is of lesser quality than the material within the limits.

7.1.2 HAPAS overview

The Highways Authorities Product Approval Scheme (HAPAS) was set up by the Highways Agency, CSS (County Surveyors' Society) and the British Board of Agrément (BBA) in 1995, with the objective of developing national approval arrangements for innovative products, materials and systems for use in highways and related areas. This removed the need for individual authorities to carry out their own assessments and tests.

The scheme is also supported by the Scottish Executive Development Department, the National Assembly of Wales, the Department for Regional Development in Northern Ireland, CSS and TAG (Local Government

Technical Advisers Group). It is also aided by TRL, other specialist laboratories, trade and professional bodies and industry experts.

The decision on which products fall within the scope of HAPAS and the most appropriate route to be followed rests with the Highways Industries Technical Advisory Committee (HiTAC), whose members are drawn from organisations with technical expertise in highways work. HiTAC is chaired by the BBA's chief executive.

If the number of products in a particular area is sufficiently large, HiTAC convenes a specialist group to develop a guideline setting out the procedures to be followed and the requirements to be met. HAPAS specialist groups draw up guidelines, documents giving details of the tests, assessment criteria and quality assurance requirements, which the BBA then uses in its evaluations.

The BBA's approval process for HAPAS involves laboratory and witness testing, site inspection and evaluation of the source of production of the material under assessment. Existing test or performance information is also investigated.

Data generated from these key elements is then considered by the BBA's HAPAS project managers/ assessors in the context of the requirements set out in the guideline. If the data is acceptable, the 'HAPAS certificate' is drafted. The draft certificate is then circulated to members of HiTAC for their comments and observations. Comments are considered by the BBA and the certificate is then formally issued. The BBA's interest in the product does not end with the award of its certificate, since surveillance visits are normally conducted twice-yearly to ensure that the specification of the product is being maintained. At the end of each five-year period, a formal review process is undertaken to confirm that the original assessment remains valid and the product's performance is satisfactory. Therefore, specifiers can be sure that the product covered by the HAPAS certificate will perform as set out in the document for as long as the certificate is valid.

The BBA's HAPAS certificates are called up where appropriate in the specification for highways works. They provide an independent opinion of the product's performance, enabling highway engineers to specify innovative products in the knowledge that they have been thoroughly evaluated.

Regarding the assessment directed towards the issue of a BBA HAPAS 'Roads and Bridges Agrément Certificate', confirming a thin-surfacing system's compliance with the requirements as defined by specialist group 3, 'Thin-surface course systems', and agreed by HiTAC, assessment and certification is undertaken in the following six stages:

- Stage 1 – Assessment of applicant's data
- Stage 2 – Assessment of production control
- Stage 3 – Laboratory testing
- Stage 4 – System installation trial
- Stage 5 – System performance trial (if required)
- Stage 6 – Certification.

Generally, each stage is to be successfully completed and, where appropriate, a report issued prior to the commencement of the next stage.

All systems are to demonstrate satisfactory performance on at least three sites of appropriate nominal installation depth classification over a period of at least two years. One of the sites is to be monitored by the BBA or their agent during the two-year period.

7.1.3 Warranties

7.1.3.1 Definitions

The following different types of warranties are defined in D'Angelo et al (2003):

> *Warranty: A type of performance-based contract that guarantees the integrity of a product and assigns responsibility for the repair or replacement of defects to the contractor.*
>
> *Warranty period: The prescribed time in which the contractor is required to repair defects in the product. Warranty periods vary by type of warranty and type of product. The ideal warranty period should be long enough to provide assurance of pavement performance, but not so long as to unnecessarily inflate contract prices.*
>
> *Materials and Workmanship Warranties: The contractor is responsible for correcting defects in work elements within the contractor's control during the warranty period. This includes distresses resulting from defective materials and/or workmanship. The owner is responsible for the pavement structural design. The contractor assumes no responsibility for pavement design or those distresses that result from design. Some responsibility is shifted from the owner to the contractor for materials selection and workmanship.*
>
> *Performance Warranties: The contractor assumes full responsibility for pavement performance during the warranty period. In effect, the contractor guarantees that the pavement will perform at the desired quality level. The contractor assumes some level of responsibility, depending on the specific project, for the structural pavement or mix decisions.*

7.1.3.2 Background

There is some suspicion and apprehension about warranties, from all parts of the industry. The following summarises perceptions about warranties and points to their continued use as a positive way forward for the industry (Kopac 2002):

> *Ironically, warranties were seen in the early days of road construction as the logical specifications due to lack of knowledge and technology (i.e., there was nothing better). They were later discontinued primarily for being a "source of endless litigation," as Roger L. Morrison put it in a report for The Asphalt Institute. Now, warranties are once again emerging, but this time as specifications whose success depends on obtaining more knowledge and technology. The National Performance-Related Specifications (PRS) Action Plan recognizes that PRS and warranties have much in common. They are actually dependent on each other and are both aimed at evolving into cost-effective, end-result performance specifications.*

Kopac (ibid) also mentions that performance-related specifications grow out of quality assurance specifications.

Warranty specifications can be broken into two types (Dukatz et al 2001: one is the workmanship and materials warranty; the other is the performance warranty. These are explained in the next two sections. Further information on the Wisconsin experience with warranties is included, and a summary of information from the 2003 report by D'Angelo et al on *Asphalt pavement warranties – technology and practice in Europe.*

7.1.3.3 Workmanship and Materials Warranty

The workmanship and materials specifications are based on the theory that the historical knowledge of how different materials perform in pavements is best understood by the contracting agency (Dukatz et al 2001). Based on that knowledge, the agency writes specifications that, when followed, will produce pavements that meet certain performance criteria. The pavement is warranted to reduce the amount of risk to the agency that the pavement will fail prematurely in some fashion under traffic. Common workmanship and materials specifications typically warrantee the pavement for up to a year after construction. To reduce the agency's risk further, the warranty period is extended to a period of two to five years. Under an extended workmanship and materials warranty specification, the contractor is held responsible for items perceived to be under contractor control, cracking, ravelling, rutting, flushing and so forth. The project is then built under tight agency control, with agency approval and acceptance required on materials, mix design and in-place pavement density (Michigan Department of Transportation 2003; Minnesota Department of Transportation 2004).

Penalties (disincentives) are assessed for non-conformance with the specifications. Some states have provided bonus (incentives) provisions for achieving or exceeding minimum test results, such as meeting both a specified in-place pavement density and lab voids within a specified tolerance.

The agency then accepts the pavement if:

1 the HMA mix meets the HMA mix Quality Control tolerances

2 there are no pavement defects

3 acceptable in-place pavement densities are achieved.

The contractor is then held responsible for repair of distresses caused by factors unrelated to materials and workmanship.

> *These include, but are not limited to, chemical and fuel spills, vehicle fires, snowplowing and ice control, and destructive testing done by the Department during the warranty period. Other factors considered to be beyond the control of the Contractor that may contribute to pavement distress will be considered by the Department (i.e. agency) on a case-by-case basis upon receipt of a written request from the Contractor* (Michigan Department of Transportation 2003; Minnesota Department of Transportation 2004).

The contractor is responsible for making warranty repairs using materials and methods as specified by the agency.

7.1.3.4 Performance Warranty

Performance warranties are based upon the premise that once the level of performance is set by the agency, the contractor is responsible for obtaining materials and constructing the pavement to meet the level of performance. Under a typical performance warranty, the agency maintains the responsibility of designing the pavement section to carry the expected traffic loads. The contractor has the option to use as little or as much of the agency's HMA specifications as they deem necessary to meet the specified performance criteria. The choice of materials, mix designs and methods of construction are at the contractor's discretion. The contractor is required to develop a Quality Management Program (QMP) for agency review. Suggested QMP guidelines are offered in the warranty specification. The scope and range are developed by the contractor, based on the contractor's perceived level of acceptable risk. This provides the contractor with options on the level of testing. For a project that is running smoothly, the amount of testing may be reduced, resulting in cost savings to the contractor with no increase in risk to the agency.

A performance specification puts an enormous amount of pressure on the contracting industry to insure that the public's best interests are served. So, under a performance specification there is incentive for the contractor to check the agency's design parameters and site conditions prior to construction. Projects with contractor perceived defects tend to be bid higher to cover defect repair costs – so much higher than the engineer's estimate that some warranty projects in Wisconsin were dropped.

7.1.3.5 Keys to success with warranties

The Wisconsin warranty experience (Dukatz et al 2001) is a successful demonstration that quality systems, improved technology and improved client–industry relations can lead to improved performance of asphalt in a warranty environment.

The keys to success in the Wisconsin warranty experience included the following:

- In 2000, Wisconsin adopted the SHRP gyratory (SGC) compactor for laboratory-mix compaction. Since volumetric requirements were already part of the Wisconsin HMA specification there was no learning curve on use of volumetric parameters.
- Key to the success of the programme was adhering to the partnering model used to develop and improve the HMA QC/QA programme.
- The warranty thresholds were developed from the analysis of the Wisconsin Pavement Management System (PMS) data. A system-wide approach was used so that state-wide practices were represented. The data was analysed using MiniTab statistical software. The historical data for smoothness, rutting and cracking was summarised and graphed.

7.1.3.6 Developing HMA warranty thresholds

The key indicator in WisDOT's[3] PMS is the pavement distress index (PDI). To determine the PDI, the HMA pavement is evaluated based on 12 identifiable surface distresses (alligator cracking, block cracking, transverse cracking, longitudinal cracking, flushing, edge ravelling, surface ravelling, patching, rutting, transverse distortion, longitudinal distortion and segregation).

These distresses are divided into tolerable and non-tolerable. A tolerable distress is defined as one where a small amount does not pose a safety or performance concern. Further, that zero-tolerance would increase costs without improving performance. A non-tolerable distress would be one that posed a safety or performance concern.

7.1.3.7 Warranty performance

Figures 7.2, 7.3 and 7.4 are from Dukatz et al (2001) and demonstrate the performance of the various mix types and specifications on the Wisconsin network.

In figure 7.3, The Marshall data starts at a higher initial IRI and stays rougher. The Superpave data set from conventional contracts is initially smoother than the Marshall mix projects and stays smoother. The warranty pavements are even smoother still. In Dukatz et al (2001), the predicted pavement life was extended from 18 years to 23 years. With another three years of data, it appears that the predicted pavement life could be extended to another three years, to 26 years.

The historical pavement distress data (PDI) is shown in figure 7.3. As shown in figure 7.2, the predicted pavement life is substantially increased over that of the traditional Wisconsin pavements. This graph also shows that the rate of distress is slower for warranty projects.

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3 Wisconsin Department of Transportation.

The rutting data presented in figure 7.4 shows some interesting trends. As expected, the warranty projects have less rutting and the rate of rutting is less than for the other pavement projects. These graphs show that the contractor has been able to balance improvements in rutting resistance, as well as cracking resistance, without increasing other pavement distresses.

Figure 7.2 Mix type pavement ride trends (Dukatz et al 2001)

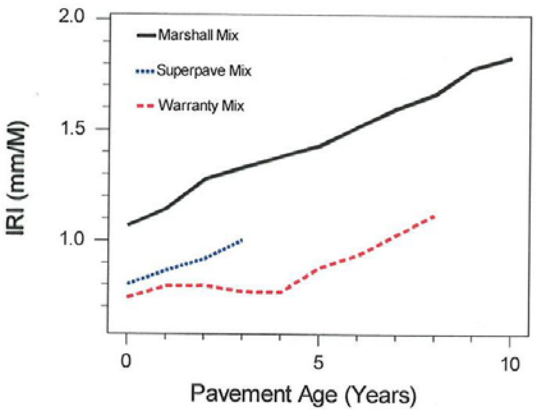


Figure 7.3 Mix type pavement distress-rating trends (Dukatz et al 2001)

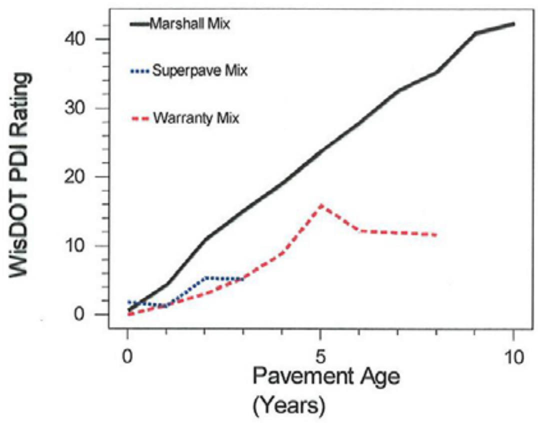
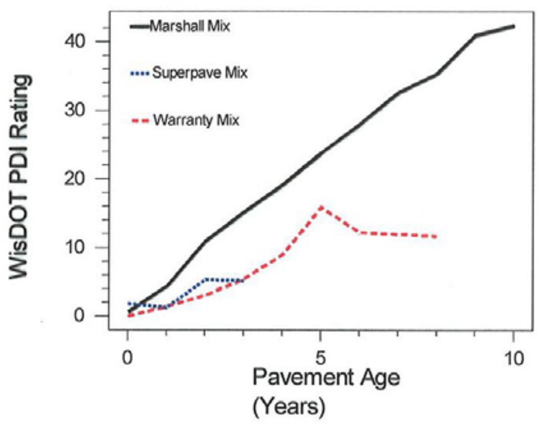


Figure 7.4 Mix type pavement rut trends (Dukatz et al 2001)



7.1.3.8 The agency perspective

Dukatz et al (2001) summed up the positives relating to warranties as follows:

- By 1987 the HMA industry in Wisconsin was in crisis. HMA Pavements were exhibiting premature cracking, ravelling and rutting.
- Early in 1994, a team from WisDOT, WAPA and the Federal Highway Administration Wisconsin office began to develop a Wisconsin HMA Warranty Specification. The team was given the following mission guidelines (Volker 1996):

  Mission of the Team:

  a. Improve the quality of our pavements.

  b. Work toward performance based specifications.

  c. Take a fresh non-restrictive look at construction practices and contracting.

  d. Reduce supervision.

The first issue was to define the current quality of the Wisconsin pavements. This was accomplished through the state's pavement management program, which was based on PDI. The team was concerned about developing thresholds that WisDOT, the contractors and the bonding companies felt comfortable with in terms of risk. WisDOT wanted the thresholds set so that the risk of premature pavement failure was minimised and pavement quality enhanced. The contractors wanted thresholds that were achievable and provided a reasonable chance of success. The bonding companies wanted the thresholds and the remedies clearly defined so that their risks could be assessed. The previous section describes how the threshold levels were set. The next issue for the bonding companies was the calculation of the bond amount.

The bond amount was set to minimise WisDOT's risk. The bond amount was based on a typical worst-case scenario determined from a review of the historical pavement maintenance data. That bond amount is calculated as the cost of a 1¾in HMA overlay over the total pavement width for the length of the project, including the cost of the asphalt binder and tack coat. Over the last nine years, this bond calculation has been successfully used on 55 warranty projects. The process has worked well enough that in 2003, bonding was made available for WisDOT's first seven-year warranty projects.

The risk with a performance specification based on agency structural design and contractor mix design is applying the warranty to inappropriate projects. A pavement with an extensive history of distress and repair that was bid without repairs to correct the underlying problems would not be a candidate for a multiyear warranty. This is controlled first by WisDOT-developed warranty selection guidelines. Second, exemptions were added to the warranty specification to account for unplanned conditions.

7.1.3.9 The contractor perspective

As previously discussed, warranties entail risks for the agency as well as for the contractor. One area in which the contractors can protect themselves is the project bid price, which reflects the contractor's perceived risk. The interesting part of the WisDOT Warranty reports (Heintz 2003; Heintz et al 2003) is that mix quality has improved without additional specifications. Indeed, some may argue that quality has improved with less oversight.

The key appears to be the switch from conflict over paperwork and non-performance issues to the focus on teamwork. The start of a warranty project requires a preconstruction meeting with the state. At this meeting the tone for partnership is set by a common goal: a good, long-lasting pavement. The contractor's QMP plan is finalised. This fosters teamwork between the agency and the contractor.

The next step is that the paving crew is empowered to do a good job. This switch in focus is not directly measurable, but depends upon human nature. The focus is on producing a good pavement, with each individual responsible for their performance. The crew has ownership. When problems occur, then the contractor's crew can quickly and efficiently correct them. Problem correction is fast, compared with a conventional project where the response is slowed by agency review and response. Also, on conventional projects (especially those with disincentives only), too often the crew's attitude is 'If it's really important, the state will check it. If the state doesn't check, then it must not have been that important.' Attention to detail becomes more important to the crew under a warranty project.

Another key is that all quality and project data collected by the contractor is available for review by the state, and vice versa. This promotes discussion about results, instead of conflict over whose data is correct, and thus improves morale.

7.1.3.10 Summary of the Wisconsin warranty experience

The Wisconsin data shows that pavements built under a performance warranty have better performance histories. The initial data shows that pavement life is increased by a minimum of five years.

The key to achieving better performing pavements with warranty specifications is empowerment, or ownership of the project by the crew, and a working atmosphere of teamwork between state and contractor to produce a long-lasting pavement.

The overall costs of performance for warranty projects are less than those for conventional projects, due to the differences in required time and materials. This does not mean the agency gets a cheaper product – often, the HMA per-ton cost is more for warranty mix. However, the state is spending less money on supervision, testing and pavement maintenance. The contractor has to spend less time working on (perceived) specification issues and can spend more effort on obtaining the best possible results from available materials. The net result is that performance warranty projects are cost effective.

7.1.4 Performance-based pay factors

Performance-based pay factors in California (Monismith et al 2004b) were calculated using performance models for fatigue and rutting based on analysis of accelerated pavement tests from the Caltrans Heavy Vehicle Simulator (HVS) and the WesTrack accelerated pavement performance test programme.

The performance-based pay factors were calculated on the basis of valuing the difference in present worth of rehabilitation costs between as-constructed and as-specified (as-expected) work, to provide a rational basis for setting the level of penalty or bonus for inferior or superior construction quality.

This research took the view that to compute the differential present worth of the future rehabilitation required two different types of model:

1. a performance model for determining the effect of construction quality on expected pavement performance
2. a cost model for translating these effects into rehabilitation dollars.

7.1.4.1 The performance model

The Californian research showed two modes of distress – rutting and fatigue – which were related to performance, as shown in table 7.3.

Table 7.3 Mode of distress and factors affecting AC performance

Mode of distress	Factors affecting performance
Rutting	Asphalt (bitumen) content Air void content Aggregate gradation
Fatigue	Asphalt (bitumen) content Air void content Asphaltic concrete thickness

7.1.4.2 The cost model

The cost model developed in Monismith et al's research (2004b) assesses the present worth of moving the first rehabilitation cycle from its on-target position, TY, to its off-target position, OTY. The net present worth, expressed as a percentage of the rehabilitation costs (in current-year dollars) is described as follows:

Assume that:

- C is the resurfacing/rehabilitation cost in current-year dollars
- TY is the target pavement life
- r is the annual rate of growth in resurfacing/rehabilitation cost, ie the construction cost index
- d is the annual discount rate
- OTY is the pavement life due to off-target construction.

Then on-target construction will result in:

- a future cost of C(F/P, r%, TY) at the end of the TY-year target period

or

- an annual equivalent of C(F/P, r%, TY)(A/F, d%, TY) for the specified target years

where:

F = future worth

P = present work

A = annuity amount.

Off-target construction will result in:

- a future cost of C(F/P, r%, OTY) at the end of the off- target period

or

- an annual equivalent of C(F/P, r%, OTY)(A/F, d%, OTY) for the OTY years.

7.1.4.2 Discussion of performance-based pay factors

As mentioned in Monismith et al (2004b), inferior construction hastens the need for future rehabilitation and may increase the cost of rehabilitation as well. Inferior construction increases the present worth of future rehabilitation costs. Superior construction reduces the present worth of these costs, largely by deferring the future rehabilitation.

It is desirable to reproduce the work described in Monismith et al (ibid) for the New Zealand environment. To compute the differential present worth of the future rehabilitation, the two different types of model discussed earlier (performance model and cost model) should be used. The emphasis on asset management in road maintenance management in New Zealand should make it relatively easy to develop the cost model.

Developing the performance model will be a little harder. An initial thought was to review CAPTIF research and observe the distress modes of asphalt of different mix design properties (reproducing the Caltrans Heavy Vehicle Simulator (HVS) and the WesTrack accelerated pavement performance test programme data). However, investigation has shown that CAPTIF almost exclusively uses unbound granular pavements with a chipseal surface, and no suitable data is available.

8 Guidelines

8.1 Purpose

The following guidelines for choosing the asphalt type were prepared as part of this research. These are suggested to be complementary to the 'Treatment selection flow chart' prepared by the Roading NZ working group (available on www.roadingnz.org.nz/).

The road surfaces and underlying pavements first have to deliver safety and be sufficiently durable to be economically effective. Lower noise performance is therefore limited to those surface and pavement types that fulfil these primary criteria. Pavement and surface design are extensively addressed within other guides. The purpose of this section within this guide is to provide non-specialists in roading design with an understanding of the constraints that lie around choice of surfaces with the derived acoustic performance.

The surfacing of a pavement is supported by the underlying structure and different surfaces have different support requirements. Thus all surfacing types cannot be used on any pavement. Besides providing support for the surfacing, it has to provide adequate skid resistance. Adequate skid resistance is obtained by using an appropriate polishing-resistant aggregate and sufficient texture so that water can be expelled from under vehicle tyres. The minimum texture required depends on the traffic speed. At higher traffic speeds more texture is required to resist aquaplaning.

In turning or braking traffic stress areas, the surfacing has to have higher strength than in low-stress areas, and some surfacings have significantly greater strength than others.

As well as the engineering properties, the surfacing must be affordable.

8.1.1 Functions of the pavement

One of the most important functions of a pavement is to withstand the loading imposed by traffic and the resulting stresses. The surfacing engineer's task is to design a pavement that performs well under those stresses.

Compressive stresses are the vertical stresses generated by vehicles, which are dissipated down through the pavement layers. The deformation in the subgrade reflects through to the surface in the form of longer wave length depression – roughness and rutting.

Figure 8.1 Compressive stress under a wheel load (TNZ et al 2005)

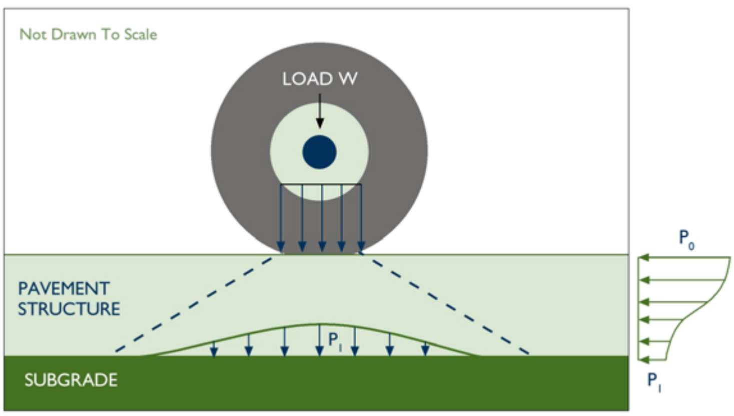


Figure 3-3 Compressive stress exerted under a wheel load.

Shear stresses are generated by shear in the pavement during braking, acceleration, etc, by the loaded vehicle tyre. The deformations cause shorter wave-length bumps on the surface.

Figure 8.2 Shear stress exerted by a braking wheel (TNZ et al 2005)

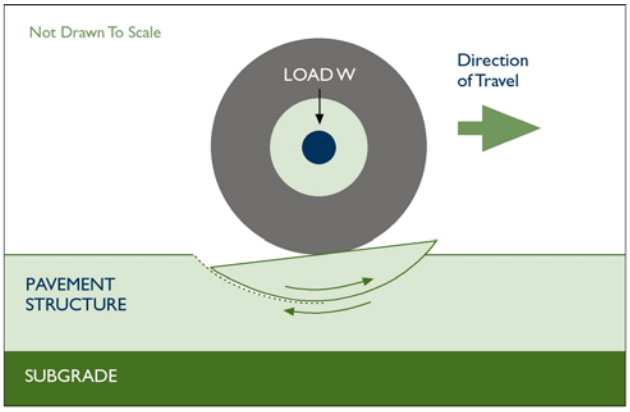


Figure 3-4 Shear stresses exerted by a braking wheel.

Figure 8.3 Tensile stress under a wheel load (TNZ et al 2005)

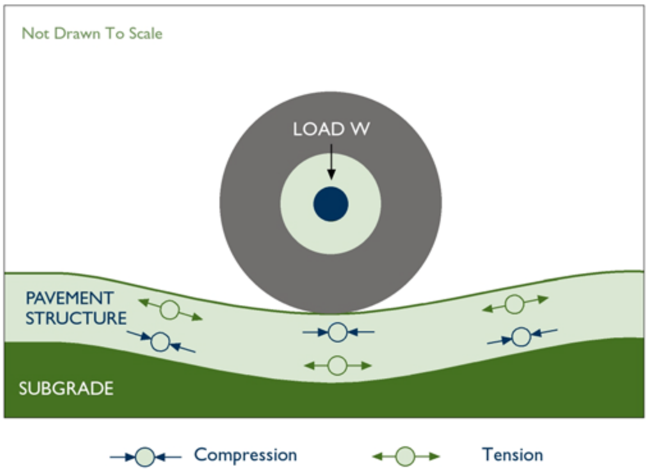


Figure 3-5 Tensile–compressive stresses exerted by a load in a bitumen-bound layer (from Transit NZ 1993).

Tensile stresses are generated by deflection of the pavement surface by the loaded vehicle's tyres. The tensile stresses can only be generated in bound materials such as cement- and bitumen-bound materials, and are not present in granular basecourse. These stresses result in the surface cracking.

The basic philosophy of pavement structural design is to choose, for the different layers, materials that have sufficient shear strength to carry the traffic loading, and to use those materials to limit both compressive stress on the subgrade, and tensile strain at the bottom of any bound layer.

8.1.2 Pavement deflection

As heavy traffic passes over the pavement, the surface deflects. This deflection can result in cracking of any bound material. The extent of the deflection is determined by the underlying pavement structure and the strength of the subgrade. A chipseal is significantly more flexible than an asphalt mix and can withstand significantly higher pavement deflection. It is therefore often not economic to apply a thin asphalt-type mix to an existing chipseal. The suitability of the pavement for an asphalt mix needs to be determined by a pavement engineer and based on the traffic volume and pavement structure.

For a new pavement the noise requirements will be factored into the initial design. If a low-noise asphalt surfacing is a requirement then the pavement engineer will design an appropriate pavement structure, based on the subgrade strength.

8.1.3 Skid resistance

The skid resistance of the surfacing is of paramount importance and the requirements are covered in the *NZTA T10: 2012 Specification for state highway skid resistance management*. The balance of the skid resistance requirements with noise is associated with the texture requirements. In high-speed areas, a minimum texture is required to ensure that water can be displaced by the tyre and aquaplaning does not occur.

The T10: 2012 requirements permit the use of all the surfacing types described here for in urban areas where the traffic speed is 50kph or less. For the 50–70kph range, the smaller-sized dense-grade mixes using 10 or 14mm maximum particle size and the slurries will not have the minimum texture requirement of 0.7mm. For high-speed roads (>70kph), the minimum texture requirement of 0.9mm for an asphalt mix can only be consistently met with SMA or OGPA. A macadam-type material can often be borderline in meeting the texture requirements.

All the chipseal types will meet the texture requirements over the whole speed range.

8.2 Limitations on surface suitability

8.2.1 Stress ratings

Pavement surfaces fail on corners, intersections and steep gradients where the high stresses are caused by braking or turning traffic. If severe enough, traffic stresses can tear out chips from the chipseal surface and the surface then rapidly loses its integrity.

These high-stress sites can be divided into those that are associated with:

- cornering
- braking and slow-speed turning.

In practice, the pavement designer would inspect the site in order to assess the stress levels (always assuming a sound substrate), and could increase or decrease the classifications for cornering, braking and turning based on site-specific conditions.

8.2.1.1 Cornering

High-stress corners can be rated 1–6 according to the advisory speed, supplemented by the gradient, as proposed in the following classification (table 8.1), which is taken from *Chipsealing in New Zealand* (TNZ et al 2005).

Table 8.1 Stress rating from *Chipsealing in New Zealand* (TNZ et al 2005)

Table 6-2 Classification for stress rating on corners				
Gradient (%)	**Advisory speed (km/h)**			
	Less than 30	30 to 50	50 to 70	More than 70
Less than 5	4	3	2	1
5 to 10	5	4	3	2
More than 10	6	5	4	3

In this classification, the lower the number, the lower the stress on the site, so that a classification of 1 implies that the stress is very low and the section is flat and relatively straight.

The number of heavy commercial vehicles (HCVs) would affect the ranking because HCVs with multiple axle groupings or spaced axle trailers can cause additional shear forces (often called 'tyre scrub') by the non-steering wheels being partially dragged around turns. This effect is exacerbated where speeds are low.

8.2.1.2 Braking and turning

Braking and turning areas that may require specialised treatment include:

- roundabouts
- intersections
- commercial driveways (in industrial areas)
- railway crossings
- pedestrian crossings.

Using the same rating system of 1–6 as for cornering, but based on traffic volume per lane per day (HCV/l/day), the following classification (table 8.2) is proposed.

Again, the comments regarding tyre scrub generated by HCVs apply here, and in all cases, the rating should be adjusted accordingly to account for this.

Table 8.2 Stress rating from *Chipsealing in New Zealand* (TNZ et al 2005)

Table 6-3 Classification for stress rating for braking and turning areas					
Number of heavy vehicles per lane per day	**Roundabout**	**Intersection**	**Commercial driveway**	**Railway crossing**	**Pedestrian crossing**
Less than 20	5	3	5	2	2
20 to 50	6	5	6	3	4
More than 50	6	6	6	4	4

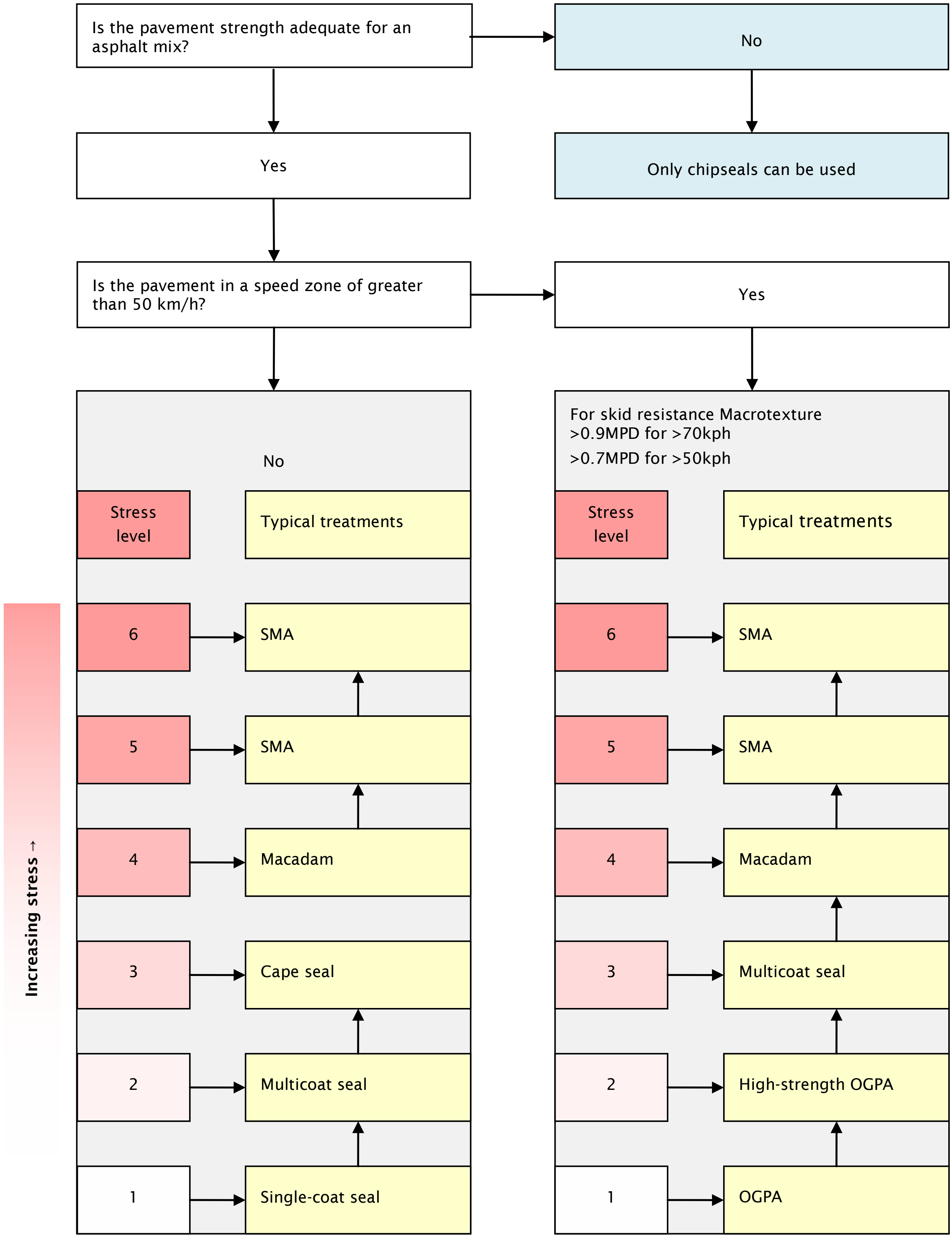
8.3 Selection flowchart

The following flow chart gives an outline of the decision-making process.

It starts with a decision on the pavement strength and whether an asphalt mix can be used. It then considers the texture requirements and then the site's traffic stress.

The site's traffic stress is shown by the ranking according to the above criteria. A surfacing suitable for a high-stress area is obviously also suitable for a lower-stress area. Thus the surfacing for a stress-level 1 site can have any of the surfacing from levels 1–6.

Figure 8.4 Surfacing selection guide



8.4 Costs

Guidance on the appropriate costs for intangibles is given in the NZTA EEM (2010), where the cost of noise is defined as $410 per year x dB change x number of households affected.

Using typical values of noise for the various chipseals and asphalt mixes and their expected lives in an urban situation, the whole-of-life costs cannot support the use of an asphalt-type mix until the traffic volumes are greater than about 8000vpd. This is based just on the cost of the surfacing and assumes the pavement has sufficient strength. In many urban situations the build-up of successive layers affects the pavement geometry. In these cases the old surfacing needs to be removed before asphalt can be applied. The extra costs in urban areas can push the justification limit to over 15,000vpd. On the state highway system less than 5% of the length has traffic volumes greater than 8000vpd, and thus the use of surfacings such as OGPA is usually cannot be justified on total costs and is therefore based on a policy decision.

The whole-of-life concept would limit the use of asphalt surfacing to very high-traffic-volume roads when the decision is based on the economic cost of noise.

In most cases the use of quieter surfacing will be a policy decision rather than an economic decision.

9 Conclusions and recommendations

This research project had the objective of identifying areas where changes could be made in the use of thin layers of asphalt so that improvements in performance could be obtained. The project was not designed to investigate quality issues, but was to concentrate on materials and selection. The conclusions regarding the various aspects studied are outlined in the following sections.

9.1 Current lives

The current lives being obtained on thin surfacings on the state highway system reflect the widespread use of OGPA on the motorways. The relatively high percentage failure of surfacings within the first four years prompted this investigation and the RAMM data does not provide information to assist in identifying the reasons.

Premature failures are often blamed on factors such as:

- treatment selection
- construction quality
- material problems.

However there is no database that records the reasons or gives any information that can be used to prevent the reoccurrence.

Overall, cracking and low skid resistance stand out as major motivations for resurfacing.

ACs and OGPAs last, on average, for 82.4% and 86.7% respectively of their standard default lives; however, there is a wide range of lifetimes for both surface types.

These average lives are similar to those reported in the US and Australia, but significantly shorter than those reported in Britain. The higher British lives could be associated with their analysis being on surfacings on structural asphalt pavements, whereas New Zealand thin surfacings are often over a granular basecourse.

In the early 2000s, changes to OGPA in Auckland through the use of polymers appear to have increased the life of surfacings, and this initiative will not have been reflected in the statistics.

Recommendations

- Premature failures should be investigated more fully and a system devised to alert NZTA National Office of the occurrence of premature failure, and an appropriate level of investigation should be performed to identify the reasons for it.
- A system to record the reasons for the choice of surface treatment and the reasons for any estimate of life is needed.

9.2 Whole-of-life costs

With the cost of thin asphalt layers being four to six times higher than a chipseal, and the lives not significantly greater, then the justification of using hotmix asphalt cannot be made in terms of cost in areas where both chipseals and asphalt would perform.

Where the traffic stress is such that a chipseal would fail very early in its life then asphalt is the obvious choice.

The main benefit of hotmix asphalt is associated with factors such as ride, rolling resistance and noise, as well as the aesthetic and functional benefits associated with its use in pedestrian areas and CBDs.

Recommendation

- The cost of noise detailed in the NZTA EEM needs to be evaluated to ensure it is reflecting current user expectations.

9.3 Fatigue

The analysis demonstrated that fatigue cracking for the top of the surface can occur in thin (less than 40mm) layers. The estimation of the level of strain is very dependent on the tyre footprint that is used in the analysis, as well as the pavement structure.

The level of strain on the surface of a thin layer is approximately the same as the level that is estimated at the bottom of a layer 40–60mm thick. This applies for the FEM analysis using a CAPTIF tyre, as well as for the Circly analysis. This implies that the fatigue relationships in Austroads (Jameson 2008) could be used for layers less than 40mm by estimating the maximum strain at the surface or at the bottom of the asphalt layer.

Recommendations

At this stage of knowledge it is recommended that the Circly-calculated results for strains at the top of the pavement, where they are higher than those at the bottom of the asphalt layer, be used as the 'critical' strains for fatigue in the pavement design.

9.4 Shear strength

The strength of asphalt is a function of:

- the aggregate grading
- aggregate particle properties eg % crushed, shape
- binder properties
- degree of compaction
- volume of voids.

New Zealand asphalt mixes are typically composed of crushed coarse and fine aggregate. The literature indicates that the binder properties can play a major role in determining the shear strength and can be more influential than changing the grading from fine to coarse. The changes introduced through the Austroads mix design methods have resulted in a denser grading and a higher level of energy in the compaction of the mix design. It is considered that the changes in the compaction energy, rather than the changes in the aggregate grading, may have more effect on the shear performance of the asphalt produced under the Austroads guide.

The use of the Servopac method of compaction at 120 cycles results in lower air voids in a mix than 75-blow Marshall compaction, and thus mixes designed with this method will have lower binder contents than if Marshall compaction had been used.

Recommendations

The relationship between the plateau density of asphalt that is achieved under different traffic conditions, and the density obtained in the Servopac method of compaction, needs to be quantified

- The benefits of changing to the US Superpave method design, which is continuing to be developed and fine-tuned, should be investigated.
- The effect of changes in binder grade and type on rut resistance, and the effect this could have on fatigue resistance, needs to be quantified and included in the NZTA asphalt guides.
- The use of the Asphalt Mixture Performance Tester in the New Zealand context should be investigated.

9.5 Acceptance schemes

The physical works contract form and the specification requirements can act as an incentive to improve the performance of asphalt. The use of warranties, percent within limits combined with proportional payments, and certified products all have a place in a matrix of methods that can be used, depending on the project.

For thin layers of asphalt, the use of a certification scheme such as HAPAS could encourage the use of proprietary surfacing systems that British experience suggests will result in enhanced lives.

The use of a proportional payment system requires that there is an agreed relationship between a performance or test measure and its effect on life for the scheme to be accepted by industry.

Recommendations

- An approval scheme based on the HAPAS model should be developed and introduced.
- A performance-related specification for asphalt should be developed.
- More emphasis should be placed on 5–10-year performance warranties, initially as trials.
- A statistical approach to acceptance and payment for thin asphalt surfacings should be developed.

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