

# **Investigation of Long-life Pavements in New Zealand**

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# Executive Summary

## Introduction

Most road pavements in New Zealand comprise layers of unbound aggregate with a thin bituminous seal coat or asphalt surface. This type of pavement structure has generally performed well under the traffic loading and environmental conditions of roads in New Zealand.

Where the road is subject to heavy duty loading conditions, the conventional pavement structure described above may not be appropriate. Structural asphalt pavements have been used extensively overseas and are becoming more common in New Zealand for heavy duty applications. A structural asphalt pavement is one where most of the pavement stiffness and load resistance is provided by a series of asphalt layers. Most commonly a structural asphalt pavement consists of three or four layers of asphalt. The lower layer provides high fatigue resistance, the intermediate layers provide rut resistance, and the upper layer is a durable wearing surface providing the required characteristics of skid resistance, stability, noise abatement, etc.

Structural asphalt pavements can provide a number of benefits to road controlling authorities, the most pronounced of which can be greatly extended design lives compared with conventional pavements. It is well established that bitumen oxidises with time and as a result becomes stiffer. This stiffening of the bitumen increases the stiffness modulus of the asphalt mix providing increased distribution of the applied stresses from traffic loading and improved rut resistance of the asphalt layers. The net result is a pavement which is expected to last a long, if not an indefinite, period and can be classified as a long-life pavement.

Note that the word 'asphalt' has been used in this report as a generic term for a mixture of bitumen and aggregate. Asphalt has also been referred to as the 'mix' or 'asphalt mix'. Where the term 'binder' has been used, this shall have the same meaning as 'bitumen'.

## Objectives

The objectives of this project, carried out in 2001-2002, were to evaluate the status of current heavy duty pavements in New Zealand, assess the influence of the long-life pavement concepts on whole-of-life costings, and to develop recommendations for design, construction and maintenance of long-life pavements.

The project involved a literature review of research into the changes in the properties of the bitumen and asphalt mix with time, predominant failure modes in structural asphalt pavements, and the concept of long-life pavements.

## Literature Review

The literature review of overseas research suggests that an increase with time in viscosity of the binder and stiffness modulus of the asphalt mix is common. Research also suggests that these changes vary with depth from the asphalt surface and are less pronounced in the lower asphalt layers.

The review identified surface-initiated cracking as the most common mode of cracking failure in a structural asphalt pavement. Research suggests that the surface cracking resulted from age hardening of the binder, thermal fatigue, and load-induced tensile surface stresses. The concept of surface-initiated cracking is contrary to New Zealand current mechanistic design practice which assumes cracks initiate at the bottom of an asphalt layer caused by fatigue, and propagate upwards.



## **Testing**

Cores were extracted from three sites which comprise structural asphalt pavements of different ages, all from within the Auckland Region. At each site, two test locations were chosen and within each test location cores were extracted from the inside wheel track and between the wheel tracks of the left-hand lane (closest to the shoulder).

Falling Weight Deflectometer (FWD) tests were also performed at each location within the wheel track and between the wheel tracks.

The asphalt cores recovered from the three sites were subjected to both visual inspection and a testing regime to determine the properties of the binder and asphalt mix.

## **Results**

The results of the testing showed no significant difference between the cores extracted within the wheel track and those taken between the wheel tracks. A general trend observed was an increase in the viscosity of the recovered binder with age of the road section. The results also showed a general relationship of increasing stiffness with age.

A semi-log relationship between resilient modulus and binder viscosity is proposed, although this result is qualified by the limited sample size.

The resilient modulus results from the FWD back analysis were found to be reasonably consistent with the laboratory results at all locations except between the wheel tracks at one site.

Cores extracted from cracked sections of structural asphalt pavement showed that cracking in the asphalt had initiated at the surface. No instances of cracking propagating from the asphalt surface layer into the rut-resistant structural layers were found in the study samples.

## **Heavy Duty Pavements in New Zealand**

Of the three pavements investigated, two should achieve a considerable life or long-life status, and one, which had already received numerous asphalt reseals, has not achieved a 25-year design life. Given the limited sample size, no generalisations about the status of heavy duty pavements in New Zealand could be made.

## **Whole-of-Life Costs**

A whole-of-life cost assessment using a hypothetical example showed that the current Transfund New Zealand project evaluation procedure favours structural asphalt, over unbound granular options, on an existing pavement that requires rehabilitation. Because they have higher initial construction costs, thicker long-life pavements are not favoured over thinner structural asphalt pavements using the current process of evaluation.

## **Considerations for a Long-life Pavement**

Research suggests that the structural asphalt pavement should be designed above certain thresholds so that the influence of rutting and fatigue cracking of the structure is minimised.

The concept of the long-life pavement is that resurfacing only will be needed to rejuvenate the pavement as it will remain structurally sound for an indefinite period, provided that loading from traffic is not increased.

This presupposes that the asphalt pavement will be constructed with the best available materials using best practice construction techniques.

Maintenance is required to ensure that any deterioration of the surface, in terms of rutting or cracking, is repaired before the structural capacity of the pavement is compromised. Ensuring adequate pavement drainage is also an important maintenance consideration.

### **Conclusions**

- Top-down cracking was confirmed as the dominant mechanism of fatigue failure in a structural asphalt pavement.
- Core testing showed no significant difference in resilient modulus from cores recovered within the wheel track and between wheel tracks. This suggested that modulus was not influenced by traffic loading.
- The viscosity of the recovered binder increased exponentially with age which resulted in an increase in the resilient modulus of the asphalt mixes.
- A whole-of-life cost assessment showed that the current Transfund New Zealand project evaluation procedure favours structural asphalt, rather than conventional unbound granular options, for an existing pavement that requires rehabilitation. Because of high initial costs, this evaluation process does not, however, favour the long-life pavement over the thinner asphalt pavement.

### **Recommendations**

The following recommendations are based on the results of this research project:

- Further investigation of the design threshold concept should be considered along with an ongoing programme of testing structural asphalt pavements in New Zealand, to gain an improved understanding of their performance.
- Contrary to the current mechanistic design model used in New Zealand, the predominant form of cracking distress in an asphalt layer is initiated at the surface. Further research is required of top-down cracking, and its impact on current design practices should be evaluated.

## **Abstract**

This research report presents a literature review on the topic of long-life pavements and provides details of an assessment of heavy duty structural asphalt pavements in New Zealand, carried out in 2001-2002.

Long-life pavements are structural asphalt pavements which maintain their strength or become stronger with time. This is caused by curing of the binder and results in a pavement which is expected to remain structurally sound for an indefinite period.

Three structural asphalt pavements were identified in the Auckland region from which cores were extracted for testing. These cores were taken from the inside wheel track and between the wheel tracks of the left-hand lane. They were tested to assess the changes in properties of the bitumen and asphalt mix with time, and the effects of traffic loading.

No significant difference was recorded in results between the core samples extracted from in the wheel tracks compared with those taken from between the wheel tracks. This suggests that traffic has little effect on the properties of the asphalt mix. Viscosity of the recovered binder and the resilient modulus of the asphalt mix both increased with age. Cores from cracked sections of pavement were extracted and the cracks were found to initiate at the surface. The reasons for top-down cracking are discussed.

A whole-of-life cost assessment showed that the current Transfund New Zealand project evaluation procedure favours structural asphalt, rather than conventional unbound granular options, for an existing pavement that requires rehabilitation. Because of high initial costs, this evaluation process does not however favour the long-life pavement over the thinner structural asphalt pavement.

## **1. Introduction**

### **1.1 Background**

Most road pavements in New Zealand comprise layers of compacted crushed aggregate with a thin bituminous seal coat or asphalt surface. This type of pavement structure has generally performed well for the traffic loading and environmental conditions usually encountered in New Zealand.

However, where the road is subject to heavy duty loading conditions the conventional pavement structure may not be appropriate. Structural asphalt pavements have been used extensively overseas and are becoming increasingly popular in New Zealand for heavy duty pavements, both in new construction and for rehabilitation. Heavily trafficked motorways and busy city streets are examples of where this type of construction has been used.

A structural asphalt pavement is one where most of the pavement stiffness and load resistance is provided by a series of asphalt layers. Most commonly a structural asphalt pavement consists of three or four layers of asphalt. The lower layer provides high fatigue resistance, the intermediate layers provide rut resistance, and the upper layer is a durable wearing surface providing the required characteristics of skid resistance, stability, noise abatement, etc.

The use of structural asphalt can provide a number of benefits for road controlling authorities, road users and society as a whole, e.g.:

- increased pavement life;
- less use of aggregate resources;
- reduced maintenance requirements;
- short construction time;
- reduced pavement depth;
- minimal conflict with existing buried services;
- reduced road user costs; and
- improved ride quality.

With the increasing use of structural asphalt in New Zealand, the roading industry is challenged with gaining an improved understanding of the criteria for design, construction and long-term performance of structural asphalt. This paper focuses on these issues, overseas experience and research, and discusses their relevance to New Zealand conditions.

Throughout this report, the word 'asphalt' has been used as a generic term for a mixture of bitumen and aggregate. Asphalt has also been referred to as the 'mix' or 'asphalt mix'. Where the term 'binder' has been used in the report, this shall have the same meaning as 'bitumen'.

## 1.2 Objectives

The objectives of this research carried out in 2001-2002 are as follows:

- to carry out a literature review on the topic of long-life pavements;
- to evaluate the status of current heavy duty pavements in New Zealand;
- to assess the influence of long-life pavement concepts on whole-of-life costings; and,
- to develop recommendations for the design, construction and maintenance of long-life pavements.

The project seeks to complement the UK-based research into long-life pavements reported by Nunn (1998) by examining the topic in a New Zealand context.

## 1.3 Current Design Procedure

The current mechanistic design procedure used in New Zealand for designing structural asphalt pavements (AUSTROADS 1992) uses the asphalt fatigue criterion developed by Shell (Claessen et al. 1977) to determine the allowable number of repetitions of a load for a given tensile strain produced by that load. The tensile strain at the bottom of the asphalt layer(s) is determined (using the asphalt fatigue criterion) by the CIRCLY computer model. The design premise is that fatigue cracking will occur at the bottom of an asphalt layer when the horizontal tensile strains, from repeated loading, exceed a tolerable limit.

This design process requires an asphalt fatigue constant to be determined from the stiffness of the asphalt mix and volume of binder for input into the CIRCLY model. The stiffness of the asphalt mix and volume of binder used in the design are those which should be achieved at the time of construction. This asphalt fatigue constant is derived from the following relationship published by Shell 1978:

$$N = \left[ \frac{6918 (0.856V_B + 1.08)}{S_{mix}^{0.36} \mu\epsilon} \right]^5$$

where: N = allowable number of repetitions of the standard load  
 (Equivalent Standard Axle, ESA)  
 $\mu\epsilon$  = tensile strain produced by the load (microstrain)  
 $V_B$  = percentage by volume of bitumen in the asphalt  
 $S_{mix}$  = mix stiffness (modulus) MPa

However, these engineering parameters which govern the service life of the structural asphalt pavement are not constants. It is well established that bitumen oxidises with time and as a result becomes stiffer.

Stiffening of the bitumen influences the stiffness modulus of the asphalt layer and tensile strain at the bottom of the asphalt layer(s) produced by the load. This in turn has an effect on the number of allowable load repetitions for fatigue to occur.

This paper reviews current literature on the topic and presents resilient modulus and binder viscosity test results for core samples taken from three New Zealand pavements.

#### **1.4 Long-life Pavements**

Nunn (1998) reported that, under certain circumstances, structural asphalt pavements maintain their strength or become stronger with time. This is explained by the change in asphalt properties, which stiffen with time. This means that the structural integrity of the pavement can be assured for a very long period, if not indefinitely. Nunn states that pavement design lives of at least 40 years can be cost-effective, even for heavily trafficked routes.

Traditionally structural asphalt pavements in New Zealand have been designed to last 20 to 25 years. Yet with little change to our current design philosophy, significantly increased design lives can be achieved. The concept of long-life pavements is further discussed, and key criteria for their design are reported from overseas research.

#### **1.5 Long-term Performance**

An important implication of Nunn's research is that the performance of structural asphalt pavements is not consistent with the mechanistic model currently used in New Zealand to design the layer thicknesses. The conventional design process involves proportioning the layers so that the horizontal tensile strain at the underside of the asphalt layers is limited to an acceptable level. This presupposes that cracks are initiated at the underside of the asphalt layers and the cracks propagate upward. However, research (Hugo & Kennedy 1985, Gerritsen et al. 1987, Dausatz & Rampal 1987, and others) shows that cracks generally start at the top of the pavement and propagate downward. The surface-initiated cracks can be longitudinal or transverse, and they generally penetrate a distance of up to 100 mm into the asphalt layers.

This top-down cracking in asphalt layers is generally attributed to a combination of environmental and thermally induced stresses, aging of the asphalt at the pavement surface, and the tensile stresses associated with tyre-pavement interaction. This means that the mode of behaviour is quite different from that assumed in the conventional mechanistic design model currently used in New Zealand.

Personal correspondence with Dr Bryan Pidwerbesky, former manager of the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF), in New Zealand, confirmed that all cracking observed at this site has initiated at the surface. The observed cracking at this site was in both thick structural asphalt pavements and thin asphalt surfacings over crushed unbound aggregate.

Cores from cracked pavements in New Zealand have been extracted to confirm the observations and research from overseas and the effect of these findings on current design practices are discussed.

## **2. Literature Review**

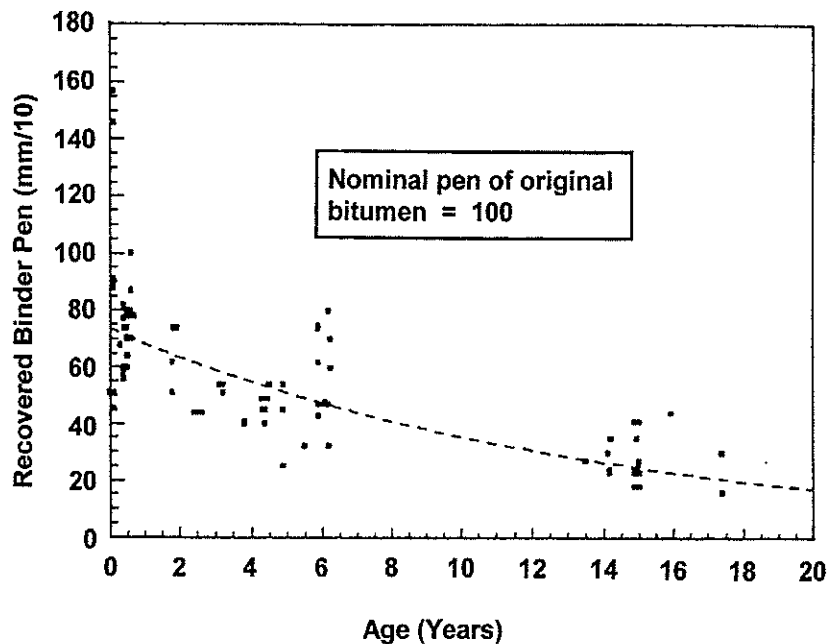
### **2.1 Asphalt Properties**

The Asphalt Handbook (Asphalt Institute 1989) describes asphalt concrete as a high quality-controlled hot mixture of asphalt cement and well-graded, high quality aggregate, thoroughly compacted into a uniform dense mass. The asphalt cement (binder) is described as a dark brown to black cementitious material in which the predominating constituents are bitumens. These bitumens are a by-product of the petroleum industry and contain volatiles which are released over time, causing the bitumen to harden. Most of the changes occur at the time of construction, although the curing of the bitumen continues to occur with time.

Hugo & Kennedy (1985) showed that the aging of asphalt causes an increase in the viscosity and a decrease in the volume of bituminous binders in the asphalt layers of a pavement structure. These changes were not found to be uniform but rather they varied throughout the depth of the asphalt layer with the effect being greatest at the surface. The variation in the viscosity of the binder caused a similar variation in the stiffness of the asphalt.

To gain an improved understanding of how hardening of the binder changed the properties of the asphalt, Hugo (1987) laboratory-aged asphalt samples by oven heating them at 100°C for 7 days followed by UV-radiation for 54 hours. These asphalt samples had been prepared by compacting Marshall briquettes and cutting cores from rolled asphalt slabs. Indirect tensile strength, resilient modulus and viscosity tests were carried out at a range of temperatures. Results of the testing showed a 20% increase for indirect tensile strength and a 17% increase for the resilient modulus, while the viscosity at the surface increased by an average of 500%. The increase in viscosity reduced with increasing depth of the sample below the surface.

The hardening of asphalt was also studied by Chaddock & Pledge (1994) who found that, over a 12-month period, the stiffness modulus of samples from an untrafficked test pavement could increase by over 100%. Most of these changes were attributable to hardening of the binder.



**Figure 2.1** Change in penetration (pen, mm/10) of recovered binder with time (in years) (from Nunn 1998).

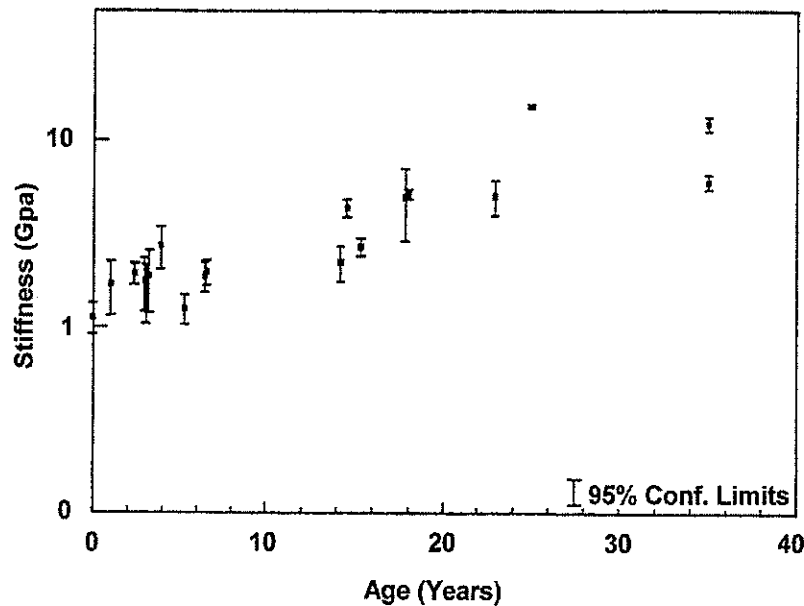
More recently, Nunn (1998) reported on data collected from in-service roads by the Transport Research Laboratory (TRL) in the UK. This database contains the measured properties of the recovered binder and stiffness of roadbase materials from pavements of different ages as shown in Figures 2.1 and 2.2 respectively. Note that the roadbase has similar mix properties to the structural ‘rut-resistant’ asphalt layer used in New Zealand which provides most of the pavement stiffness.

The nominal penetration of the original bitumen was 100, compared with an average recovered binder penetration of 74 at the time of construction. This highlights the significant influence that the production of the asphalt mix and construction practices have on the properties of the asphalt.

The results clearly show a reduction in binder penetration and increase in stiffness as the asphalt ages. Nunn (1996) concluded that “stiffness modulus of macadam roadbase containing 100 penetration grade binder can increase by a factor of 4 or more over 20 years, and the rate of increase decreases with time”.

Potter & Youdale (1998) suggest that, for Australia’s more temperate environment, a stiffness increase of about 6 times may be more appropriate. New Zealand may well fall somewhere in between.





**Figure 2.2** Change in roadbase stiffness (GPa) over time (in years) (from Nunn 1998).

Sharp & Tepper (2001) carried out a study of in-service heavy duty asphalt pavements in Australia and found that the large scatter of results and/or insufficient data prevented any conclusions being reached. There was however evidence of an increase in modulus over time. Little, if any, change to the modulus of the lower layers was found. This supported neither Nunn’s work nor the suggestion made by Potter & Youdale (1998).

## 2.2 Long-life Pavements

The philosophy of the long-life pavement concept is that the pavement has certain threshold strengths beyond which the rate of rutting and fatigue deterioration become negligible. These benefits can be attributable to the asphalt curing with time.

With an increasing stiffness modulus, the asphalt provides increased distribution of the applied stresses and improved rut resistance of the asphalt layers. Pavement performance, as a whole, improves.

In contrast to improved rut resistance, increasing the stiffness modulus of an asphalt layer generally corresponds with a reduction in fatigue resistance and reduced fatigue life of that layer. However, using the relationships developed in the UK study, Nunn found that the increase in stiffness with age produced a reduction in the traffic-induced tensile strain responsible for fatigue at the underside of the roadbase. The net result was that fatigue life can increase with time.

The consequence of the long-life pavement concept is that greatly extended structural lives can be achieved for a given level of axle loading, provided that:

- the pavement has sufficient strength to survive the initial trafficking phase of its life;
- the magnitude of the axle loads remains constant; and
- any deterioration of the surface, in terms of rutting or cracking, is repaired before the structural capacity of the pavement is compromised.

Another way to view this concept is that, even at the end of the pavement's design life, it can be rejuvenated simply by replacing or repairing the surface course. The residual value in the whole-of-life cost analysis will be high due to the relatively low cost to rehabilitate (rejuvenate) the pavement. A conventional unbound granular or thinner structural asphalt pavement will have high rehabilitation costs at the end of the pavement's design life because it has to be modified or replaced. This also raises an interesting point that, if the pavement is rejuvenated by replacing or repairing the surface, there is no such thing as a design life.

To ensure that a pavement achieves a long life, an improved understanding of the predominant failure modes is required so that timely and effective maintenance strategies can be implemented. Uhlmeier et al. (2000) describes a questionnaire completed by 22 European countries in 1999 for the Directorate General Transportation, European Commission. It included a question asking for the most common types of asphalt pavement deterioration. The top three responses were rutting, loss of skid resistance, and surface-initiated cracking, in that order.

Observation and current practice in New Zealand seems to indicate that rutting is not a problem but that surfaces are replaced because of a loss of skid resistance, durability, or environmental issues.

### **2.3 Rutting**

Rutting is typically the result of densification or deformation of one or more of the pavement layers. Nunn (1998) states that deformation within the upper asphalt layers does not have a serious effect on the structural integrity of the pavement. Deformation of the subgrade however is an indication that the load-spreading ability of the pavement is insufficient to protect the subgrade and, if unchecked, will lead to the break-up of the pavement structure. Limiting the vertical compressive strain at the top of the subgrade is an important design criterion. Also the ongoing protection of the subgrade is an important maintenance consideration to achieve long-life status, in particular to ensure that appropriate pavement drainage is maintained.

A summary of the mean rutting rates of pavements with dense bitumen macadam roadbases, reported in the UK study by Nunn, is shown in Figure 2.3.

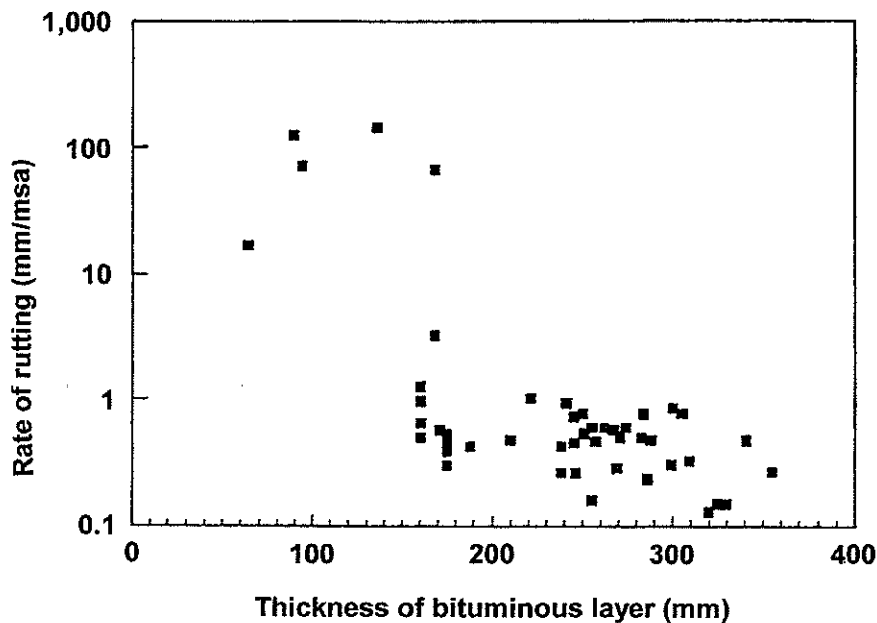


Figure 2.3 Rate of rutting (mm/msa) of trunk roads (from Nunn 1998).  
(msa - million standard axles)

The rate of rutting in Figure 2.3 is the measured rut depth (mm) divided by an estimate of the cumulative traffic (msa, or million standard axles) at the time the rut was measured.

Nunn (1998) states that the threshold pavement thickness for rut resistance in the UK is approximately 180 mm. He found that pavements with more than 180 mm of asphalt showed a rate of rutting about two orders of magnitude lower than pavements with less than 180 mm of asphalt (Figure 2.3). Nunn suggested that the increased rate of rutting in pavements with less than 180 mm of asphalt was caused by the much higher traffic-induced strains in the subgrade.

The UK study by Nunn also found that the rate of rutting was significantly greater on roads with a subgrade CBR (Californian Bearing Ratio) of less than 5% compared with roads with a subgrade CBR greater than 5%. The rate of rutting averaged 0.58 mm/msa where the subgrade CBR was less than 5 compared to an average rate of rutting of 0.36 mm/msa where the subgrade CBR was in excess of 5.

## 2.4 Skid Resistance

Number two on the European list of common modes of pavement distress is loss of skid resistance. Over time, polishing or loss of texture of the surface can be expected to occur under heavy axles or large volumes of traffic and resurfacing will be required. This can be adequately managed by monitoring of SCRIM (Sideways-force Coefficient Routine

Investigation Machine) data and providing triggers for resurfacing to ensure that an appropriate level of skid resistance is maintained, like that used in New Zealand on the State Highway network. However, a review of the loss of skid resistance of pavements is outside the scope of the current research.

## **2.5 Top-down Cracking**

Top-down cracking is the predominant cracking mode of distress recorded on European roads and appears to be common elsewhere overseas. Top-down cracking is not consistent with the current mechanistic design theory used in New Zealand, which assumes that cracks initiate at the bottom of an asphalt layer caused by fatigue, and propagate upwards. This raises the question “are we designing against the predominant mode of failure?”

Hugo & Kennedy (1985) presented findings from a study of surface cracking in South Africa. They found that cracks initiated at the surface as a result of load or non-load associated factors or the interaction of the two. Surface-initiated cracks were observed underneath the rubber pressure pads of a heavy vehicle simulator after the pavement had been subjected to repeated stationary cyclic loads. They attributed the cracks to the presence of horizontal stresses induced by the rubber pads. Thermal effects on the aging asphalt were also found to be a cause of crack initiation.

Gerritsen et al. (1987) reported on the problems that the Dutch Highway Authorities have had with cracking of the surface layers in asphalt roads. These cracks generally occurred in the top layers only and not in the bituminous base layer. Cracks were often observed soon after construction and were located both inside and outside the wheel paths. Where cracking occurred in the wheel track, Gerritsen concluded that radial inward shear forces under the tyres had initiated cracking at the surface near the edges of the tyres. They found that surface cracking outside the wheel paths often indicated that the asphalt had low strength characteristics at low temperatures. The thermally induced stresses were found to be most severe at these low temperatures. Their conclusion was that surface cracking was being caused by both load- and thermally induced stresses.

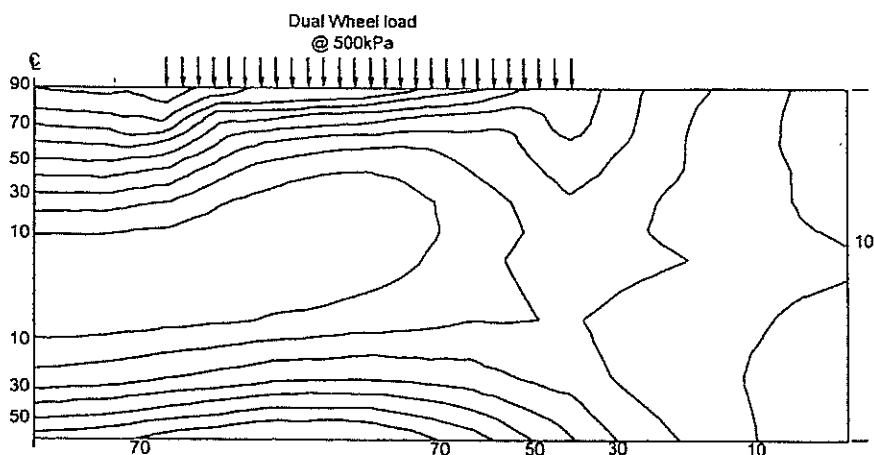
Dauzats & Rampal (1987) reported that assessment of the asphalt pavements on the highways and superhighways in France over a 10-year period had revealed irregularly shaped cracks in the wearing course. These cracks, which were both longitudinal and transverse, were found to affect part or all of the wearing course without threatening the pavement structure. They concluded that cracks were initiated by low temperatures or thermal fatigue and that repeated surface loading caused these cracks to propagate down. The rapid hardening of the binder was also noted as a likely contributor to the pavement distress.

The rationale proposed by Himeno et al. (1987) for surface cracking in Japan was that fatigue failure can initiate at the top of the asphalt layer when the mix stiffness modulus is low. Claessen et al. (1977) had previously indicated that when the mix stiffness

modulus is comparatively low in summer, the largest strain can occur at the top of the asphalt layer. Consequently Himeno concluded maximum fatigue damage happened in the day time in summer. These results were reported as coinciding well with visual observations made by former researchers in Japan who noted less surface cracks where the shadow of an overpass bridge made the pavement cooler.

The conclusion from this early research suggests that the occurrence of surface cracking is a result of traffic loading and thermally induced stresses. Dausatz & Rampal found in Europe that cracking was most likely to occur at low temperatures, while Himeno suggested that cracking was most prevalent during summer in Japan. The effect of the temperature extremes on the asphalt will depend on the temperature sensitivity of the bitumen. At low temperatures the binder is prone to becoming too hard or “brittle” while at high temperature, as Hugo (1987) reported, shrinkage due to heat and evaporation may cause cracking.

The concept of fatigue damage at the top of the asphalt layer is shown in the results of the PACE program described by Rowe et al. (1995). This program uses an elasto-viscoplastic model to compute contours of dissipated energy which relates to fatigue cracking failure and rut profiles. Contours of the percentage fatigue damage in a 200-mm thick, dense bitumen macadam layer are shown in Figure 2.4.



**Figure 2.4** Contours of percentage fatigue damage in 200-mm thick, dense bitumen macadam layer, at 20°C, under 500kPa dual wheel loads (after Rowe & Brown 1997).

The above model indicates that maximum fatigue damage to the asphalt pavement occurs at the top of the layer with the percentage of fatigue damage reducing quickly with depth. This would suggest that cracks which initiated at the surface would penetrate into the asphalt layer until the fatigue energy was no longer of a magnitude sufficient to continue propagation of the crack. The model in Figure 2.4 suggests that, if the stresses are sufficiently high, cracking may also occur at the bottom of the asphalt layer.

A study by Myers et al. (1998) found that surface-initiated cracking in Florida accounted for 90% of the observed cracking in road pavements that were scheduled for rehabilitation. The asphalt thicknesses of the pavements studied were in the range of 50 to 200 mm. The observed cracking was usually longitudinal, with surface crack widths of 3 to 4 mm, decreasing with depth from the asphalt surface. With the use of a computer model, Myers et al. concluded that tensile stresses under the treads of the tyres and not the edges were the primary cause of surface cracking. Wide-base tyres were found to have the highest tensile stresses. They noted that these stresses dissipate quickly with depth and this may be the reason for the cracks to stop growing, as Figure 2.4 suggests. Myers et al. concluded that the surface stresses were relatively independent of pavement structural characteristics and were more related to mixture composition. Specifically they concluded that more fracture-resistant asphalt mixes were required.

Myers et al. (1998) further investigated the concept of contact stresses under truck tyres as a cause of surface cracking in Florida. Tyre contact stresses (vertical, longitudinal and transverse) were measured for both bias-ply and radial truck tyres. They reported that longitudinal surface-initiated cracking in the wheel paths was caused by tensile stresses generated by radial truck tyres. Conversely the bias-ply tyres were not found to induce surface tensile stresses. This conformed well with the observation that surface cracking had become more prevalent as the use of radial tyres increased. Horizontal stresses under the radial tyre were found to be directed away from the centre of the tread resulting in tensile stress of the pavement directly under each tread. Conversely the stresses under the bias-ply tyre were of much lesser magnitude and were directed towards the centre of the tread.

Nunn (1998) found that the longitudinal cracks were either single or multiple cracks in the wheel paths suggesting that traffic-induced stresses played a significant role in their formation. Areas with transverse cracking were often found to have lower binder penetration values compared with uncracked sections. From this Nunn concluded that age hardening of the binder in the wearing course played a part in surface cracking, with hardening over time progressively reducing the ability of the wearing course to withstand the thermal and traffic-generated stresses at the surface.

In summary, the research to date suggests that surface cracking can result from:

- age hardening of the binder which makes the asphalt more susceptible to temperature variations, in particular to low temperatures;
- thermal fatigue;
- load-induced tensile surface stresses.

However, as Nunn stated, “the mechanism of surface cracking is complex and there is no satisfactory explanation of this phenomenon”. Although this calls into question the basis for the current mechanistic design procedure used in New Zealand, still further research is required to better understand the key triggers for the initiation of top-down cracking.

## 2.6 Depth of Cracks

Having established that cracks generally initiate at the surface and propagate downward, a secondary issue is “to what depth do the cracks extend?”

A study by the Dutch Ministry of Transport (Schmorak & van Dommelen 1995) of 176 pavement sections revealed that in pavements with a thickness of asphalt greater than 160 mm, cracks initiated at the surface and penetrated to approximately 100 mm. In thinner pavements the cracks penetrated the full thickness of the asphalt. Structural analysis of the samples suggested that the cracks which had penetrated the full thickness had initiated at the surface and propagated downwards.

This was in agreement with Nunn (1998) who reported that, in the UK, cracking in structural asphalt pavements, which could be longitudinal or transverse, generally only penetrated to 100 mm down from the asphalt surface.

Uhlmeier et al. (2000) extracted cores from a number of cracked sections of road pavement in Washington State, USA. The cores which contained top-down cracks typically were from a thicker pavement structure than the cores that contained full depth cracks. On average, where partial depth cracking was found, the cores were 161 mm thick compared with 107 mm for full depth cracking. The average depth of crack in the thicker asphalt pavements was 47 mm and this was mostly within the wearing course of the asphalt. Engineers in Washington State observed that, where paving operations left surface distress behind the paver screed, top-down cracks were occurring as early as 2 to 3 years after construction. This distress appeared as longitudinal marks (or streaks), occurred in the wheel paths, and gave the mix an open textured appearance. This reinforces the need to ensure that adequate quality control measures are in place during construction to ensure design assumptions are met and appropriate construction techniques used.

The pavement-depth threshold above which full depth cracking was no longer observed has significant maintenance and pavement life implications. Where full depth cracking may require replacement of the pavement, surfacing cracking can be dealt with much more simply by overlays, or mill and replacements, depending on the severity of distress.

## 2.7 Time for Crack Initiation

Generally surface cracks take some time to appear. The reported ages as noted by Uhlmeier et al. (2000) ranged from 1 to 5 years (Japan), 3 to 5 years (France), 5 to 10 years (Florida), to 10 years (UK). New Zealand’s more temperate environment and lower traffic volumes would suggest that the time required for crack initiation should be at the longer end of the range.

This report investigates surface cracking and determines whether such observations made overseas are consistent with our findings in New Zealand.

### 3. Testing

#### 3.1 Core Sampling

Cores were taken for testing from three sites which comprise structural asphalt pavements of different ages, and all within the Auckland Region. These were:

1. The Southern Motorway, northbound, between the Ellerslie/Penrose and Greenlane interchanges;
2. The Strand, northbound, between St Georges Bay Road and Gladstone Road;
3. St Johns Road, northbound, between St Heliers Bay Road and Felton Mathew Avenue.

Two test locations were identified at each site, each separated by approximately 50 m. At each test location, three cores were taken from the inside wheel track in lane 1 and three cores from between the wheel tracks in that lane (left-hand lane closest to the shoulder). All testing was confined to lane 1 as it carries most of the heavy commercial traffic and it minimised traffic disruption during the coring operation. In each instance, lane 1 was the westernmost lane.

An additional two cores were taken from cracked sections of carriageway on The Strand.

#### 3.2 Laboratory Testing

The asphalt cores recovered from the three sites listed above were subject to visual inspection and the following testing regime:

- Resilient modulus  
*(test standard AS 2891.13.1-1995: this test was undertaken to provide an assessment of the stiffness of the asphalt mix and to assess the influence of the hardening of the binder)*
- Binder viscosity  
*(test standard AS 2341.5-1997: to assess how the properties of the binder vary with age)*
- Bulk density  
*(test standard AS 2891.9.2-1993)*
- Maximum density  
*(test standard AS 2891.7.1-1993)*
- Binder content  
*(test standard AS 2891.3.3-1997: to assess whether the volume of binder influenced other properties of the mix)*



- % Voids  
*(test standard AS 2891.8-1993 contains voids and density relationships used to determine % voids in the mix)*

These tests were undertaken on cores recovered from within the wheel track and between the wheel tracks to determine whether traffic loading has had an influence on the properties of the asphalt layers. The different ages of the road sections should also provide an insight into the changes in properties of the bitumen with time.

Testing was carried out at the South Australia Department of Transport Laboratory in Adelaide.

### **3.3 Falling Weight Deflectometer (FWD) Tests**

FWD tests were taken at 10 m intervals on the inside wheel track, and between the wheel tracks, of lane 1 at each site. The tests, which numbered 20 in total, commenced 20 m in advance of the first coring location and terminated 20 m beyond the second coring location.

Deflection readings are a common design tool, used to determine the strength of a pavement and its remaining service life. These results are provided for each road section for comparison with the back-analysed subgrade CBR and asphalt resilient modulus, providing an indication of the stiffness of the pavement structure.

## 4. Results and Discussion

### 4.1 Core Details

Typical details of the cores extracted from the three road pavements in Auckland are provided in Table 4.1.

**Table 4.1 Details of the cores from the three sites on Auckland roads.**

Site	Age <sup>(1)</sup> (years)	Asphalt Depth/ Core Length (mm)	No. of Asphalt Layers	Approx. Layer Thickness (mm)	Layer details (name)
Southern Motorway	33	265 - 280	4	40	20 mm open-graded porous asphalt surfacing (friction mix)
				20	Levelling layer (or previous surfacing)
				100	40 mm max. nominal aggregate size, bitumen-treated basecourse (upper structural)
				120 - 135	40 mm max. nominal aggregate size, bitumen-treated basecourse (lower structural)
The Strand	19	140 - 190	3 - 4	30	Mix 14 wearing course surfacing
				30 - 60	Mix 10 or mix 14 old asphalt surfacings
				80 - 100	Mix 40 structural asphalt layer
St Johns Road	2	180 - 220	3	35	Mix 14 wearing course surfacing
				95 - 125	Mix 40 structural asphalt layer
				50 - 60	Mix 20 high fatigue asphalt layer

Notes:

<sup>(1)</sup> Age of the structural asphalt layers. The open-graded surface on the Southern Motorway was constructed in 1992, and The Strand was resurfaced most recently in 1999. Only the structural and high fatigue layers are of interest as they provide most of the pavement stiffness.

Photographs of a typical core from each of the three sites have been included in the Appendix.

### 4.2 Laboratory Results

The results from the laboratory testing carried out by the Australian laboratory are presented in Table 4.2.

**Table 4.2 Summary of laboratory test results from cores taken at the three sites.**

Site	Layer	Core location (IWT/ BWT)	Number of Cores	Resilient Modulus (MPa)	Voids %	Binder Viscosity (log Pa.s)	Binder Volume (%)
Southern Motorway	Friction mix surfacing	IWT	1	5448	19.4	7.49	13.3
		BWT	2	5028	18.2		
Southern Motorway	Upper structural	IWT	6	9341	4.8	5.43	12.4
		BWT	6	8094	4.9	5.7	11.4
Southern Motorway	Lower structural	IWT	6	10388	3.4	6.98	12.2
		BWT	6	11317	3.5	7.31	12.7
The Strand	Structural	IWT	6	3019	4.4	5.02	12.7
		BWT	5	4368	4.9	4.87	12.2
St Johns Road	Surfacing	IWT	1	1793	8.5	5.1	-
		BWT	2	2350	11.1		
St Johns Road	Structural	IWT	6	2359	2	4.46	12
		BWT	6	2031	4.2	4.39	11.3
St Johns Road	High fatigue	IWT	6	1928	4.5	4.79	13.2
		BWT	3	1827	1.9	4.51	14.7

IWT - in wheel track; BWT - between wheel track

Table 4.2 shows no significant difference in results between the core samples taken in the wheel tracks and those taken outside the wheel tracks. The core samples extracted from the wheel tracks did not consistently have a lower voids content or higher resilient modulus as could be expected. With such a limited sample, drawing too many conclusions from it is unwise, although the results would tend to suggest that traffic loading has had a minimal effect on the properties of the asphalt.

The study by Nunn (1998) found that the level of traffic was not a major factor affecting the residual fatigue life of the road base layers. The differences in fatigue life were accounted for by variations in material composition rather than by differences in traffic loading. Volume of binder accounted for the largest amount of variability followed by the penetration of the recovered binder. Unlike the UK study, the volume of binder at each site did not change significantly depending on age of the mix, nor was there any apparent relationship between the volume of binder and resilient modulus or viscosity.

It would therefore seem reasonable to consider the same to be true for the resilient modulus of the asphalt mix. That is, stiffening of the asphalt mix with age is governed by the properties of the binder and environmental factors, and not by traffic loading. Of

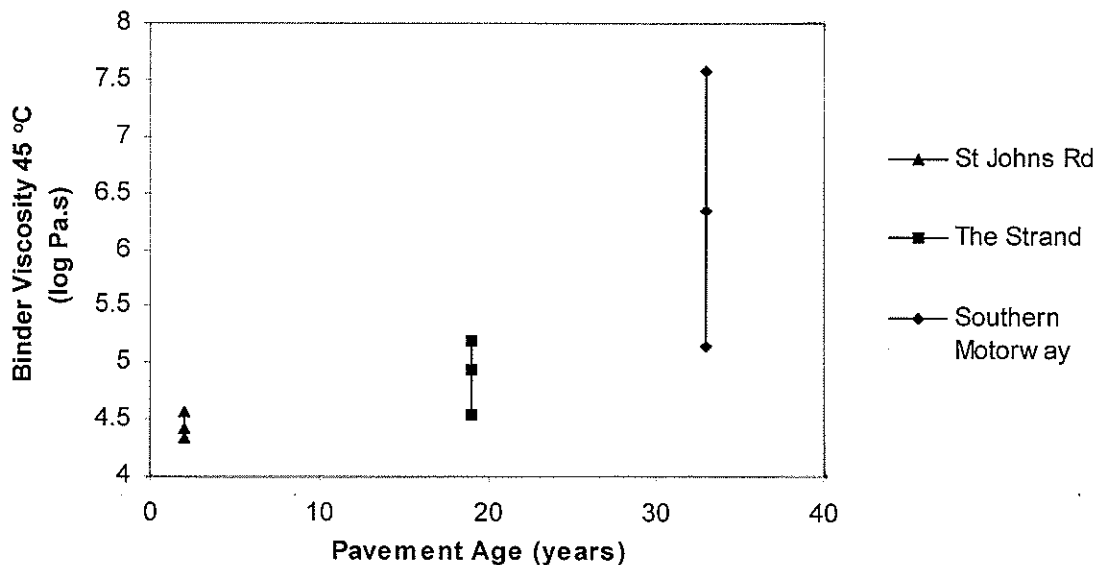
interest in this regard is the stiffness of the friction mix surfacing on the Southern Motorway which, with a modulus of 5 GPa, is very high for an open-graded mix. The viscosity result would suggest the binder has hardened significantly which may explain the high modulus results. Note however that the voids in the friction mix over time fill with grit which may provide additional stiffness.

#### 4.2.1 Viscosity of Binder

A general trend observed from the results was an increase in the viscosity (in Pa.s) of the recovered binder with age of the road section. The viscosity of the binder in the asphalt surface layer that was measured at St Johns Road and the Southern Motorway was found to be higher than the viscosity of the other lower asphalt layers. This was of interest on the Southern Motorway which was resurfaced in 1992 and is therefore much younger than the structural asphalt layers.

The results at the Southern Motorway also showed higher viscosity results for the lower structural layer compared with the structural layer directly above, suggesting that the lower layer is older. Correspondence with past Ministry of Works employees indicated that the road may have been opened to traffic for a relatively short period (1-2 years) before completing the full depth of asphalt layers. The viscosity results would suggest that the lower structural layer was laid initially, and used for about 1 to 2 years before the rest of the asphalt layers were constructed, although this could not be confirmed. The range of recovered binder viscosity at each of the three sites is plotted in Figure 4.1.

**Figure 4.1** Range of recovered binder viscosities (log Pa.s) for structural asphalt layers of different ages at the three sites.

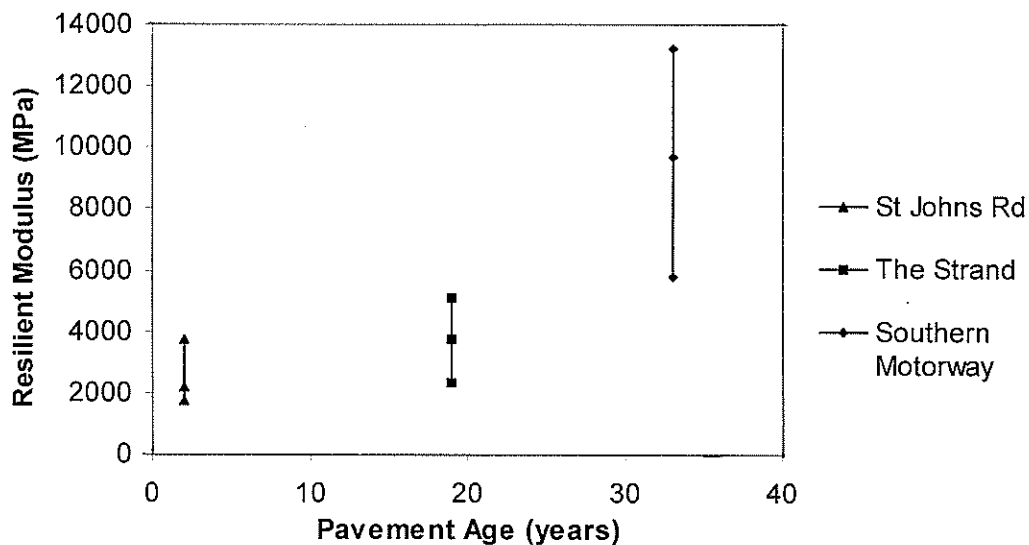


Assuming little variation in the viscosity of the binder at the time of construction, Figure 4.1 clearly illustrates an exponential increase in binder viscosity with time. The viscosity of a typical 180/200 or 80/100 grade binder at 45°C is expected to be about 2.9 and 3.3 log Pa.s respectively. A viscosity increase of 3- to 4-fold could be realised during the mixing and paving operations taking the viscosity of a typical 180/200 and 80/100 grade binder to as high as 3.4 and 3.9 log Pa.s respectively. At St Johns Road a slightly lower penetration 60/70 grade binder was used for the structural layer which seems to correspond well to the results in Figure 4.1. The corresponding changes in the resilient modulus of the asphalt mix are shown in Figure 4.2.

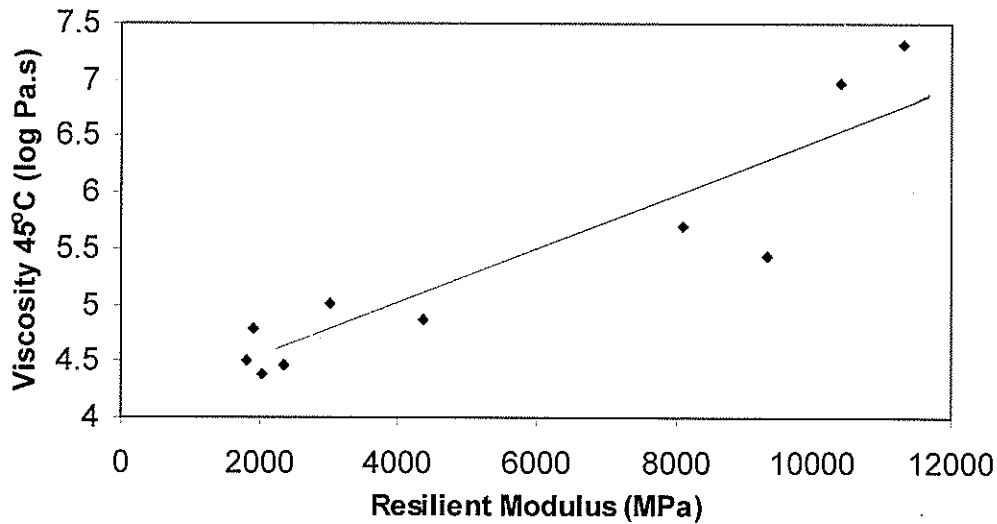
#### 4.2.2 Resilient Modulus

The lines plotted in Figure 4.2 show the spread and mean of the resilient modulus results at each site. Assuming a similar asphalt stiffness for all three sites at the time of construction, Figure 4.2 indicates a general relationship of increasing stiffness with age. The measured stiffness of the asphalt mixes at St Johns Road and The Strand show a lower stiffness increase with time than overseas research would suggest. In fact, there is little improvement from a stiffness of 2000 to 3000 MPa, which could be expected at the time of construction.

Figure 4.2 Range of resilient moduli (MPa) for structural asphalt layers of different ages at the three sites.



The study by Nunn in the UK found that, over a 20-year period, a stiffness increase of 4-fold could be expected. This increase was expected to be conservative for New Zealand's more temperate environment. However the level of stiffness increase found by Nunn was not confirmed by the results at The Strand. The resilient modulus results for the Southern Motorway were within the range of results as those found for pavements of a similar age in the UK.



**Figure 4.3** Change in resilient modulus (MPa) with increasing viscosity (log Pa.s) for the structural and high fatigue asphalt layers.

Figure 4.1 confirms the significant influence that hardening of the binder has on the resilient modulus of the asphalt mix. This relationship has been plotted in Figure 4.3.

The results in Figure 4.3 indicate a semi-log relationship between binder viscosity and resilient modulus. The slope of the above regression line is  $2.39 \times 10^{-4}X + 4.0$ , and the data has a Pearson's correlation ( $r$ ) of 0.9. Any apparent relationship between the viscosity of the binder and resilient modulus of the mix should be qualified in recognition of the limited sample, the inconclusive volume of binder results, and limited knowledge of the particle size distribution.

Of the three sites from which the cores were extracted, St Johns Road was the only site where we were able to obtain results of the asphalt mix testing before construction.

#### 4.2.3 Design Verification

Samples of the asphalt mix from the mixing plant were sent to Australia for testing to confirm the design assumptions, made by the pavement designers, at the time of construction. The asphalt was supplied to the South Australia Department of Transport laboratory, Adelaide, as loose mix, where it was reheated, and immediately compacted into a slab using a slab compactor with footpath roller (AUSTROADS AST 05:1999). Cylinders of asphalt for testing were extracted from the slab. The results of these tests compared to the testing carried out on the core samples is presented in Table 4.3.

The test results show a higher resilient modulus for the asphalt mix at the time of construction when compared with the aged core samples. This appears to be in conflict with the aforementioned research from overseas, and with the core samples extracted

from the three road pavements in New Zealand which show an increase in resilient modulus with time.

**Table 4.3 Laboratory test results of plant mix compared with core samples.**

Site	Asphalt mix	Plant Test Results <sup>(1)</sup>		Core Sample Test Results <sup>(2)</sup>	
		Resilient Modulus (MPa)	Voids %	Resilient Modulus (MPa)	Voids %
St Johns Road	AC40 (Structural mix)	4526	3.8	2359 (IWT)	2
				2031 (BWT)	4.2
	AC20 (High Fatigue mix)	2465	5.5	1928 (IWT)	4.5
				1827 (BWT)	1.9

- Notes: (1) Asphalt plant mix test results before construction.  
 (2) Core samples tested were from either IWT (in the wheel track), or BWT (between the wheel tracks) 2 years after construction.

Possible reasons for the higher than expected test results of the plant mix could include age hardening of the plant mix during the reheating process in the laboratory before compaction. The largest difference was noted for the AC40 mix which could suggest that segregation is occurring with the larger aggregate.

Generally the voids in the laboratory-tested plant mix are higher than the voids in the core samples as expected.

### 4.3 FWD Results

Nunn (1998) proposed that with an increase in elastic stiffness the resulting improvement in load spreading ability should manifest itself as a reduction in deflection over the life of the road. The FWD results for each road section are presented in Table 4.4.

The resilient modulus results from the FWD back analysis are reasonably consistent with the laboratory results at all locations except between the wheel tracks at The Strand where the FWD result is about half that of the laboratory test (Table 4.2).

The deflection results for the Southern Motorway are very low and a reasonable conclusion would be that they have become lower since the time of construction. Without historical deflection data, an assessment of the magnitude of any reduction in deflection can not be made.

Discussion with past Ministry of Works employees indicates that a significant portion of this section of motorway was constructed on rock, which may also explain the low deflections.

**Table 4.4 FWD test results for the entire pavement of the three sites.**

Site	Core location IWT, BWT	Back calc'd Resilient Modulus (MPa)	90 <sup>th</sup> Percentile deflection	Representative Subgrade CBR
Southern Motorway	IWT	9278	0.13	5
	BWT	8669	0.10	6
The Strand	IWT	2961	1.09	2
	BWT	2179	0.82	3
St Johns Road	IWT	1752	0.49	2
	BWT	1847	0.50	3

IWT - in wheel track; BWT - between wheel track

The deflection data indicate that the pavement at the Strand has less inherent strength than either the Southern Motorway and St Johns Road. This could be the result of insufficient subgrade protection and perhaps explains why the pavement has received numerous asphalt surface layers as shown in Table 4.1.

Repeated deflections of the magnitude found at The Strand may be sufficient to form microcracks within the lower asphalt layer caused by fatigue. This would explain the lower than expected resilient modulus laboratory test results in the wheel track.

#### **4.4 Top-down Cracking**

Two cores were taken from a section of pavement that had transverse cracking on The Strand. Photos of these cores, and one other from a heavy duty container yard with surface cracking are presented in Figure 4.4.

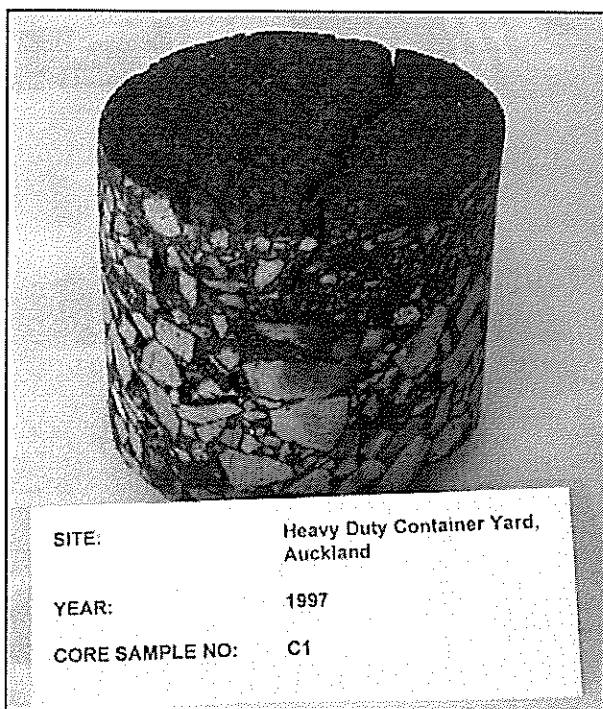
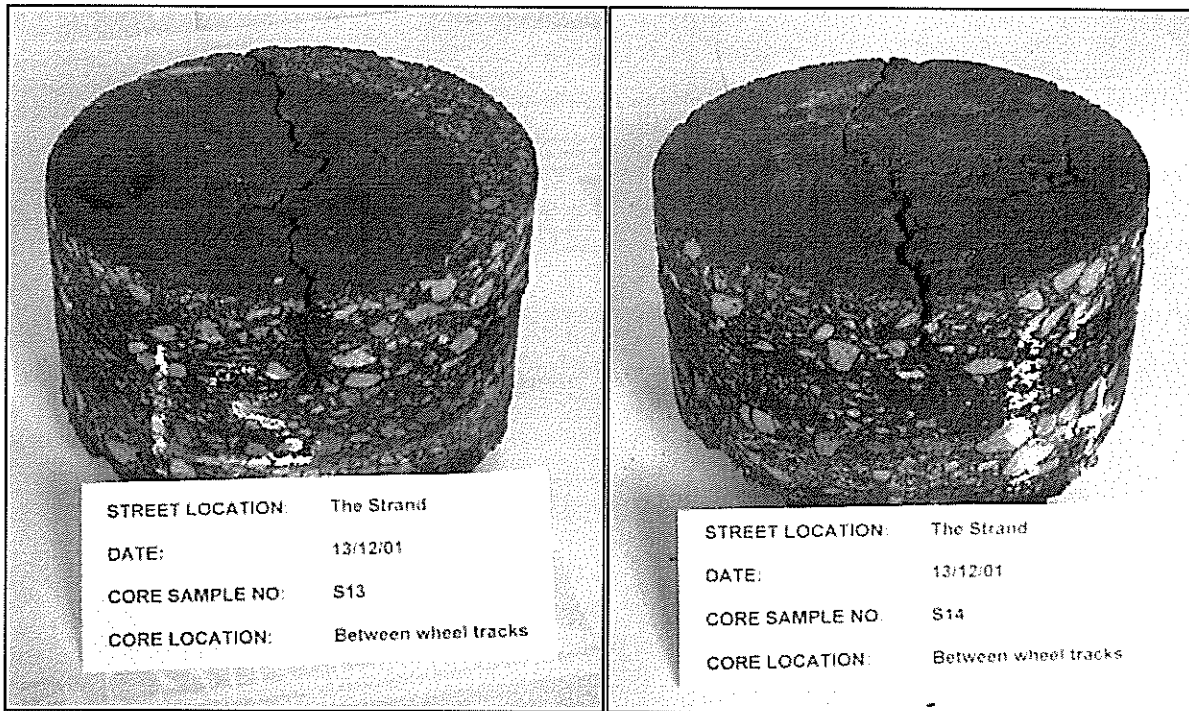
The cores in Figure 4.4a, b show the multiple asphalt surface layers noted earlier on The Strand. Cracking has occurred through the top wearing course layer which was laid in 1999, without propagating further. The cracks were up to 4 to 5 mm in width, and were 20 to 35 mm in depth, all within the upper wearing course layer.

Independently of this project, cores were extracted from an asphalt pavement constructed for a container yard with heavy duty loading conditions. The core in Figure 4.4c clearly shows surface cracks which were up to 3-4 mm in width, and to 30 mm in depth.



Figure 4.4 Examples of top-down surface cracking.

a, b. Surface cracking between wheel tracks in The Strand (upper left & right).



c. Surface cracking from heavy duty container yard, Auckland City (left).

This is consistent with observations made overseas which found that cracking in the asphalt was initiated at the surface. No instances of cracking in the asphalt surface layer that propagated into the rut-resistant structural layers were found in the limited study sample.

The limited number of cores exhibiting surface cracking that could be recovered from pavements in Auckland, reflect two situations: very few structural asphalt pavements have been constructed in Auckland; and even fewer of them show signs of cracking distress. This may also reflect the relatively young age of many of the structural asphalt pavements in Auckland which have been constructed within the last 3 years.

## 5. Heavy Duty Pavements in New Zealand

The three road pavements from which the cores were extracted are quite dissimilar in respect to age, depths of asphalt layers, and the roading industry's knowledge of design and construction practices at the time these pavements were built.

### 5.1 St Johns Road

Heavy duty pavements are now designed in accordance with the current mechanistic procedures described in AUSTROADS (1992). These procedures were used to design St Johns Road utilising a 3-layer approach, i.e. a durable surface wearing course overlying a rut-resistant structural layer, which in turn overlies a high-fatigue bitumen-rich layer. Laboratory testing was arranged by the Contractor to verify key performance parameters before construction. Laboratory testing is now widely used by designers and contractors alike to verify design assumptions and to gain an improved understanding of the performance of asphalt materials. Although the laboratory aims to ensure that the specimens they prepare for testing have properties as similar as possible to those of the asphalt placed on the road, the results at St Johns Road show that significant variation in results can still occur.

The results from the laboratory testing are inconclusive to decide if St Johns Road will achieve a design life of 40 years or more, i.e. a pavement which Nunn calls as 'long life'. The results do not show an improvement in the stiffness of the asphalt materials since construction, and the resilient modulus is perhaps below that which would be reasonably assumed in the design process. However the thickness of the asphalt layers exceeds that which Nunn found to be prone to increased rutting and full depth cracking as observed overseas. Therefore, as long as the pavement is able to withstand the current traffic loads, and the structural properties of the asphalt improve with time, then the pavement should have a considerable life.

### 5.2 The Strand

The structural asphalt layer at The Strand was constructed in 1983 and since then further asphalt surface layers have been provided. This pavement was constructed before the National Roads Board (NRB) *State Highway Pavement Design and Rehabilitation Manual* was first issued in 1987. Research at that time which may have been available to the designers included *The Shell Pavement Design Manual* (1978) and *Recommended Procedure for Design of Asphaltic Concrete Pavements in New Zealand* by Boon (1979).

Clearly the test results and subsequent maintenance programme show that this pavement has not achieved a 25-year design life. This pavement was the only site showing visible signs of cracking distress that were found to be confined to the surface layer. The Strand is an arterial road linking the Southern Motorway with Auckland Port and carries

significant volumes of heavy commercial traffic. By current design standards, the pavement is simply too thin to cater for the applied loading.

### 5.3 Southern Motorway

The section of Southern Motorway where the cores were taken is believed to have been reconstructed with asphalt in 1969. The extent of information available about the design and construction of this pavement is very limited. This pavement is one of the first sections of bitumen-treated granular pavement constructed in New Zealand. The pavement has lasted over 30 years and the test results suggest that the load-carrying ability of the pavement structure is improving with time. This pavement should achieve long-life status.

Assuming that the Southern Motorway was to be built today, using the current mechanistic procedures in AUSTROADS (1992) and the CIRCLY computer program, a simplified pavement design model has been provided to demonstrate the impact of the increasing resilient modulus of the asphalt with age. Three iterations are provided (Table 5.1), with Pavement Model 1 assuming a set of parameters at the time of construction, Model 3 utilising the parameters obtained from the laboratory testing of the core samples, and Model 2 representing a set of intermediate values.

**Table 5.1 Comparison of the three pavement models with new versus aged resilient modulus for structural layers, and related to traffic volumes.**

Pavement Model	Layer	Thickness (mm)	Resilient Modulus (MPa)	Volume of Bitumen <sup>(1)</sup> (%)	Output Damage Factor <sup>(2)</sup>	Design Traffic (ESA) <sup>(3)</sup>
1	Asphalt surface <sup>(4)</sup>	40	2700	13.7	n/a	3.5 x 10 <sup>7</sup>
	Structural layer	230	2500	12	0.84	
	Base layer	150	150	n/a	n/a	
	Subgrade	∞	50	n/a	0.001	
2	Asphalt surface <sup>(4)</sup>	40	2700	13.7	n/a	1 x 10 <sup>8</sup>
	Structural layer	230	5000	12	0.83	
	Base layer	150	150	n/a	n/a	
	Subgrade	∞	50	n/a	0.002	
3	Asphalt surface <sup>(4)</sup>	40	2700	13.7	n/a	7 x 10 <sup>8</sup>
	Structural layer	230	9000	12	0.85	
	Base layer	150	150	n/a	n/a	
	Subgrade	∞	50	n/a	0.005	

Notes to Table 5.1:

- (1) The volume of bitumen has been assumed to be constant for the asphalt surface layer (13.7%) and for the structural asphalt layer (12%) for the purpose of this exercise.
- (2) Output damage factor of less than 1 is required to ensure the service life meets or exceeds the design life.
- (3) Design traffic is in ESAs (equivalent standard axles)
- (4) Asphalt surface layer is assumed to have resilient modulus of 2700 MPa.

Table 5.1 shows that increasing the resilient modulus of the structural asphalt layer by a factor of 2 (Model 2) and nearly 4 (Model 3), has resulted in an increase in the design traffic loading by approximately 3 and 20 times respectively. This suggests that the southern motorway has a very long, if not indefinite, residual life.

Given the limited study sample, generalisations are difficult to make about the status of current heavy duty pavements in New Zealand. The philosophy of the long-life pavement concept, as previously mentioned, is that certain pavement strength thresholds exist beyond which the rate of rutting and fatigue deterioration become negligible. To gain a better understanding of the status of New Zealand heavy duty pavements, an improved understanding is needed of the predominant mode of pavement distress caused by fatigue and by deformation of the pavement layers. Overseas research and local observations have shown that this fatigue distress causes top-down cracking, with the depth of the cracks dependent on the thickness of the asphalt layers. Pavement deformation (rutting) was also shown by Nunn to be a function of pavement layer thicknesses and subgrade support. This suggests that thresholds exist beyond which the rate of rutting and fatigue deterioration become negligible. The results of the research discussed in this report will provide a good basis for comparison with further local research which is required in New Zealand and Australia.

#### **5.4 Whole-of-Life Costs**

A pavement which is designed to be above the threshold strength that is required to achieve a long life, as Nunn reports, should remain structurally sound for at least 40 years. A long-life pavement has an increased depth of the same asphalt product as opposed to a thinner pavement which is designed to last 20 to 25 years. The long-life pavement will therefore have a greater initial construction cost than a thinner pavement, although this will be balanced over a period of time by lower periodic maintenance and replacement costs. The main benefits from a long-life pavement, as opposed to those of a thinner pavement however, are not realised until the thinner pavement requires replacement. At this stage the thinner pavement will incur the construction cost of replacement, as well as the road-user costs related to delays during construction.

To gain a better understanding of the influence of the pavement life on the whole-of-life costs, an assessment of these costs has been made in this research for a sample 1000 m<sup>2</sup> rehabilitation site, based on similar assessments made in actual projects. The existing pavement to be rehabilitated has been assumed to be unbound granular and the proposed rehabilitation treatment options include unbound granular and structural asphalt.

**Option 1** involves removing the existing pavement and undercutting to achieve 500 mm of compacted granular material.

**Option 2** involves removing the upper, say, 160 mm of existing pavement, and providing a structural asphalt pavement of the same depth. For this exercise the assumption is that the pavement in Option 2 will achieve a 25-year design life with sufficient maintenance.

**Option 3** involves removing the upper 190 mm of existing pavement and providing a structural asphalt pavement of the same depth. For this exercise we have assumed that this depth of pavement will achieve a design life of 40 years or more, and can therefore be classified as a long-life pavement. The present value costs of Option 3 will be assessed over a 25-year period and residual (salvage) value of the pavement will be assessed in year 26.

This cost analysis shows that the discounted whole-of-life costs for Options 1 and 3 are about 20% and 6% respectively higher than Option 2. For the purpose of this exercise we have assumed that Options 1 and 2 will have no residual life at the end of the 25 years and will require rehabilitation.

In contrast, Option 3 will achieve a design life of 40 years and, depending on the integrity of the structural layers of the pavement at that time, may continue to perform satisfactorily well after 40 years with periodic replacement of the surface.

The current process evaluates a pavement treatment option over a 20- or 25-year period. This example has used a 25-year period and on this basis Option 2 would be most favourable, even though the pavement would need to be replaced at the end of this period.

**Option 1**

Item	Description	Amount (NZ\$)	Present Worth factor	Discounted Cost (NZ\$)
1	Cost of Works (\$100/m <sup>2</sup> )	100,000	0.91	91,000
2	Cost of Annual Maintenance in year one			500
3	Cost of Annual Maintenance following works	250	8.57	2,143
4	Periodic Maintenance Costs - Mill, Levelling course, and AC reseal • year 8 • year 16 - Holding seal • year 24	20,000 20,000 5,000	0.4665 0.2176 0.0923	9,330 4,352 462
5	<i>less</i> Year 26 Salvage value			0
	<b>Total Discounted Option Costs</b>			<b>\$107,787</b>

**Option 2**

Item	Description	Amount (NZ\$)	Present Worth factor	Discounted Cost (NZ\$)
1	Cost of Works (\$80/m <sup>2</sup> )	80,000	0.91	72,800
2	Cost of Annual Maintenance in year one			500
3	Cost of Annual Maintenance following works	250	8.57	2,143
4	Periodic Maintenance Costs - Mill, Levelling course, and AC reseal • year 8 • year 16 - Holding seal • year 24	20,000 20,000 5,000	0.4665 0.2176 0.0923	9,330 4,352 462
5	<i>less</i> Year 26 Salvage value			0
	<b>Total Discounted Option Costs</b>			<b>\$89,587</b>

**Option 3**

Item	Description	Amount (NZ\$)	Present Worth factor	Discounted Cost (NZ\$)
1	Cost of Works (\$90/m <sup>2</sup> )	90,000	0.91	81,900
2	Cost of Annual Maintenance in year one			500
3	Cost of Annual Maintenance following works	250	8.57	2,143
4	Periodic Maintenance Costs - Mill, and AC reseal • year 8 • year 16 • year 24	17,000 17,000 17,000	0.4665 0.2176 0.0923	7,931 3,699 1,569
5	<i>less</i> Year 26 Salvage value	90,000 x(15/40)	0.0839	-2,832
	<b>Total Discounted Option Costs</b>			<b>\$94,910</b>

The following additional considerations are worthy of note:

- This assessment is sensitive to the discount factor used, which in this instance was 10%. A lower discount factor would increase the salvage value of Option 3 and increase the maintenance costs of Options 1 and 2, compared with Option 3.

5. *Heavy Duty Pavements in NZ*

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- The vehicle operating costs (VOC) would be higher for Option 1 as a result of the increased pavement deflection and deterioration. VOCs would also be higher during the reconstruction of Options 1 and 2 in year 26.

The above assessment shows that current procedures for determining life-cycle costs favour structural asphalt, over unbound granular options, where an existing pavement requires rehabilitation. The current process does not, however, favour the long-life asphalt pavement over a thinner asphalt pavement because of the higher initial cost.



## 6. Considerations for a Long-life Pavement

### 6.1 Design

As previously discussed (Sections 4.2.1 and 4.2.2), the properties of the asphalt change with time because the binder undergoes age hardening. This results in an improvement in the stiffness or resilient modulus of the asphalt and a reduction in the critical horizontal stresses responsible for structural fatigue cracking. The net effect is that the pavement becomes less vulnerable to traffic load-induced stresses with time. Nunn reported that, as long as the road is built strongly enough initially and its structural properties have time to improve, then the structural integrity of the pavement can be assured for a very long time. Therefore the initial strength of the road should be a major factor in determining its future life.

The asphalt pavement must also be designed above certain thresholds above which the influence of rutting and fatigue cracking of the structure of the pavement is minimised. The concept of the long-life pavement is that only resurfacing will be required to rejuvenate the pavement because it will remain structurally sound for an indefinite period.

Nunn found that pavements with an asphalt thickness of at least 180 mm deformed at a much slower rate than thinner pavements. The increased level of deformation in the thinner pavements implies that structural damage was occurring to the pavement, while rutting of the thicker pavements was non-structural and contained within the upper pavement layers. This implies that, as a rule of thumb, a pavement consisting of at least 180 mm of asphalt is less likely to sustain structural deformation distress which could lead to the break up of the pavement. This will obviously be very dependent on the stiffness of the underlying pavement layers, in particular of the subgrade.

The pavement must also be designed against occurrence of the predominant cracking mode of pavement distress which initiates at the surface and continues downwards. This is contrary to the current New Zealand mechanistic design practice and requires further research to develop an appropriate theoretical model for predicting this type of fatigue.

Overseas research revealed that pavements with an asphalt depth of less than 160 mm were prone to develop cracks that penetrated the full depth of the asphalt. Where cracks developed in pavements having an asphalt depth in excess of 160 mm, they generally only penetrated up to approximately 100 mm from the surface. This implies that a pavement with an asphalt layer thickness less than 160 mm is prone to develop cracking which will compromise the structural integrity of the pavement.

The concept of designing above certain minimum thresholds that have been identified from field studies is worthy of consideration and will require further field testing to confirm studies overseas.

## **6.2 Construction**

Much of the variation observed in the performance of pavements both in New Zealand and overseas is a result of variable material quality and different construction techniques. The process of obtaining an increased life from an asphalt pavement requires use of the best available materials, utilising best practice construction techniques. This will limit the potential for failure early in the life of a pavement before the structural properties of the asphalt have had time to improve.

## **6.3 Maintenance**

The ongoing maintenance requirements for a structural asphalt pavement include:

- ensuring that any surface cracking is repaired before it starts to compromise the structural integrity of the pavement,
- maintaining adequate drainage provisions to ensure softening of the lower pavement layers does not occur, in particular the subgrade.

## 7. Conclusions

The objectives of this research paper that have been achieved were:

- to carry out a literature review on the topic of long-life pavements;
- to evaluate the status of current heavy duty pavements in New Zealand;
- to assess the influence of long-life pavement concepts on whole-of-life costings; and,
- to develop recommendations regarding the design, construction and maintenance of long-life pavements.

The main conclusions to be drawn from this study are:

- Top-down cracking was confirmed as the dominant mechanism of fatigue failure in a structural asphalt pavement;
- Core testing showed no significant difference in resilient modulus from cores recovered within the wheel track and between wheel tracks. This suggested that modulus was not influenced by traffic loading.
- An increase in resilient modulus with age was shown to be consistent with the work undertaken by Nunn (1998).
- Viscosity of the recovered binder increased exponentially with age which resulted in an increase in the resilient modulus of the asphalt mixes.
- A whole-of-life cost assessment showed that the current Transfund New Zealand project evaluation procedure favours structural asphalt, rather than conventional unbound granular options, for an existing pavement that requires rehabilitation.

However, because of high initial costs, this evaluation process does not favour the long-life pavement over the thinner structural asphalt pavement.

## 8. Recommendations

- Further investigation of the design threshold concept should be considered, along with an ongoing programme of testing structural asphalt pavements in New Zealand, to gain an improved understanding of their performance.
- Contrary to the current mechanistic design model used in New Zealand, the predominant form of cracking distress in an asphalt layer is initiated at the surface. Further research is required of top-down cracking, and its impact on current design practices should be evaluated.

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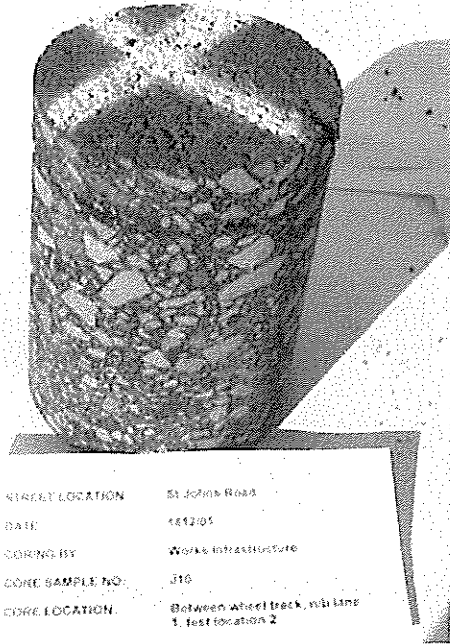
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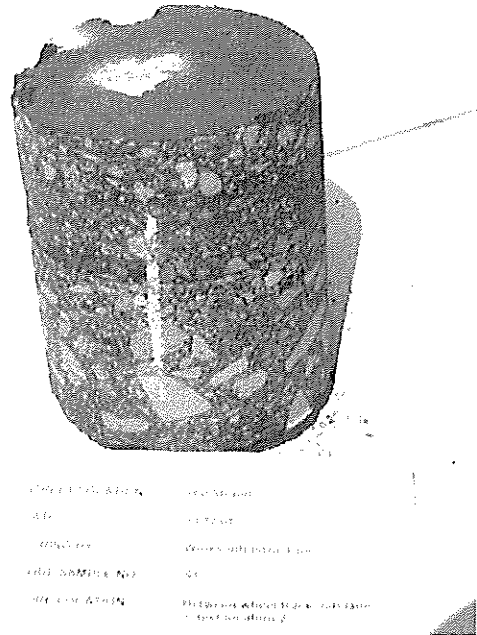
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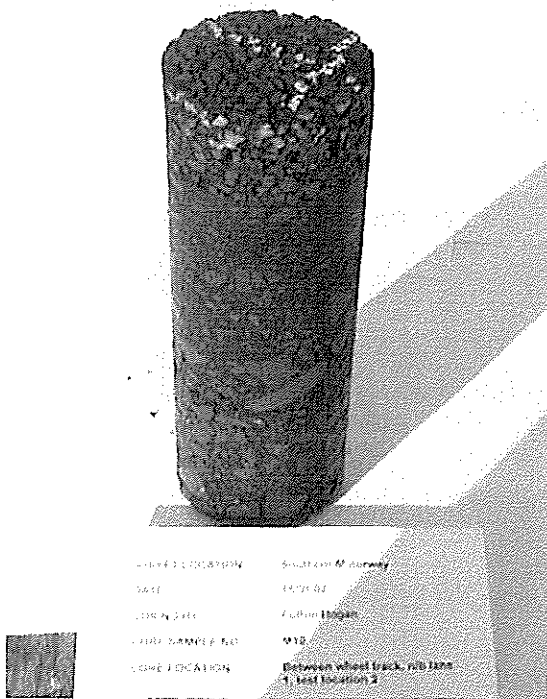
## Appendix Photographs of Typical Extracted Cores



**Photo1: St Johns Road**



**Photo 2: The Strand**



**Photo 3: Southern Motorway**



