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Abbreviations and acronyms

Austroads	Australian Road Research Board
dTIMS	Deighton's Total Infrastructure Management System
EU	European Union
FWD	falling weight deflectometer
ITS	indirect tensile strength
ITT	indirect tensile test
LTNZ	Land Transport New Zealand
MPD	mean profile depth
NAASRA	National Association of Australian State Road Authorities
OGPA	open graded porous asphalt
Pen	binder penetration (bituminous binder @ 25ºC)
PI	penetration index of bituminous binder
PSV	polished stone value
RAMM	Road Assessment and Maintenance Management
RIMS	Road Information Management Systems
SAMI	strain-alleviating membrane interlayer
SASW	spectral analysis of surface waves
SMA	stone mastic asphalt
SNP	modified structural number
TNZ	Transit New Zealand Ltd
TRL	Transport Research Laboratory
TR&B	ring-and-ball softening point in °C (bituminous binder)

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Executive summary

Research background and objectives

Open graded porous asphalt (OGPA) pavement surfacing is widely used on New Zealand motorways because of the benefit it provides in terms of low noise pollution and its ability to drain water from the road surface during rainstorms. The current maintenance strategy that is applied to these surfacings involves the overlaying of the existing OGPA surface, or the milling off and replacement of the surfacing.

One of the factors affecting the service life of an OGPA surfacing layer is the hardening of the bituminous binder in the mixture. The predominant failure mechanism in these layers occurs when the binder is no longer able to sustain the level of strain induced by the repetitive wheel loads or thermal stress build-up within the OGPA layer. Increasing levels of ravelling and texture depth are generally indicative of OGPA failure. A measure of the binder stiffness within the surfacing mixture would provide valuable information about the point at which the OGPA layer is likely to fail.

The objectives of this research, which was carried about between 2006 and 2007, were to:

- propose methods for disseminating and incorporating local as well as international knowledge of OGPA performance into the current asset management systems
- determine the terminal condition of an OGPA mix
- determine the rate of stiffening of OGPA surfacings on the South Auckland State Highway Network.

The study included a literature review on maintenance practices in Europe, the UK, Australia and New Zealand; a statistical analysis of the current data set of the Auckland South State Highway Network; and laboratory testing of samples retrieved from selected sites on the network and production mixes used on the 2006/07 maintenance contract.

Literature study

During 2004, the US Federal Highway Administration (FHWA) undertook a study tour of five European Union (EU) member countries in order to study their practices regarding pavement systems. The study concluded that these countries had moved away from using surfaces with predominantly noise-reduction properties, in favour of more optimised mixes in terms of both functional and structural properties. EU-funded projects were underway to classify noise-reduction surfaces with improved structural and functional durability, with the objective of providing decision makers with a rational planning document.

In Southern Australia, OGPA was only being used on high-speed freeway-type roads, for its ability to reduce splash and spray. However, stone mastic asphalt (SMA) mix was more popular because of its longer life and acceptable functional properties.

New Zealand experience

In New Zealand, the maintenance approach has been to overlay the existing OGPA surface with a membrane seal prior to the application of the subsequent overlay. This methodology has resulted in the upper pavement structure comprising multiple (up to four) OGPA layers, which has been performing well. The focus of this approach did not originally consider any increase in the structural capacity of the pavement due to the increase in total pavement depth, but was on restoring the functional properties of the surfacing layer.

The experience was that the stress ratio in the base course decreased with each consecutive OGPA overlay, accompanied by a decreased rate of rut progression. In 1995, pavement rehabilitation was halted in favour of multiple OGPA layering. This practice, based on observed good performance, prompted research to understand the increase in apparent pavement strength. This practice, which appears to be unique to New Zealand, carries the potential risk of various mechanisms of failure occurring over the lifetime of this composite structure, and up until the time of this study, these had not been researched.

Success of the New Zealand approach

The success of the multiple-OGPA surfacings on the South Auckland State Highway Network could be partly because a large proportion of the pavements on the network have relatively low deflection and curvature values (70% of the network has a deflection value less than 600μm and curvature values of less than 200μm). Also, the steps taken to minimise water ingress and to ensure that water cannot pond within the OGPA layers would be beneficial.

Maintenance planning

Current maintenance planning utilises an optimisation process to produce an annual works programme that is field-verified prior to implementation. The analysis is based on a life cycle cost approach and consists of strategy generation, optimisation and programme development. This research proposes a maintenance decision-making tree that considers the structural soundness of a pavement with cognisance of site constraints and functional requirements. The use of this decision-making structure aims to formalise the decision-making process and contribute towards the improvement of the network condition in the long term, thus meeting the objectives of this research.

Statistical analysis of the South Auckland State Highway Network

The time till initial crack development in an OGPA surfacing layer was found to be 7.5 years (50% of the data was distributed over 5 to 11 years). In addition, the analysis showed that the time to crack initiation was not dependent on the structural strength of the pavement. Binder stiffening seemed to occur on all pavements, irrespective of the structural strength of the pavement. In addition to the ageing of OGPA layers, the data also indicated that the mean texture depth on the OGPA network increased over time. At the time of writing, overseas development was underway to use changes in surface texture to predict ravelling.

Stiffening of OGPA mixes due to binder ageing

One of the modes of failure reported on OGPA mixes is the stiffening of the binder. A study conducted by Transport Road Research (TRL) in the UK proposes that the ultimate failure of an OGPA mix is likely to occur rapidly after the binder penetration has fallen below 15Pen, or its stiffness has increased to a value in excess of 100MPa. These ultimate values are reported for all binder types, regardless of whether they are modified or not.

Measurement of OGPA stiffness

Bitumen stiffness can be measured by the penetration on recovered binder; however this measurement involves a complicated process. This study attempted to measure the stiffness of OGPA mixes by means of the repeated-load 'indirect tensile strength' (ITS) test. This was done on cores extracted from the road network as well as on cores prepared from production mixes used during the 2006/07 maintenance contract.

In an attempt to relate measured stiffness of the OGPA mixes to the ultimate binder stiffness, established rheological models and the Shell nomograph were used. Stiffness values were derived from chipseal failures in New Zealand, OGPA failures in the UK, and asphalt failures reported in the US. Stiffness values measured on core samples extracted from the road were compared to the above terminal stiffness values.

A pilot study was conducted using the ITS test on laboratory-prepared OGPA specimens. These specimens were prepared from a plant-produced mix and the measured stiffness values were compared against the terminal OGPA stiffness values.

Relating field performance to resilient modulus

The proposed method for determining OGPA failure was used to evaluate the stiffness of OGPA mixes from cores extracted from the existing pavement on the Auckland South State Highway Network. The visual condition of the sites varied from good to poor. The ITS results showed some correlation between the visual pavement condition and OGPA stiffness.

The method can be used to track the stiffening of an asphalt layer over time by extracting core samples from the layer at various age intervals and predicting the time at which the layer will reach a terminal stiffness. Research in the UK suggests a decrease in binder penetration of approximately 20% per year. Stiffness values measured on core samples extracted form the Auckland South Network indicated an increase in OGPA stiffness of 16% per year. This approach can be used to enhance decision-making in a network pavement management system.

Recommendations

A maintenance decision-making tree that considers the structural soundness of a pavement, taking cognisance of site constraints and functional requirements, is proposed.

Measuring the stiffness of in-service OGPA layers will assist in identifying the point in time at which intervention is required. These measured stiffness values may be used in conjunction with the modelled outputs during the works programme stage to aid decision makers during their site validation stage. Once the relationships are tested, future applications might involve the incorporation of this method into the New Zealand dTIMS[1] optimisation model.

1 Deighton's Total Infrastructure Management System

Abstract

The objective of this research, which was carried out between 2006 and 2007, was to propose methods to disseminate and incorporate local as well as international knowledge of OGPA performance into the current New Zealand asset management systems.

Compared with asphalt overlay practices in Europe and the UK, the current New Zealand maintenance practice of multiple OGPA overlays is not considered to be optimal from a structural point of view. This study undertook a literature and database review, and determined the terminal stiffness of OGPA mixes.

An analysis of the South Auckland State Highway dataset showed that the life of multiple OGPA overlays shortens with successive overlays. Statistical evidence suggested that binder ageing was the limiting factor in this phenomenon, and a correlation was found between pavement surface condition and mix stiffness. Repeated-load indirect tensile strength (ITS) tests were undertaken on cores retrieved from network pavements and on production mixes from the 2006/07 resurfacing contract. Terminal OGPA mix stiffness was found to be in the order of 2300MPa at 25°C, with corresponding binder penetration of 11Pen as determined from back-calculation through the use of the Shell nomograph.

This research recommends a gradual move away from the current multiple-overlay approach, and the utilisation of OGPA for its functional rather than structural integrity.

1 Introduction

1.1 Background

The aim of this research was to develop methods to assist with maintenance planning for pavements surfaced with open graded porous asphalt (OGPA), based on the analysis of data contained in the Road Asset Maintenance Management (RAMM) data base of the South Auckland State Highway Network. In order to try to relate OGPA layer stiffness to field performance, the research reviewed the findings and test sites of a previous study of the performance of multiple OGPA layers that was undertaken for the NZ Transport Agency (NZTA, previously known as Transit NZ) during 2005/06. This new study also contains a brief discussion on overseas experiences of using OGPA mixes.

A supplementary study on the stiffness of plant-produced OGPA used during the 2006/07 maintenance works on the Auckland South Motorway Network was also undertaken. The research puts forward a method of predicting OGPA stiffness from its volumetric properties, and comparing it to a terminal stiffness for OGPA mixes, by using the Shell (Shell 1978) and Van der Poel (Van der Poel 1954) nomographs and other models.

Stiffness moduli of several OGPA core samples were measured by means of the indirect tensile strength (ITS) method at various temperatures. These stiffness values are presented in a diagram that compares ITS-measured stiffness values with stiffness values predicted by the above-mentioned nomographs. A terminal stiffness limit is also indicated on the same diagram and is based on previously published failure stiffness values for bitumen binders. This approach enables a designer to predict the road temperature at which an OGPA mix will fail, depending on the stiffness and volumetric properties of the mix.

Although the proposed method has not been tested to its full extent, it is based on sound bitumen rheological principles and will assist the designer in understanding asphalt behaviour.

Various other failure modes exist within an OGPA layer – for example, contamination (which causes blockage of the voids), chemical spillage and mechanical damage. This report focuses on binder ageing and layer stiffening as primary causes of failure of OGPA layers.

1.2 Problem statement

With much of the New Zealand road network now well developed, the emphasis has shifted from the construction of new roads to the maintenance and improvement of existing road networks. Effective maintenance strategies for road pavements require a sound understanding of the behaviour and performance under loading of the materials within the pavement structure and surfacing. Regardless of the maintenance strategy used, it should be based on sound principles that are related to the performance of the pavement structures and surfacings on the road network.

The useful life of a pavement under increasing traffic loadings is dependent on the performance of each component within the pavement structure and the interaction of these components with each other. For instance, the life of an asphalt surfacing layer is affected not only by the climatic conditions/environment under which it operates, but also by the level of structural support provided to it by the underlying pavement.

The life of an OGPA surfacing layer is dependent on the hardening of the bituminous binder within the mixture. Failure of the layer occurs when the binder is no longer able to sustain the level of strain induced by the repetitive wheel loads on it. Increasing levels of ravelling, fretting and texture depth are generally indicative of OGPA failure, as the stiffness of the mix increases with age as a result of the hardening of the binder. The ageing period of in-service OGPA layers varies significantly – data from the South Auckland State Highway Network indicates an average life of eight years, although some mixes may last longer – up to 14 years.

The prediction of remaining life of OGPA layers would greatly enhance the optimisation process in formulating network maintenance strategies for these surfacing layers.

1.3 Study objective and methodology

The main objective of this study was to relate the in-field stiffness of an OGPA layer to its performance on the road, in order to optimise the way that current maintenance strategies for OGPA layers are formalised and managed. The research focused on defining the useful life of asphalt surfacing layers (particularly OGPA), and predicting the remaining life of these layers.

The specific objectives were to:

- propose methods for disseminating and incorporating local as well as international knowledge of OGPA performance into the current asset management systems
- determine the terminal condition of an OGPA mix
- determine the rate of stiffening of OGPA surfacings on the South Auckland State Highway Network.

The methodology involved the evaluation of previously published failure stiffness values of bitumen binders and their relation to a terminal stiffness value for OGPA layers, using the Shell nomographs and other published binder stiffness prediction models.

The stiffness of the plant-produced OGPA mix that was used in the 2006/07 maintenance works on the Auckland South State Highway Network was also evaluated.

2 Understanding open graded porous asphalt

2.1 Volumetric properties of OGPA mixes

Porous asphalt is an open graded mixture of coarse and fine aggregates, mineral filler and bituminous-based binder, produced hot in a mixing plant. From a road user's point of view, the main advantage of OGPA surfacing lies in its excellent functional properties, including the reduction of noise, wet-weather spray and glare, and skid resistance. In some countries, OGPA is valued purely for its functional properties rather than its structural properties.

The quality of these functional properties can be ascribed to the large void ratio in the mix. The other variables that can be selected to change the functional properties of the mix are the stone size used in the mix, and the layer thickness. The optimal OGPA layer thickness for tyre-noise reduction is in the order of 40mm; beyond this thickness, there is very little improvement in noise reduction. OGPA surfacing layers do not reduce engine noise.

A twin-layer OGPA system has been developed in the Netherlands and Denmark. It aims to improve surface drainage and reduce wet-weather spray, and consists of a bottom layer that uses stones of a large size, and a top layer constructed with smaller stone. These mixes have the best of both worlds as far as drainage and noise reduction are concerned.

Skid resistance on an OGPA surface is also an important property, and goes hand-in-hand with drainage of the layer. Because of the high void ratio of the mixes, aquaplaning is very unlikely to occur on OGPA surfaces – the voids in the mix are interconnected and hydraulic pressures cannot develop between the vehicle tyre and the OGPA surface.

The volumetric properties of a typical OGPA mix, compared to those of a typical continuously graded asphalt mix, are summarised in table 2.1.

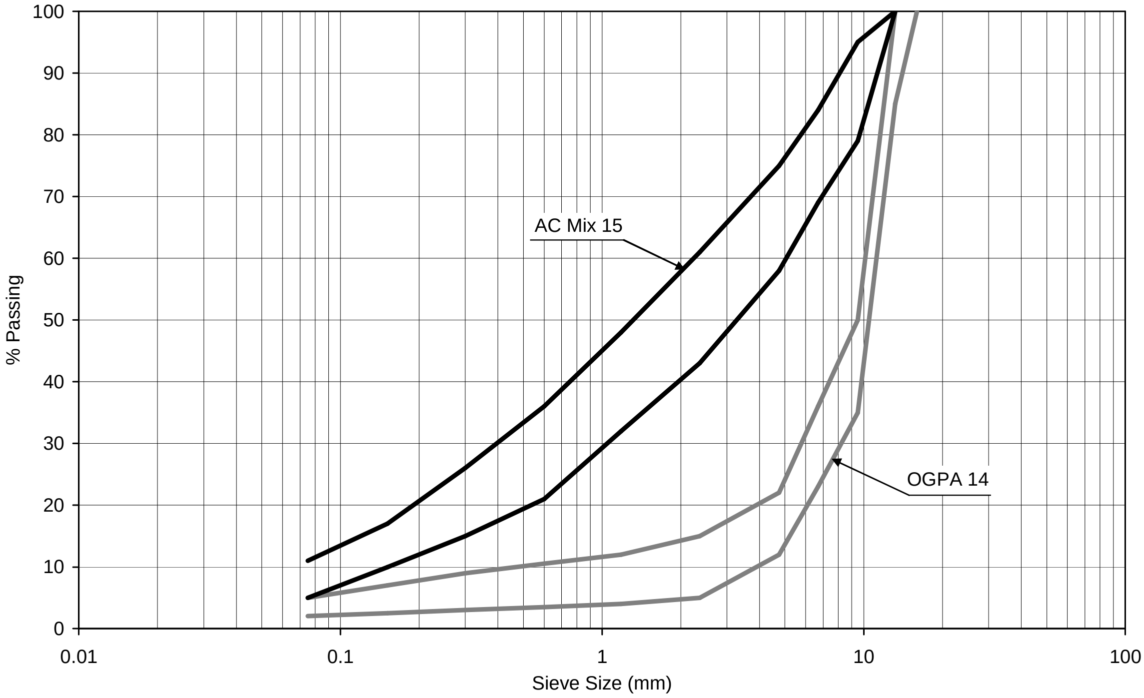
Table 2.1 Volumetric properties of typical OGPA and AC mixes

Volumetric properties	OGPA	Continuously graded AC
Void ratio	20–23%	4–6%
Binder content (m/m)	±5.5%	±5.5%
Voids in mineral aggregate	30–33%	14–17%
Voids filled with binder	30–35%	70–75%
Binder film thickness	10μm	7μm

While the most outstanding property of an OGPA mix is its void ratio, the percentage of voids filled with binder is critical, as this property is an indication of the bond between the aggregate particles. This bond can be described as chip glued together with bituminous cement (binder).

The grading of an OGPA mix is more 'open' than that of a dense, continuously graded mix, as illustrated in figure 2.1 following. Because of its open nature, binder in OGPA is more prone to oxidation or ageing than binder in a continuously graded asphalt mix. Another disadvantage of OGPA as a surfacing layer is the reduction in its functional aspects over time as the surface voids become choked with debris or road dust.

Figure 2.1 Particle size distributions of OGPA and dense graded asphalt



In order to understand the volumetric properties of an OGPA mix, the mechanism through which the grading influences the mix properties needs to be understood.

With reference to the particle size distribution of a standard OGPA mix, approximately 85% of the material is retained on a 2.0mm sieve. The voids in the mix are thus created by a stone skeleton, while the sand, filler and bitumen fractions fill the voids to the desired level. For an OGPA mix, the fraction of aggregate passing a 2.0mm sieve is approximately 15% less than for a continuously graded asphalt mix. This structure accounts for the slightly greater film thickness of the OGPA mix when compared with other mixes with similar binder contents.

The ability of an OGPA mix to retain binder during the mixing and placing process is therefore less than for other asphalt mixes, which increases the importance of having sufficient volume of binder in the mix to ensure durability for in-service conditions. On the other hand, the binder should not be allowed to drain down as a result of low binder-holding ability of the sand and filler fractions in the mix. The drain-down test is included in the design process of an OGPA mix, with the aim of optimising the volume of bitumen being added to a mix.

The general design method for OGPA layers optimises binder content within the specified limits for drain-down and the Cantabro durability limits.

2.2 Literature study

None of the international experience documents multiple OGPA layering; thus, the New Zealand approach is unique. The information that follows relates to single OGPA layers.

2.2.1 Netherlands experience

The Netherlands' literature described OGPA as a highly open graded asphaltic concrete (ZOAB) (Rijkwaterstaat 2006). It was further described as small-sized stones glued together with a bituminous binder, and it was only used for its functional properties. In a country like the Netherlands – where all road aggregate is imported from other countries – ZOAB had found a place in the market only because of its functional properties, and no structural value was attributed to the material.

At the end of its life, ZOAB was being recycled for use in other asphalt mixes. Modified binders were not permitted in these mixes because of problems associated with the reheating of recycled mixes that contained modified binders. Special attention was paid to the detailing of the drainage of ZOAB layers; provision was made for special edge drains to ensure effective drainage of the layered system.

At the time of this research, development work in the Netherlands was focusing on laser-enhanced technology in order to detect ravelling in an OGPA surface – the early detection of ravelling would assist in the maintenance management of its network.

2.2.2 Danish experience

At the time of this research, the Danish Road Institute (DRI) was participating in the European Union-funded (EU) research project SILVIA, with the aim of developing noise-reducing pavements (Danish Road Institute 2006). The purpose of their study was to provide decision makers with a tool that would allow them to plan traffic noise-control measures rationally, improve the functional and structural durability of low-noise surfacing mixes, and develop maintenance techniques and a full life cycle cost–benefit analysis procedure for this type of pavement. The final product was published in 2006 as *European guidance manual on the utilisation of low-noise road surfacings*.

2.2.3 South African experience[2]

The first open graded porous surfacing, 'whisper asphalt', was introduced into South Africa during 1993 under the authority of the Gautrans Department of Public Works (previously called Transvaal Roads Department). This project entailed the construction of a total of approximately 1,000,000m^2 of OGPA surfacing on a carriageway carrying 25,000 vehicles per lane per day.

During the design phase it had been decided to use a highly modified binder because of its good track record in asphalt mixes, and a bitumen rubber binder was selected. The objective of the design was to optimise the mix's binder content versus air voids, abrasion loss and binder drain-down. This design produced mixes with typical void ratios of 20–23%, with 5.8% binder content.

The functional properties of the mix had to be maintained by monthly cleaning of the surface by water jetting and vacuuming, in order to prevent the surface from silting up in the dry, dusty environment. Under high-rainfall conditions, OGPA layers were expected to be self-cleaning because of the suction forces of fast-moving wheel tyres over the surface.

The surfacing performed well for six to seven years before the first surface ravelling became visible. The ravelling developed in the slow lane – the lane with the highest traffic loading. As soon as signs of

2 Project experience of Enrico Fletcher, one of the authors of this research paper.

ravelling emerged on the surface, cores were taken for laboratory testing. Binder extracted from these cores showed considerable stiffening, and the surface ravelling was ascribed to binder ageing in the mix. Because of budget constraints, the OGPA surface was not milled off and replaced, but after experiments with various emulsion blends, was resurfaced with a specialised bituminous mixture. This surface treatment penetrated the existing open grade porous layer and rendered it a pseudo stone mastic asphalt (SMA).

The life of the surfacing layer was extended through this treatment, and at the time of writing, the surfacing was still performing well, 14 years after its introduction. However, the level of water spray and noise reduction was similar to that of an SMA surfacing. Skid resistance was measured immediately following the treatment, and again after six months. The average grip number after treatment was 0.44, but after the film coating on the exposed chip had worn off after six months of service, the grip number improved to 0.48.

2.2.4 South Australian experience

The following is an extract from correspondence with Hugo van Loon, the senior asphalt engineer of South Australia, with regards to the use of open graded asphalt in that country.

> *In the past in South Australia a lot of OG was placed on important roads to reduce splash and spray etc. These surfacings had a life of approx. 10 years and when a polymer-modified binder was used, the life extended to say 15 years, although some have been down 20 years and still performing.*
>
> *However today, OG14 is only used for high-speed freeway type roads, principally for its splash- and spray-reduction characteristics. All our OGs have a SAMI[3] seal under them acting as a water barrier for the water to flow along and out at the ends. The SAMI needs to be level with the gutter and thus our OGs are above the kerbline, which creates a ridge which the cyclists don't like. For this reason and the relatively short life, it is more popular for SMA (stone mastic asphalt) to be placed instead of OG, as this also has a high negative texture but an expected 30-year life. Some noise reducing is attributed to SMA, but OG still provides the best noise reduction.*
>
> *The gradings are as required in AS2150, and our main mix is OG14, but some OG10 has been used as a thin surfacing. Our design is to APRG18, and our spec. requires Type II, to 'medium-duty design' (ie 80 gyratory cycles – Gyropac). The design in APRG18 is based on the 'particle loss test' to provide a minimum binder content, voids at 20% providing the maximum binder content, and the mean adjusted up by the 'binder drain-down test' to give the nominated design binder content. Our 'production air voids' tolerance is 18 to 23%, and voids are measured by mensuration. The in-situ air voids are to be between 18 and 25%.*
>
> *We do not allow OG to be placed over winter (April to Oct), as during these colder months the PMB binder on top of the stone takes too long to wear off and very slippery conditions occur. The same applies to SMA; however we can grit SMA to wear the bitumen off the stone for good initial skid resistance. We cannot grit OG as it would then fill up the 20% approx. air voids. Our dominant PMB is 35P for dense mix, and I think it has been used successfully for*

3 Strain-alleviating membrane interlayer

OG as well. Recently we had a situation where the placement temperature was too high (about 165°C) and binder drain-down occurred, and cooling somewhat did not change the problem, so the binder was changed to 15E, and this worked well.

I am not sure of standard thickness, but I think 35–40mm is used for OG14, and 25mm for OG10.

One of my main issues with OG and SMA is the polishing of the aggregates. Our current requirement for PAFV (vertical wheel) is 48, but I want to raise this to 55 for OG and SMA mixes. We are currently in the process of trialling high-pressure water blasting, rotating steel discs and captive shot-blasting to reinstate the microtexture of the aggregate to improve marginal skid resistance. The only one trialled to date is water blasting, and this only had a three-month benefit.

As for maintenance, we have in the past on a double carriageway repaired large patches with dense mix on the kerbside lane, and then the median lane does weep through the OG onto the dense mix (and possibly down into the pavement at the joint) even though the old OG is 'completely blocked'. We always remove both the OG and SAMI before reinstating, and this is another reason for it falling out of favour (Van Loon, pers comm, 2007).

2.2.5 Tasmania

The following is an extract from correspondence from Barry Walker, the manager of asset management at the Department of Infrastructure, Energy and Resources of Tasmania.

Open graded asphalt (OGA) mixes have had only the occasional application in Tasmania, mainly to counter traffic noise and problems associated with excess surface water. We tend to follow National Asphalt Specifications for this product.

Our mix requirements are generally: aggregate size 10mm, depth laid, nominal 40mm.

Normal maintenance practice is to repair failed areas using 'dense graded asphalt', and whilst this is possibly not the best option, it is the most practical given the difficulties in sourcing small quantities of OGA mix from suppliers.

Cleaning of OGA surfaces is recommended; however this has not been undertaken on any sites, with no obvious detrimental effect on the surfacing performance.

Prior to resurfacing over existing OGA, removal of the OGA is recommended (Walker, pers comm, 2007).

2.2.6 US Federal Highway Administration

During 2004, the US Federal Highway Administration (FHWA 2006) undertook a tour of five EU member countries, with the aim of studying current practices in the design, construction, maintenance and monitoring of quiet-pavement systems. Details of this investigation were published on the FHWA website www.international.fhwa.dot.gov.

The visiting team focused on the following areas relating to the above:

- policy

- design
- noise analysis
- construction
- maintenance
- research.

The panel started each visit with a general discussion of current noise policy and applicable noise-measuring and noise-monitoring systems. The following sections summarise the team's findings.

2.2.6.1 Belgium

Road authorities in Belgium favoured ultra-thin and SMA surfaces because of the high traffic loading on their road networks. These surface types were optimised for noise reduction and were preferred above open graded asphalt mixes. The authorities were confident that these mixes provided a better combination of durability and noise reduction than open graded asphalt mixes.

2.2.6.2 Denmark

The use of low-noise surfaces on road networks in Denmark was limited. Single and twin-layer porous asphalt mixes were also in use. However thin open graded mixes – which were more durable – were favoured above open graded asphalt.

Pilot studies were carried out and extensive ongoing monitoring was underway to formulate the effectiveness of quiet-pavement systems.

2.2.6.3 France

France had a quiet-pavement policy in place. However, the implementation of quiet-pavement systems was not likely for cities like Paris, which have historic cobblestone roads. Structural and functional characteristics of porous asphalt were separated and no structural value was assigned to a porous asphalt layer in a pavement system. Finer graded mixes (6mm and 10mm) were preferred over the 14mm mixes because of their better skid- and noise-reduction properties.

New pavements were generally constructed with a structural asphalt base layer, and an ultra-thin functional layer as a wearing surface.

2.2.6.4 The Netherlands

During the late 1970s, initial research was undertaken on porous asphalt to reduce 'splash and spray' on road surfaces. Noise legislation required the placement of porous asphalt on all roads carrying in excess of 25,000 vehicles per day. At the time of the research, up to 60% of the strategic road network in the Netherlands was surfaced with porous asphalt.

A classification system, which favoured porous asphalt on national highways and ultra-thin layers on inner city roads, was used. This system was based on the extent of vehicle braking, acceleration and turning movements on a particular road section.

2.2.6.5 Italy

Italy had advanced legislation in terms of the reduction of road noise. The first use of porous asphalt was intended to increase skid resistance and reduce wet-weather spray. When it was observed that these pavements also resulted in reduced pavement noise, an effort was mounted to optimise the noise-reduction effect without loss of safety or surface durability.

2.2.6.6 United Kingdom

In the past, the focus in the UK has been on providing excellent skid-resistant road surfaces. Recently, efforts have been made to find a balance between safety and noise reduction. At the time of the research, the overall goal was to resurface 60% of the strategic road network in the UK with quieter overlays within 10 years. Thin-layer quiet-surfacing types had overtaken the use of older, open graded asphalt technology, because of its higher cost effectiveness. These surfacing types were even being used to overlay concrete pavements.

More than 32 approved proprietary surfacing systems that met safety and noise requirements were available on the UK market. In general, these systems were similar to SMA, but were proprietary formulations using modified binders and closely controlled aggregate mixes. These mixes had reported lives of 12 years, compared with the eight years of the open graded mixes.

Porous asphalt mixes were assigned 50% structural credit at the standard thickness of 50mm. The proprietary surfacing systems at 20–35mm were considered to have the same structural capacity as the 50mm OGPA.

The noise-control policy in the UK was scheduled for a review in 2008, and would include the findings of the EU-funded research projects. These projects are summarised in table 2.2 below.

Table 2.2 Current EU projects

Project	Objectives	Principal research authority
SILVIA	· Classification and conformity of noise-reducing surfaces · Improving structural and functional durability · Life cycle cost–benefit	Belgium Road Research Centre
SIRUUS	· Low-noise multi-layer system · Optimising texture, roughness, hydraulic conductivity and sound absorption for different structural functions	Autostrade Italy

2.2.6.7 Summary

In general, ultra-thin functional mixes were gaining popularity over porous mixes. The noise-reduction capability of ultra-thin mixes was not as significant as in open graded mixes, but concern about the durability and the high cost of maintenance of the open graded mixes were some of the main reasons why these mixes were losing favour with road authorities.

The wet-weather performance of open porous asphalt surfacings was the main reason for the selection of these types of mix. Twin-layer mixes – with a coarser mix at the bottom and a finer mix on the surface – appeared to be the most effective in terms of noise reduction and water-spray abatement.

The selection of these mixes in terms of life cycle or cost–benefit ratios was thus quite complex, and striking a balance between functionality and durability criteria was even more complex.

2.2.7 New Zealand experience

The NZ Transport Agency is the Crown entity responsible for state highways in New Zealand. The national road network includes approximately 5.5 million square metres of OGPA surfacing.

The first OGPA surfacing layers were introduced into New Zealand during the early 1970s, and have since been used successfully. At the time of writing, the oldest section of OGPA still in service on the Auckland South State Highway Network was 11 years old.

Until now, the preferred maintenance approach on these layers has been to overlay the existing OGPA surfacing without removing the underlying layers, undertaking relatively little pavement rehabilitation in the process, even on pavements where the maintenance measures included structural asphalt overlays. The maintenance of OGPA-surfaced pavements has mainly involved sealing the existing OGPA surface with a waterproofing membrane, followed by a 25–35mm OGPA overlay. The membrane seal consists of a single-coat grade 5 chipseal (6mm nominal size aggregate) on 1.0L/m^2 bitumen.

This methodology has resulted in pavement structures that consist of multiple OGPA layers. Despite a substantial increase in traffic volume, these structures have performed well and there are suggestions that this multiple-layered approach contributes to the structural capacity of the pavement.

This multiple-layered approach is unique to New Zealand – in other countries, OGPA surfacing layers are milled off and replaced.

The New Zealand approach takes cognisance of the condition of the surface that needs to be overlaid, and the consecutive overlay is mainly done to restore functional aspects as well as rutting and riding quality. Structural improvement on sites treated in this manner has been proven by falling weight deflectometer (FWD) testing. This practice has been extended from granular flexible structures to deep-lift asphalt structures and cement-stabilised pavement structures.

The following is a typical chronology of the multiple-OGPA approach:

- Chipseal grade 3/5 (also known as a 13mm/6.7mm double seal) is usually placed as a first surfacing.
- After the first year, a levelling or rut-filling layer is placed if required (TNZ M/10 Mix 10, also known as nominal 6mm continuous graded asphalt).
- The first OGPA layer is placed on a grade 5 membrane seal. Typically, after 5–7 years the OGPA will have clogged and signs of loss of functionality will be in evidence.
- A second OGPA layer will be placed on a levelling course if required, along with another grade 5 membrane seal.
- This process is repeated until practical constraints, such as bridge clearance and safety limits on New Jersey Safety Barriers, are reached.
- To prevent water ingress, the lower OGPA layer is sealed off with a dense graded asphalt mix on each side. Also, any area that could pond water is pre-levelled prior to the application of the membrane seal – *unless* there is the slightest possibility of sealing in moisture, in which case this practice should not be used.

While this unorthodox practice is estimated to have resulted in considerable whole-of-life savings for the network, there is the possibility that multiple OGPA layering may lead to pavement failure in the future.

Figure 2.2 shows a core extracted from a multiple-OGPA structure that has a seal and asphalt levelling layer between consecutive OGPA layers (a two-layered system).

FIgure 2.2 Two-layered system

Figure 2.3 shows a core extracted from a multiple-OGPA structure with seal and asphalt levelling layer between consecutive OGPA layers (three-layered system).

Figure 2.3 Three-layered system

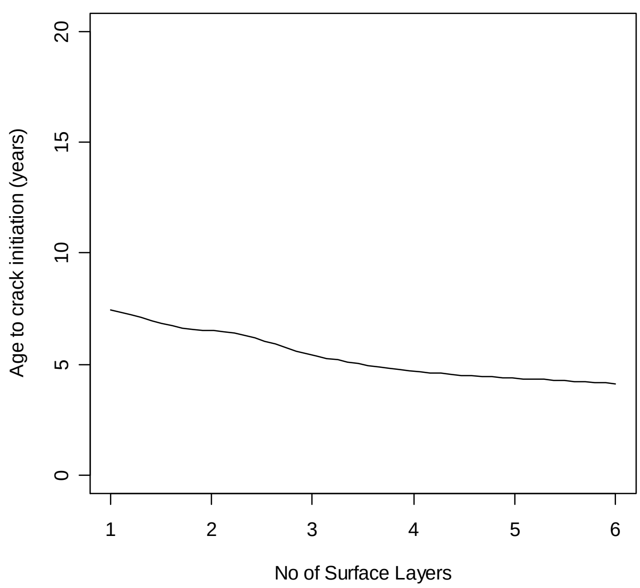
OGPA layers in New Zealand are currently designed in accordance with TNZ P/11 or P/23 specifications. The TNZ P11: 2003 specification (TNZ 2003) optimises binder content versus air voids, abrasion loss and binder drain-down, while the TNZ P23: 2005 specification (TNZ 2005) lays down design values for layer thickness, binder type, aggregate polished stone value (PSV), texture depth, mix designation and other properties. Performance requirements for texture depth, surface shape and ravelling are also included in this specification.

The current OGPA draft specifications allow the use of OGPA on new projects, with the requirement that OGPA is placed on a grade 3/5 seal and has a six-month running-in phase. A second coat of grade 5 seal is required prior to the construction of the OGPA layer. The aim of this approach is to achieve a watertight membrane seal below the OGPA that has a residual binder content of more than 3L/m^2.

This procedure has become standard practice on the South Auckland State Highway Network, and no significant failures have been attributed to following this methodology, except for areas where the shoulder was over-filled with dense graded asphalt (for safety reasons) and free drainage was impaired by the edge constraint.

Over the years, as many as six OGPA overlays have been placed on some sections of the South Auckland State Highway Network. Statistical evidence suggests that the time until initial crack development of the top surfacing layer reduces with each consecutive overlay, as illustrated in figure 2.4.

Figure 2.4 Crack initiation with age (OGPA layers)



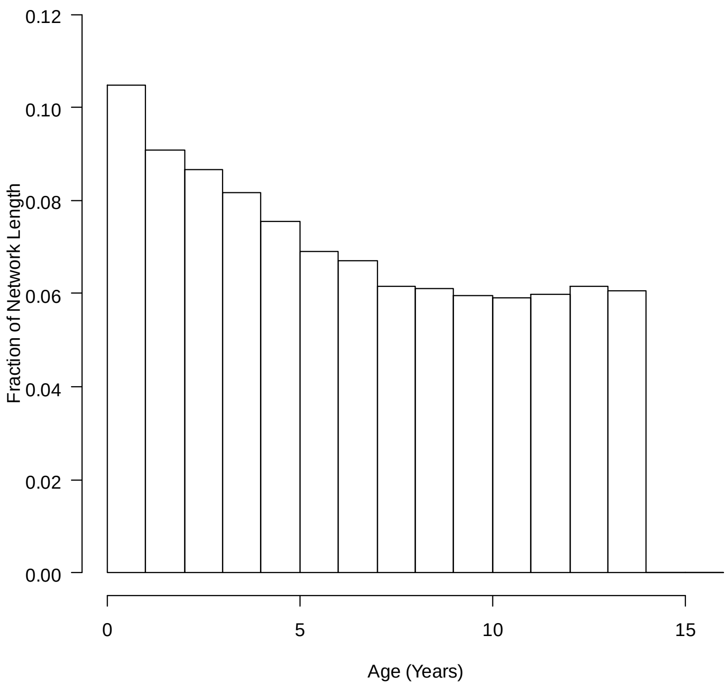
3 Statistical analysis of the Auckland South State Highway Network

The South Auckland State Highway Network consists of approximately 3.34 million square metres of OGPA surfaced pavements. A statistical analysis of these pavements has been undertaken, in an effort to provide greater insight into the behaviour of OGPA surfacings at network level. This analysis is based on data from the 2006 RAMM database of this network, and excludes road sections on ramps and roundabouts.

3.1 Surface age

The age distribution of the South Auckland State Highway Network OGPA surfacings is illustrated in figure 3.1. In 2006, approximately 42% of the network's OGPA surfacings were more than 8 years old and approximately 20% were aged 4–8 years. The remaining 38% were less than 4 years old.

Figure 3.1 Surfacing age



The average life of an OGPA surfacing layer is generally expected to be around 8 years, so this information indicates that a large percentage (42%) of the network surfacing will probably require increased maintenance in the short term. The portion that is aged 4–8 years will require a higher level of monitoring than the remaining 38% that is less than 4 years old.

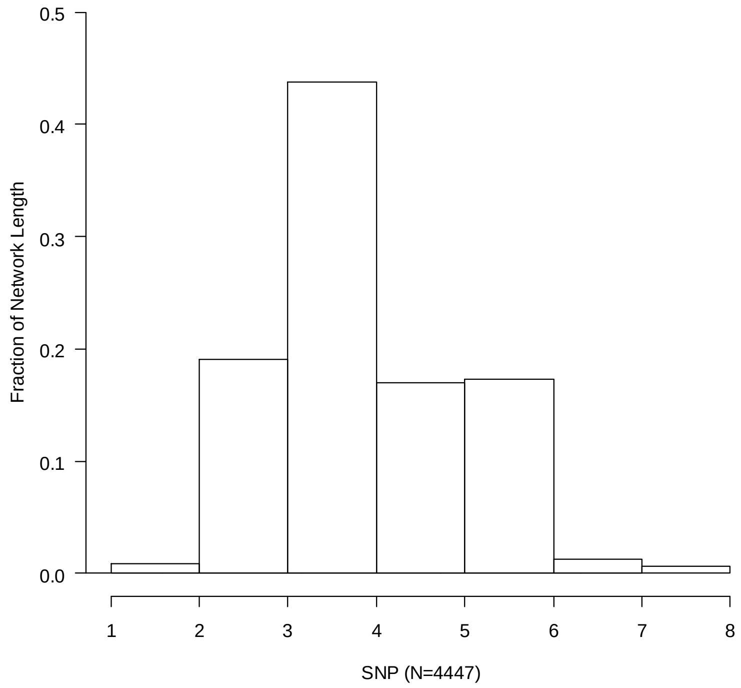
However, age may not be the only factor that determines the service life of an OGPA layer. Other factors (eg binder ageing) can also influence the service life of OGPA surfacing, and are investigated in this report.

3.2 Structural strength and crack initiation

The structural strength of a pavement on the road networks is obtained from its relationship with pavement deflection. The adjusted structural number (SNP) derived from this relationship is used to define the structural capacity of pavement sections on a network level.

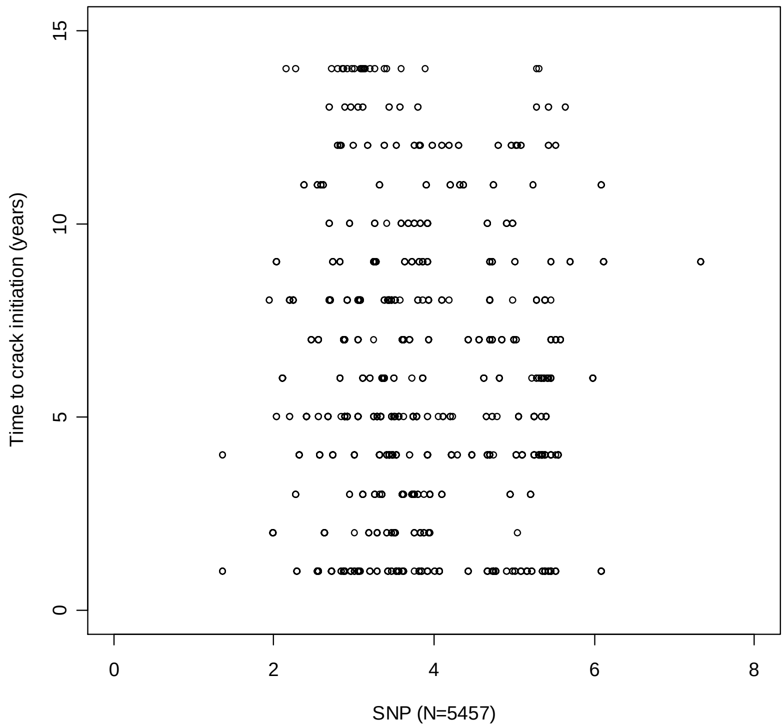
The distribution of the structural strength across the South Auckland State Highway Network is illustrated in figure 3.2.

Figure 3.2 Network structural strength



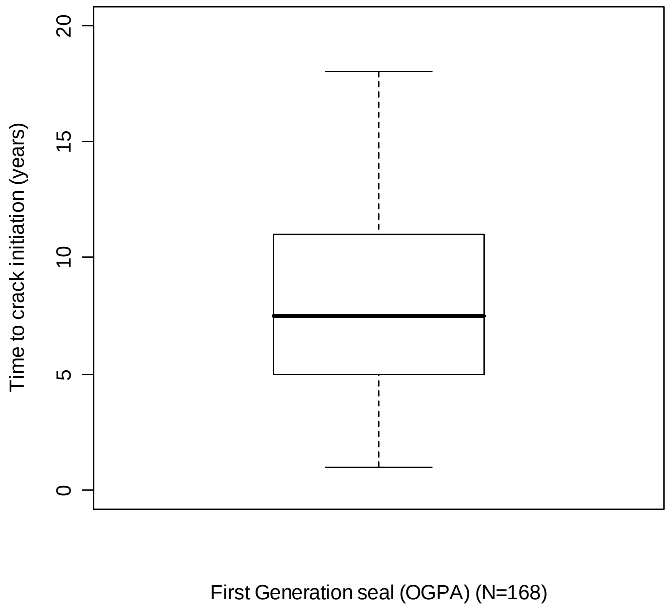
Considering the relationship of time to crack initiation versus SNP, as shown in figure 3.3, the time to crack initiation is not influenced by the structural strength of the pavement as measured by the SNP.

Figure 3.3 Time to crack initiation versus SNP



A distribution of the time to crack initiation of OGPA layers on the network is illustrated in figure 3.4. The median value for time to crack initiation on the first OGPA overlays is 7.5 years, while the time to crack initiation for 50% of the layers lies between 5 and 11 years. The skew distribution in this chart might be as a result of the range in structural integrity of the pavements and improved performance of some of the bitumen binders used in these overlays.

Figure 3.4 Time to crack initiation distribution

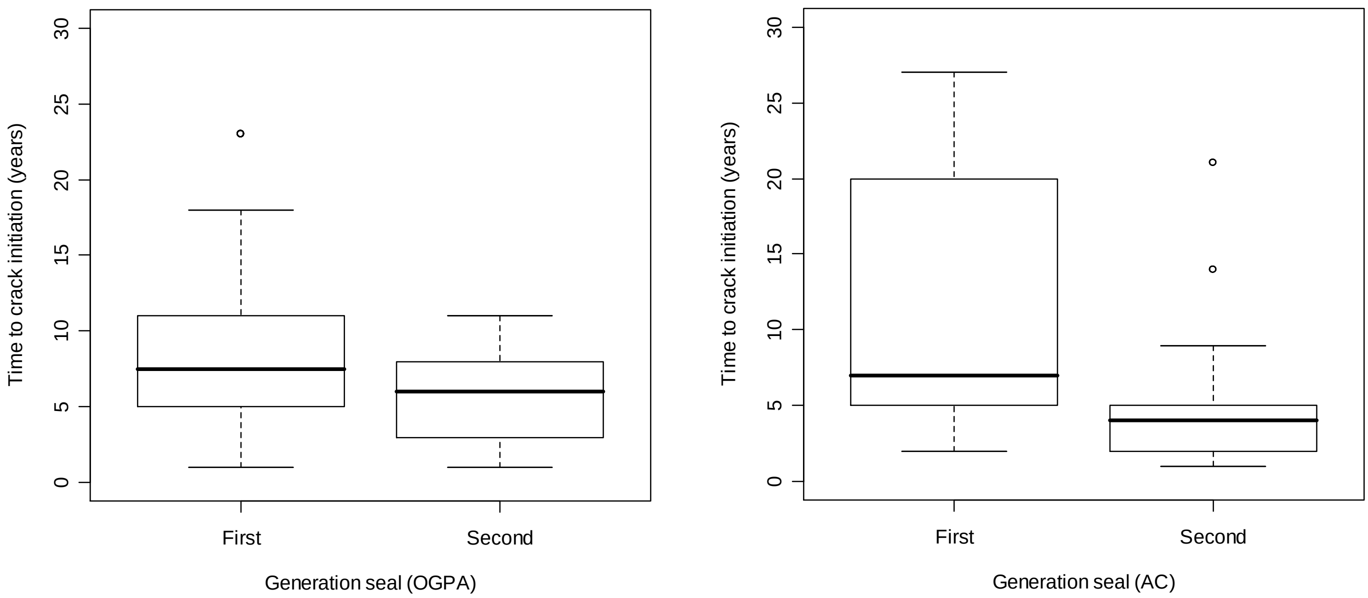


The poor relationship between OGPA performance, pavement strength and surface age might be confirmation that this layer should be regarded as a non-structural layer. It would appear that factors other than surface age and pavement strength affect the performance of OGPA surface layers.

3.3 Comparison of OGPA and dense graded mixes

Valuable information regarding crack initiation of OGPA and dense graded asphaltic concrete (DGAC) mixes was extracted from the network RAMM database, and is presented in figure 3.5.

Figure 3.5 Comparison of time to crack initiation for OGPA and DGAC



The first dataset in both graphs represents the distribution of the data for the first overlay, and the second set represents the data from the second overlay. The median values of time to crack initiation for the first set are 7.5 and 7 years for OGPA and DGAC layers respectively, and 6 and 4 years respectively for the second set.

The life of the first overlay might be ascribed to the binder ageing process, while the decrease in time to crack initiation on the second overlay might be as a result of reflective cracking from the underlying surfacing layer, due to stress concentration at the crack.

The difference in time to crack initiation between the OGPA and DGAC layers might be due to the fact that it is more difficult to detect cracks on an OGPA layer because of its coarse surface texture. Another reason for the slower rate of crack development might be the flexible nature of an OGPA mix, which could inhibit crack initiation.

4 Current New Zealand practice

4.1 Maintenance practice

Since its introduction in New Zealand, OGPA surfacing has been maintained by the reinstatement of the functional properties of the surfacing layer by overlaying the existing layer. An OGPA layer is selected for treatment as a result of its condition, which is generally described by the degree of ravelling and cracking visible on the surface. This approach results in multiple-OGPA structures of up to three or more consecutive layers.

This resurfacing approach does not consider any increase in the structural capacity of the pavement that is due to the increase in total pavement depth, but focuses on restoring the functional properties of the surfacing layer.

The maintenance regime comprises the sealing of the existing surface by the application of 1.0L/m^2 bitumen followed by a layer of grade 5 (6mm nominal size) chip. The purpose of this layer is to seal off the voids in the existing surface prior to overlaying it with a 25mm nominal-thickness porous asphalt layer. This membrane seal is applied immediately prior to the OGPA overlay.

In some instances, rut filling is undertaken as a pre-seal treatment and to ensure there are no areas that will pond water. This involves the application of a layer of dense graded asphalt mix prior to the OGPA overlay.

Because of the high traffic volumes on the Auckland motorway (up to 25,000 vehicles/lane/day), maintenance works are carried out during the night. It is a requirement that all maintenance activities are completed on the same night.

Constraining factors such as concrete kerb levels, barrier heights and vertical bridge openings influence the selection of the appropriate maintenance treatment. In cases where these factors limit the overlaying of the existing road, the existing surface is milled off prior to the application of the membrane seal and OGPA layer.

In areas where the structural capacity of the pavement is suspect, alternative treatment options are considered – these may include rehabilitation or reconstruction of the pavement.

This multiple-layer approach in maintaining the network has contributed a certain amount of structural strength to these composite structures. Beams extracted from a two-layer OGPA pavement on the South Auckland State Highway Network were subjected to fatigue testing using the third-point loading beam fatigue test in the laboratory (MWH 2006). The fatigue mechanism observed during these tests can be described as two-fold:

1. Inspection of the test specimen revealed that the bottom half of the beam (the bottom layer of the two-layered structure) failed in tension, which was clear from the cracks that appeared directly under the load points. These cracks propagated upwards through the lower layer to the interface between the two layers (membrane seal), at which point the crack progression ceased.
2. Although the beam stiffness declined with an increase in load repetitions, the beam remained intact and could be tested well beyond 1 million load repetitions under a strain level of 200μm.

The above observation may hold the key to a better understanding of the behaviour of multiple-OGPA structures in a road pavement. Consider the following scenario – the underlying OGPA layer may provide minimal structural contribution once it is cracked, yet the membrane seal between the OGPA layers may act as a stress-absorbing membrane interlayer (SAMI) that prevents the cracks from migrating upwards into the overlaying layer. At this stage, the overlaying OGPA layer may start to contribute towards the structural integrity of the system in bearing capacity, rather than in flexural strength. This structure may effectively protect the underlying layers from the pressure from tyres, acting as an equivalent granular layer.

This rationale is supported by research undertaken by the Transport Road Research Laboratory in the UK (TRRL), which showed that 50mm of porous asphalt has load-spreading abilities equivalent to that of a 20mm layer of rolled asphalt (Potter and Halliday 1981).

There is a view that this maintenance approach helps to slow down the bitumen-hardening process in the lower layers of the multiple structure – they are shielded from the degrading effects of ultraviolet radiation, which may either slow down or stop the ageing process of the binder.

However, the potential risk of this approach is that the following mechanisms of failure can occur over the lifetime of this composite structure:

- The binder in the lower layers can age, resulting in the loss of cohesive strength in the layer, with consequent rutting or potholing on the surface.
- A binder-stripping mechanism with the same consequences can also occur.
- The membrane seal can lose its integrity, allowing water to penetrate into the lower layers and cause structural failure of the granular base layer.
- The voids in the mixture can silt up, leading to a loss in functional properties such as the ability to dampen noise and to drain freely.

These mechanisms are worldwide phenomena and have been recorded on pavements with ordinary asphalt-layered systems. However, none of these failure mechanisms have been observed or documented on the OGPA-surfaced networks in New Zealand.

According to the South Auckland State Highway Network manager, the multiple OGPA layers have performed well in the past. This success may be due to the relatively thin nature of the OGPA surfacings on the network – they are traditionally constructed to a depth of 25–35mm, while the void ratio of the mix is based on a 65mm thick block. It has been shown that when mix is placed at 25mm thick, the void ratio increases and the mix become more open on the road.

The concept of a traditional OGPA layer – which is selected solely for its functional properties – is also changed with the addition of polymer modifiers to the bitumen binders in the mix.

4.2 OGPA retexturing

In recent years, several attempts have been made to restore road surfaces to improve their skid resistance. However, at the time of this research, none of these had been successful in restoring the microtexture (ie aggregate polishing) of OGPA.

In general OGPA surfaces need very little maintenance. The rate of clogging of OGPA surfaces depends largely on the physical environment. In dry, dusty environments, OGPA requires more maintenance than in a high-rainfall environment. In New Zealand, OGPA shows signs of clogging within two years of construction, and signs of polishing towards the end of its functional life.

4.2.1 Cleaning and retexturing trials

Captive water blasting and removal of water and dirt through a mechanical vacuum system were trialled on a section of the Auckland network during April 2005. The water pressures tested during the trial ranged from 7–35MPa. Water pressure of 35MPa was found to be excessive; pressure of 20MPa was found to be more satisfactory. It was also found that five repeated runs over the same area at a pressure of 7–20MPa did not cause damage to the surface.

Some of the results, especially on the older OGPA surfaces, showed a 50% improvement in permeability after cleaning. Two-year-old sites also showed an increase in permeability after cleaning, and the trials indicated that cleaning had a generally positive effect on OGPA permeability. The trial also indicated that course OGPA (large-stone OGPA 20) showed little sign of clogging and had the ability to self-cleanse.

Retexturing of OGPA surfaces that had low skid resistance (ie lacked in SCRIM requirements) was attempted. These trials were conducted with the use of specially designed equipment that was designed to mechanically abrade the surface, thereby restoring the surface texture. On OGPA mixes, the retexturing damaged the older surfaces – the process was found to be more suited to SMA mixes.

Figure 4.1 Retextured OGPA surface (note the newly formed faces)

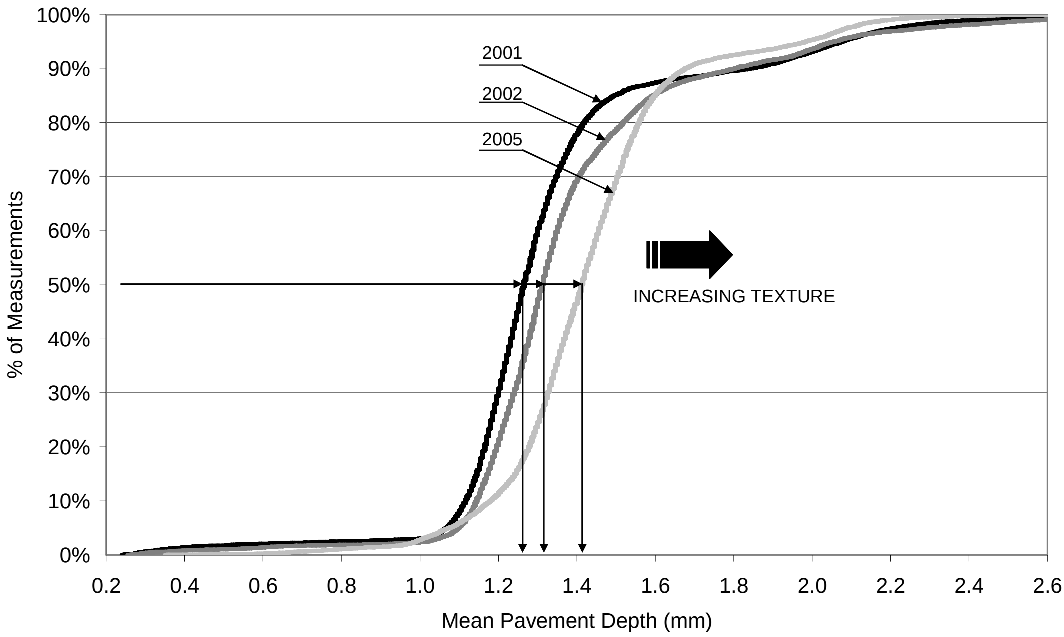
4.2.2 Surface texture

Surface texture is measured as the difference between the profile (also the average peak level) and a horizontal line through the top of the highest particle within a certain profile length/width. On a network basis, surface texture is measured by using high-speed devices that are equipped with laser profilometers. The texture depth measured with these devices is reported in terms of the mean profile depth (MPD).

The cumulative distribution of MPD on the network is illustrated in figure 4.1, which presents distribution plots from several consecutive years. Data used in this assessment was obtained from the high-speed data surveys that are undertaken annually on this network.

It was expected that the texture depth of an OGPA layer would increase over time, due to an increased level of ravelling and cracking on the surface because of binder hardening. This is confirmed in the trends presented in figure 4.2.

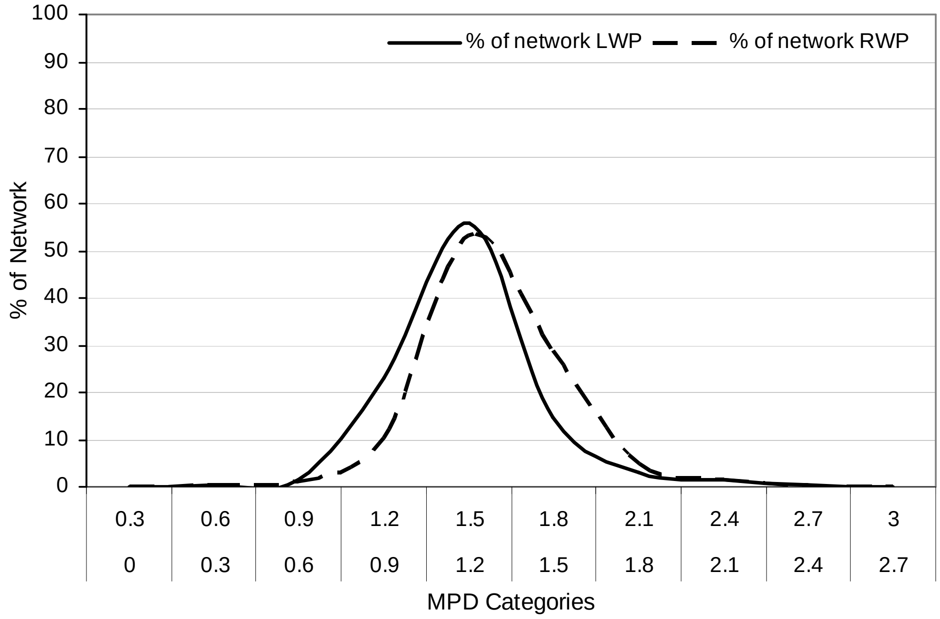
Figure 4.2 Surface texture (MPD)



The average MPD increased from 1.27mm in 2001 to 1.41mm in 2005, suggesting that the mean texture depth on the OGPA network was increasing with time. Although the reliability of the MPD measuring system is unknown, it shows some potential in measuring the condition of OGPA, which can be useful as a decision-making tool in the management of the OGPA road network.

Figure 4.3 shows a marked difference in the texture of the wheelpaths – the right wheelpath seems to be coarser than the left wheelpath. Closer inspection of the surface revealed that the left wheelpath was more silted up than the right wheelpath, which contributed to its lower texture depth.

Figure 4.3 Distribution of mean pavement depth (MPD)



Maintaining skid resistance to the required level remains a challenge for any road network maintenance manager. The traditional way to address deficient micro- and macro-texture is to resurface the road. This approach is costly, especially when the deficiency occurs within the design life of the surface layer.

When remedial measures are investigated, the expected effect has to be measurable in terms of years of service life added to the road surface, as compared against the value of the resources expended to effect the remedial action. In other words, the cost of maintenance should never outweigh the value gained by the improvement to the road surface.

The following techniques are available to the network manager:

- **Low-pressure high-volume captive water blasting:** This does not alter the surface texture, and is merely a cleaning action that can be used after spillages have occurred. It includes a vacuuming technique to dispose of the contaminants or loose material on the surface. Chemical cleaning agents can also be used.
- **High-pressure high-volume captive water blasting:** This can be used to unclog and restore the functional properties of the surface, and is specifically suited to cleaning OGPA layers. A vacuuming technique is used in conjunction with this treatment to dispose of the contaminants or loose material that is produced in this process.
- **Ultra-high-pressure low-volume captive water cutting:** This can be used to remove paint markings from the surface and restore its macro-texture. The method might also enable restoration of the micro-texture of the aggregate. Because of the abrasion caused by the high water pressure, this method can damage the pavement surface and should be used with caution. The high cost of water cutting means that the process is normally limited to a small portion of a site.
- **Mechanical processes such as shot- or sand-blasting and milling:** These can be used to abrade the surface in order to restore the micro-texture. These techniques do not perform well on flushed

surfaces, and flushing needs to be removed in order to expose the aggregate before the abrasive action can be applied. High-pressure shot- or sand-blasting can also be used to remove paint markings from road surfaces. Fine milling of the surface can be used to improve the macro/micro-texture of an asphalt surface, but should be used with caution as it might cause damage to brittle surfaces and OGPA layers. These techniques have not generally been found to be effective on OGPA.

These methods were all tested on the South Auckland Motorway Network. However, the results of these trials were not conclusive, and further trials are required.

The challenge is to find a restoration method that is economical, efficient (ie optimum utilisation of limited resources) and effective, and that has sustainable results. At the same time, the methodology should only require brief and simple road closures (for minimal impact on road users) and should produce little wastage.

4.3 OGPA production

In New Zealand, OGPA is produced and constructed under two specifications:

- TNZ P/11, which is a conventional-method specification
- TNZ P/23, which is a performance-based specification.

OGPA is produced from high-quality stone, and both modified and unmodified binders are used in its production. Crushed steel slag has been utilised in OGPA production, mainly because of the high PSV that it can produce. Table 4.1 summarises the properties of typical New Zealand aggregate, the required binder properties, and the typical mix properties required under the TNZ P23 specification.

Table 4.1 Typical OGPA mix properties

Item description	TNZ P/23 requirement	Typical value
Course aggregate		
Crushing resistance (kN)	≥230(kN) to produce	≥230
Weathering resistance	AA	AA
Polished stone value	TNZ T/10 specification (project-specific)	55–61
Los Angeles Abrasion Loss (%)	Report value	7–30
Wet/dry strength variation (%)	Report value	8–53
Water absorption	Report value	0.8–1.8
Fine aggregate		
Crushing resistance (kN)	200 min. parent rock	≥200
Sand equivalent	35 min.	64
Clay index	3 max.	1.3
Water absorption (%)	Report (NAS specification 2 max.)	1.1
Degradation factor	Report (NAS specification 60 min.)	93
Binder properties		
Penetration grade bitumen	Compliance with TNZ M/1	B50 43–48
Modified binder – softening point (°C)	Report value contract specification 62 min.	AB4 70–85
Modified binder – torsional recovery (%)	Report value contract specification 12 min.	31–46
Modified binder viscosity (Pa.S)	Report value contract specification 1 max. @ 165°C	0.4–0.5

Item description	TNZ P/23 requirement	Typical value
Mix design properties		
Drain-down (%)	0.3 max. @ 8°C more than mix temperature	0.2–0.3
Asphalt particle loss (%)	15 max.	2–9
Retained tensile strength (%)	80 min.	78–94
VMA (%)	Not specified	28–33%
Refusal air voids (%)	2 min. AC	18–23
Binder film thickness (μm)	7.5 min. AC	7–9
Field permeability (litre/min)	12 max.	8–9
MPD (mm)	Contract specification	1.2–1.4
Wheel tracking (mm)	Report value	2–5
Binder content (%)	Report value	4.5–5.5
Voids (%)	Report value	18–23

4.4 Summary of OGPA specification and approval process

Prior to construction, the mix for the OGPA to be used at that site is calculated based on a lay-down trial. The mix is accepted according to its volumetric, textural and permeability properties. During the production of the OGPA, the contractor is required to provide data on the mix variations in agreement with the contract quality plan. Parameters such as improvements in riding quality and rutting, and other site information, is treated as report-only information. Properties such as MPD and PSV are contract-specific, and SCRIM requirements are covered in the TNZ T/10 specification.

Figure 4.4 Free-draining OGPA on the South Auckland Motorway

4.5 Improvement in riding quality

The post-surfacing improvement in riding quality is usually specified to be less than a NAASRA[4] count of 50. Because of the nature of an OGPA overlay and its relative thinness, the post-surfacing results were found to be dependent on the roughness of the underlying surface – improvements of 15–30% in NAASRA roughness were reported on pre-surfacing values of 100 and 80 respectively.

4.6 Additional testing

The following additional asphalt tests were conducted on OGPA samples from the network:

- resilient modulus testing
- fatigue testing
- wheeltrack rutting
- torque bond test.

4.6.1 Resilient modulus testing

The testing was carried out at two locations on the network – the Southern Motorway at East Tamaki and the Northwestern Motorway at Henderson. The considerations for selecting these sites were as follows:

- **Southern Motorway at East Tamaki:** This section of motorway was constructed and opened in 1955. The SNP of the site was 6.8 and the base was cement-stabilised. The test cores showed significant pre-levelling with what appears to be Mix 10 asphalt. The base consisted of three consecutive OGPA layers, with the most recent surface being five years old at the time of coring.

  Six cores were taken and the resilient modulus averaged 2050MPa, with a range of 1616–2439MPa.

- **Northwestern Motorway:** Because of the 1970s oil shock, sections of the Northwestern Motorway did not receive a structural asphaltic concrete overlay. It was considered to be a test of what contribution multiple OGPA layers could provide on a pavement, conceivably outside its design life. The SNPs were between 3.5 and 4.5.

  Twelve samples were tested for this resilient modulus, with an average of 1900MPa and a range of 1600–2600Mpa. This implies the core had the same strength as normal dense-mix asphaltic concrete.

4.6.2 Fatigue testing

This process tests the pavement's flexure strength and involves multiple deflection loadings. The results were inconclusive, as although 4 of the 11 beams performed well, 7 failed relatively early; ie after a relatively low number of repeated load cycles. It is thought that beams cut from an OGPA slab may not be appropriate for testing, given the potential for damage from either pre-existing defects or during the beam's extraction.

4 Measure of road roughness in terms of NAASRA counts/km (National Association of Australian State Road Authorities)

4.6.3 Wheeltrack rutting

This testing simulates deformation of the asphaltic mix from wheel loadings and is carried out at elevated temperature (60°C). The 12 samples had an average of 2.8mm of deformation and a range of 1.9–4.4mm – the Austroads guide *Selection & design of asphalt mixes* (1998) ranks deformation depths of less than 3.5mm as superior.

4.6.4 Torque bond test

This test was one of the most intriguing, not only in terms of the results, but in the way the samples failed. The intention of the test was to get an indication of the bonding strength between the layers. Of the 13 samples tested, 10 exhibited good torsional strength over 3 layers. Torsional strengths on 10 of the samples ranged from 350kPA to 520kPA; the other 3 were between 140kPa and 170kPa. Most of the samples failed through the OGPA layer (as opposed to the chipseal layer), indicating that the chipseal layer provided a bonding action, allowing the transfer of the wheel load into the lower OGPA layers, which is the probable key to the strength of this composite structure.

4.7 Failure mechanisms observed in OGPA layers

Typical failure mechanisms observed on the South Auckland State Highway Network were as follows:

- loss of functionality – loss of skid resistance and water transitivity
- ravelling of the surface – stone loss due to brittle binder and mechanical damage
- pavement rutting – OGPA have excellent rut resistant properties, but the granular bases seem to be the weak link
- surface cracking and pumping – usually related to the pavement structure rather than the OGPA surfacing.

The above failure mechanisms are depicted in the photos below.

Figure 4.5 Ravelling due to ageing of the bituminous binder

Figure 4.6 Cracking and pumping of the pavement surface due to pavement rutting and moisture retention in the OGPA surface

Figure 4.7 Pavement rutting and subsequent failure of the OGPA surfacing

Figure 4.8 Multiple OGPA exposed to constant wet conditions in the bottom layer because of impeded side drainage (stripping of binder has occurred)

4.8 Mechanistic analysis of multiple OGPA layers

In order to quantify the contribution of consecutive OGPA overlays to a pavement's structure, a mechanistic analysis focusing on the basecourse stress ratio was performed. Actual data obtained on State Highway 16 (section RP 7/8.58 to 7/8.92) was selected as the analysis section, owing to the availability of long-term traffic, overlay and rut information. The first step was to model the pavement and each of its OGPA overlay phases with CIRCLY design software, focusing on the triaxle stress induced in the basecourse under a standard 80KN axle load. The pavement was modelled as follows:

1 as a sealed pavement

2 as a pavement with a single OGPA overlay

3 as a two-layer system with stiffening of the initial OGPA overlay

4 by overlaying it with a third overlay and progressively stiffening the initial and second overlays.

The stress ratio was defined as the triaxial stress under the design load divided by the triaxial stress at failure. Assuming an internal angle of friction between 44 and 46 degrees, and a nominal cohesion of 50kPa for the basecourse material, the stress ratio could be calculated.

The pavement composition and historic traffic loading are illustrated in figures 4.9 and 4.10 respectively.

Table 4.2 summarises the CIRCLY inputs (layer properties) and date of overlay. The results of the analysis indicated that with each consecutive overlay, the rut capacity of the base increased as the stress ratio in the middle of the basecourse decreased.

It was concluded that as the number of OGPA layers increased, the structural capacity of the composite structure increased and the rate of rut progression of the basecourse decreased significantly, as illustrated in figure 4.11.

Figure 4.9 Traffic loading

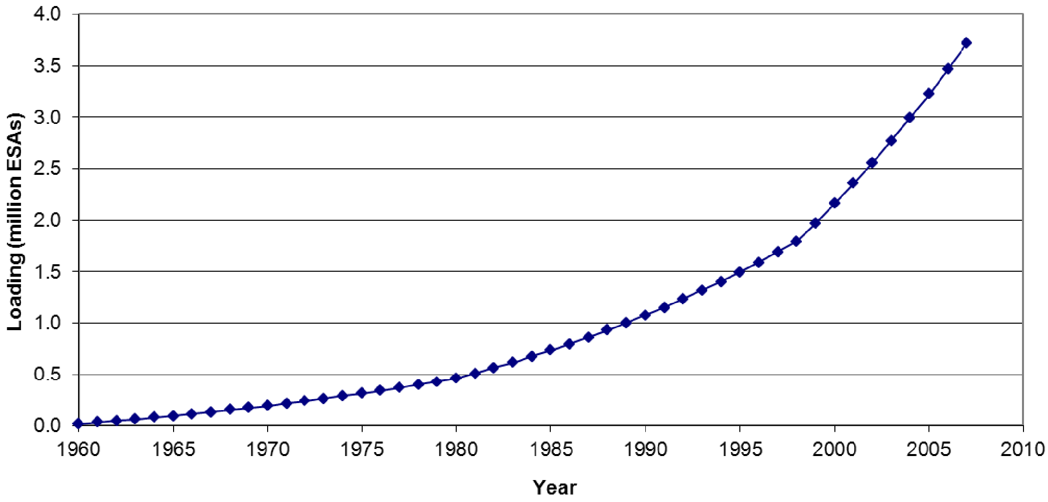


Figure 4.10 Pavement composition

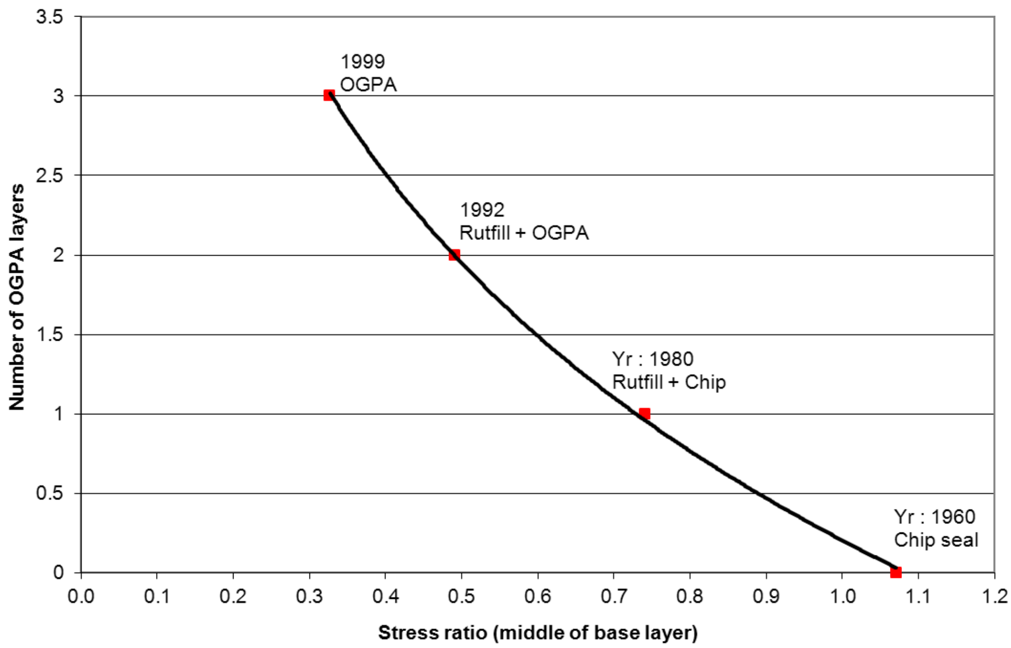
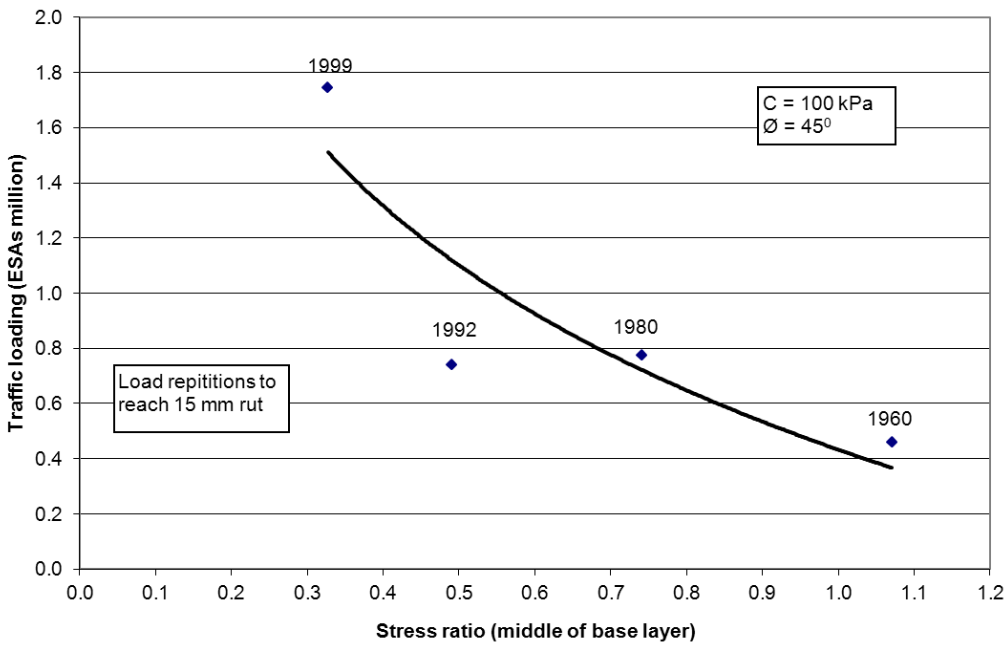


Table 4.2 Mechanistic analysis summary

Year of construction	1960	1980	1999	2007
3rd OGPA stiffness				1800MPa
2nd OGPA stiffness			1800MPa	2000MPa
1st OGPA stiffness	Seal	1800MPa	2000MPa	2500MPa
Base stiffness	550MPa	550MPa	550MPa	550MPa
Sub-base stiffness	250MPa	250MPa	250MPa	250MPa
Subgrade CBR	4%	4%	4%	4%
Cumulative traffic loading (ESA)	0.46×10^6	1.2×10^6	1.9×10^6	3.9×10^6
Rut as measured		10-15mm	5-8mm	<3mm
Base stress ratio (SR)	1.07-0.89	0.74-0.62	0.49-0.62	0.32-0.27

Figure 4.11 Performance curve



The age/stiffness relationship illustrated above (and confirmed by the 1997 Transport Research Laboratory (TRL) study) allows the reader to model a multiple-OGPA system and calculate the stress reduction in the other pavement layers (as demonstrated in this section. We are confident that the stiffness values are accurate and measured stiffness fits as expected from classic rheological models. Terminal stiffness was also derived from measurement (ITT[5] on cores and beams from ravelled sections and confirmed through known universal terminal bitumen stiffness). By following these principles, overlaying of sections that have reached terminal stiffness can be avoided, or pre-treatment of such sections can be undertaken (filling with emulsion membrane sealing etc). If stiffness measurement (ITT or beam test) is undertaken at regular intervals on a network level, the condition of an OGPA surfacing can be tracked with time and overlays can be programmed accordingly.

5 Indirect tensile test

5 Network maintenance management

5.1 NZTA pavement management system

The 'pavement management system' through which the NZTA manages the New Zealand state highway network incorporates a long-term pavement performance modelling process that uses the dTIMS[6] CT computer software as framework to find the optimal set of maintenance strategies. This system determines the optimal overall levels of service based on a target benefit–cost ratio, and ensures that the maximum return is derived from investment in the road infrastructure. Data is obtained from the RAMM database, which is the depository of all asset information such as inventory, performance and maintenance data.

The Road Information Management Systems (RIMS) Group is responsible for providing leadership and strategic advice to the New Zealand road management industry on best practice and asset management information systems for roads. In this role, the RIMS Group is responsible for the customisation of the dTIMS CT application software to reflect the conditions and pavement performance in New Zealand.

The analysis within dTIMS CT uses the 'life cycle cost analysis' approach to compare different strategies for maintaining or improving road elements – ie it estimates the future costs that would result from various strategies. The three basic functions within the dTIMS CT analysis are strategy generation, optimisation and programme development.

Maintenance strategies that employ a range of treatments are generated for each road segment. The system predicts how each strategy would affect the condition of the road section over the analysis period (20 years). To find the best treatments for any given year, the system uses triggers that describe the feasible range of treatments that represent every reasonable course of action that could be performed on each road element during the analysis period.

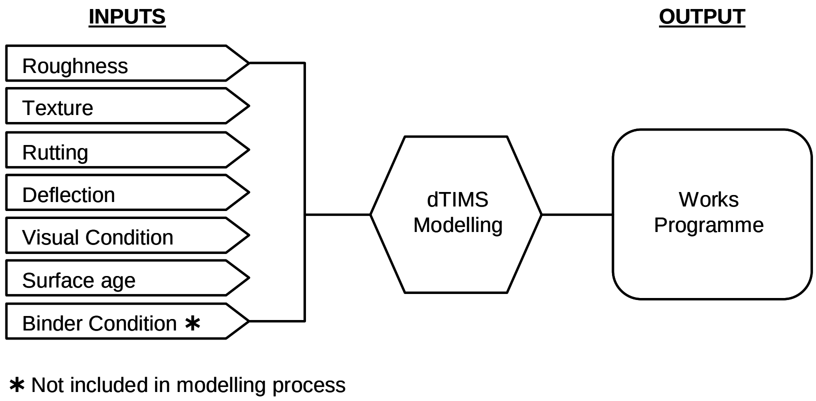
The cost stream generated by each given strategy is automatically calculated by dTIMS CT and summarised for different types of treatments occurring over the analysis period. dTIMS CT uses the area under the curve (condition over time) to calculate the benefit for each strategy.

To find the optimal maintenance regime under a given funding scenario, dTIMS CT evaluates the life cycle cost of the range of maintenance strategies and uses an optimisation process, considering the objectives and constraints, to determine the strategy for each road section that will minimise the total transport costs for a given budget scenario. The selected strategies then form the construction programme or works programme.

The network-level pavement management process is illustrated in figure 5.1.

6 Deighton's Total Infrastructure Management System

Figure 5.1 Network-level pavement management



5.2 Finalising the works programme

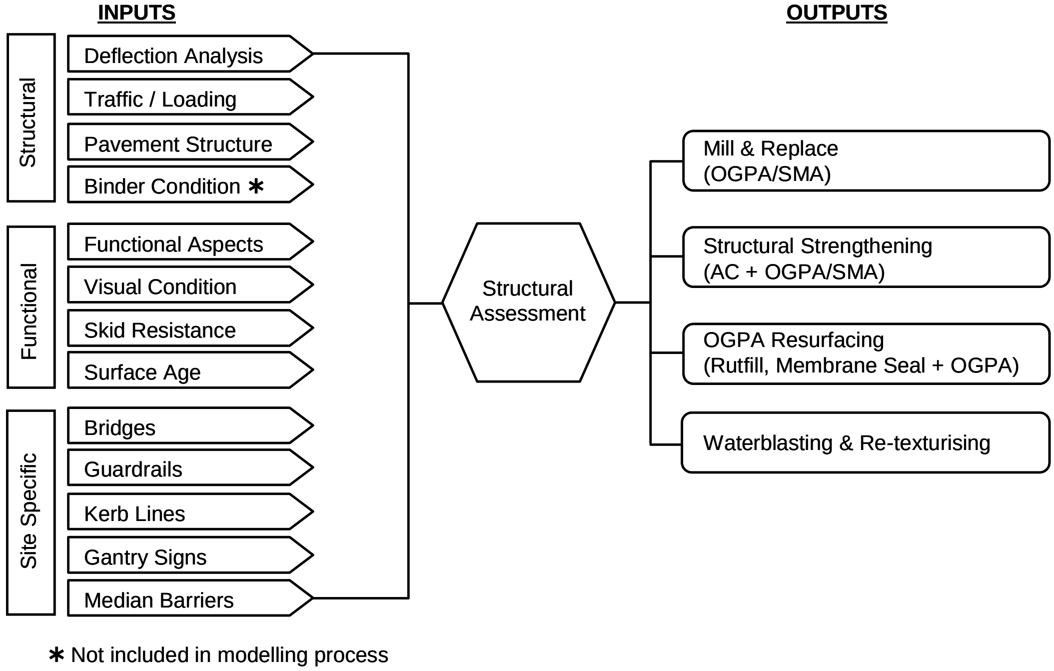
Once the strategies are generated for each treatment length, a 10-year condition forecast and forwards works programme are produced, followed by a field validation exercise of the works programme. The validation is undertaken by the network manager and a panel of experienced evaluators, with a focus on the outputs of the first three years of the programme.

During the network-level assessment, a panel of evaluators considers several factors to decide on the final treatment. All the relevant link information, such as traffic loading, surface attributes, several condition parameters, and maintenance costs, is available to the panel during this evaluation exercise. In formulating its final decision, the panel also considers site constraints such as the available kerb and barrier lip heights, guardrail heights, and the structural number of a treatment length. For instance, in cases where overlaying of the existing road surface will result in the barrier lip height not conforming to the applicable safety standard, milling off the existing surface is selected prior to resurfacing with an OGPA overlay.

A detailed structural analysis of pavements is generally not undertaken at network level. Road sections that are earmarked for resurfacing are also checked for structural capacity, so that only sections with a SNP ≥2 are considered for overlay. Sections with SNP <2 are expected to pose a risk of early OGPA failure due to expected high pavement deflections and curvature, and are considered for structural strengthening prior to resurfacing. The structural numbers used in the analysis are derived from network pavement deflection surveys measured by an FWD apparatus.

The process for finalising the works programme is illustrated in figure 5.2

Figure 5.2 Finalising works programme



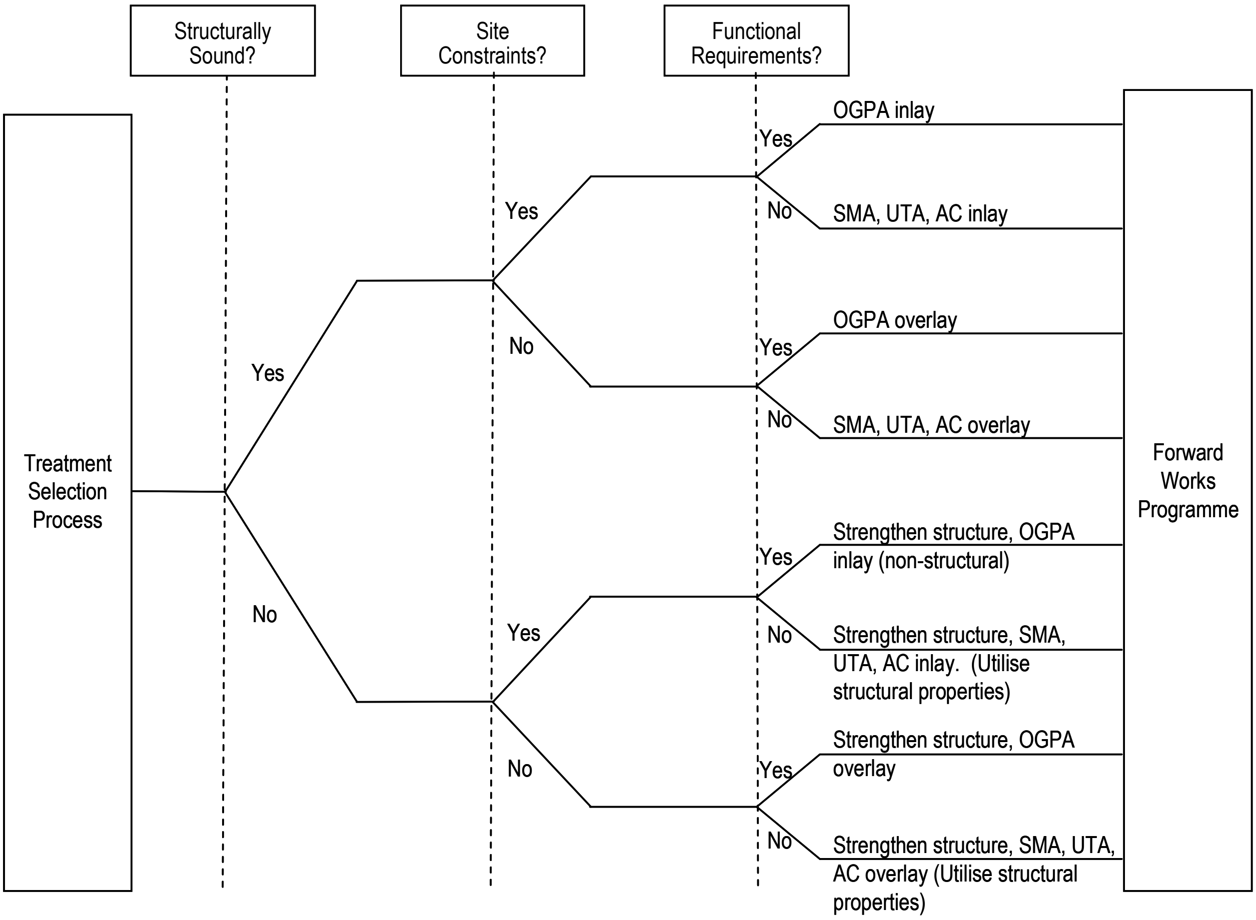
The finalised works programme then serves as input into dTIMS CT for a specified analysis run. The outputs of this analysis are used to confirm the budgetary spending, and to compare the predicted network condition against other budget scenarios.

5.3 Proposed improvement to optimisation process

From the above, it is clear that the dTIMS CT system is a comprehensive and complex model. Although the model does not use binder condition or binder ageing as a direct input, it utilises other parameters, such as surface condition, surface age and maintenance costs, as an indirect measure of binder condition.

A maintenance decision-making tree that considers the structural soundness of a pavement with cognisance of site constraints and functional requirements is proposed in figure 5.3. The use of this decision-making structure would formalise the decision-making process, and would contribute towards the improvement of the network condition in the long term.

Figure 5.3 Proposed network maintenance decision-making structure



In the case of a sound pavement section (SNP ≥2) on which the functional properties of an OGPA surfacing are required, and where the section has no site constraints such as kerb lines, guardrails, etc, the decision-making structure suggests milling off the existing OGPA surfacing prior to replacing it with new OGPA. The current maintenance practice under these conditions (no site constraints) would not necessitate milling off the existing surfacing. The long-term implications of this approach need to be monitored, as the potential of a water reservoir sandwiched in the multiple layers could lead to bitumen stripping and consequent failure. However, there has not yet been any evidence of this occurring to any significant degree.

6 Methods of measuring OGPA stiffness

The aim of the stiffness measurement of a multiple-OGPA structure is to determine the relationship between mix stiffness/binder stiffness and in-service layer performance, aided by previous experience and published terminal binder-stiffness values.

The stiffness of an existing asphalt layer can be measured through various methods – a short overview of the available tests is necessary in order to select the best method.

6.1 Non-destructive methods

6.1.1 Falling weight deflectometer (FWD) method

This device is considered to be the most efficient method for testing the condition of a pavement structure. The technology developed around FWD testing – such as the back-calculation of layer moduli and pavement life prediction – is a worldwide standard.

Results from the FWD testing can be used to back-calculate the stiffness of an asphalt layer, but the equipment is not considered sensitive enough to calculate the stiffness of the surfacing layers to the accuracy required for this study – owing to the scale of the test and the spacing of the geophones or accelerometers of the device, it might not be able to back-calculate the stiffness of a thin asphalt layer.

With its current set-up, the FWD provides useful information from which the pavement curvature can be calculated. Crack activity on an existing pavement can also be determined with this device – this parameter enables the assessment of the degree of reflective cracking in the pavement, which is linked to the success of consecutive overlays. This device remains a valuable tool in the structural assessment of pavements on both network and project design levels.

6.1.2 Spectral analysis of surface waves (SASW)

SASW measurement is based on the velocity of seismic waves through the pavement layers. The principle behind this procedure is the analysis of the complex series of waves produced by a disturbance created at the surface of the pavement. In order to measure the surface stiffness of the pavement, the baseline or distance between the geophones needs to be short. Also, the frequency at which the geophones operate needs to be high enough to measure the velocity of the surface waves. In Texas, this system was mounted on a FWD device and used in combination with this technology, to supplement the FWD data.

In 2003, Land Transport NZ (now NZTA) funded a research project that investigated the application of SASW for the measurement of layer thickness and material moduli of pavements, in order to assess pavement strength (Furlong et al 2004). Although the equipment used during this project produced positive results, the researchers recommended modifications to render the device more useful for the measurement of pavement-surface stiffness. The proposed enhancements include improved measurement resolution, simplified data interpretation, and converting the SASW equipment into a rolling measuring device.

The technology is not yet available in this form in New Zealand, but promises to be a valuable pavement-analysing tool that can be used effectively at network level.

6.2 Destructive methods

6.2.1 Measurement of binder stiffness

This measurement is applied to a sample of bitumen binder recovered from core samples extracted from the pavement. A solvent is used to extract the binder from the mix; the solution of solvent and bitumen is then put through a 150μm sieve, centrifuged for 20 minutes, and filtered under vacuum in order to purify the solution. The volatiles in the solution are allowed to evaporate in a nitrogen atmosphere, leaving the recovered bitumen. The modulus of the binder is determined in a parallel plate rheometer, or the binder penetration can be determined in a penetration test apparatus.

Binder recovery on a network level is not deemed feasible because of the high cost involved, and is usually applied to research or specific problem-solving situations.

Recovering binder from multiple-OGPA-layered samples is complicated because of the relatively thin layers, and the presence of the inter-layered membrane seals in these samples.

6.2.2 Bending beam test

This test was developed to determine the fatigue life of asphalt samples, and can be used to establish the stiffness of a mix. Failure of a test specimen is defined as the point at which its stiffness has reached 50% of the initial measured stiffness value. The test can be performed at a constant strain level or a constant stress level, the latter being the more severe situation.

The disadvantage of this test lies in the level of care and precaution required when retrieving and preparing an undisturbed sample for testing, which makes it an expensive test. The testing of samples retrieved from multiple-OGPA structures is problematic with regards to the interpretation of the results because of the presence of membrane seal layers between successive OGPA layers.

6.2.3 Measurement of modulus by means of indirect tensile measurement (ITS)

The basis of the ITS test is the measurement of the tensile strain that is exerted on the asphalt sample by the application of a compressive force on the sample. Test specimens with diameters of between 100mm and 150mm, and specimen heights between 35mm and 70mm, can be tested. Specimens with a resilient modulus between 600MPa and 28,500MPa can be tested.

During this study the ITS test was conducted successfully on both laboratory-prepared OGPA samples and multiple-layered samples retrieved from the road surface.

6.2.4 Other performance tests

Cantabro durability testing of laboratory specimens has been used extensively throughout the world. Herrington et al (2005) proposed that the Cantabro test should be specified for design purposes on unaged and aged OGPA specimens. In this method, the newly prepared specimens are aged in a pressure-ageing vessel at 80°C for three days under an air pressure of 2070kPa (300psi). The results published in their report indicated a weight-loss increase by a factor of 1.5 to 2.1 after ageing. However, the applicability of the Cantabro test on cores retrieved from the road is limited because of the required sample thickness of 65mm.

6.3 Discussion

The test methods described above are summarised in table 6.1 in terms of complexity and relevance to the measurement of OGPA stiffness. The destructive nature of the tests, and whether the test method is available in New Zealand, are also assessed.

Table 6.1 Summary of test methods

Method	Destructive	Applicable to OGPA samples?	Complexity	Available in NZ?
Binder recovery	Yes	Yes	Not easy to perform	Yes
FWD	No	No	Easy to perform	Yes
SASW (high frequency)	No	Yes	Easy to perform	No
Bending beam	Yes	Yes	Not easy to perform	Yes
Resilient modulus (ITS)	Yes	Yes	Easy to perform	Yes
Cantabro durability	Yes	Yes – thick OGPA layers	Easy to perform	Yes

Although the FWD test method rates highly as a non-destructive and easy-to-perform test, its accuracy in measuring the in-situ stiffness of a relatively thin OGPA surface layer is considered to be poor, owing to the spacing of the geophones (200mm minimum). Further research on this aspect is needed. Binder recovery and stiffness measurement is an alternative (but expensive) test, and is not considered feasible at network level. High-frequency SASW promises to be a quick and accurate means to measure in-situ stiffness, but the technology available in New Zealand has not yet matured and further development is required to enable the measurement of multiple OGPA layers.

Bending beam tests have been performed on OGPA layers in New Zealand, but the sample preparation is costly and difficult, and the results vary.

ITS testing presents a quick and reliable means of measuring OGPA layer stiffness, but samples with thin OGPA layers pose a problem because the minimum sample thickness required by the test method is in the order of 50mm.

Cantabro testing offers to test the durability of a mix on samples retrieved from the road surface but again, the sample thickness is the limiting factor, as the test method requires a sample thickness of 65mm.

Testing with the aim of measuring stiffness values of OGPA samples retrieved from the road is therefore problematic, and this necessitates the development of alternative ways to obtain in-situ stiffness values.

7 Relating field performance to resilient modulus (stiffness)

7.1 Background

7.1.1 Investigation of multiple OGPA layers

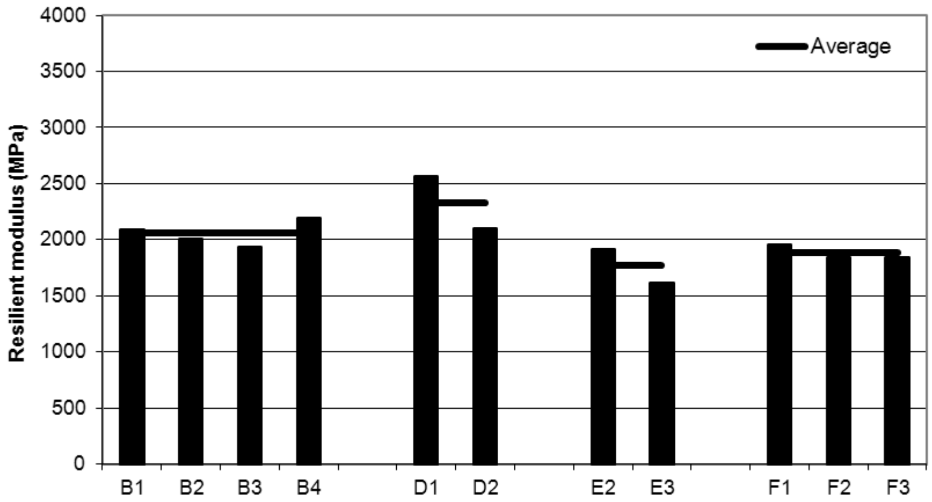
During 2005/06, MWH New Zealand Ltd investigated the performance of pavements with multiple OGPA layers on the South Auckland State Highway network (MWH 2006). Samples extracted from four test sites on State Highway 16 (SH16) were subjected to standard laboratory testing procedures to determine the mix stiffness, by means of the repeated-load ITS test, as well as the bending beam (fatigue) test. A description of the condition of the sites at the time of testing is presented in table 7.1.

Table 7.1 Multiple-OGPA study

Reference	Layer description	Visual condition
SH16 – site B	3 OGPA layers on bitumen-treated base layer	Good
SH16 – site D	2 OGPA layers on AC	Poor – cracking & ravelling
SH16 – site E	2 OGPA layers on granular base	Fair – limited cracking & ravelling
SH16 – site F	2 OGPA layers on granular base	Fair – limited cracking & ravelling

The apparently good visual performance of site B can be ascribed to the low deflection (0.33mm – 90th percentile and curvature (0.06mm – 90th percentile of the pavement in relation to the other sections. The deflection and curvature results, together with the pavement profile, are summarised for each section in appendix A. The results of the resilient modulus testing are presented in figure 7.1.

Figure 7.1 Resilient modulus results from SH16 cores



The surface on site D was milled off and replaced during the 2005/06 maintenance operations on the network. A visual inspection of the remaining sites during this research found that further deterioration of the surface layers had occurred in the form of ravelling and cracking. This deterioration was particularly evident on sites E and F, which had a high degree of ravelling and cracking on localised areas.

7.1.2 Pilot study

As part of this research project, a pilot study was conducted on freshly produced OGPA samples for which the volumetric properties were known. The mix stiffness of these samples was determined by means of the repeated-load ITS test under varying temperature conditions. The reason for this modification to the standard test method was to establish a master curve for OGPA stiffness in which OGPA mix stiffness is related to temperature.

The results from this testing are summarised in table 7.2. This table lists the resilient modulus, loading time, temperature and volumetric properties for the test specimen from the pilot study, as well as for the samples retrieved from the experimental sections on SH16.

The resilient modulus of the SH16 samples varied between 1619MPa and 2561MPa at the standard ITS test temperature of 25°C.

Samples from site D produced the highest resilient modulus values of all the field samples, with modulus values ranging between 2097MPa and 2561MPa. At the time when the cores were retrieved from the road, the OGPA surface on this site exhibited extensive cracking and ravelling, and was consequently milled off and replaced during the 2005/06 maintenance operations on the network.

The stiffness moduli of the other SH16 samples varied between 1600MPa and 2100MPa. The surfacing on these sites was in a moderate to good condition at the time of sampling, although some ravelling was noticed during a visual site evaluation in February 2007.

The above observations point to a relationship between the visual condition of the OGPA surface in terms of the extent of ravelling, and the stiffness of the layer as measured by means of the ITS test.

The results of the ITS testing on samples prepared for the pilot study indicated a significant variation in resilient modulus, which varied between 478MPa and 6155MPa, at test temperatures ranging between 30.2°C and 5.5°C respectively. This illustrates the effect of temperature variation on OGPA stiffness and raises the importance of careful consideration with regard to the application of these mixes in different temperature regimes. This observation also highlights the possibility that OGPA mixes are more prone to ravelling during the cold winter period, as the mix ages.

Table 7.2 Resilient modulus at varying temperatures (ITS test results)

Sample no.	Site ref.	Temp (ºC)	Loading time (sec)	Resilient modulus (MPa)	Air voids (%)	Notes	
1323	SH16-E1	25.4	0.100	1914	-	Site E1	Fair condition, some ravelling, Feb 2007
1325	SH16-E2	25.3	0.100	1619	-	Site E2	
1327	SH16-F4	25.3	0.100	1957	-	Site F4	Fair condition, some ravelling, Feb 2007
1328	SH16-F5	25.3	0.100	1846	-	Site F5	
1329	SH16-F6	25.4	0.100	1848	-	Site F6	
1209	SH16-D3	25.0	0.100	2561	-	Site D2	Poor condition ravelled & cracked. Milled & replaced Feb 2006
1210	SH16-D1	24.8	0.100	2097	-	Site D1	
1212	SH16-B4	24.9	0.100	2096	-	Site B4	Good condition, some ravelling, Feb 2007
1213	SH16-B1	25.1	0.100	2012	-	Site B1	
1214	SH16-B2	25.3	0.100	1940	-	Site B2	
1215	SH16-B3	25.4	0.100	2196	-	Site B3	
3	Lab	5.2	0.097	5336	20.3		Pilot study – production mix, Nov 2006
1	Lab	5.5	0.102	6155	20.2		
21	Lab	6.8	0.097	5645	19.7		
23	Lab	6.8	0.099	5376	20.7		
24	Lab	12.7	0.098	3042	21.4		
29	Lab	13.2	0.100	2871	20.7		
12	Lab	19.5	0.100	1404	20.4		
10	Lab	19.6	0.100	1608	20.4		
11	Lab	19.6	0.100	1329	20.4		
17a	Lab	24.8	0.100	949	-		
17	Lab	24.9	0.100	908	-21.5		
18	Lab	24.9	0.100	771	21.3		
16a	Lab	24.9	0.100	880	-		
18a	Lab	24.9	0.100	725	-		
8	Lab	25.1	0.100	888	20.4		
16	Lab	25.2	0.100	995	21.4		
9	Lab	25.2	0.100	962	21.0		
7	Lab	25.2	0.100	815	19.1		
14	Lab	29.8	0.100	543	21.6		
13	Lab	30.2	0.100	478	19.9		
15	Lab	30.3	0.100	540	21.1		

The samples tested from the pilot study stiffened up more than expected at a temperature of 5°C, which indicated that the binder used in this mix might have been sensitive to low temperatures. This emphasises the value of resilient modulus testing at various temperatures on proposed asphalt mixes. Temperature sensitivity may be identified through this approach.

7.2 Failure of OGPA mixes due to binder stiffening

Based on the above observations, it is suggested that the failure mechanism for an OGPA surfacing layer is that the mix stiffens with time because of ageing of the binder, to the point where cracking and ravelling occurs on the surface.

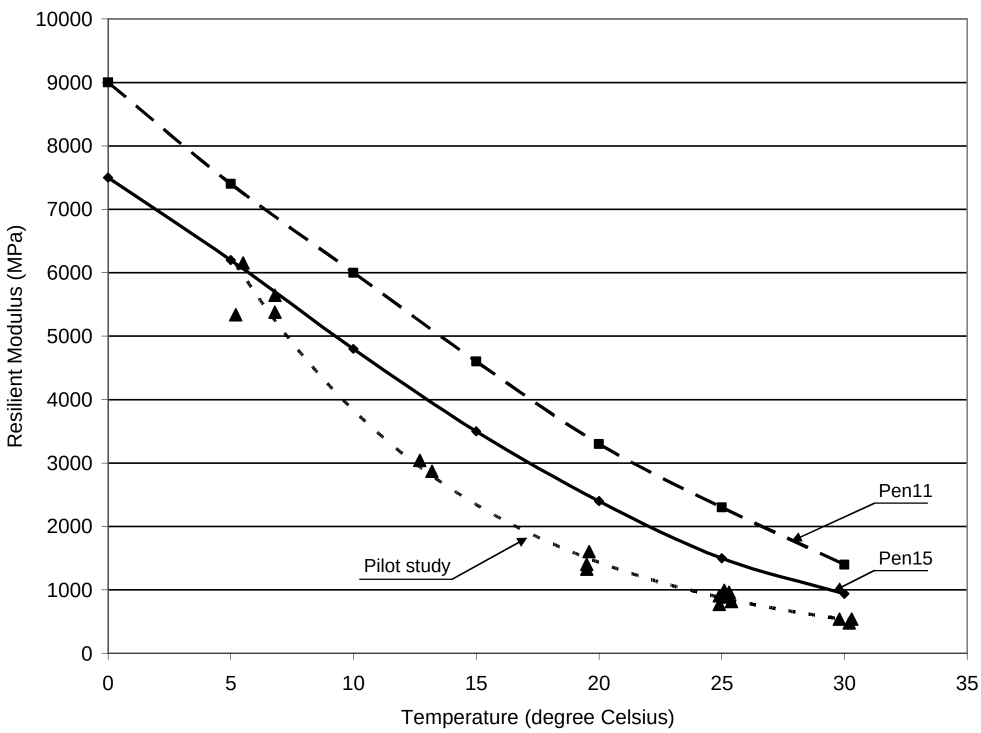
Exploring the wide range in time to crack initiation, several other failure mechanisms might be at work. This section is focused on the binder stiffness, with the aim of being able to predict the point at which brittle failure of the binder in the mix occurs.

7.3 Predicting OGPA failure from binder stiffness

From the reported binder failure properties as shown in appendix C, the terminal binder stiffness can be calculated for a range of temperatures, using the same loading time as for the repeated-load ITS and the proposed equations noted in appendix C.

Using the above-mentioned relationships, figure 7.2 was prepared to estimate the mix stiffness at failure for temperatures ranging from 0–30°C. The stiffness values obtained from laboratory ITS tests on the pilot study samples were then superimposed onto this chart, to determine the critical temperature at which the various samples would 'fail' or reach terminal stiffness.

Figure 7.2 OGPA resilient modulus versus temperature



The relationships in figure 7.2 were prepared by using a load frequency of 0.1sec corresponding to the loading time of the ITS test. Varying the temperature from 0–30°C, the resilient modulus of an OGPA mix with volumetric properties, as mentioned in section 2.1 of this report, was calculated by using the proposed equations and Shell nomograph, and a failure criteria of Pen = 15, PI = 0.4 and TR&B = 70°C to calculate the first failure envelope (Pen15). These values were based on the binder properties of failed sections of OGPA that were published in TRL report 264 (Nicholls 1997), and summarised in appendix C. The corresponding OGPA stiffness of 1500MPa at 25°C was calculated at the same loading time as for the repeated-load ITS by using the proposed equations and Shell nomograph.

Similarly, a second envelope (Pen11) was calculated using Pen = 11, PI = 0.5 and TR&B = 75°C. This was based on an observed failed section of asphalt as reported in appendix C (Bahia and Nam 2004). The corresponding OGPA stiffness of 2300MPa at 25°C was calculated at the same loading time as for the repeated-load ITS by using the proposed equations and Shell nomograph.

Ravelling was observed on the test sections of back-calculated bitumen penetration values ranging between Pen15 and Pen11. These values correspond to an OGPA stiffness of 1500MPa and 2300MPa. Based on a terminal binder stiffness of Pen11, a terminal OGPA stiffness of 2300MPa is proposed.

The resilient modulus results obtained from the pilot study were then superimposed onto this chart. The stiffness moduli of the samples tested during the pilot study showed a strong correlation with the predicted stiffness values for a 40/50 Pen binder and volumetric properties of a typical OGPA mix.

This method has not been proven, and no claim can be made that the predicted failure stiffnesses are absolute values. The method does, however, show potential in predicting OGPA stiffness based on the binder properties within the mix. This principle was explored by Van de Ven (2004) as presented at the 8th Conference on Asphalt Pavements for Southern Africa (CAPSA) in 2004.

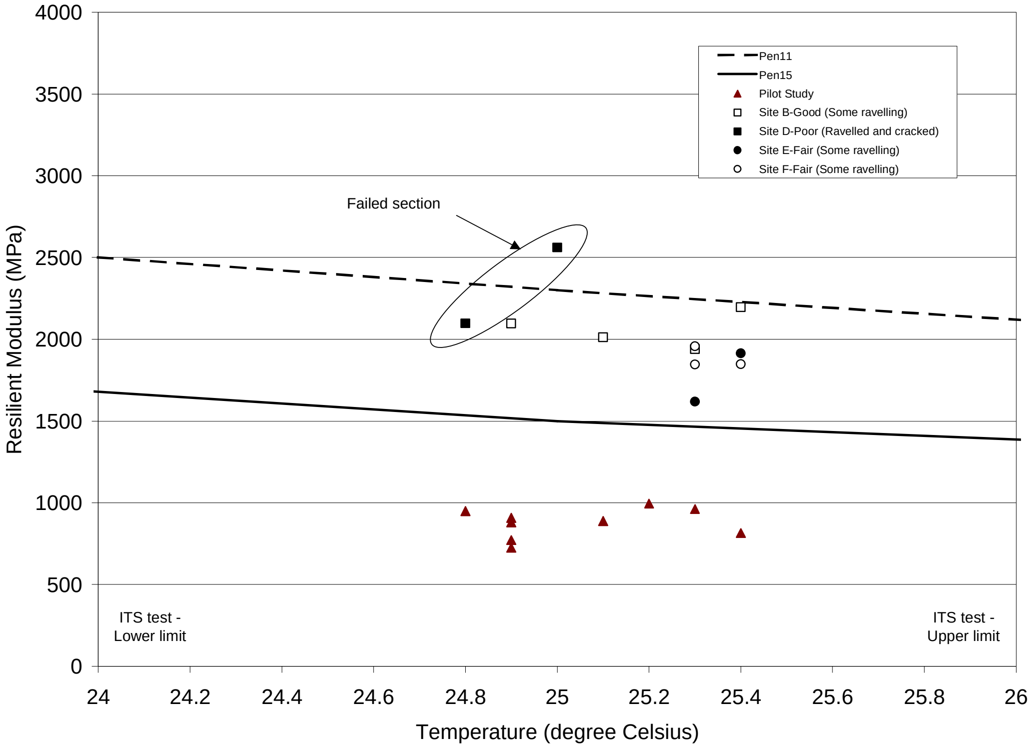
The purpose of this form of data presentation is to provide more insight into the temperature sensitivity of the asphalt mix, and allows ITS test results to be evaluated in case they were performed at temperatures other than the standard test temperature of 25°C. Owing to the nature of the ITS test (tensile strain caused by a compressive force), some researchers promulgate testing temperatures below 20°C, in order to eliminate permanent deformation of the sample during loading.

7.4 Application of method

The proposed method was used to evaluate the stiffness of laboratory-prepared samples from the pilot study, as well as of OGPA cores extracted from an existing pavement.

The resilient modulus of an OGPA mix at varying temperatures within the test specification is illustrated in figure 7.3, and indicates the failure zone above the upper contour. The chart allows for values to be plotted against various temperatures in order to be more accurate when evaluating a specific test result. Stiffness moduli from the pilot study are presented on this chart, as well as moduli obtained from core samples on the network. These cores were taken from sections that were showing various levels of distress at the time of sampling, as described in section 7.1.1.

Figure 7.3 OGPA modulus versus temperature

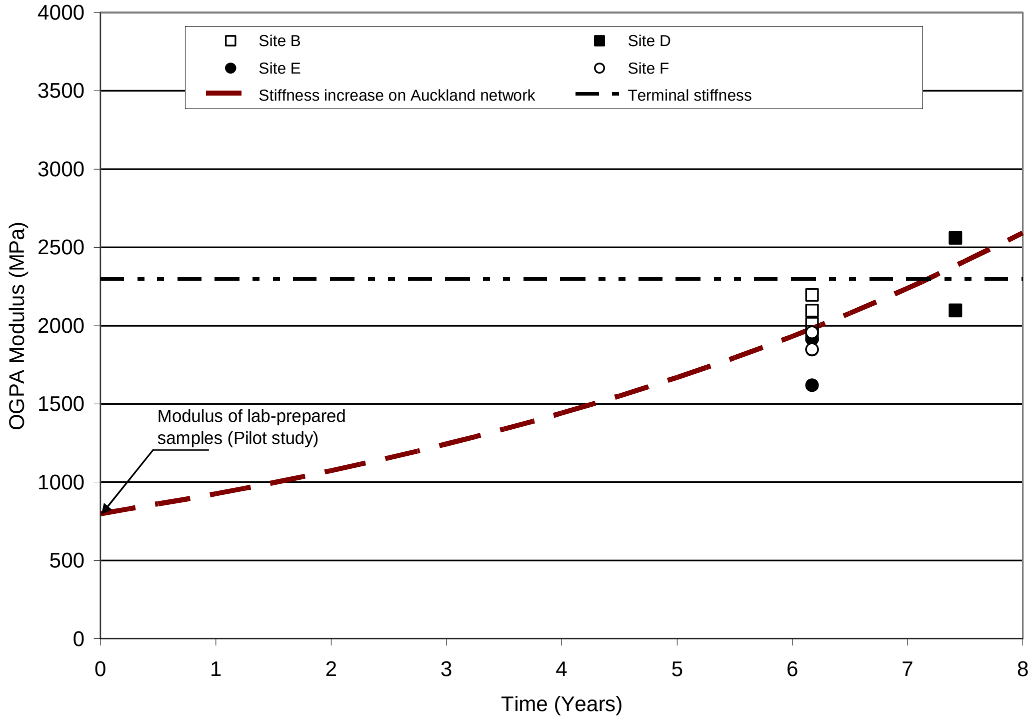


It can thus be concluded that the ITS test gives a very good indication of the condition of an OGPA mix.

This method can also be used to track the stiffening of an asphalt layer over time, by testing core samples from the pavement at various age intervals over the lifespan of the layer, and predicting the time at which the layer will reach a terminal stiffness. This approach has the potential to enhance the decision making in a pavement management system.

The stiffness moduli of samples retrieved from the South Auckland State Highway Network are plotted against age in figure 7.4.

Figure 7.4 OGPA modulus versus surface age



Research undertaken in the UK recommended a decrease in binder penetration at approximately 20% per year (Nicholls 1997). The increase in OGPA stiffness described in figure 7.4 is in the order of 16% per year, which is comparable to the rate of stiffening suggested in the above-mentioned TRL research report.

The remaining life of an OGPA layer can thus be predicted based on the rate of stiffening, as per the above model. The terminal OGPA stiffness was found to be 2300MPa at 25°C, as measured with the ITS test. The corresponding binder stiffness is in the order of 220MPa or Pen 11.

8 Conclusions

The principal conclusions that can be drawn from this study, which was based on the data obtained from the South Auckland State Highway Network, are as follows:

- The OGPA surfacing on the South Auckland State Highway Network has an average life of eight years to first crack initiation, which is a slightly longer life than that of the dense graded asphalt surfacings.
- The performance of OGPA appears to be related to the structural integrity of pavement structures with high SNP values. Factors other than the surface age and pavement strength affect its performance, and are probably related to binder ageing and environmental influences.
- The mean texture depth on the OGPA network increased over time – the average MPD increased from 1.27mm (in 2001) to 1.31mm (in 2002) and 1.41mm (in 2005). This may be indicative of increased levels of ravelling of the surface.
- Network maintenance management is undertaken by means of the dTIMS CT optimisation software and a maintenance decision tree – this caters for structural soundness of the pavement and also considers the site constraints and functional requirements of the new maintenance intervention.
- The current asphalt design methods in New Zealand (TNZ P11 & TNZ P23) optimise binder content within the specified limits for drain-down and Cantabro durability limits. The upper limit for stone loss in the Cantabro test is 20% and no lower limit is specified at this stage. The ITS test could be useful for determining the low-temperature performance of an OGPA mix that is not captured in this Cantabro test. This suggestion is based on the observation of early cracking on some of the OGPA surfaces on the South Auckland State Highway Network.
- The ultimate life of the porous asphalt is limited by binder hardening, with failure likely to occur when stiffness of the binder at 25°C reaches a value of between 100 and 200MPa, with an equivalent penetration value of between 15 and 11. The corresponding OGPA mix stiffnesses are in the order of 1500MPa and 2300MPa.
- Measurement of OGPA stiffness by means of ITS testing on samples cored from the road surface was found to be the most reliable method of determining the resilient modulus of the OGPA layer. Some idea of the remaining life of the surfacing could be estimated by using a 20% pa increase in stiffness. The ultimate stiffness at 25°C of between 1500MPa and 2300MPa was measured by means of ITS testing on OGPA layers that had failed by visual standards.
- A relationship between the visual condition of the OGPA surfaces and their stiffness measurements (by means of the ITS test) was found on the test sections on SH16. The results from the pilot study indicated a significant variation in resilient modulus between 478MPa and 6155MPa at temperatures ranging between 30.2°C and 5.5°C. This illustrated the effect of temperature variation on OGPA stiffness, and highlighted the importance of care with regards to the application of these layers in different temperature regimes. This also highlighted the fact that as the mix ages, it becomes more prone to ravelling during cold winter periods.
- Tracking the stiffening of asphalt over time shows potential in measuring the condition of OGPA, which could be useful in the early detection of ravelling to assist in the maintenance management of a road network.

9 Recommendations

This research has resulted in the following recommendations:

- The effects of the multiple-OGPA approach in road network maintenance could be monitored by utilising the decision-making structure proposed in figure 5.3, with the view of gradually moving away from the multiple-OGPA approach and utilising OGPA surfacing for its functional, rather than structural, properties.
- The downside of the inclusion of the Cantabro durability test in the design procedure is that mixes can be designed to comply with durability requirements, but fail at low temperatures because of excessive built-in stiffness. The ITS test could be utilised during the design process to develop a master curve for OGPA mixes, in order to find the cold-temperature susceptibility of these mixes, and the curve derived from the ITS testing could be used to define the lower limit of stone loss in the Cantabro test.
- Since there is a correlation between the visual condition and the remaining life of the OGPA, a visual survey of the pavement surface could be utilised as a management tool and for the development of a network-wide index of visual condition.
- Further research should be undertaken to develop a condition index, based on MPD measurements of the OGPA, which can be useful in the early detection of ravelling to assist in the maintenance management of the network.

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Appendix A SH16 investigation

During 2005/06, MWH New Zealand Ltd undertook an investigation for the NZTA (previously known as Transit NZ) into the performance of multiple OGPA layers on the South Auckland State Highway Network (MWH 2006).

The 90th percentile central deflection (D_0) for all test sites ranged from 0.33mm to 0.74mm. The deviation from the mean was considered to be low, which pointed to good uniformity along the various test sites – refer to appendix C for a visual illustration of the deflection results. The pavement deflection measured at site B (which was on a stronger subgrade) was the lowest, with a 90th percentile deflection of 0.33mm. Site E (which was overlaying the weakest subgrade) yielded the highest deflection of 0.74mm (90th percentile).

The curvature function values that were derived from the deflection bowl parameters are shown below. The low curvature values are indicative of a stiff upper pavement.

Table A.1 Summary of pavement curvature

Curvature function (mm)					
Test site	**Min.**	**Avg.**	**Max.**	**St.dev.**	**90th percentile**
B	0.03	0.05	0.07	0.01	0.06
D	0.04	0.09	0.14	0.02	0.11
E	0.10	0.14	0.19	0.02	0.16
F	0.09	0.11	0.14	0.01	0.13

Table A.2 Summary of central deflection

Central deflection (mm)						
Test site	**Min.**	**Avg.**	**Max.**	**St.dev.**	**90th percentile**	**Inferred subgrade CBR**
B	0.20	0.29	0.36	0.04	0.33	8
D	0.27	0.51	0.67	0.09	0.58	6
E	0.46	0.64	0.90	0.09	0.74	4
F	0.46	0.59	0.72	0.07	0.70	5

The deflection analysis is summarised in table A3 in context with the visual condition, stiffness and rut results.

Table A.3 Summary of visual condition and FWD testing

	Section	B	D	E	F
Visual assessment	Visual condition	Good	Poor, cracking & ravelling	Fair, limited cracking & ravelling	Fair, limited cracking & ravelling
	Number of OGPA layers	3	2	2	2
FWD testing	Central deflection (90th%)(mm): Low: $D_0 < 0.3$ Medium: $0.3 < D_0 < 0.7$ High: $D_0 \geq 0.7$	0.33mm Low	0.58mm Medium	0.74mm Medium-high	0.70mm Medium
FWD testing	Curvature (90th%)(mm): Low: CF < 0.4 Medium: 0.4 < CF < 0.7 High: CF ≥ 0.7	0.06mm Low	0.11mm Low	0.16mm Low	0.13mm Low
Resilient modulus	M_R	2061MPa	2329MPa	1767MPa	1884MPa
Rut resistance	Wheeltracking	2.5mm	4.0mm	2.7mm	2.3mm

The next table summarises the pavement composition on the various sites.

Table A.4 Summary of pavement composition at the various sites

Site B

Description	Layer depths (mm)			
OGPA (6yrs)	30	30	25	25
Membrane Chip	10	10	10	10
OGPA (14 yrs)	30	35	27	30
Membrane Chip	10	10	10	10
OGPA (26 yrs)	20	15	25	25
Membrane Chip	-	-	-	-
OGPA	-	-	-	-
BTB	120	85	80	95
AC	-	25	25	-
Chip	-	10	7	-
Stabilised	-	15	65	-

Site D

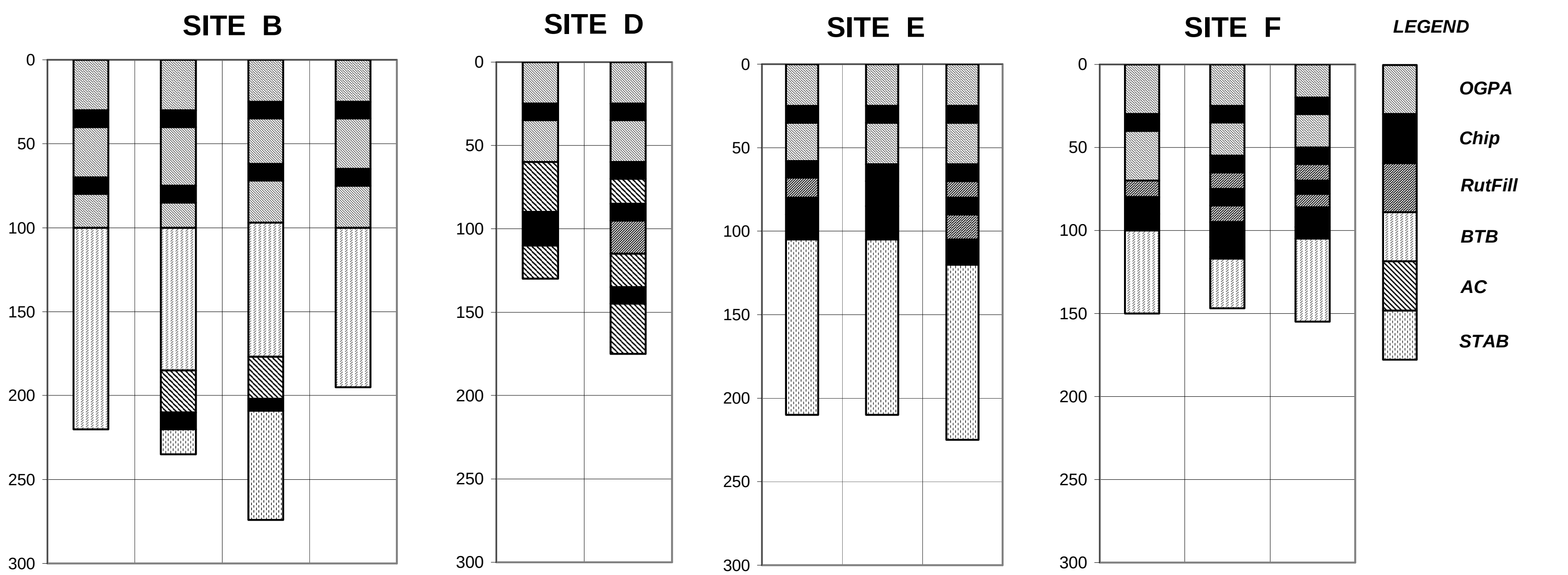
Description	Layer depths (mm)	
OGPA (8yrs)	25	25
Membrane Chip	10	10
OGPA (14yrs)	25	25
Membrane Chip	0	10
AC	30	15
Chip	20	10
Rut fill	0	20
AC	20	20
Chip	0	10
AC	0	30

Site E

Description	Layer depths (mm)		
OGPA (7yrs)	25	25	25
Membrane Chip	10	10	10
OGPA (14yrs)	23	25	25
Membrane Chip	10	10	10
Rut Fill	12	0	10
Chip	10	15	10
Rut Fill	0	0	15
Chip	15	20	15
Stab	105	105	105

Site F

Description	Layer depths (mm)		
OGPA (7yrs)	30	25	20
Membrane Chip	10	10	10
OGPA (14yrs)	30	20	20
Membrane Chip	0	10	10
Rut Fill	10	10	10
Chip	10	10	8
Rut Fill	0	10	8
Chip	10	15	12
Chip	0	7	7
BTB	50	30	50



Appendix B Clarification of terms

An explanation of some of the terms found in the literature is given below in order to eliminate confusion.

The difference between a resilient modulus and a dynamic complex modulus test is based on the loading sequence. For the resilient modulus test, loadings with a given rest period are used, while a sinusoidal or haversine loading with no rest period is applied in the dynamic complex modulus test.

The term stiffness modulus is used by Shell bitumen in lieu of the dynamic modulus. The stiffness modulus of bitumen can be determined by either a creep test with a loading time *t*, or a dynamic test under a sinusoidal loading with a frequency *f*. Van Der Poel (1954) found that similar stiffness moduli are obtained when *t* is related to *f* by:

$$t = \frac{1}{2\pi f}$$

where:

t = loading time (seconds)

f = frequency of sinusoidal loading (Hz).

A loading time (t) of 0,1sec for the resilient modulus test would thus be equal to 1.59Hz for the dynamic test.

Appendix C Literature study on terminal stiffness of bitumen binder

The stiffening of the bitumen binder in asphaltic mixes has been the focus of several worldwide studies. Terminal stiffness values at which failure generally occurs are available from the following publications:

- **Bitumen durability – *LTNZ research report 291* (Herrington et al 2006)**
  This study reported bitumen failure moduli ranging between 50 and 150MPa, with an average of 100MPa for New Zealand bitumen (Safaniya bitumen).

- **Review of UK porous asphalt trials – *TRL report 264* (Nicholls 1997)**
  This report refers to a terminal penetration value (on recovered binder) of 15Pen for bitumen binders in OGPA layers. It is interesting to note that a binder penetration of 15Pen at 25°C is equivalent to a binder stiffness of 100MPa. The study also found that the critical binder penetration remains at 15Pen, irrespective of the presence or type of modifier.

- **Effect of binder hardness on rate of texture change in chipseals – *LTNZ research report 284* (Ball 2005)**
  The focus of this study was to determine the binder stiffness on old seals at the point at which failure occurred. The project was based on binder stiffness measurements by means of a Dynamic Shear Rheometer (DSR) on binder recovered from the seal chip. Sample preparation for this test is very complex. The results confirmed a terminal binder stiffness value of 100MPa.

  Oliver (2004) also investigated binder stiffness in chipseals in Australia and found no significant difference in the viscosity of the bitumen after eight years in the field.

- **Investigation into the cause of cracking of a pavement section in South Africa – Gautrans internal report (Bahia and Nam 2004)**
  A study that was undertaken in the US on continuously graded asphalt that failed prematurely on a high-profile project in South Africa yielded the following results using the DSR on the recovered binder.

Table C.1 Results of DSR test

Sample reference	Temp (°C)	Frequency (Hz)	Phase angle (degrees)	Complex modulus (MPa)	Notes
Ch150	25	9.65	10.0	217	Failed section
Ch150M	25	9.65	11.7	162	
ND32	25	9.65	45.0	12	
ND38	25	9.65	47.5	10	
SB	25	9.65	39.3	22	
SBM	25	9.65	42.5	17	
NB	25	9.65	21.6	74	

The results show that the stiffness of the binder retrieved from the failed section (Ch150 and Ch150M) was much higher than that of the other sections. This result was confirmed by the low phase angle values of the stiffer binder.

This study concluded that a high level of thermal stress build-up could be expected within the bitumen because of these high stiffness values, and that stress relaxation would not be possible, owing to the low phase angles measured on the samples. These properties could also result in fatigue damage to the mix.

C1 Discussion

In simple terms, it can be concluded that the bitumen samples that were tested had stiffened to a point where brittle failure would occur.

Stiffness varies with the grade of binder and (if modified) the degree of modification. Other factors – such as the source of the binder and whether the bitumen was overheated during processing in the manufacture of the asphalt mix – also contribute to the ageing/stiffening process.

The penetration/stiffness can be determined by extracting binder from core samples taken from the road surface. The penetration/stiffness of the binder in future years can then be estimated by assuming a cumulative increase of 20% per year (Nicholls 1997). Ultimate failure is likely to occur rapidly after the binder penetration has fallen below 15Pen, or the stiffness has increased to a value in excess of 100MPa.

Considering the amount of research that has been undertaken, as described in the above publications, an attempt was made to obtain a common denominator from the conclusions reached by all these studies. Since the publication of the Shell nomographs (Shell 1954) (which are used to derive the stiffness modulus of an asphalt mix), many research projects have been undertaken to develop tools that can be used to estimate the complex or stiffness modulus of bitumen in relation to bitumen type. The type of bitumen is described in terms of its 'penetration index' (PI), 'pen' and 'ring-and-ball softening point' (TR&B).

Since the Van der Poel nomograph (Van der Poel 1954) cannot be used to accurately define the binder stiffness, several researchers have developed equations that allow a more accurate estimate of stiffness. Some of these equations have been developed by Shahin (1972) and validated at the Delft University (Sabha et al 1995). They are:

If:

$$10^{-8} < S_{bit} \leq 10 - 30MPa$$

$$\log_{10} S_{bit} = -1.35927 - 0.06743T - 0.90251\log_{10} t + 0.00038T^2 - 0.00138T\log_{10} t + 0.00661PI * T$$

(Equation C1)

where

Sbit = bitumen stiffness (in 0.1MPa)

T = test temperature minus TR&B (°C)

T = loading time (sec), t >0.2

PI = penetration index.

The ranges for which the equation was derived were:

-2 < PI < +2

-100°C < T < 50°C

-10^{-2} < t < 10^{5} sec.

If:

$$10-30MPa < S_{bit} \leq 2000MPa \text{ and } t < 0.2\,\text{sec}$$

$$\log_{10} S_{bit} = -1.90072 - 0.11485T - 0.38423PI - 0.05643\log_{10} t - 0.00879T\log_{10} t - 0.05643PI\log_{10} t \\ - 0.02915(\log t)^2 - 0.51837\times10^{-3}T^2 + 0.00113PI^3\times T - 0.01403(PI\times T^3)\times10^{-5}$$

(Equation C2)

The ranges for which this equation was derived were:

-1.5 < PI < +2

-100°C < T < 50°C

-10^{-2} < t < 10^{5} sec.

The PI can be solved by using the following equation:

$$\frac{20-PI}{10+PI} = 50\frac{\log PenT_1 - \log PenT_2}{T_1 - T_2}$$

(Equation C3)

where

$PenTa$ = penetration at temperature Ta

T_1 = test temperature in degrees Celsius

T_2 = softening point in degrees Celsius.

These equations were applied to the results from the above-mentioned research studies. The comparative results are summarised in table C.2.

Table C.2 Comparative results

Sample reference	Temp (°C)	Freq (Hz)	Phase angle	Complex modulus (MPa)	Material type	Road condition	PI	TR&B	Pen	SBit
Ch150	25	9.65	10.0	217	Recovered binder	Failed	0.05	75	11	217
Ch150M	25	9.65	11.7	162	Recovered binder	Failed	0.5	75	11	162
ND32	25	9.65	45.0	12	Bitumen	Sound	-0.6	48	80	11
ND38	25	9.65	47.5	10	Bitumen	Sound	-0.5	48	80	10
SB	25	9.65	39.3	22	Bitumen	Sound	0.1	53	60	22
SBM	25	9.65	42.5	17	Bitumen	Sound	0.7	53	73	17
NB	25	9.65	21.6	74	Recovered binder	Moderate	0.8	70	19	72
TRL264	25	10	NA	100	Recovered binder	Failed	0.4	70	15	99
LTNZ291	-	-	NA	50–150	Recovered binder	Failed	-	-	-	-
Safaniya 180/200 Pen	0	9	NA	38	Bitumen	-	1.5	43	200	37

The above table represents measured stiffness values (complex modulus) of recovered binders and retained bitumen samples, and represents bitumen used in South Africa (reference Ch150, Ch150M, ND32, ND38, SB, SBM, and NB), bitumen used in Britain (TRL264), bitumen used in New Zealand (LTNZ 291), and Safaniya 180/200 bitumen.

The equations that were used to calculate the binder stiffness present a good fit to the stiffness values published in the various studies, and are therefore proposed instead of the Van der Poel nomographs to predict the terminal stiffness for non-modified bituminous binders in OGPA mixes.

Since the stiffness of the bitumen binder can now be calculated, a relationship between this stiffness value and that of an asphalt mix is required. For this purpose, it is proposed that the Shell nomographs be used in order to arrive at the ultimate stiffness, or stiffness at failure, for an OGPA mix. Inputs required for this nomograph include binder stiffness and the volumetric properties of the mix.

The binder properties of the samples retrieved from the failed sections in the above table (Ch150 and CH150M) were used to predict the ultimate stiffness at which the OGPA mix would show signs of cracking and ravelling. In order to compare the measured stiffness of the OGPA samples extracted from the road against the laboratory-prepared samples, the predicted failure stiffness at similar loading times were used as for the ITS test.

The equation proposed by Sabha (1995) for the calculation of bitumen stiffness allows for the variation of the temperature at which the binder stiffness is required (or the test temperature). This allows the calculation of the mix stiffness for specific binder properties, allowing for the loading time and variation in test temperature.