

ASPHALT FATIGUE PREFORMANCE CRITERIA FOR THE NATIONAL WAR MEMORIAL PARK

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Technical Expert Paper - Asphalt Fatigue Performance Criteria For the National War Memorial Park

ABSTRACT

The New Zealand Government is building the National War Memorial Park, to commemorate the 100th anniversary of Gallipoli landings during the First World War on Anzac Day – 25th April 2015. The Memorial Park Alliance is constructing the works on behalf of NZ Transport Agency and Ministry of Culture Heritage. The project has four distinct elements, the Underpass, the Park, the Inner City Bypass Improvements and the Basin Reserve Bridge. For the pavement works, there are number of constraints: realignment of State Highway 1 to allow for the park's construction; traffic delays due to greater volumes of earthworks; and relocating shallow services along the existing road corridor. A thinner structural asphalt pavement than usual was necessary due to the potential costs, delays and risks in relocating these ground services. As a result, a new fatigue performance criterion was developed for a high rut and fatigue resistance asphalt mixture, which was highly modified by incorporating a polymer. Performance criteria were derived for the polymer modified asphalt laboratory using controlled stress and strain flexural beam fatigue testing. These alternative criteria were the basis for validating a thinner pavement structure. The accepted design saved 270 mm of total depth, including 80 mm of structural asphalt. Polymer modification to the asphalt binder allowed these savings. Overall, the alternative results in significant cost savings, less construction time and public disruption for a project that will play a role in New Zealand's identity.

INTRODUCTION

A laboratory derived, asphalt performance criteria has been accepted for use in the pavement design of a New Zealand State Highway road for the first time. An alternative asphalt fatigue performance criterion was researched, tested and applied to the Memorial Park Alliance. This paper illustrates an example of how research can drive efficiency.

Background: Memorial Park Project

The National War Memorial Park was conceived in 2008, and with the 100th anniversary in mind a decision to build the Park was made in the autumn of 2012. The Memorial Park is going to be an important venue for national centenary celebrations. The opening of the park in April 2015 provides one of the key venues for New Zealand as a nation to commemorate the 100th anniversary of Gallipoli landings, Anzac Day. As well as improving the public space to celebrate significant remembrance days, the site will contribute to both New Zealand's sense of national identity, and enhance Wellington's landscape.

The Park is one of two elements of the project which covers the length of State Highway 1 between the existing Terrace and Victoria tunnels, these are:

- Underpass – the undergrounding of the State Highway(SH) 1 along its existing alignment which allows for the construction of the Park and integration of the local roads above.
- Inner City Bypass – geometric improvements to existing intersections and links which provide significant traffic benefits for state highway and local traffic

To build this park, the current Buckle Street, SH1, needs to be diverted, and will be done so by an underpass. Buckle Street underpass and the Memorial Park, along with the existing National War Memorial can be seen in Figure 1.

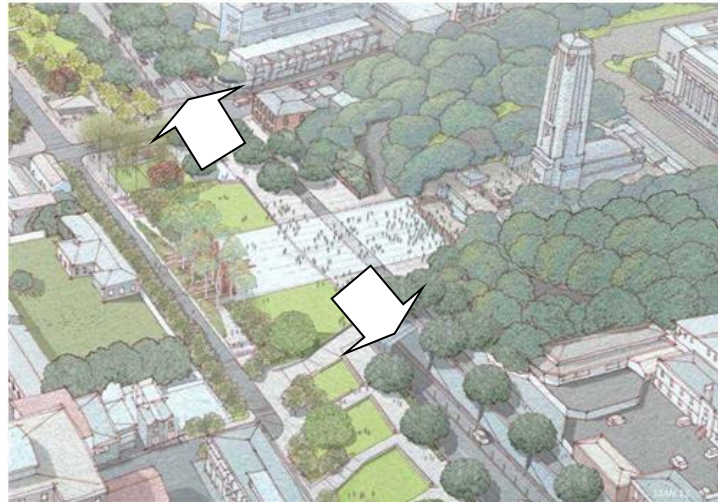


Figure 1: Artist Impression of the National War Memorial Park

The adjoining sections for this underpass have difficulties typical of urban infrastructure projects, i.e. relocating shallow services if they are within the pavement structure. Relocating these will significantly increase the cost of the project, on an already expensive build, and create significant delays to both the project and the travelling public.

Given this situation, a thinner pavement was desirable, so this was presented as an alternative pavement design. This thinner pavement will minimise the impacts of relocating ground services. Examples of some of the unexpected shallow ground services for the project are shown in Figure 2. Stabilising the sub-base to improve the support was considered impractical and uneconomic due to the limited areas/volumes. One concern was that a thinner pavement can increase risks (i.e. a perception that carries a reduction in pavement strength). The other is the thinner design is not standard practice in the current guidelines. An alternative criterion was therefore needed, which would then conform to these design principles. Therefore, a new asphalt fatigue criterion was derived from flexural beam fatigue tests instead of using an empirical equation developed by Shell researches. The use of an alternative strain criterion derived from beam fatigue tests has not been used before on a New Zealand State Highway but is an accepted method in the AUSTROADS guide provided the lab performance is correlated with the field. Correlation with field performance is proven from other international full scale pavement test trials where the actual fatigue life in the field is always greater than 4 times the fatigue life predicted from lab tests. In this design a shift factor was not applied to the beam fatigue test to ensure the resulting design is conservative.

Objective

The aim of this paper is thus to validate the structural integrity of this thinner alternative pavement design. This aims to reduce the risks, plus, if successful, could also help avoid cost and time to relocate shallow ground services.



Figure 2: Examples of some “Unexpected” Shallow Ground Services in Section A

PROPOSED PAVEMENT DESIGN OPTIONS

The following section presents the various pavement design options that were proposed. The advantages and disadvantage of the options are discussed below. These designs are:

- Option 1 – full depth structural asphalt (AC)
- Option 2 – structural AC on a stabilised sub-base
- Option 3 – Polymer modified structural AC

Table 1: Proposed Pavement Design Cross Sections

	Full Depth Structural AC	Structural AC on a Stabilised Sub-base	Polymer Modified Structural AC
	Option 1	Option 2	Option 3
Surface Layer	40 mm	40 mm	40 mm
Base Asphalt Layer	160 mm	135 mm	90 mm
Fatigue Layer	50 mm	-	50 mm
Asphalt Thickness	250 mm	175 mm	170 mm
Subbbase	600 mm	250 mm	400 mm
Subbgrade	CBR 3%	CBR 3%	CBR 3%
Total Pavement depth	850 mm	425 mm	580 mm

Note - the design traffic for the site is 29 million equivalent standard axles.

Option 1: Full Depth Structural Asphalt

This option uses traditional full-depth structural asphalt, and requires a total asphalt thickness of 250 mm. No stabilising of the sub-base is needed, thus hoeing into the existing soil is not required. The major disadvantage of this option is a significant amount of undercutting of existing soil could be required, with high excavation and disposal costs. This would impact on the ground services. In addition, there is a greater use of more natural resources with a higher carbon footprint.

Option 2: Structural Asphalt on a Stabilised Sub-base

Conventional asphalt over a stabilised sub-base is constructed for this option. This requires milling of an existing pavement down to the depth of the asphalt layer and then in-situ stabilising the existing pavement layers. Or if there is insufficient pavement material then this could be replaced by imported granular material to build the stabilised sub base layer. An advantage of a cement stabilised sub base is the ability to bridge any soft subgrade and the asphalt depth is reduced to 175 mm as per the AUSTROADS Pavement Design Guide to prevent reflective cracking.

This option is not preferred as it involves mixing existing (or imported) soil with cement to stabilise the pavement - adding constructability issues. If the services are located between a depth of 175 mm and 425 mm depth, services would need to be relocated; ultimately increasing lane closure time and adding greater programme pressures to have the project ready for Anzac Day 2015. Stabilisation is not a viable option on this small site where sections are constructed separately to allow for continued movement of traffic. Site constraints and phasing dictate very limited quantity of work at each visit; hence, stabilisation is considered impractical and cost ineffective option.

Option 3: Polymer Modified Structural Asphalt

Option 3 uses highly modified polymer asphalt. Compared with Option 1, Option 3 is thinner by 270 mm, resulting in significant savings. This avoids the need to dig into the shallow services. Thus less asphalt is needed and often would only require the milling of an existing pavement down to the depth of the asphalt layer, provided there is sufficient granular material underneath. To validate this as a suitable option, the structural integrity needs to be proven; hence, some testing was conducted to show the polymer modified asphalt has adequate strength and fatigue performance.

BACKGROUND INFORMATION ON ASPHALT FATIGUE

Current Structural Asphalt Design Philosophy

According to the shell fatigue performance criterion in AUSTROADS, the major concern with Option 3 is that the asphalt layer will fail by fatigue cracking (if conventional unmodified asphalt).. This is because compared with the others, Option 3 assumes an alternative asphalt fatigue relationship than the AUSTROADS Shell fatigue criterion. In the AUSTROADS Pavement Design: A Guide to the Structural Design of Pavements (2010) and its New Zealand Supplement to the AUSTROADS (2007) it stipulates for the design of structural asphalts, when there is no known fatigue relationships, this Shell criteria shall be used (AUSTROADS, 2010).

Shell Fatigue Performance Criterion

The Shell fatigue criterion, Equation 1, is a fatigue life model to predict field performance. It predicts the number of loads to failure for a certain tensile strain level, stiffness modulus and volume percentage of bitumen in the mix. The fatigue criterion was derived in the laboratory (AUSTROADS, 2012; Baburamani, 1999). No field shift factor has since been used in the equation to relate to in-service performance (Jameson, 1992). The laboratory relationship was based on controlled strain sinusoidal loading fatigue tests on 13 typical asphalt mixes from various countries (Shell International Petroleum Company Ltd., 1978; Van Dijk., 1975). Table 2 gives details on the mix types. Asphalt mixes vary from open, gap and dense graded with voids

from 1.7% to 33.2%. Compared with NZ structural asphalt, with target design air voids between 3-5% (New Zealand Transport Agency (NZTA), 2010), there is a great level of variation. Given this variation, questions are thus raised on the reliability to predict fatigue.

Equation 1

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

where:

N_f = allowable number of loading repetitions until fatigue cracking failure

V_b = percentage by volume of bitumen in the asphalt mix (%)

S_{mix} = asphalt stiffness (flexural) modulus (MPa)

$\mu\epsilon$ = tensile strain at the bottom of the asphalt layer (microstrain)

Table 2: Composition of Asphalt Mixes used in the Development of the Shell Life Prediction Model (Claessen et al., 1977).

Mix Type	Binder Grade	Binder Volume (%)	Air Voids (%)
Asphalt Concrete State of California	40/50	14.2	1.7
Dense Asphaltic Concrete	40/50	11.4	1.9
Gravel Bitumen French	40/50	9.3	9.3
Dense Bitumen Macadam	40/60	11	3.6
Rolled Asphalt Base Course Mix	40/60	14.1	2.2
Bitumen Sand Base Course	45/60	8.9	20.3
Gravel Sand Asphalt, Dutch	45/60	11	11
Rich Sand Sheet	45/60	19.3	7.8
Gravel; Sand Asphalt Dutch	50/60	13.3	6.6
Dense Bitumen Macadam	80/100	11	3.4
Lean Bitumen Macadam	80/100	4.9	33.2
Lean Sand Asphalt	80/100	10.5	8.4
Asphalt Base Course Mix German (Stuttgart)	80/100	9.3	2.6

Figure 3 shows the gradation of the various mixes. Gradation type and size is reported to have an effect on fatigue life. In addition, the geology can have an influence. Indeed, Peploe (2008) found that Greywacke had a longer laboratory fatigue life compared with Basalt for New Zealand's asphalts. For the development of the Shell fatigue performance criterion, notice the different aggregate gradation used compared with the dense graded New Zealand structural asphalt mix. In addition, all 13 mixes are penetration grade binders.

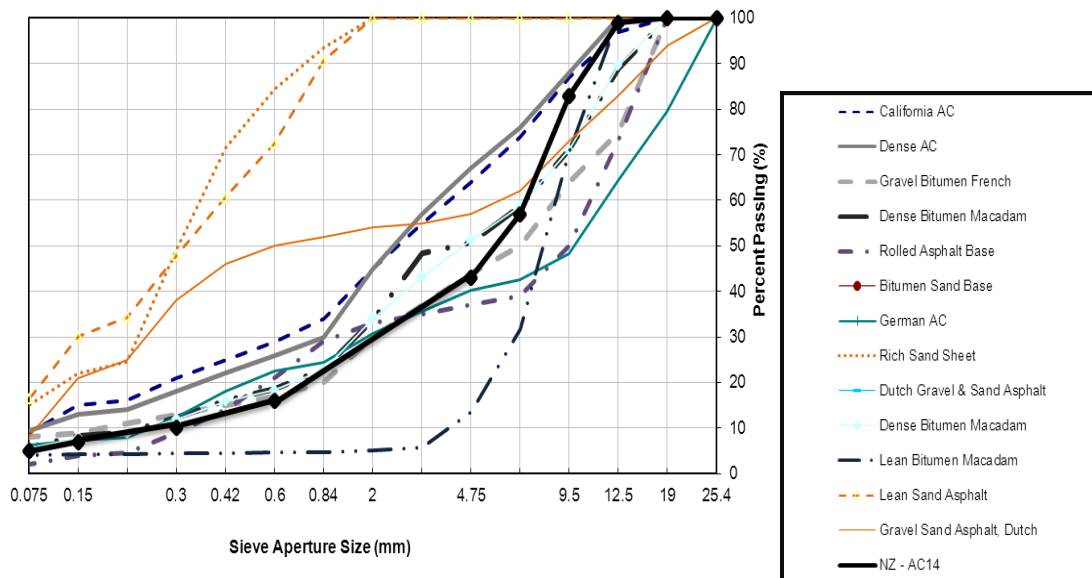


Figure 3: Aggregate Gradation Curves for the Various Asphalt Mixes used in the Development of the Shell FTF

In summary, the composition of the asphalt mixtures used in the development of the Shell model are different from New Zealand's structural asphalts. Considering the influence of the factors affecting fatigue of asphalt, a "general" equation is unlikely to be accurate of asphalt fatigue; furthermore, when polymer is added to enhance fatigue performance then the Shell fatigue performance criteria is bias towards such mixes. International studies have demonstrated superior asphalt fatigue performance with the addition of PMBs (Criterion et al, 2012; and Tim et al., 2012). Clearly this results in an unfair contest between mixes when judging their engineering properties and economic life.

Inherent Issues with Shell Fatigue Performance Criterion

Within the New Zealand and Australian roading industry there is uncertainty with regard to the validity of the Shell model for predicting the asphalt fatigue life of the country's structural asphalts. These industry beliefs are in line with the discrepancies of the variables affecting asphalt fatigue – as mentioned above. Furthermore, field evidence suggests that this model is overly conservative. Practitioners have stated that the Shell fatigue relationship appears to be overly-conservative (Pidwerbesky, 2010; Stubbs, 2010; Transit New Zealand, 2005; and Gribble and Patrick (2008). Even back to 1982 it was noted that "[a] strong case would be made for research effort to establish design charts and formulae for New Zealand conditions and materials" (Saunders, 1982). Today New Zealand's roading industry still continues to request characterisation of asphalt's modulus and fatigue behaviour (Gribble & Patrick, 2008). Although, industry would like lab characterisation data on their asphalt it is often not available and the modulus and fatigue behaviour is assumed.

Experience tends to indicate that the Shell FTF is inappropriate for a New Zealand context. Two thirds of the Wellington and Auckland motorway network was constructed with structural asphalt having been designed using the earlier guideline, the State Highway Pavement and Rehabilitation Design Manual, in which the thickness of asphalt is 30 per cent less than the Shell model. Yet they are performing well past their design lives with minimal structural maintenance required (Transit New Zealand, 2007).

Laboratory fatigue testing carried out at the University of Canterbury also demonstrated that the Shell model is overly conservative. Stubbs et al. (2010) showed that the Shell model underestimates the laboratory fatigue life of a typical New Zealand roading hot mix asphalt – AC14 60/70 by an average of 5.5 times (range 3.1–8.9). Their laboratory model when used as a performance criterion resulted in a potential cost saving of \$90,000 per lane kilometre. If a field calibrated model is used, even greater savings could be made. Saleh (2012) showed typical New Zealand asphalt, AC10 80/100 had an even greater laboratory fatigue life than predicted by

the Shell model. The AC10 80/100 endures a fatigue life on average by 9.3 times (range 5.9–14.6) greater than the prediction from the Shell fatigue performance criterion.

Loading Mode: Controlled Strain vs. Controlled Stress

In testing for fatigue, there are two types of loading modes: controlled strain and controlled stress. Controlled strain testing is defined by maintaining a constant deformation during cyclic loading throughout the test; hence, controlled strain is also known as controlled displacement testing. In this test, the load is decreasing over time to keep a constant deformation. In contrast to controlled strain testing, controlled stress testing is achieved by maintaining a constant loading stress throughout the test. It is therefore referred to as controlled force testing. In this case, the deformation increases during the test as a result of cracking; hence, failure is defined when the specimen fractures. It could be argued that controlled stress testing is analogous to what actually happens on the road since the stress (traffic loading) does not reduce as the pavement deflects more.

Within the literature, controlled strain testing is said to be more applicable for relatively “thin” asphalt pavements (less than 100 mm thick) (Baburamani, 1999), on the other hand, controlled stress testing is more relevant for “thick” pavements. Huang (2004), states that controlled strain is more suitable to thicknesses less than 2 inches (51 mm); and controlled stress is more suited for thicknesses greater than 6 inches (152 mm). Nonetheless, it is generally agreed that controlled strain testing is best for thin pavements because, the level of strain at the bottom of the asphalt layer is more sensitive to the modulus and thicknesses of the underlying pavement layers. In addition, Pellinen et al. (2004) notes softer and more flexible mixes perform best for thin pavements as they provide superior performance. As mentioned before, the Shell FTF was developed from controlled strain testing, and this was one of the reasons why it was adopted in the AUSTROADS guidelines.

For the proposed testing methodology, both controlled strain and stress modes were tested to gain a better understanding, and provide greater confidence in the design.

PROPOSED ASPHALT AND TEST METHOD

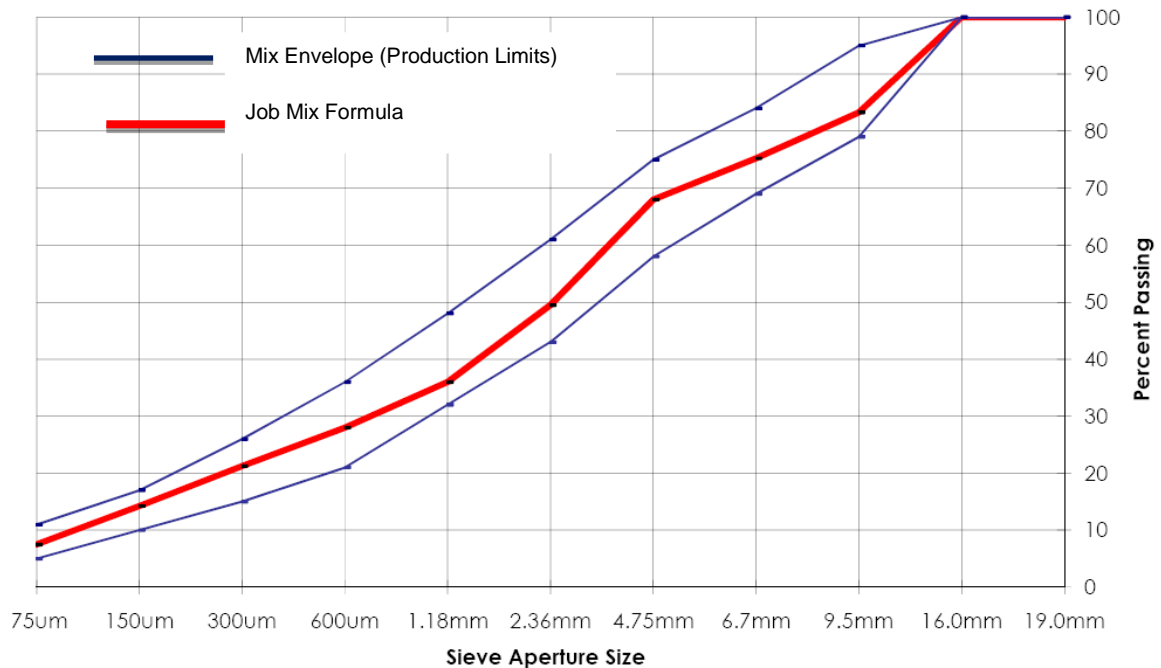
Materials, Mix Design and Volumetric Properties

It was decided to use Downer’s NZTA Mix15 60/70. Historically, in the Wellington region, this conventional penetration grade binder asphalt is the best mix in fatigue and rutting (Vercoe, 2012) for Downer. A polymer, RS 3 High Strength, was added to the mix to further improve the performance properties of the mix. Therefore a new asphalt mix design was carried out with the RS 3 High Strength polymer prior to the performance testing. This mix design meets the NZ Transport Agency (NZTA) M/10 (2005) Specifications. The mix was labelled: NZTA 15 RS 3 HS KP 13. This notation denotes the of NZ Transport Agency asphalt mix with a maximum aggregate size of 15 as the smallest sieve size which allows 100 percent of the material to pass. RS 3 HS is the highly modified binder which stands for Road Science 3 High Strength. The binder is supplied by Road Science in Mount Maunganui. KP is Kiwi Point and is the source of the greywacke aggregate. 13 is the 2013 year.

The volumetric properties of the mix design are reported in Table 3 and the combined aggregate gradation of the dense graded Mix 15 is plotted on Figure 4.

Table 3: Mix Design Variables for the NZTA 15 RS 3 HS KP 13

Mix Variable	NZTA 15 RS 3 HS KP 13
Binder Content (by mass)	5.7%
Asphalt mix bulk specific gravity	2.382
Asphalt mix theoretical maximum specific gravity	2.455
Binder Content (by volume)	13.3%
Air voids in total mix (%)	3.3%
Voids in mineral aggregate (%)	14.2%

**Figure 4: Combined Aggregate Gradation for the NZTA 15 Dense Graded Mix**

Fatigue Testing

Fatigue testing was carried out by a flexural test, following the procedure of AGPT/233. The IPC Global beam fatigue apparatus was used as shown in Figure 5. Testing was carried out in a controlled temperature cabinet at 20°C, Wellington's weighted mean annual pavement temperature (Transit, 2007). It is acknowledged that temperature will have an effect on fatigue.

Two types of fatigue testing modes were performed: Controlled strain and controlled stress. For each testing mode, five different loading conditions (stress in the case of controlled stress; or strain in the case of controlled strain) were chosen. A haversine loading pattern was used for the testing and then these strains divided by two to convert them into a sinusoidal strain. This ensures that when fitting a power law equation, the five points mathematically characterise the shape of this curve. In addition, a minimum of three replicates were carried out to ensure reliability within the results. Huang (2004) states normally 8-12 specimens are required to establish the fatigue relationship for a given temperature. In this case, a minimum of 15 beam fatigue tests were carried out to establish greater statistical confidence. Fatigue failure, for both controlled stress and strain testing, was defined at a 50% reduction in the initial stiffness.

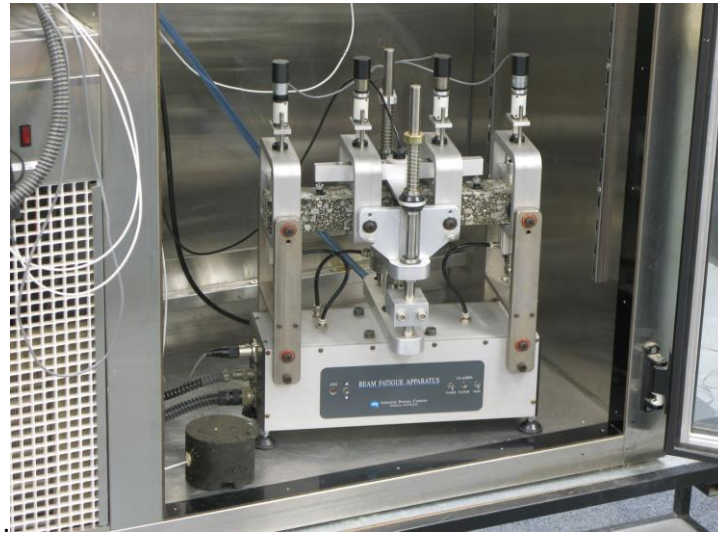


Figure 5: *Beam Fatigue Apparatus – within Temperature Cabinet*

FATIGUE RESULTS

The following fatigue relationships were determined. These are fatigue performance criteria and are expressed as equations 4 and 5. Equation 4 is developed from controlled strain testing (haversine tested converted into a sinusoidal strain), and Equation 5 is based on controlled stress testing. The models predicts the number of loading cycles to fatigue failure based on either the maximum critical horizontal strain at the bottom of the asphalt layer (strain based) or the maximum critical horizontal stress at the bottom of the asphalt layer (stress based). Note the fatigue curves can be seen in Figure 7 and Figure 8

Equation 4	$N = \left(\frac{1549 .6}{\mu\epsilon} \right)^8$	Strain based
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Equation 5	$N = \left(\frac{4670 .5}{\sigma} \right)^{10.87}$	Stress based
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where:

N = Number of loading repetitions until fatigue cracking failure

k and b = Material constants empirically derived.

$\mu\epsilon$ = Tensile strain amplitude (microstrain)

σ = Tensile stress amplitude (kPa)

PAVEMENT DESIGN VALIDATION

A CIRCLY analysis was carried out based on the proposed pavement cross-section to verify the design. A summary is presented in Table 4 for various modulus values.

Modulus Choice

A cumulative distribution plot is presented for the measured initial flexural stiffness (or modulus). A total of 31 measurements were taken.

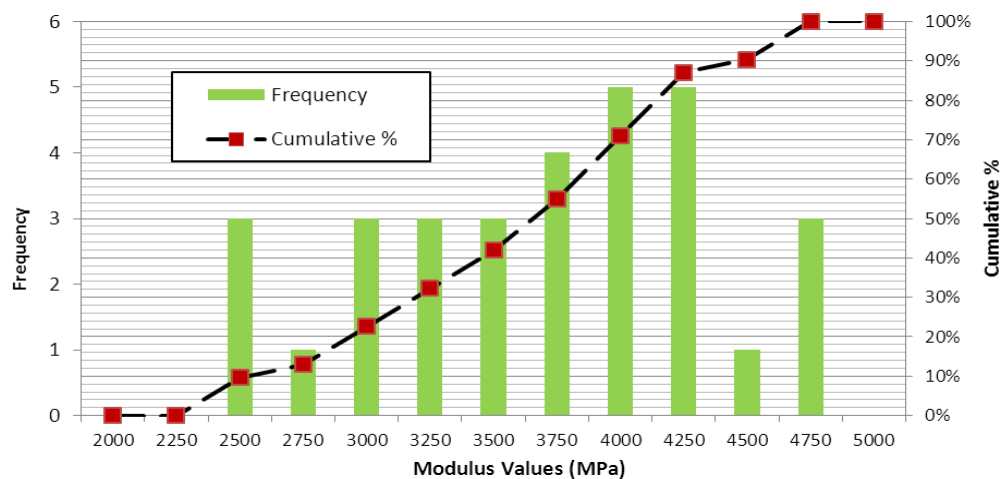


Figure 6: Initial Flexural Stiffness Moduli Results – Cumulative Distribution

Given the low speed zone of the pavement sections, being at an intersection, these laboratory measurements have been adjusted to give a design modulus of 2500 MPa. For design, the laboratory modulus is not corrected for temperature. This is because the fatigue testing was carried out at Wellington's Weighted Mean Annual Pavement Temperature (WMAPT) and is the same as the fatigue testing temperature of 20°C.

Pavement Analysis

From the design modulus values, the pavement configuration and fatigue models, a CIRCLY analysis has been carried out. Table 4 shows the key outputs. Benkelman beam testing prior to construction back-calculated a subgrade modulus of 40MPa – which is greater than the proposed design option in **Error! Reference source not found.**, which originally assumed a subgrade modulus of 30%.

Table 4: Pavement Analysis – Stress vs Strain Based

Asphalt Modulus (MPa)	Critical Tensile Strain ($\mu\epsilon$)	Critical Tensile Stress (kPa)	Predicted Fatigue Life Strain Based (ESA) ¹	Predicted Fatigue Life Stress Based (ESA) ⁴	Design Traffic (ESA)
2500	206	706	10,090,461	671,369,227	29,000,000

These theoretical design values, that are calculated in CIRCLY, shown in Table 4, are plotted against the developed fatigue curves for comparison (modulus value of 2500MPa). This is illustrated in Figure 7 and Figure 8. The analysis shows that using a stress based design criterion, the design traffic is within the fatigue life. In the case for the strain based (sinusoidal) design, the design traffic exceeds the predicted fatigue life by 2.9 times. However, considering the notable differences between laboratory fatigue life and the field (explained in the next section), an allowance was made for this project. Given that this pavement is considered “thick” i.e. greater than 150 mm (Huang, 2004 and Pellinen et al., 2004), the authors believe stress based testing is considered appropriate for this thick pavement. As a result, the design traffic is less than the stress based predicted fatigue life.

¹ Equivalent Standard Axles (ESA)

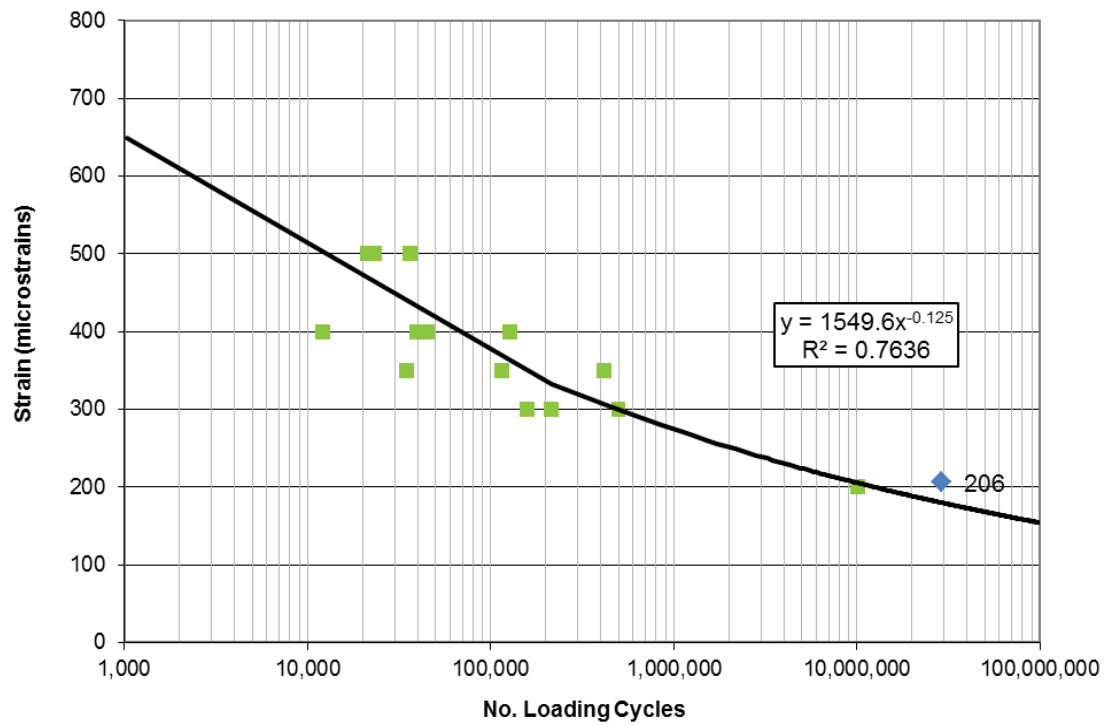


Figure 7: Asphalt Performance Data - Strain Control with CIRCLY Strains

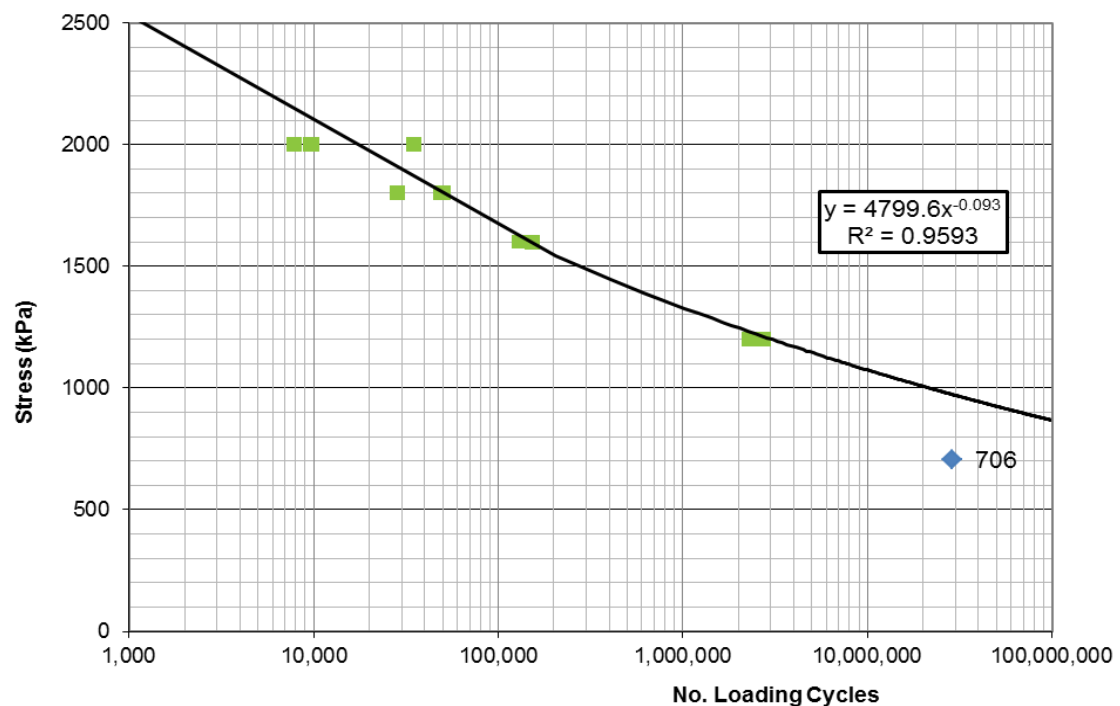


Figure 8: Asphalt Performance Data - Stress Control with CIRCLY Stresses

Fatigue: Laboratory vs. Field

Notably, the laboratory fatigue life always reported by researchers to be very conservative compared with the field. Differences exist between measured fatigue in the laboratory and the field. Baburamani (1999) stated that these discrepancies between the field and the laboratory are due to differences in the loading set-ups; establishing realistic loading times and rest periods between traffic loading; the surrounding temperature during the pavement service life; and the level of compaction of the asphalt. In addition, in the laboratory, the same *level* of load is applied in the same position – in every load cycle; the surrounding temperature is constant;

the loading rate is constant; and the asphalt beam is simply supported. Conversely, in the field, traffic loads are variable and dependent on the axle configuration. These loads “wander”, and thus are not always loaded in the same line; the air temperature is continuously changing; the traffic loading rate is dependent on the vehicle speed, which is varied continually; the asphalt layer is fully supported from the underlying layers. In addition, as the asphalt ages, it becomes stiffer. Together, these differences make laboratory fatigue tests more stringent and severe than field conditions. Indeed AUSTROADS (2012) state “the actual number of load applications producing cracking in the field may be many times the number obtained by laboratory testing.” Because laboratory conditions are known to be more conservative than field conditions, a field shift factor (FSF) is commonly applied to laboratory fatigue models to estimate field fatigue, as given by Equation 6.

Equation 6

$$N_{f(Field)} = FSF \times N_{f(Lab)}$$

Where:

$N_{f(Field)}$ = Number of loading repetitions until fatigue cracking failure in the field

FSF = Field shift factor

$N_{f(Lab)}$ = Number of loading repetitions until fatigue cracking failure in the laboratory

The shift factor depends on the level of cracking that is to be tolerated by the given transport agency (i.e. 10% cracking or 50% cracking). The literature found shift factors can vary from 10 to 20 (Baburamani, 1999) and 40 to 100 (Adhikari, Shen, & You, 2009). For polymer modified sections field shift factor of 4.2 have been used (National Cooperative Highway Research, 2010).

It is noted, that for this project, a field shift factor was applied in the strain based design. A value of 2.9 was used. This was on the basis of the comments above and was a risk that the Memorial Park Alliance was prepared to accept.

CONCLUSIONS

Overall, the asphalt fatigue testing has demonstrated that an alternative pavement design for the Memorial Park Alliance has structural integrity, and will meet the design life of 29,000,000 equivalent standard axles (heavy vehicles). This will mean that the pavement construction can avoid the shallow services, and assist in reducing the paving time. Thus assists in helping the Memorial Park project be ready for Anzac Day 2015.

The result has enabled a paradigm shift in the design of structural asphalt pavements in New Zealand to use criteria derived from beam fatigue testing rather than an empirical equation derived from Shell. AUSTROADS framework for mechanistic pavement design and the use of pavement design software like CIRCLY gives the ability for designers to use actual performance criteria on asphalt mixes derived from lab tests. Although, this design approach was relatively straight forward when using CIRCLY more guidance by AUSTROADS is needed such that this design process is more common.

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AUTHOR BIOGRAPHIES

Anthony Stubbs works for Road Science (a division of Downer) as a pavement engineer. He studied civil engineering at the University of Canterbury and went on to graduate with a master's degree. His master's thesis topic was on the fatigue performance of asphalts, which challenged the status quo on pavement design of structural asphalts. Anthony has also worked for Blacktop as their Quality and Technical Manager.

Dr. Greg Arnold is the Technical Pavement Manager for Road Science a division of Downer. Greg is well recognised as one of NZ's leading pavement design and research specialists. For the past 5 years he has been running his own pavement design, research and material testing business "Pavespec", carrying out a wide variety of work, TNZ spec pavement materials saving over \$1m. Prior to this Greg was the NZTA Engineering Policy Manager responsible for research and specifications.

John Vercoe is the Technical Manager for Road Science with over 20 years experience in the technical aspects of bitumen based roading materials and techniques, including management of manufacturing operations. He graduate from Auckland University with a Bachelor of Science (Chemistry) and worked throughout the world with Exxon. He was chairman of the AUSTROADS Bitumen Surfacing Research Group for PMB's and represented both Transit NZ & Roading NZ on the AUSTROADS committee related to chipsealing and bitumen.

David Gedney has worked on the design and delivery of major transport infrastructure projects for the last 20 years and is Civil/Roading Team Leader with the Memorial Park Alliance. With the detailed design of the Underpass and ICB substantially completed David is currently responsible for the delivery of the Park design together with construction liaison up to its completion and opening in March 2015.

John Donbavand is currently the National Pavements Manager for NZTA. In this role his principal responsibility is to maintain and develop the standards, specifications and guidelines for State highway pavements. John also takes a leading role in training and pavement related research.

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