WARRIP

WESTERN AUSTRALIAN ROAD RESEARCH AND INNOVATION PROGRAM

Asphalt Fatigue at Elevated Temperatures

2017-005

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for Main Roads Western Australia

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SUMMARY

This report presents the findings of work commissioned by Main Roads Western Australia (Main Roads) and conducted by the Australian Road Research Board (ARRB) under the Western Australia Road Research and Innovation Program (WARRIP).

The purpose of this project is to characterise the stiffness and fatigue performance of typical Western Australian (WA) asphalt mixes and evaluate opportunities for improving current asphalt mix and structural design practices in the states north, where WMAPT's are greater than 35 °C.

The work conducted to date includes:

- a review of the current asphalt thickness design procedures, including asphalt stiffness characterisation
- a review of the findings from a similar project conducted under the National Asset Centre of Excellence (NACoE) research program
- stiffness testing of four WA dense graded hot mix asphalt mixes, namely:
  - 20 mm intermediate mix with C320 binder;
  - 14 mm intermediate mix with C320 binder;
  - 20 mm intermediate mix with A35P grade polymer modified binder; and
  - 14 mm intermediate mix with A35P grade polymer modified binder.

    The mixes with polymer modified binder have the same volumetric and grading design as the mixes with Class 320 binder except for the binder type. These mixes were selected for testing, since at the time of selection, these were the mixes intended for use in full depth asphalt pavements in areas of high WMAPT (northern WA).
- fatigue testing for the same four mixes cited above
- development of master curves in accordance with Austroads test method AGPT/T274 (Austroads 2016b)
- development of mix specific asphalt fatigue performance models. The mixes were tested for flexural modulus and fatigue at high temperatures. The intention of the testing was to compare the current pavement thickness design methodology using presumptive modulus values (derived from indirect tensile tests) and the Shell fatigue equation with what would be obtained using mix specific flexural modulus and fatigue test results.

The results of the testing undertaken indicate that:

- for the 20 mm intermediate mix with Class 320 binder: the use of flexural modulus master curves resulted in lower design modulus at elevated temperatures compared to the values currently assigned in Part 2 of the Austroads Guide to Pavement Technology (AGPT) and Engineering Road Note 9 (ERN9).

- for the 14 mm intermediate mix with Class 320 binder: the use of flexural modulus master curves resulted in higher design modulus at 30 °C compared to AGPT Part 2 and ERN9 and approximately the same modulus values at 40 °C.
- The use of A35P polymer modified binder did not greatly influence the stiffness of the tested mixes but it increased the fatigue life of those mixes.
- the Shell International Petroleum (1978) asphalt fatigue model predicts longer fatigue lives than the laboratory performance of the asphalt mixes tested, except for the 20 mm intermediate mix with Class A35P binder when tested at 30°C. This particular mix showed a fatigue life result that was similar to the Shell equation prediction.

The discrepancy between the presumptive modulus derived from ITT and the flexural modulus results obtained in this study may be due to a number of possible reasons, such as fundamental differences between indirect tensile modulus and flexural modulus and the fact that the presumptive values represent a range of mixes whereas the results presented in this report represent mix specific results.

Future work recommended include:

- exploring an interim design approach for the design of asphalt layers, informed by available literature and data, that accounts for the reduced fatigue damage expected to occur at elevated pavement temperatures
- exploring a methodology for laboratory testing of asphalt fatigue with rest periods and conducting testing with rest periods for the same mixes
- recommending a program aimed at improving laboratory-to-field shift functions.

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1 INTRODUCTION

This report presents the findings of work commissioned by Main Roads Western Australia (Main Roads) and conducted by the Australian Road Research Board (ARRB) under the Western Australia Road Research and Innovation Program (WARRIP).

The aim of this project was to characterise the flexural modulus and fatigue behaviour of two typical Western Australian dense graded asphalt mixes, using the protocols developed under the National Asset Centre of Excellence (NACoE) research program, in order to determine whether improvements can be made to current asphalt mix design and structural design practices.

This project builds upon the approach and findings of a similar project undertaken in Queensland under the NACoE research program.

The work conducted to-date includes:

- a review of the current asphalt fatigue model
- a review of the findings from the NACoE project
- asphalt stiffness testing of four typical Western Australian mixes
- asphalt fatigue testing of three typical Western Australian mixes
- development of stiffness master curves for the mixes tested in accordance with Austroads test method AGPT/T274 *Characterisation of Flexural Stiffness and Fatigue Performance of Bituminous Mixes* (Austroads 2016b)
- development of asphalt fatigue performance models for each of the mixes tested.

1.1 Background

The majority of the WA road network comprise of unbound granular pavements with either a sprayed seal or a thin asphalt wearing course (< 60 mm). However, recent increases in traffic loading have led to an increase in the use of Full Depth Asphalt (FDA) pavements.

The design thickness of asphalt pavements in WA is determined using the Austroads Guide to Pavement Technology (AGPT) Part 2: *Pavement Structural Design* (2012; 2017) and the Main Roads Engineering Road Note 9 (ERN9) *Procedure for the Design of Road Pavements* (2013).

ERN9 states that the asphalt modulus selected for thickness design must not exceed the lesser of the typical Australian dense graded asphalt modulus values presented in Table 6.13 of AGPT Part 2, and the modulus value obtained from indirect tensile tests (ITT) on the design mix. In most cases, the value used in the design is the typical modulus value (i.e. presumptive value) from AGPT Part 2, incorporating corrections for in-service air voids, traffic speed and pavement temperature in accordance with the methodology described in AGPT Part 2.

Asphalt is a viscoelastic material and the stiffness is dependent on both the rate of loading (speed of traffic) and temperature. At higher temperatures and lower traffic speeds, asphalt exhibits lower modulus values. In the joint AGPT Part 2 and ERN9 mechanistic design approach, asphalt fatigue life is directly related to the asphalt modulus. In thick (generally > 150 mm) asphalt pavements, lower modulus values result in greater deflections and higher strains at the bottom of the asphalt layer, thus, leading to increased fatigue damage and a reduction in design life according to AGPT Part 2. As a result, increased asphalt pavement layer design thicknesses are required for thick (generally > 150 mm) asphalt pavements where the pavement temperatures are high and/or traffic speeds are low. However, observations of asphalt fatigue damage in service indicate that most of the damage accumulates at low temperatures (Mateos et al. 2012, Pellinen et al. 2004, Stuart et al., 2002).

Weighted mean annual pavement temperatures (WMAPTs) in Western Australia vary from 24 °C in Albany, to 42 °C in Kununurra. The hot climate in the northern areas of the state, with WMAPTs in excess of 30 °C, therefore, leads to very thick FDA pavement design outcomes using the current design procedure.

1.2 Scope and Objectives

The objective of this project is to improve asphalt modulus and fatigue characterisation for WA climate and traffic loading conditions. This is achieved by following the methodology proposed in the new Austroads test method AGPT/T274 (2016b) and recommended in Queensland's Department of Transport and Main Roads Technical Note 167 (TMR 2017). The flexural stiffness and fatigue performance of four asphalt mixes were evaluated according to AGPT/T274 (2016).

The aim of this study is to improve the cost-effectiveness of FDA pavement design outcomes in WA.

1.3 Structure of the Report

Section 2 of the report presents an overview of the current pavement design framework and describes the current fatigue model for asphalt materials. Section 3 presents the proposed methodology to improve the modulus characterisation of asphalt mixes and fatigue models based on recent Austroads and NACoE projects. Section 4 summarises the laboratory testing conducted as part of the study on four typical WA mixes and presents fatigue performance models based on the laboratory tests conducted. Section 5 assesses how the use of the mix specific fatigue results obtained would affect the total required asphalt thickness of a typical FDA pavement in WA. Finally, conclusions are presented in Section 6.

2 ASPHALT PAVEMENT DESIGN FRAMEWORK

2.1 Shell Pavement Design Manual

The in-service asphalt fatigue model presented in AGPT Part 2 was derived from the laboratory fatigue equation recommended in the Shell Pavement Design Manual (SPDM) (Shell 1978). The SPDM laboratory model was originally developed using mean laboratory fatigue performance measurements for twelve asphalt mixes considered typical for different countries, including France, Netherlands, USA, England and Germany (Van Dijk & Visser 1977). The Shell model is presented in Equation 1:

$$\varepsilon_{fat} = (0.856 \cdot V_b + 1.08) S_{mix}^{-0.36} \cdot N_{fat}^{-0.2} \quad (1)$$

where

ε_{fat}	=	strain at the bottom of the asphalt specimen in microstrain
V_b	=	percentage of binder by volume
S_{mix}	=	mix stiffness (N/m^2)
N_{fat}	=	number of load repetitions to failure

In the SPDM, correction factors (shift factors) were introduced to correlate the laboratory model with field performance. The shift factor accounts for the effect of variable load frequency, asphalt healing, load wander and temperature variations in the pavement.

According to Gerritsen and Koole (1987), the shift factor for dense graded mixes can vary by a factor of 10 to 25 (i.e. the number of load repetitions to failure in the field is 10 to 25 times greater than in the laboratory) when taking into consideration all applicable factors. The higher shift factor values apply to mixes with moderate bitumen and air void content at higher temperatures, while the lower shift factor values apply to high air void content mixes at low temperatures (Gerritsen & Koole 1987).

2.2 Pavement Design according to AGPT Part 2 and ERN9

The AGPT Part 2 fatigue model is a rearrangement of the SPDM model (Equation 1), considering (Austroads 2012, pp. 88):

- a shift factor relating mean laboratory fatigue life (SPDM Equation 1) to a mean in-service life, taking account of the differences between the laboratory test conditions and the conditions applying to the in-service pavement
- a reliability factor relating mean in-service fatigue life to the in-service life predicted at a desired project reliability, taking into account factors such as construction variability, environment and traffic loading.

These two factors are combined into a single reliability factor (RF) in AGPT Part 2 2012. It is important to note that in the 2017 update to AGPT Part 2 (Austroads 2017), this approach was amended to separate the RF and the shift factor (SF), although the principal remains the same. The latest Austroads fatigue life equation is presented in Equation 2:

$$N = \frac{SF}{RF}\left[\frac{6918\ (0.856\ V_b + 1.08)}{E^{0.36}\mu\varepsilon}\right]^5 \quad 2$$

where

N	=	allowable number of repetitions of load
V_b	=	percentage of binder by volume
E	=	flexural modulus of the asphalt (MPa)
$\mu\varepsilon$	=	tensile strain in microstrain
SF	=	shift factor (presumptive value = 6)
RF	=	reliability factor

Table 2.1 presents the suggested RF values in AGPT Part 2 for asphalt pavements at different levels of desired project reliability. ERN9 requires a project reliability level of not less than 95% (i.e. RF equals to 6.0).

Table 2.1: Suggested reliability factors (RF) for asphalt fatigue in AGPT Part 2

Desired project reliability					
50%	80%	85%	90%	95%	97.5%
1.0	2.4	3.0	3.9	6.0	9.0

Source: Austroads (2017).

In practice, adoption of a RF of 6.0 suggests that the fatigue performance observed in the laboratory will be replicated in-service for 5% of the cases. Although this approach may seem conservative, there is currently insufficient data available to support the use of a higher RF in WA.

It should be noted that the testing carried out in the development of the SPDM model was performed using a flexural beam configuration (i.e. two- and three-point bending) and was based on testing of asphalt specimens with non-modified conventional binders.

Furthermore, the presumptive design modulus values in ERN9 and AGPT Part 2 are based on ITT tests, adjusted to an equivalent in-service flexural modulus value by means of the loading rate and adjustment factor, given by Equation 3. Based on data from several studies, it was estimated that ITT modulus at 40 ms rise time was similar to a beam flexural modulus at a frequency of 14.8 Hz. Equation 3 was obtained through regression analysis of calculated loading frequencies for a range of vehicle speeds and the ratios of the moduli to the values at 14.8 Hz (Jameson 2013).

$$\frac{Modulus\ at\ speed\ V}{Modulus\ at\ test\ loading\ rate} = 0.19V^{0.365} \quad 3$$

3 ASPHALT MODULUS AND FATIGUE PERFORMANCE OF TYPICAL WA MIXES

Austroads has recently developed an updated testing procedure to characterise asphalt flexural modulus and fatigue performance at different loading frequencies and temperatures (Austroads 2016b). This test method was developed to better simulate the test conditions that were used in the development of the original SPDM model (Shell 1978).

3.1 Modulus Characterisation

The recommended procedure for determining the modulus of asphalt consists of a four-point bending test under a sinusoidal constant strain loading on prismatic specimens. Testing is carried out at different temperatures and frequencies. The measured flexural modulus for each combination of temperature and load frequency is used to develop a flexural modulus master curve.

The master curve is constructed by shifting the mean modulus test results obtained at the different load frequencies for each test temperature to form a continuous function at a reference temperature (T_{ref}). Details on the derivation of the master curve are provided in AGPT/T274 (Austroads 2016b). The function used to construct the master curve, in accordance with AGPT/T274, is presented in Equations 3, 4 and 5 and illustrated in Figure 3.1:

$$log_{10}|E^*| = \delta + \frac{\alpha}{1 + e^{\beta + \gamma \log_{10} f_r}} \tag{4}$$

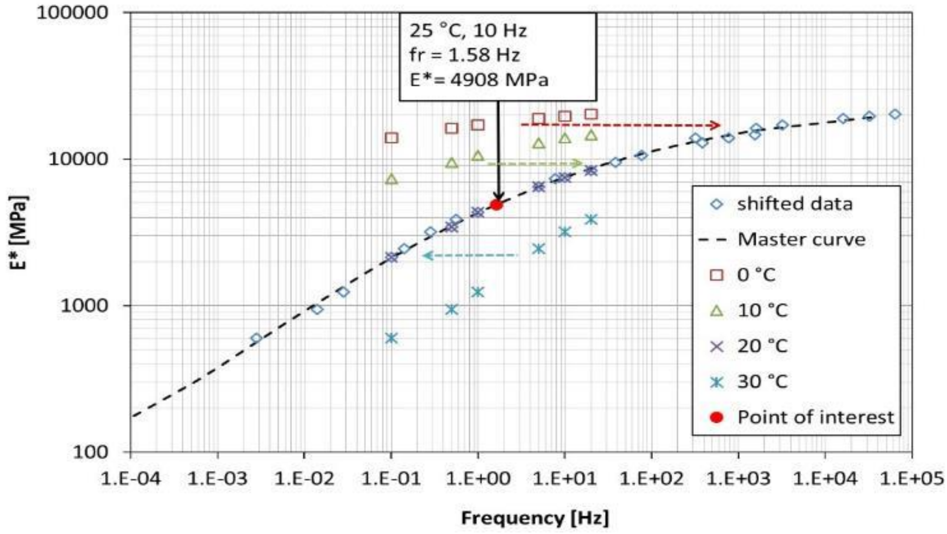
$$f_r = a_T * f \tag{5}$$

$$\log_{10}(a_T) = a(T - T_{ref})^2 + b(T - T_{ref}) \tag{6}$$

where

E^*	=	flexural modulus (MPa)
$\delta, \alpha, \beta, \gamma$	=	fitting parameters
f	=	frequency (Hz)
f_r	=	reduced frequency (Hz)
a_T	=	shift factor as a function of temperature (°C)
T	=	temperature (°C)
T_{ref}	=	reference temperature (°C)
a, b	=	fitting parameters

Figure 3.1: Flexural temperature/frequency sweep data and master curve (AGPT/T274)



Source: Austroads (2016b).

3.2 Fatigue Characterisation

The fatigue testing of each asphalt specimen was conducted using the four-point bending test in accordance with AGPT/T274 to determine the number of cycles required to achieve a 50% modulus reduction at different temperatures and strain levels.

Two different models were investigated to fit the fatigue data. The first model is based on the laboratory model used in the SPDM (1978) and is presented in Equation 6. This equation contains three regression coefficients, which is referred to in this report as the 3-Parameter Model and is retained in the 2004 US Mechanistic Empirical Design Pavement Guide (NCHRP 2004):

$$N_{f(50)} = a_1 \times E^{a_2} \times \varepsilon^{a_3} \quad (7)$$

where

$N_{f(50)}$ = number of cycles to a 50% modulus reduction

a_1, a_2, a_3 = regression coefficients

E = modulus of asphalt (MPa)

ε = strain in µm/m (microstrain)

The second model, as proposed by CROW (2010), is presented in Equation 7 and contains five regression coefficients. This is referred to in this report as the 5-Parameter Model.

$$\ln\left(N_{f(50)}\right) = c_1 \cdot ln^3(E) + c_2 \cdot ln^2(E) + c_3 \cdot \ln(E) + c_4 + c_5 \cdot \ln \varepsilon \quad (8)$$

where

$N_{f(50)}$ = number of load cycles to a 50% reduction in modulus

E = modulus of the asphalt (MPa)

ε = strain in μm/m (microstrain)

c_1, to c_5 = regression coefficients

Regression coefficients were calculated using Microsoft Excel's solver function by maximising the coefficient of determination (R^2) for the regression.

It is important to note that the proposed methodology does not consider the effect of healing in asphalt. Studies by several researchers, such as Bazin and Saunier (1967) and Van Dijk and Visser (1977), have identified that rest periods between loading allow the recovery of tensile strength by the asphalt, thus extending the fatigue life of the pavement.

Furthermore, the current work does not include an investigation of laboratory-to-field shift factors. It is proposed that future research is carried out to establish a field testing and monitoring program aimed at investigating laboratory-to-field shift functions relevant to WA conditions.

4 LABORATORY PROCEDURES

Asphalt beams were prepared for flexural stiffness and fatigue-testing using supplied aggregate and binders from WA. Asphalt beams were prepared and tested at the ARRB laboratory, located in Vermont South, Victoria and in Port Melbourne, Victoria, after the laboratory's relocation in 2018.

4.1 Aggregate

The 20 mm, 14 mm, 7 mm, 5 mm, 2 mm granite aggregates and coarse quartz sand utilized for this project, were respectively sourced from the Hanson Red Hill quarry and Gingin quartz sand quarry.

Approximately 7 tonnes of aggregates were delivered to the ARRB laboratory in 220 L drums (see Figure 4.1). The aggregates were sub-sampled using an automated rotary splitter (see Figure 4.2). Splitting the aggregates through a rotary splitter ensures the homogeneity of the sub-sampled aggregate (Austroads 2014). Aggregates were stored sealed plastic buckets post splitting. The aggregates were dried in the oven at 110 °C and stored in sealed plastic bags so that the materials could be used in the asphalt mixes.

Figure 4.1: Aggregate storage upon arrival at ARRB and after splitting

Figure 4.2: Example of splitting process

Source: Austroads (2014).

A summary of the various aggregate proportions used for the asphalt mix designs are presented in Table 4.1.

Table 4.1: Aggregate portions for the 20 mm and the 14 mm mix designs

Aggregate type	Aggregate source	Mix 1 & 3 (20mm) aggregate proportions (%)	Mix 2 & 4 (14mm) Aggregate proportions (%)
20 mm granite	Hanson, Red Hill Quarry	14.0	–
14 mm granite	Hanson, Red Hill Quarry	14.0	17.0
10 mm granite	Hanson, Red Hill Quarry	16.0	17.0
7 mm granite	Hanson, Red Hill Quarry	–	7.0
5 mm granite	Hanson, Red Hill Quarry	24.5	26.5
2 mm granite	Hanson, Red Hill Quarry	22.0	23.0
Coarse quartz sand	Gingin quartz sand	8.0	8.0
Filler (hydrated lime)	SIBELCO	1.5	1.5

4.2 Bitumen

Main Roads nominated a C320 binder as a baseline and an A35P grade polymer modified binder (PMB) binder for this study. SAMI WA supplied and shipped the binders to the ARRB laboratory (Vermont South) in 20 L containers (see Figure 4.3a). The 20 L bitumen containers were split according to AS/NZS 2341.21:2015 *Methods of Testing Bitumen and Related Roadmaking Products – Method 21: Sample Preparation* and the sub-samples were stored at room temperature in 2 L containers (see Figure 4.3b). The bitumen sub-samples (2 L containers) were later heated to the recommended asphalt mixing temperatures for the purpose of asphalt production. The binder sub-samples were only heated once, prior to asphalt production. The remaining surplus bitumen in the 2 L containers, which was left after asphalt batching, was discarded.

Figure 4.3: Binder delivery and storage

(a) Supplied binder (b) Sub-sampled binder

Source: ARRB (2017).

4.3 Asphalt

The nominated asphalt mixes consisted of two types of binder (C320 and an A35P grade PMB) and two types of aggregate matrices (20 mm intermediate mix and 14 mm intermediate mix). Preparation and mixing of asphalt specimens were conducted in accordance with AS/NZS 2891.2.1-2014 *Methods of Sampling and Testing Asphalt – Method 9.2: Determination of Bulk Density of Compacted Asphalt: Presaturation Method*. The C320 bitumen and A35P grade PMB asphalt samples were respectively heated and conditioned at 150 ± 3 °C and 160 ± 3 °C (as per AS/NZS 2891.2.1-2014)

Mixes 1 and 2 contained conventional Class 320 binder, while Mixes 3 and 4 contained an A35P grade PMB. A summary of the various asphalt mix designs used for this study are presented in Table 4.2, while, a detailed summary for each mix is presented in Appendix A.

Table 4.2: Mix design information

Asphalt mix No.	Main Roads mix registration number	Description	Binder type	Binder content (%)
1	JM 53	20 mm Dense-Graded Asphalt – Intermediate Course	C320	4.4
2	JM 54	14 mm Dense-Graded Asphalt – Intermediate Course	C320	4.8
3	JM 53	20 mm Dense Graded Asphalt – Intermediate Course	A35P	4.4
4	JM 54	14 mm Dense-Graded Asphalt – Intermediate Course	A35P	4.8

Asphalt slabs were manufactured at the ARRB laboratory in accordance with AG:PT/T220 (Austroads 2005) test method. Figure 4.4 shows the mixing and compaction apparatus used for manufacturing of the slabs.

Figure 4.4: Asphalt mix preparation and compaction

(a) Hobart mixer (b) BP slab compactor

Source: ARRB (2017).

An automated saw was used to cut the slabs into beams. All cut asphalt beams used in this study had a target air void content of 5.0 ± 0.5% when tested by the pre-saturation method as per AS/NZS 2891.9.2:2014. The beams were stored on a flat surface in a temperature-controlled room with a temperature not exceeding 30 °C.

The AGPT/T274 (2016) test method notes that “*Whenever practicable, test should be performed within 30 days of the date of compaction for laboratory-prepared slabs.*” Appendix B show the age of the beams that were tested for flexural stiffness and fatigue failure (Mixes 1, 2, 3 and 4). The age of the beams at testing varied between 2 and 55 days for flexural stiffness and 3 and 89 days for fatigue failure due to laboratory scheduling constraints. Appendix B shows that the age of the beams at testing does not appear to influence the resulting flexural stiffness or fatigue failure, as no clear correlations between age and flexural stiffness or number of cycles to failure can be inferred. Appendix E shows a fatigue model regressed from test results with and without consideration of age of the beams as one of the input parameters. It shows that the age of the beams have an insignificant effect on the results, and does not considerably improve the fit of the model.

4.4 Laboratory Testing

4.4.1 Bitumen Tests

The C320 bitumen and A35P grade PMB certificates were supplied in Main Roads report number 15 S2299/1(AN: 1982) and SAMI report number 8246 (AN: 5598). The certificates, as well as independent testing carried out by ARRB, indicate that the C320 bitumen and A35P grade PMB satisfy the requirements set by AS 2008-2013 and AGPT-T190-14 respectively.

The test properties of the A35P and C320 bitumen are summarised in Table 4.3 and Table 4.4, respectively.

Table 4.3: Typical commercial A35P grade PMB test properties

Property	Supplied A35P [2]	ARRB result	PMB specification
PMB modifier	Unknown	Unknown	–
Viscosity at 165 °C (Pa.s)	0.46	0.44	0.6 Max
Torsional recovery at 25 °C (%)	20	19	6–21
Softening point (°C)	70.0	68.0	62–74
Consistency at 60 °C (Pa.s)	2 292 [1]	3 914	2,000 Min
Consistency 6% at 60 °C (Pa.s)	1 106 [1]	1 388	Report
Stiffness at 25 °C (kPa)	62 [1]	86	120 Max
Penetration at 25 °C (0.1 mm)	Not tested	Not tested	–

1 Monthly test. Not obtained for the sample supplied
2 SAMI report number 8246 (AN: 5598)

Table 4.4: C320 bitumen properties used for this project (Main Roads report number 15 S2299/1(AN: 1982))

Property	Supplied test result	ARRB result	Bitumen specification
Viscosity at 60 °C (Pa s)	317	307	260–380
Viscosity at 135 °C (Pa s)	0.44	0.47	0.40–0.65
Penetration at 25 °C (0.1 mm)	49	51	40 min.
Viscosity at 60 °C after rolling thin film oven (RTFO) treatment (Pa s)	639	586	–
Percentage viscosity at 60 °C increase after RTFO (%)	201	191	300 max.
Toluene insoluble (% weight)	0.00	0.03	1.0 max.
Flashpoint (°C)	–	> 300	–

4.4.2 Modulus Tests

Flexural modulus tests were performed on a set of four beams according to AGPT/T274 (Austroads 2016b). Each beam was tested at five temperatures (i.e. 5 °C, 10 °C, 20 °C, 30 °C and 40 °C) and eight loading frequencies (i.e. 0.1 Hz, 0.5 Hz, 1 Hz, 3 Hz, 5 Hz, 10 Hz, 15 Hz and 20 Hz) at each temperature. It is noted, however, that AGPT/T274 advises measured moduli for test temperatures above 30 °C should be used with care, as a result of possible issues with non-linearity and also for possible creep of the specimens.

4.4.3 Fatigue Tests

Asphalt fatigue testing was carried out according to AGPT/T274 (Austroads 2016b). A minimum of three beams were tested at high, medium and low strain levels. Each beam was tested at a frequency of 10 Hz and at one of three temperatures (i.e. 10 °C, 20 °C and 30 °C). A number of fatigue tests were also carried out at 40 °C. However, these tests could not be completed due to sudden failure prior to achieving the termination stiffness (50% of initial stiffness). This is due to non-linearity and creep of specimens, as mentioned in AGPT/T274 (Austroads 2016b). Figure 4.5a and Figure 4.5b illustrates the rapid asphalt beam failure occurring between the inner and outer clamps at 40°C.

Consequently, fatigue testing at 40 °C was omitted from the laboratory fatigue testing programme.

Figure 4.5: Fatigue testing at 40 °C (Mix 1, 20mm C320)

Source: ARRB (2017).

AGPT/T274 test method recommends fatigue testing on a minimum of 18 beams at a minimum of three different strain levels. However, a statistical analysis carried out as part of the NACoE project indicated that when fitting a single model to asphalt fatigue results at different temperatures, it would be sufficient to test a minimum of 9 beams per temperature, over at least three strain levels (NACoE 2014; Denneman & Lam 2015; Denneman & Bryant 2016).

4.4.4 Summary

A summary of test methods and parameters tested in included in Table 4.5.

Table 4.5: Test methods and tested properties

Test method	Parameter tested
Bitumen	
AS/NZS 2341.21:2015	Sub-sampling of base & PMB binders
AGPT-T190-14	PMB binder specification testing
AS 2008-2013	Base binder specification testing
AS 2341.4	Viscosity at 135°C
AS 2341.7	Density
AS/NZS 2341.10 and AS 2341.2	Viscosity at 60°C after RTFO treatment
AS 2341.12	Penetration at 25°C
AS 2341.14	Flash point
AGPT/T103	Loss on heating
AGPT/T108	Segregation
AGPT/T111	Viscosity at 165 °C
AGPT/T112	Flashpoint
AGPT/T121	Consistency at 60 °C, Consistency 6% at 60 °C, Elastic recovery at 60 °C, 100 s, Stiffness at 15 °C
AGPT/T122	Torsional recovery at 25 °C, 30 s
AGPT/T131	Softening point
AGPT/T132	Compression limit at 70 °C, 2 kg
AS 2341.8	Toluene insoluble
AGPT/T121	Stiffness at 25 °C (kPa)
Asphalt	
AG:PT/T220	Compaction of asphalt slabs
AS/NZS 2891.2.1:2014	Mixing, quartering and conditioning of asphalt
AS/NZS 2891.7.1	Maximum Density (Water Displacement Method)
AS/NZS 2891.8	Air Voids
AS/NZS 2891.9.2	Bulk Density (Saturated Surface Dry Method)
AGPT/T274	Flexural stiffness and fatigue
Aggregate	
AS 1141.11.1	Particle size distribution

4.5 Test Results

4.5.1 Modulus Results

Figure 4.6 to Figure 4.10 shows the frequency sweep (flexural) modulus results for the four asphalt mixes (Mixes 1, 2 ,3 and 4) across a range of temperatures (i.e. 5 °C, 10 °C, 20 °C, 30 °C and 40 °C). The upper and lower limit envelopes (which corresponds to ±15% variation from the mean stiffness of all four mixes at the tested frequencies) are included in the figures to provide an indication of the variation in modulus between the mixes.

Figure 4.6 to Figure 4.8 indicate that the flexural modulus of all four mixes were similar for the frequency sweeps conducted at low and intermediate test temperatures (i.e. 5, 10 and 20 °C), as the curves for each mix were all within the ±15% envelope. This indicates that all four mixes have similar modulus values at low and intermediate temperatures (5 to 20 °C), regardless of the aggregate matrix (i.e. 14 mm or 20 mm) and binder type (i.e. C320 bitumen or A35P grade PMB).

Figure 4.9 shows that the modulus results at 30 °C for Mixes 1 and 3 (20 mm mix with C320 bitumen and 20 mm mix with A35P grade PMB) are slightly outside the ±15% envelope at frequencies of 5Hz and lower and 3 Hz and lower respectively. The corresponding results obtained for these two mixes were well outside the ±15% envelope (for all studied frequencies) when tests were performed at 40 °C (see Figure 4.11). This may be due to the high test temperature, for which the test is not considered to be reliable.

Figure 4.6: Flexural stiffness at 5 °C with ±15% (upper/lower limit) from the mean

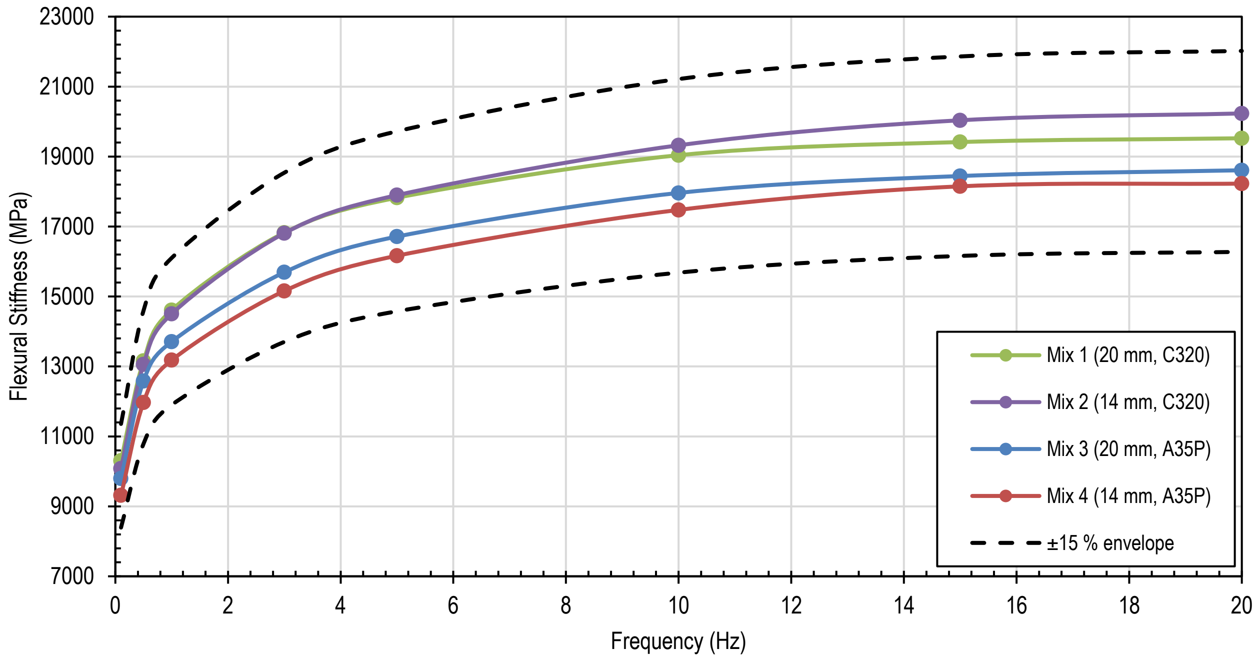


Figure 4.7: Flexural stiffness at 10 °C with ±15% (upper/lower limit) from the mean

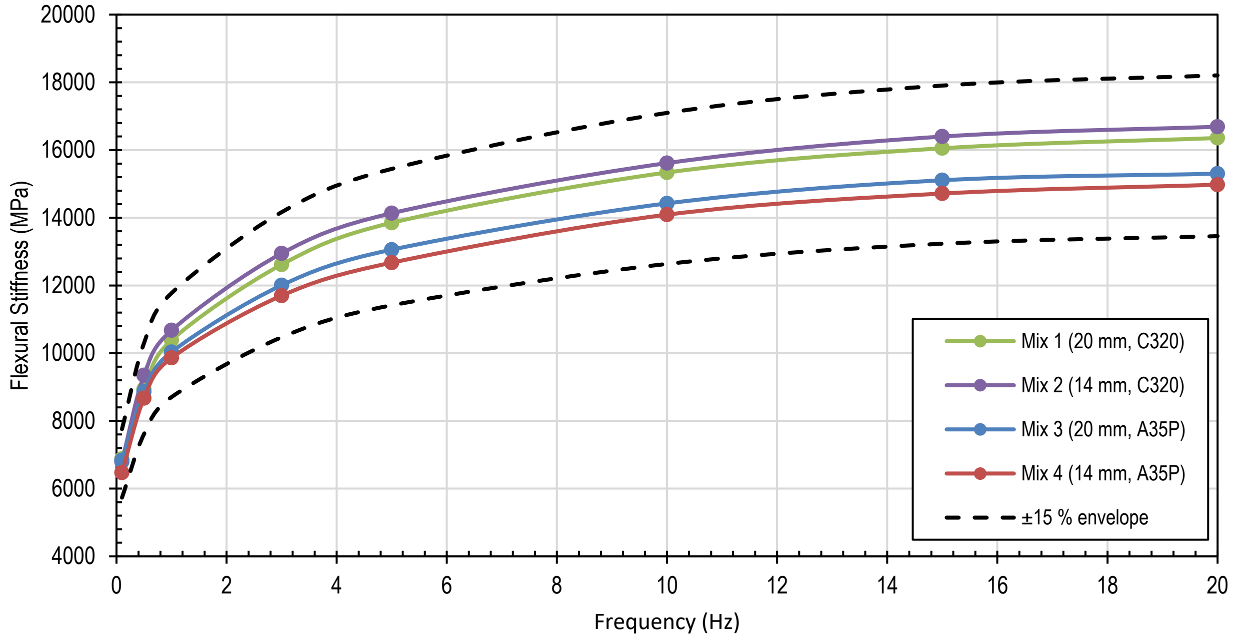


Figure 4.8: Flexural stiffness at 20 °C with ±15% (upper/lower limit) from the mean

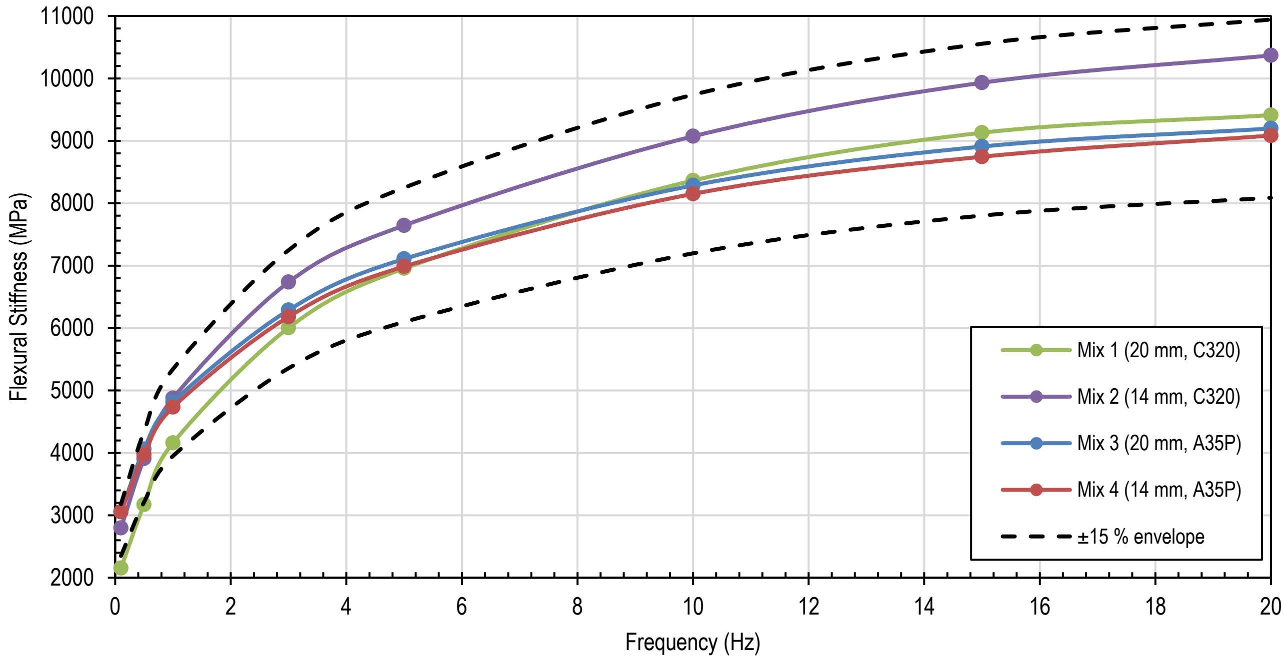


Figure 4.9: Flexural stiffness at 30 °C with ±15% (upper/lower limit) from the mean

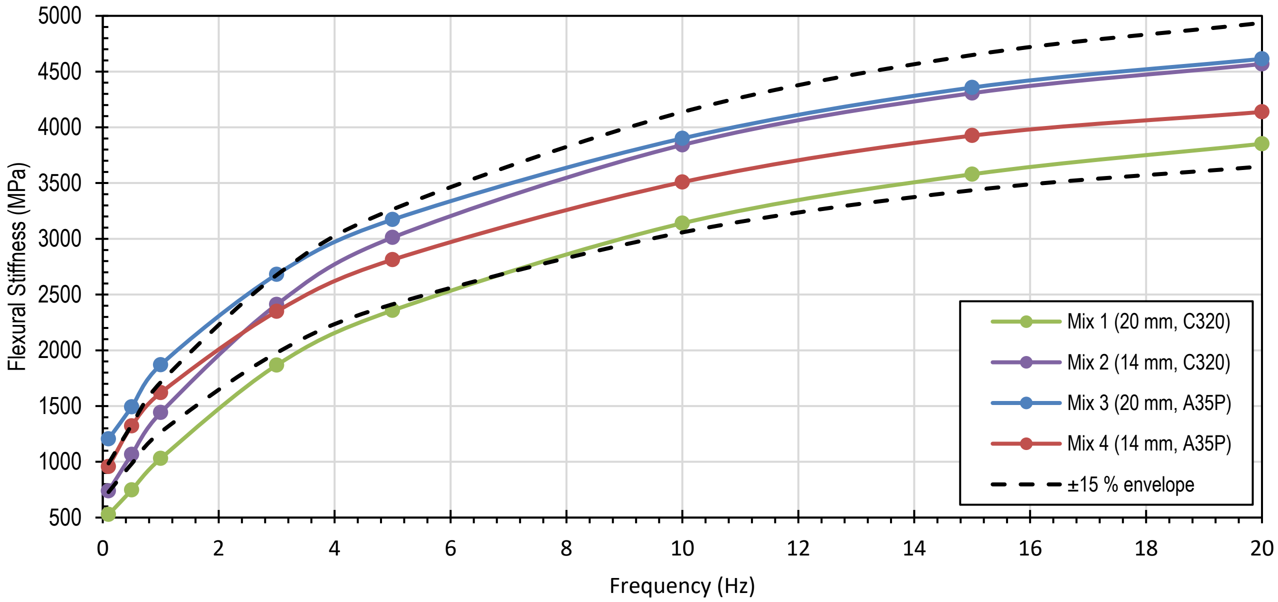
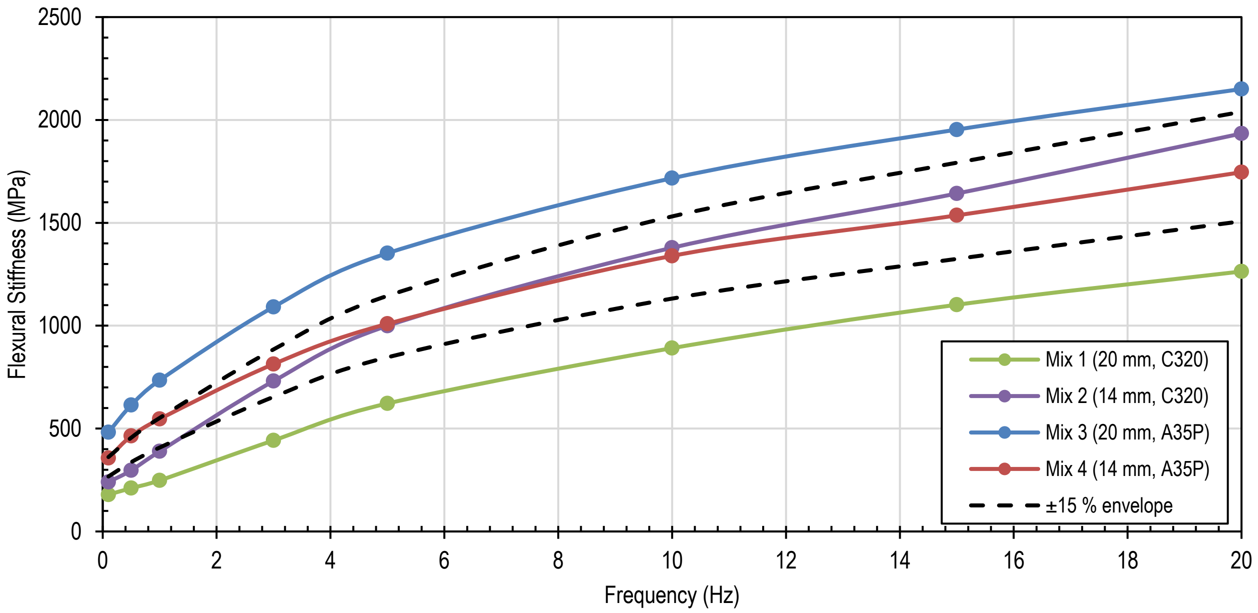


Figure 4.10: Flexural stiffness at 40 °C with ±15% (upper/lower limit) from the mean



The flexural modulus master curves (refer Section 3.1) for a reference temperature (T_{ref}) of 20 °C for Mix 1, 2, 3 and 4 are presented in Figure 4.11, Figure 4.12, Figure 4.13 and Figure 4.14. The regression coefficients used to construct the master curve are presented in Table 4.6. A comparison of the flexural master curves is presented in Figure 4.15. It is noted that although the master curves were fitted to represent flexural modulus test results of less than 1000 MPa, the Austroads and ERN9 design methodology limits asphalt design modulus values to a minimum of 1000 MPa. This limitation to the modulus value was included in the design methodology based on observations that although values lower than that are observed in the laboratory they are not typically observed in the field. Adoption of modulus values of less than 1000 MPa would result in more conservative (thicker) FDA pavements.

Figure 4.11: Flexural modulus master curve results for Mix 1 (C320 20 mm)

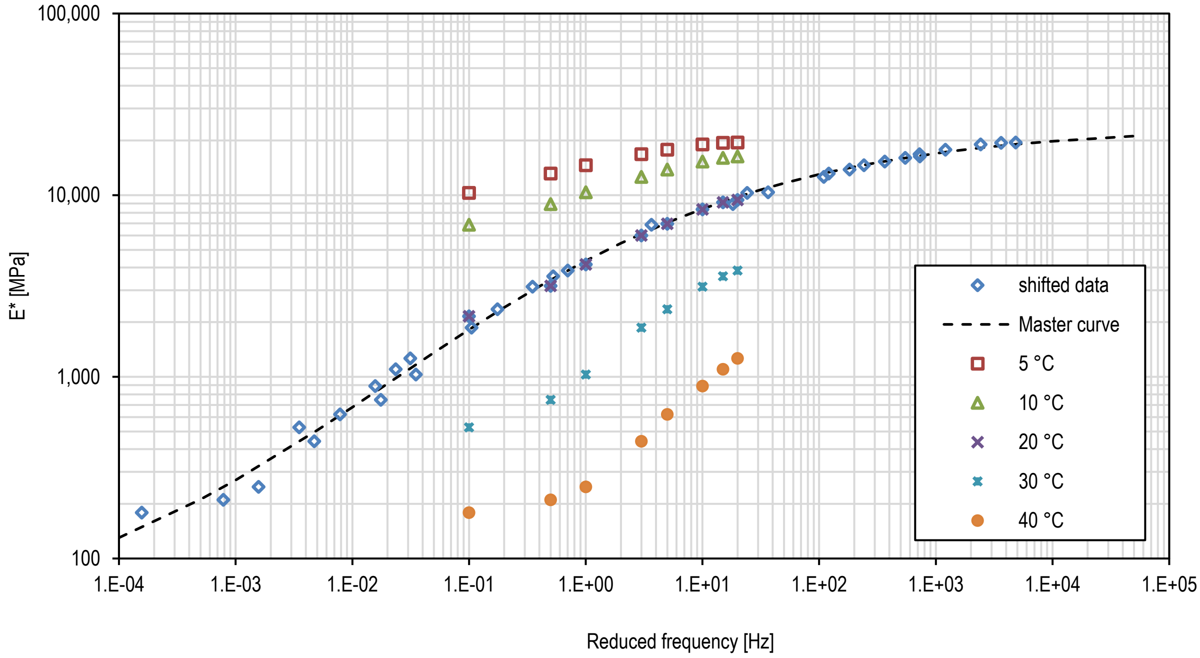


Figure 4.12: Flexural modulus master curve results for Mix 2 (C320 14 mm)

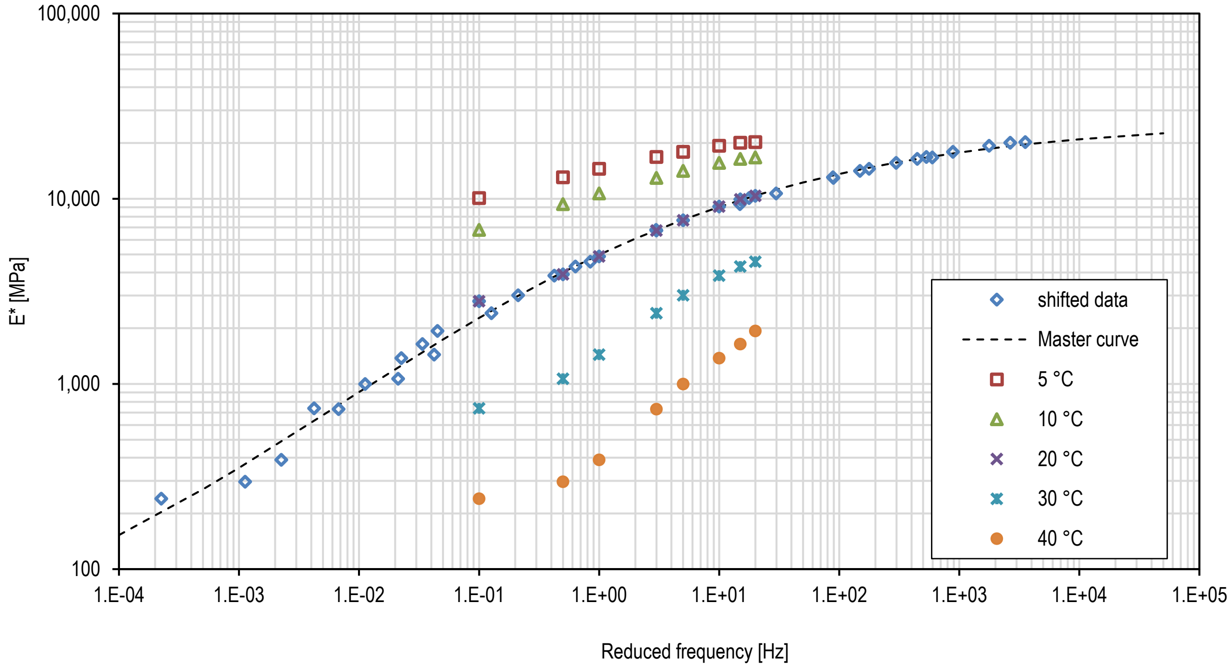


Figure 4.13: Flexural modulus master curve results for Mix 3 (A35P 20 mm)

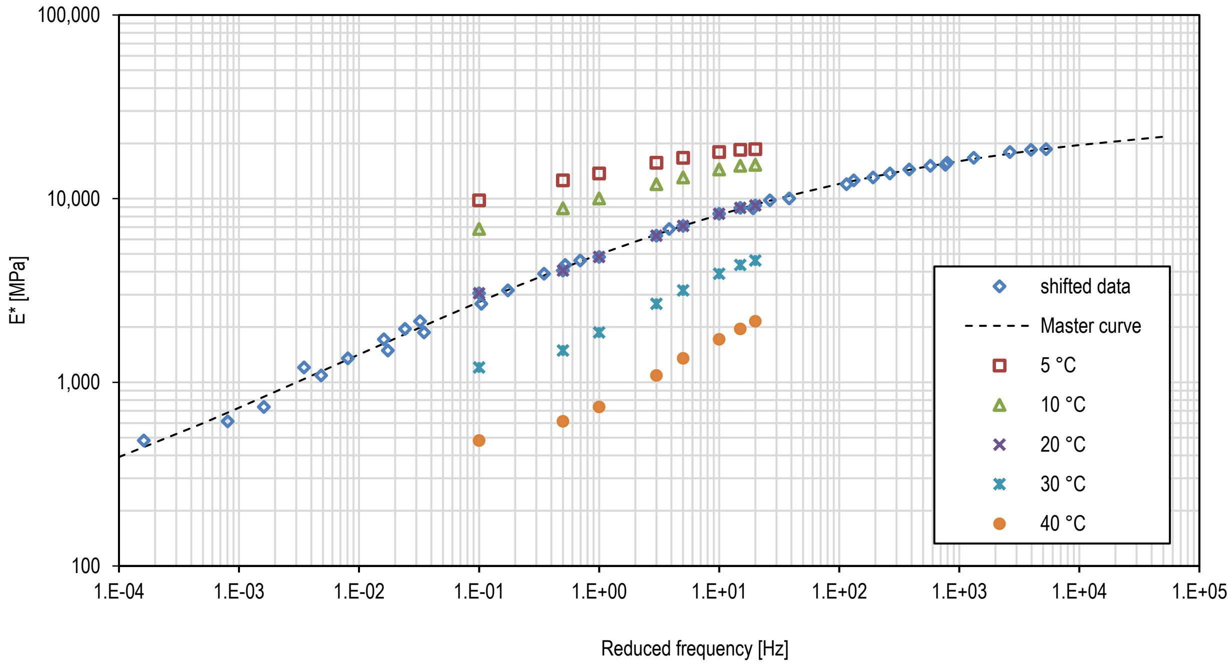


Figure 4.14: Flexural modulus master curve results for Mix 4 (A35P 14 mm)

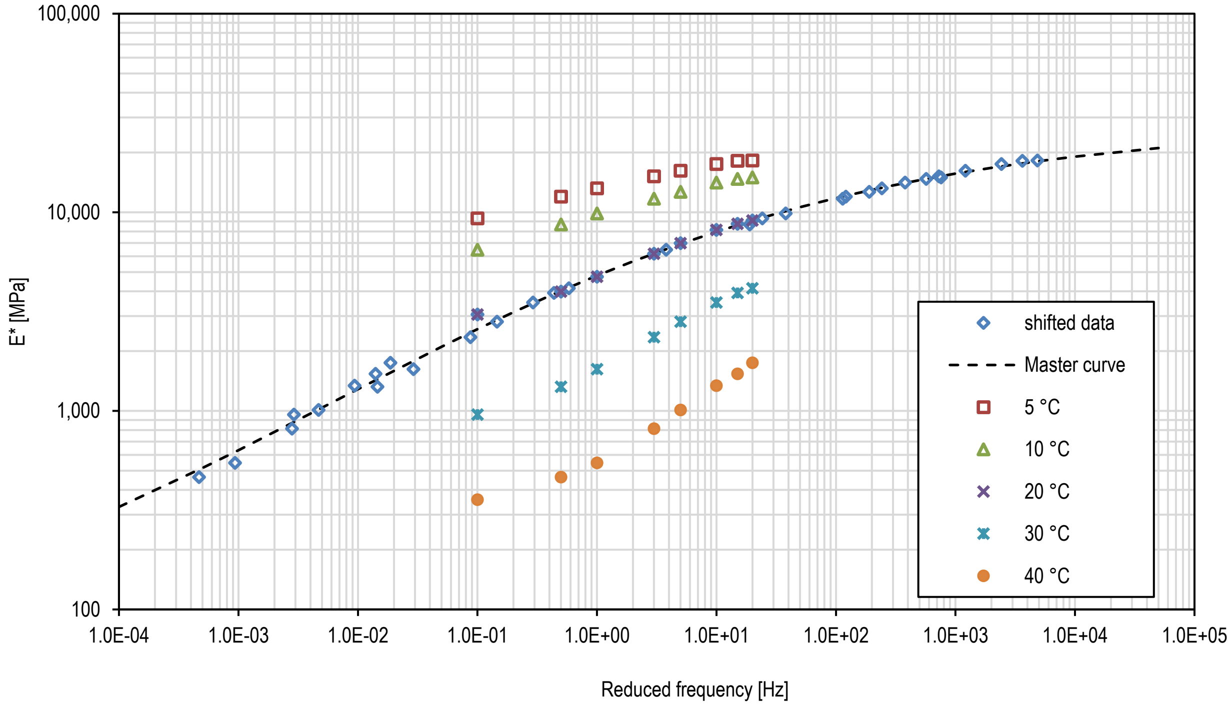


Figure 4.15: Flexural master curve results comparison

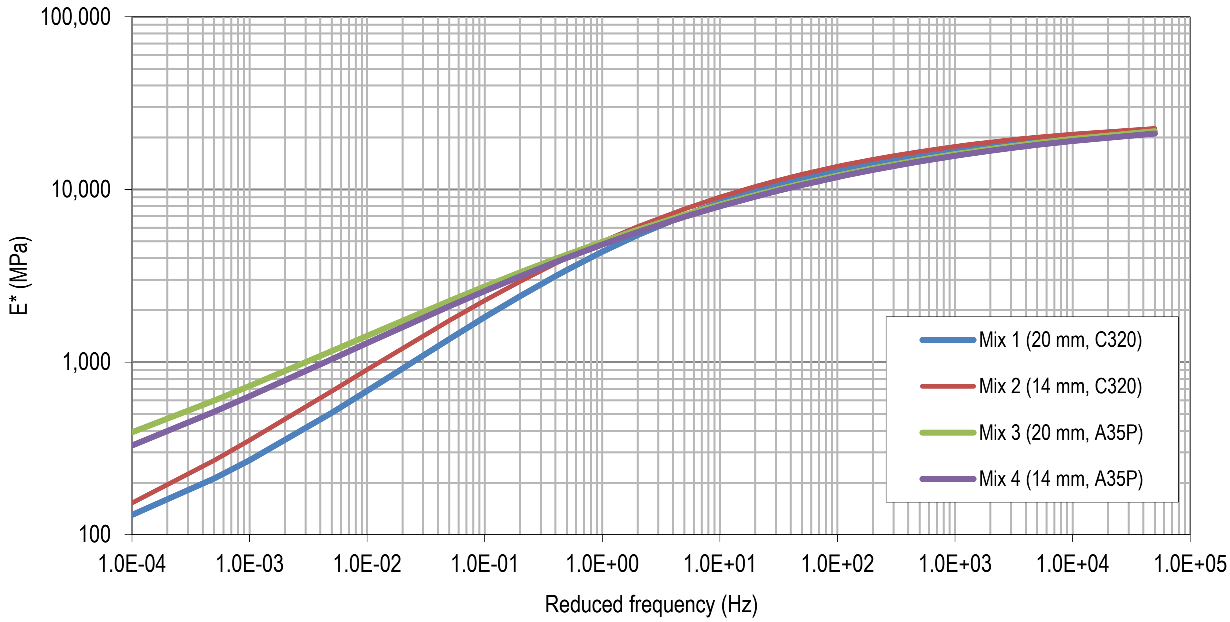


Table 4.6: Flexural master curve regression coefficients

Mix number	Mix type	δ	α	β	γ	a	b	R^2	T_{ref} (°C)
1	C320 20	1.580	2.800	–1.021	–0.616	5.34E–04	–0.1509	0.996	20
2	C320 14	1.359	3.067	–1.166	–0.542	5.00E–04	–0.1424	0.995	20
3	A35P 20	1.747	2.727	–0.919	–0.430	6.22E–04	–0.1521	0.997	20
4	A35P 14	1.640	2.810	–0.974	–0.442	2.15E–04	–0.1557	0.998	20

Figure 4.16 and

Figure 4.17 provides a comparison between the flexural modulus obtained from the laboratory testing conducted and those predicted using the presumptive ITT in AGPT Part 2 and ERN9 for a Weighted Mean Annual Pavement Temperature (WMAPT) of 30 °C and 40 °C respectively. The minimum design modulus using the Austroads design procedure is 1000 MPa (i.e. values within the shaded grey area in Figure 4.16 and

Figure 4.17 are not used in the design). The dotted lines represent the modulus that would be used in accordance with AGPT Part 2 and ERN9 prior to the application of the correction factor for in-service air void content (i.e., the air void content in service was assumed to be 5%, as used in the construction of the master curve). The values prior to the application of the correction factor for in-service air void content were used for the comparison as they correspond to the air void content of the tested specimens, which were the values used for the construction of the master curve. (If the presumptive values were corrected for the in-situ air voids, the moduli from the master curve would also need to be corrected using the same equation. The correction for in-service air voids would shift both curves similarly, and therefore, would not alter the conclusions.)

Assumed ITT at 40 ms rise time and 25 °C in accordance with AGPT Part 2 for intermediate course asphalt mixes are presented in Table 4.7, noting that there are no presumptive values for mixes containing A35P other than the adoption of AGPT Part 2 adjustment factors.

Table 4.7: Assumed ITT at 40 mx rise time and 25 °C

Mix	Assumed ITT modulus at 40 ms rise time and 25 °C
Mix 1 (20 mm, C320)	5500 MPa
Mix 2 (14 mm, C320)	5000 MPa

Figure 4.16: Comparison of measured flexural modulus master curve and presumptive modulus in AGPT Part 2 at 30 °C (for Av=5.0%)

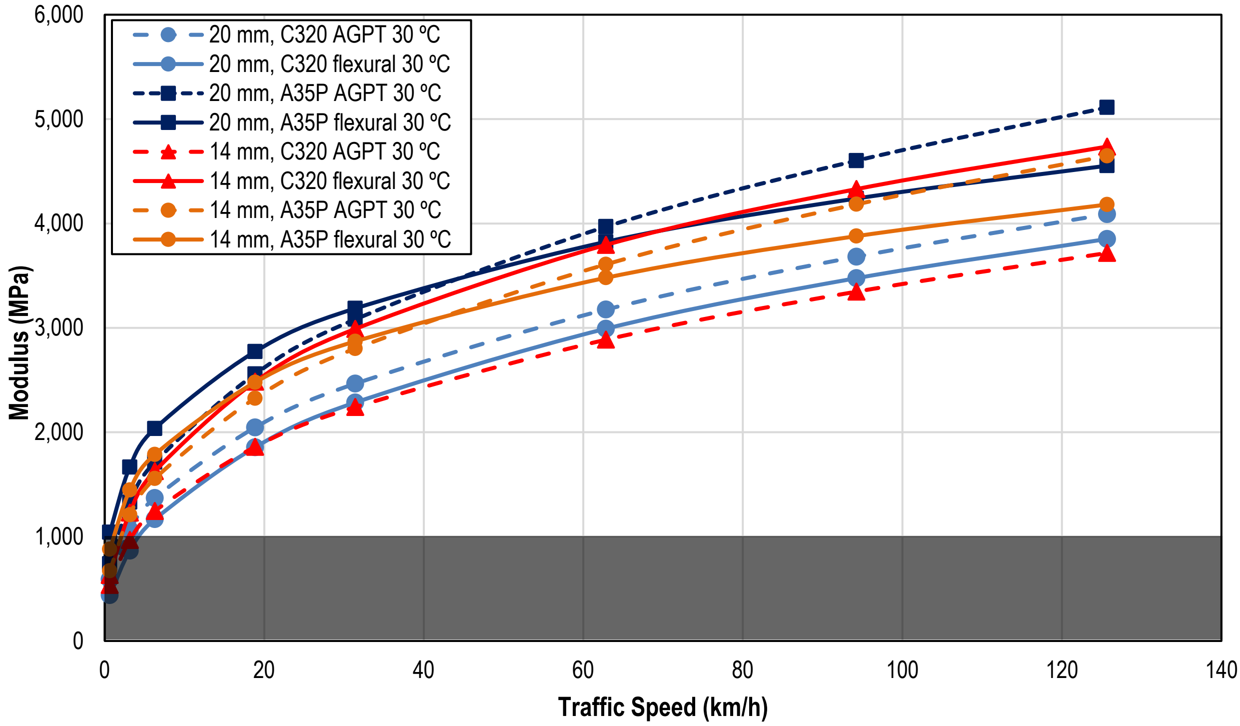
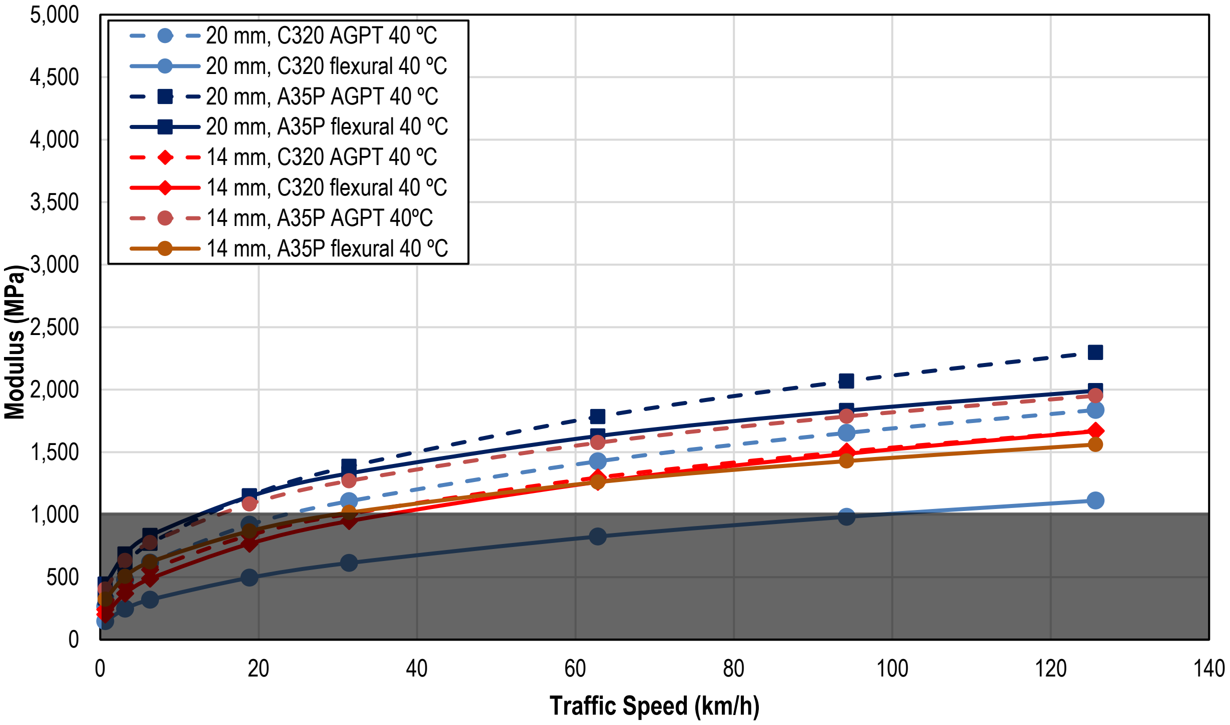


Figure 4.17: Comparison of measured flexural modulus master curve and presumptive modulus in AGPT Part 2 at 40 °C (for Av=5.0%)



According to Figure 4.16 and

Figure 4.17, at pavement temperatures of 30 °C and 40 °C, which are within the range of WMAPT values commonly observed in northern parts of WA:

- For Mix 1 (20 mm C320), the modulus values obtained using the current design methodology (presumptive ITT values) are higher than the modulus values obtained from the flexural beam testing. The difference in modulus is relatively small at 30 °C but becomes more significant at 40 °C.
- For Mix 2 (14 mm C320), the modulus values obtained using the current design methodology (presumptive ITT values) are lower than the modulus values obtained from the flexural beam testing at 30 °C and about the same at 40 °C.
- Mix 3 (20 mm A35P) has higher modulus values than the same mix without polymer (Mix 1), both at 30 °C and at 40 °C.
- Mix 4 (14 mm A35P) has modulus values similar to the same mix without polymer (Mix 2) at 40 °C. Compared with Mix 2 at 30 °C, Mix 4 shows similar modulus at lower speeds and slightly lower modulus at higher speeds.
- Mix 2 (14 mm C320) generally has a higher flexural modulus than Mix 1 (20 mm C320) for an air voids content of 5.0%.

It is noted that the observations above are based on an air voids content of 5.0%. In the field, higher air void contents are expected, leading to reduced modulus. Lower moduli values in full depth asphalt pavement design leads to higher strain values and reduced fatigue life.

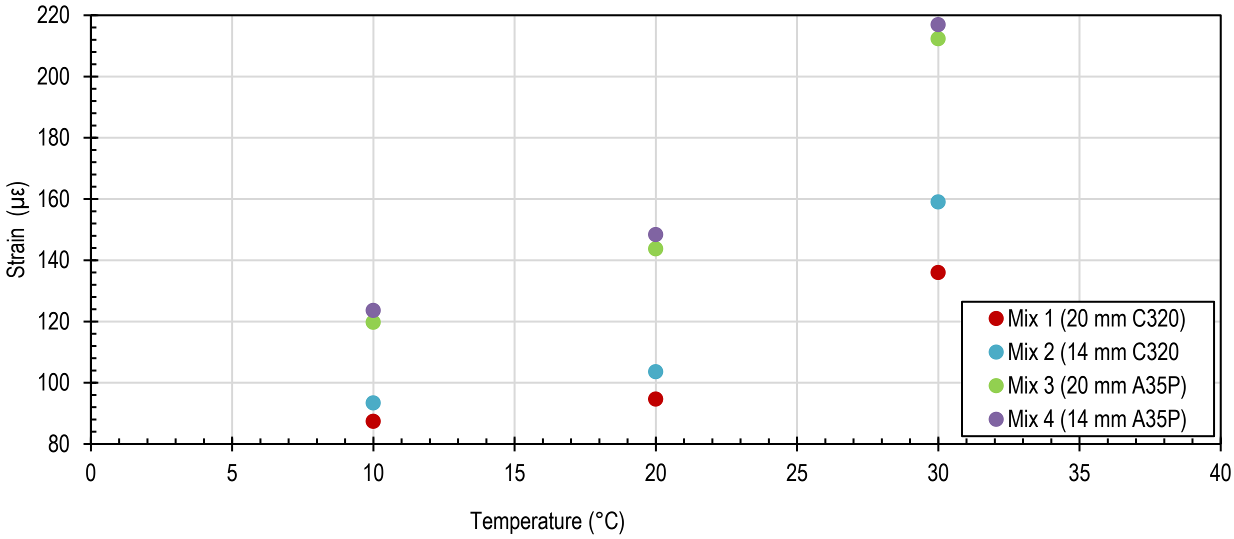
4.5.2 Fatigue Results

Figure 4.18 shows the strain level corresponding to 1 000 000 cycles to failure at various temperatures. All four mixes tested (Mixes 1, 2, 3 and 4) show improved fatigue characteristics with increase in temperature (for the same strain levels).

A35P is an asphalt class that uses a plastomeric polymer called EVA (ethylene vinyl acetate). EVA is added to increase stiffness and deformation resistance, as well as increase resistance to fuel spills where required. EVA is not expected to improve asphalt fatigue life for a given level of strain. AP-T235-13 (Austroads Guide to the Selection and use of PMB and Multigrade Bitumens) does not recommend A35P for enhancing fatigue performance. (Nevertheless, considering the increased stiffness of the mix, for the same asphalt thickness and load magnitude, a pavement with A35P mix, compared to the same mix without polymer, will result in reduced strains at the bottom of the asphalt layer. Reduced strains may result in an increased pavement fatigue life depending on the relationship between modulus and fatigue life for a given mix.)

The increased fatigue performance (for the same strain level) of Mix 3 and 4 (A35P) compared to Mix 1 and 2 (C320) could be due to the presence of some elastomeric-type polymer in the A35P grade PMB (refer to Section 4.4.1).

Figure 4.18: Strain at 10^6 cycles at various temperatures



The fatigue test results and the 5-Parameter Model curves are shown in Figure 4.19, Figure 4.20, Figure 4.21 and Figure 4.22 respectively for Mix 1, 2, 3 and 4. $N_{f\ 50}$ represents the number of cycles to failure, with failure defined as a 50% reduction in the asphalt modulus.

Figure 4.19: Fatigue results for Mix 1 (20 mm C320)

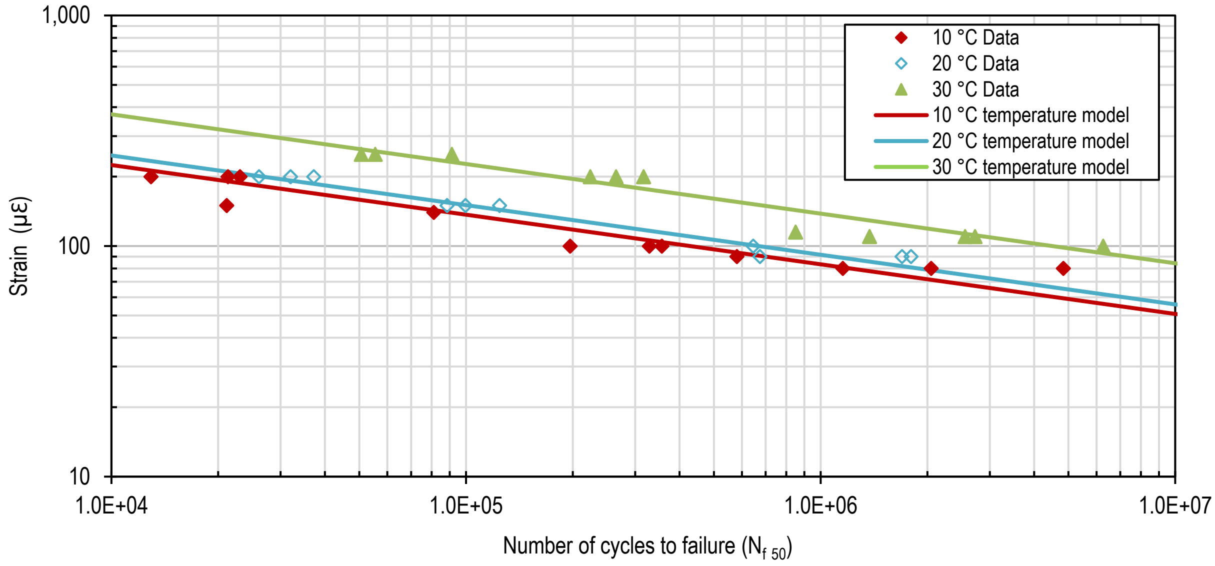


Figure 4.20: Fatigue results for Mix 2 (14 mm C320)

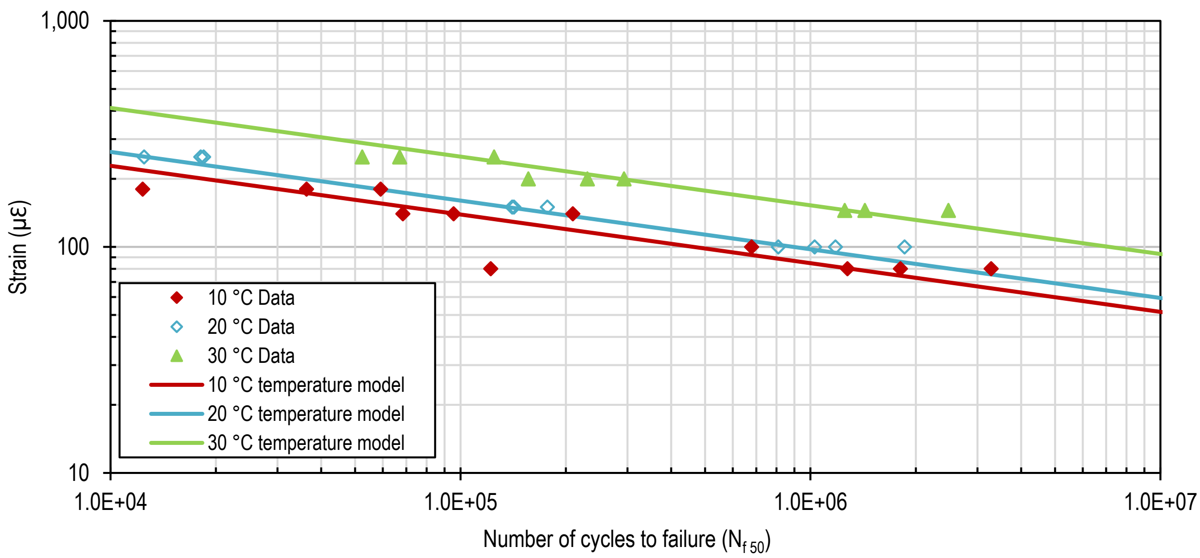


Figure 4.21: Fatigue results for Mix 3 (20 mm A35P)

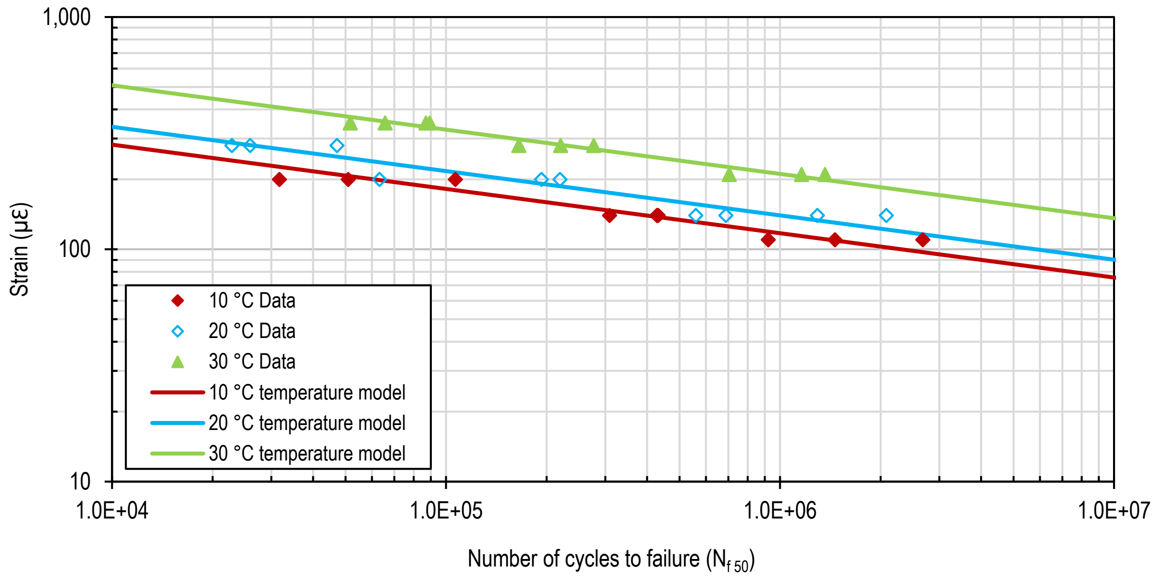
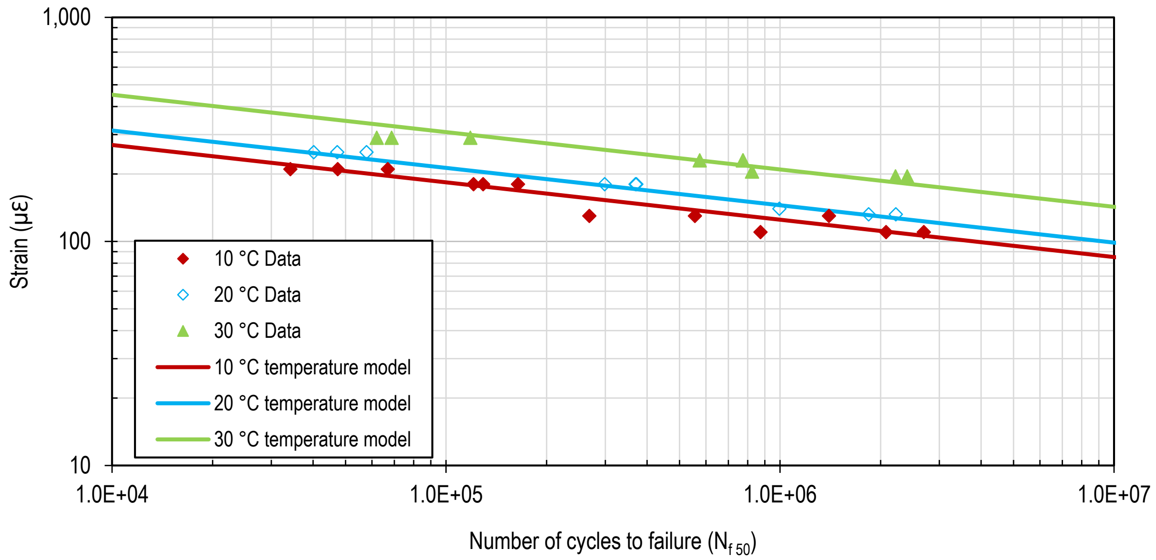


Figure 4.22: Fatigue results for Mix 4 (14 mm A35P)



As discussed in Section 4.3, AGPT-T274 recommends that '*Wherever practicable, tests should be performed within 30 days of the date of compaction for laboratory-prepared slabs*'. Due to the laboratory schedule, the age of the tested beams varied from this recommendation. A summary of beam age, N_{f50} (number of repetitions to failure), strain level, beam air voids, initial modulus (at cycle 50) and modulus calculated from the master curve at 10, 20, 30 °C (frequency of 10Hz) for Mixes 1, 2, 3 and 4 are presented in Appendix D, Table D 1, Table D 2, Table D 3 and Table D 4 respectively. Appendix E includes a comparison of asphalt fatigue models with and without consideration of beam age as one of the input parameters. It indicates that beam age did not significantly influence the results in this particular study.

4.5.3 *Temperature-dependent Fatigue Models*

The regression coefficients for Equation 4 and Equation 5 were calculated using Microsoft Excel solver by maximising the coefficient of determination (R^2) for the regression. The modulus of the asphalt for each mix was assumed using the master curves presented in Section 4.5. Regression coefficients for the 3-Parameter Model and 5-Parameter Model are presented in Table 4.8 and Table 4.9 respectively, where σ_y represents the standard deviation of the residuals. The σ_y is slightly smaller for the 5-Parameter Model fitted to Mix 3 and 4, indicating a slightly better fit to the data.

Table 4.8: Regression coefficients for 3-Parameter Model

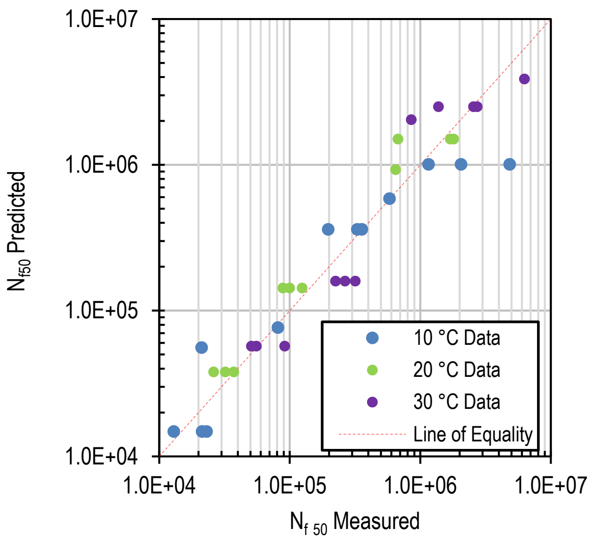
Mix type	n	a_1	a_2	a_3	σ_y
C320 20 mm	33	1.00E+21	–1.490	–4.606	0.52
C320 14 mm	31	2.91E+23	–1.976	–4.777	0.62
A35P 20 mm	29	4.47E+26	–2.371	–5.250	0.44
A35P 14 mm	30	1.72E+27	–2.173	–5.857	0.42

Table 4.9: Regression coefficients for 5-Parameter Model

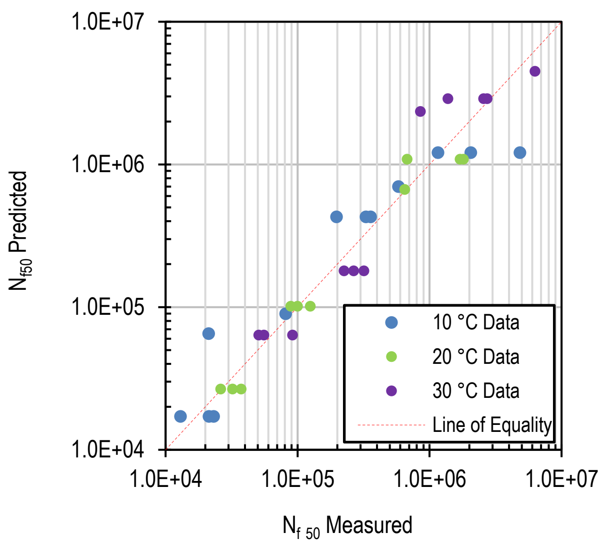
Mix type	n	c_1	c_2	c_3	c_4	c_5	σ_y
C320 20 mm	33	0.404	–9.959	79.73	–170.3	–4.646	0.50
C320 14 mm	31	0.402	–9.992	80.17	–169.8	–4.643	0.64
A35P 20 mm	29	0.423	–10.447	82.87	–168.4	–5.245	0.41
A35P 14 mm	29	0.406	–10.063	80.26	–159.7	–5.997	0.43

The mix-specific fatigue models presented in Figure 4.23 to Figure 4.26 can be used to compare the predictive fatigue performance to the measured performance. The plots indicate that there is no clear bias on the prediction for a given test temperature.

Figure 4.23: Measured vs predicted fatigue data for Mix 1



3-Parameter Model



5-Parameter Model

Figure 4.24: Measured vs predicted fatigue data for Mix 2

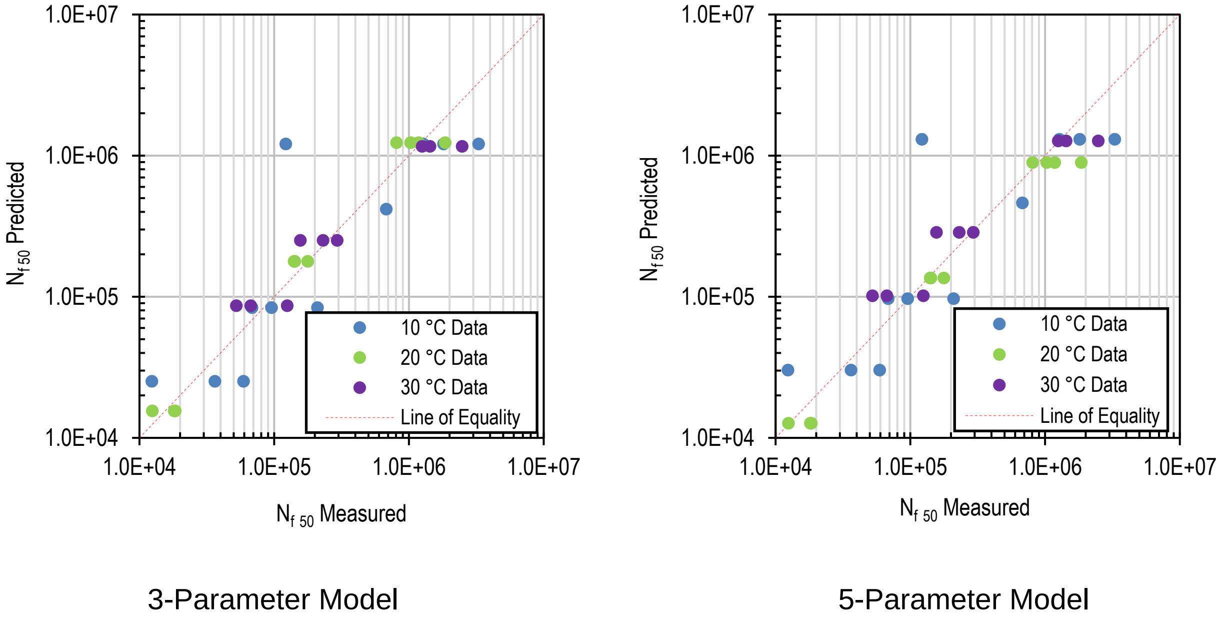


Figure 4.25: Measured vs predicted fatigue data for Mix 3

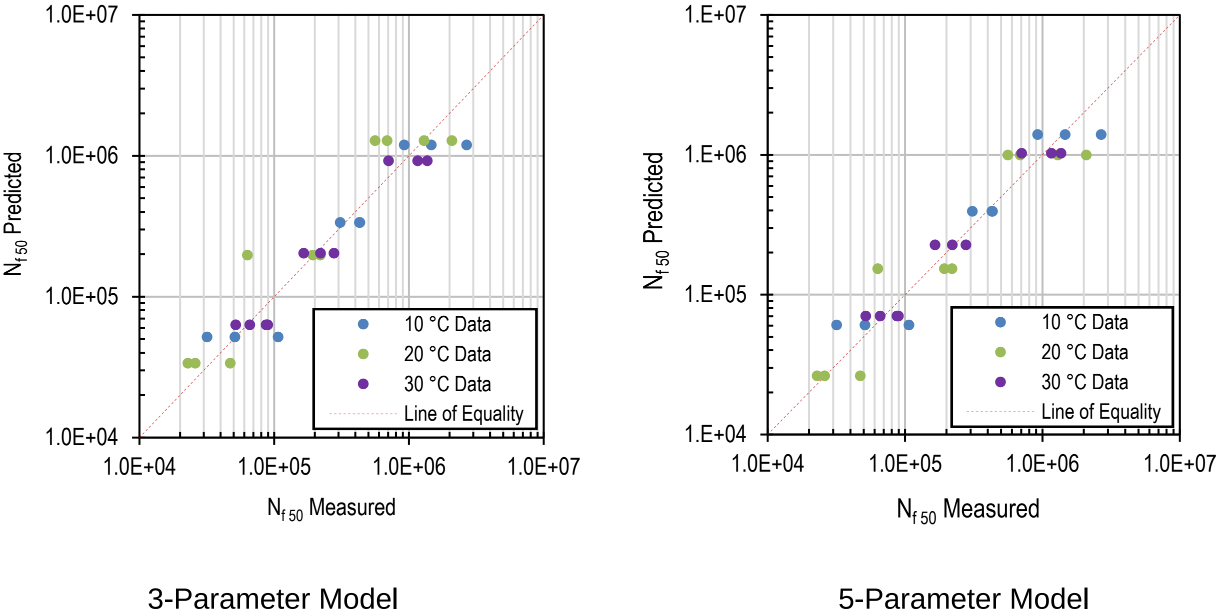
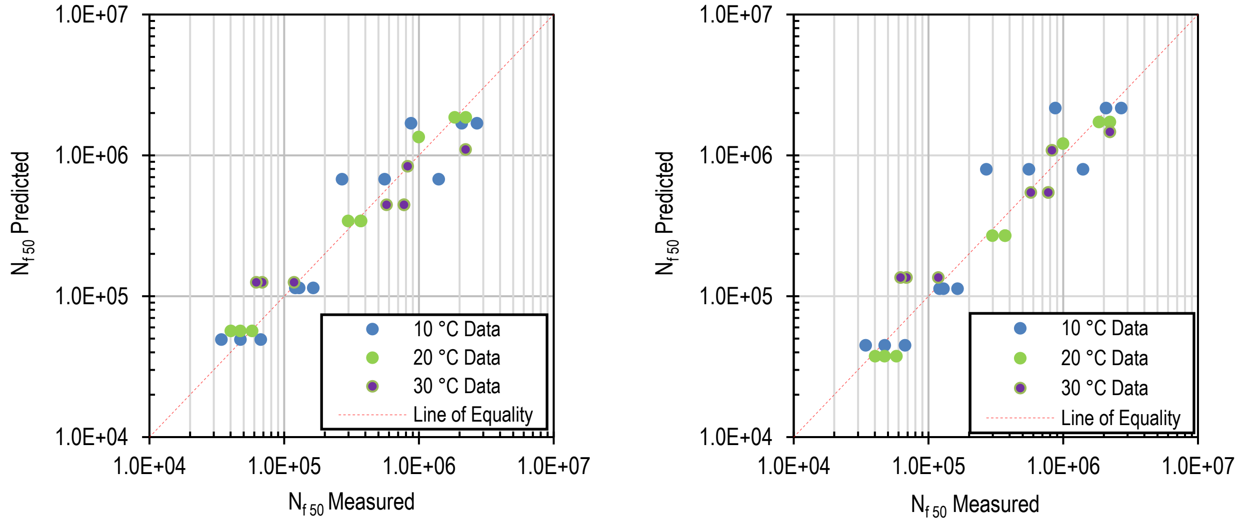


Figure 4.26: Measured vs predicted fatigue data for Mix 4



3-Parameter Model 5-Parameter Model

4.6 Comparing the Mix-specific Models to SPDM

A comparison between the Shell laboratory model in Equation 1 and the laboratory derived fatigue models are presented in Figure 4.27 to Figure 4.30 for Mix 1 to 4 respectively.

Figure 4.27: Comparison between 3-Parameter Model, 5-Parameter Model and SPDM for Mix 1 (20 mm C320)

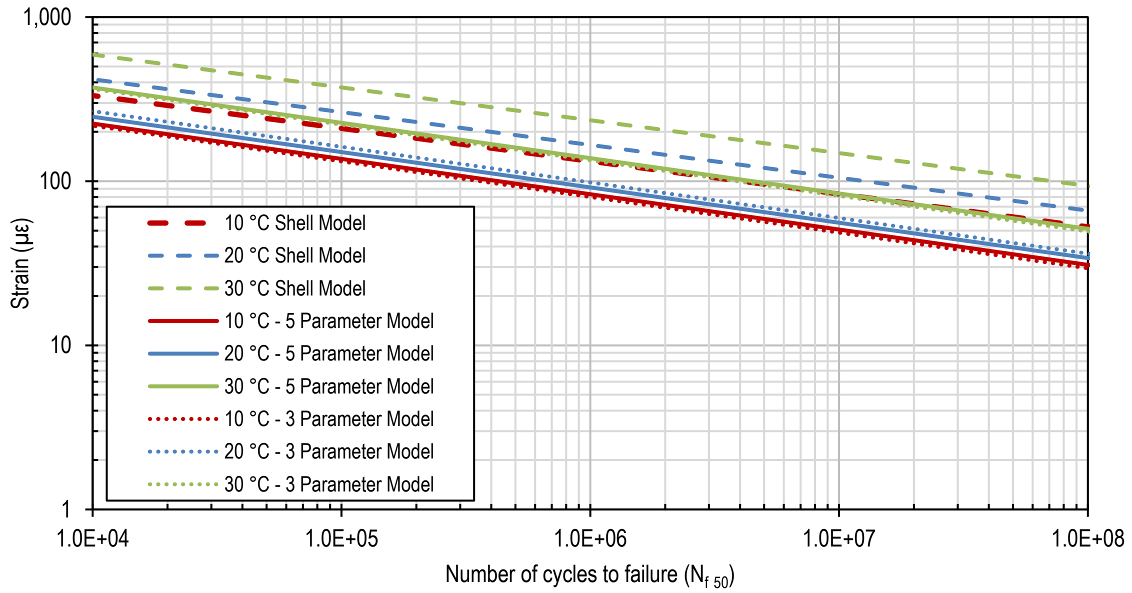


Figure 4.28: Comparison between 3-Parameter Model, 5-Parameter Model and SPDM for Mix 2 (14 mm C320)

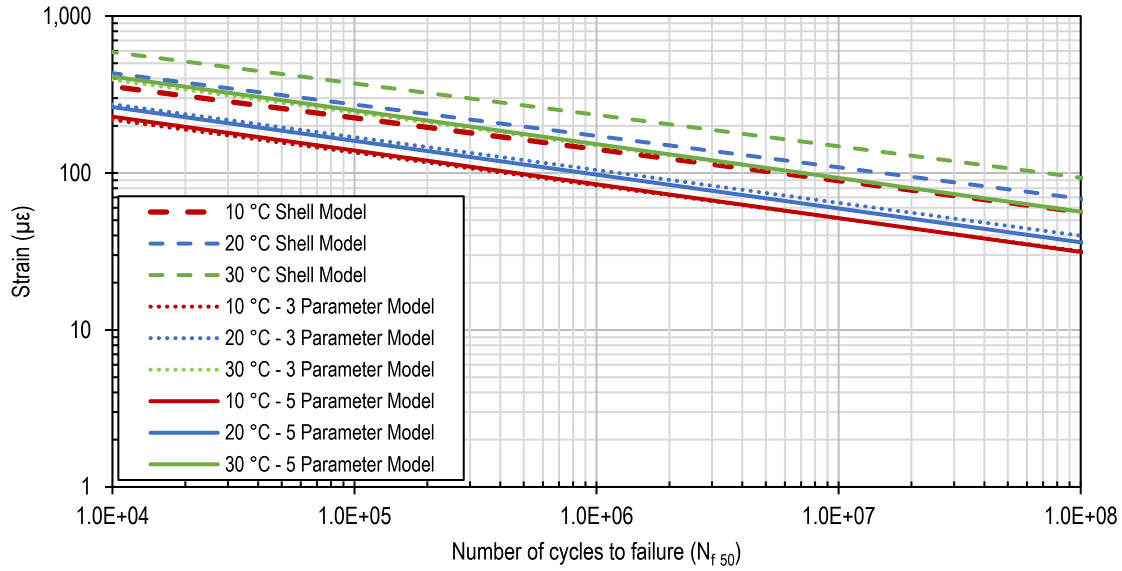


Figure 4.29: Comparison between 3-Parameter Model, 5-Parameter Model and SPDM for Mix 3 (20 mm A35P)

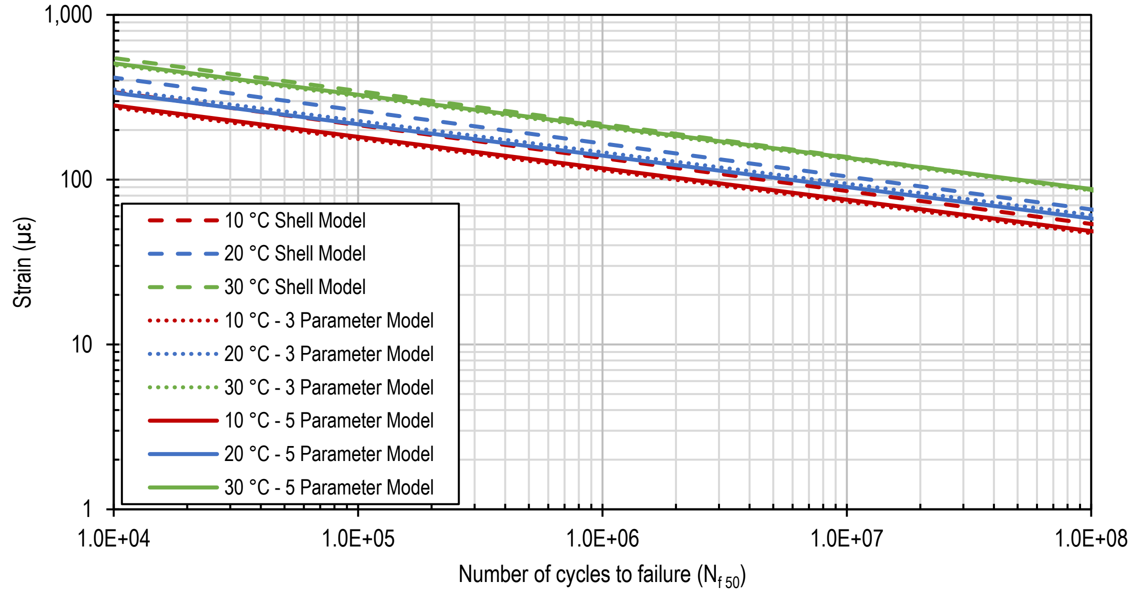
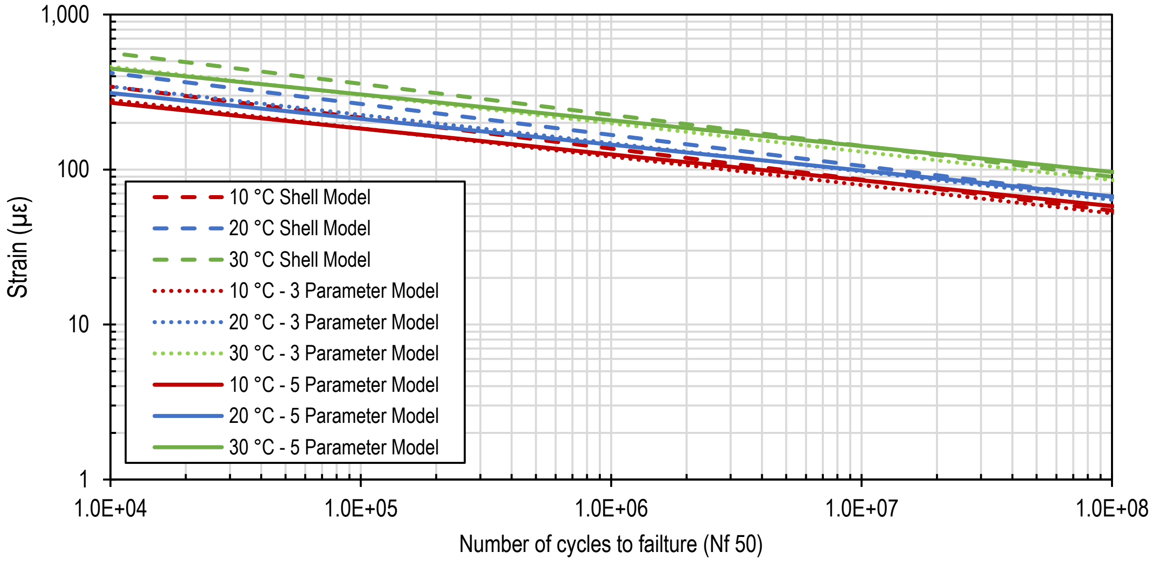


Figure 4.30: Comparison between 3-Parameter Model, 5-Parameter Model and SPDM for Mix 4 (14 mm A35P)



The results indicate that for the asphalt mixes tested, the fatigue lives measured in the laboratory were generally lower than the performance predicted by the Shell laboratory model (with the exception of Mix 3 and 4 tested at 30 °C, which presented similar fatigue performance to what is predicted using the Shell equation). This is in agreement with other recent studies (Austroads 2015 & 2016a) and indicates that the Shell laboratory fatigue model may be inappropriately applied to current Australian asphalt mixes. The Shell equation was developed more than 35 years ago for European mixes. The results obtained indicate that it may not accurately represent WA (or Australian) mixes.

5 INFLUENCE IN PAVEMENT THICKNESS DESIGN

To assess how the use of mix specific measured flexural moduli and laboratory fatigue results obtained would affect the total required asphalt thickness of a typical FDA pavement in WA, a linear elastic analysis (using CIRCLY 5.1) was conducted for a case study with a design traffic speed of 10 km/h and design traffic volume of 5 x 10^7 ESAs. The design traffic speed represents the value that would be used in the design of intersections, which is where FDA would typically be adopted outside of the metropolitan area.

The range of temperatures analysed varied from 25 °C to 40 °C, which is within the range of WMAPTs expected in WA (i.e. Albany 24 °C, Perth 29 °C, Geraldton and Kalgoorlie 30 °C, Dampier and Broome 40 °C).

The following case studies were analysed:

- Case study 1: asphalt modulus and fatigue characterised based on the AGPT Part 2 (Austroads 2012) and ERN9 methodology.
- Case study 2: asphalt modulus and fatigue characterised based on the AGPT Part 2 (Austroads 2012) and varying the modulus of the 14 mm and 20 mm intermediate mixes based on the range of typical values included in AGPT Part 2 (Austroads 2012) as summarised in Table 5.1 .
- Case study 3: asphalt modulus characterised based on the new proposed methodology for both asphalt mixes tested (20 mm and 14 mm asphalt intermediate mix).
- Case study 4: asphalt modulus and fatigue characterised based on the new proposed methodology for both asphalt mixes tested (20 mm and 14 mm asphalt intermediate mix).

Table 5.1: ITT modulus at 40 ms rise time and 25 °C

Mix	Range (MPa)	Typical (MPa)
Mix 1 (20 mm, C320)	3000–7500	5500
Mix 2 (14 mm, C320)	2000–7000	5000

Source: Austroads (2012).

The modulus values were corrected to the minimum design in-situ air voids according to the latest unpublished version of ERN9 (i.e. 6.0% for Mix 1 and 7.0% for Mix 2). The binder volume used in the Shell fatigue model (AGPT Part 2 (Austroads 2012) and ERN9 methodology) was assumed as 10.5% for Mix 1 and 11.0% for Mix 2. In all cases, the design asphalt modulus was limited to a minimum of 1 000 MPa, as per AGPT Part 2 and ERN9.

ERN9 does not allow consideration of beneficial effects of polymers in thickness design procedures. When analysing case studies 1 and 2, the effect of the polymer was therefore not considered.

The subgrade was assumed to comprise of a non-cohesive sand with CBR 12% and the granular subbase was assumed to have a modulus of 150 MPa, in accordance with Table 6.4 of AGPT Part 2 (Austroads 2012), referenced in ERN9. The assumed pavement structure is summarised in Table 5.2.

Table 5.2: Assumed pavement cross section and design modulus values

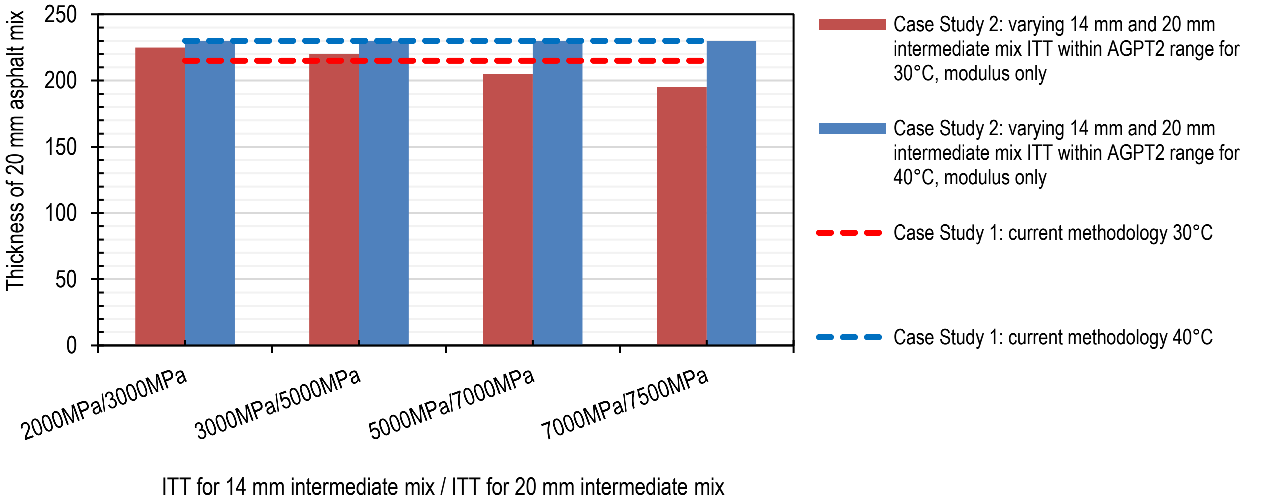
Pavement Layer	Nominal Thickness (mm)	Design Modulus (MPa)																			
		Case Study 1				Case Study 2								Case Study 3				Case Study 4			
WMAPT		**25 °C**	**30 °C**	**35 °C**	**40 °C**	**30 °C**	**30 °C**	**30 °C**	**30 °C**	**40 °C**	**40 °C**	**40 °C**	**40 °C**	**25 °C**	**30 °C**	**35 °C**	**40 °C**	**25 °C**	**30 °C**	**35 °C**	**40 °C**
14 mm intermediate mix ITT/ 20 mm intermediate mix ITT (MPa)*		**5000/ 5500**	**5000/ 5500**	**5000/ 5500**	**5000/ 5500**	**2000/ 3000**	**3000/ 5000**	**5000/ 7000**	**7000/ 7500**	**2000/ 3000**	**3000/ 5000**	**5000/ 7000**	**7000/ 7500**	–	–	–	–	–	–	–	–
14 mm, A35P asphalt mix	40	1680	1130	1000 (750)	1000 (510)	1130	1130	1130	1130	1000 (750)	1000 (750)	1000 (750)	1000 (750)	1680	1130	1000 (750)	1000 (510)	1680	1130	1000 (750)	1000 (510)
14 mm, A35P asphalt mix	50	1930	1290	1000 (870)	1000 (580)	1000 (520)	1000 (770)	1290	1930	1000 (230)	1000 (350)	1000 (580)	1000 (870)	2980	1800	1060	1000 (620)	2980	1800	1060	1000 (620)
20 mm, C320 asphalt mix	variable	2270	1520	1020	1000 (680)	1000 (830)	1380	1940	2210	1000 (370)	1000 (620)	1000 (870)	1000 (990)	2610	1330	1000 (670)	1000 (360)	2610	1330	1000	1000
Granular sub-base	150	150																			
Subgrade	infinite	120																			

Notes: values in parenthesis represent the modulus values calculated without limiting it to a minimum value of 1000 MPa, for the analysis, a modulus value of 1000 MPa was used.

*Initial uncorrected presumptive modulus values based on Table 5.1.

The required thickness of the 20 mm intermediate mix asphalt for Case Study 1 compared to Case Study 2 is summarised in Figure 5.1.

Figure 5.1: Thickness of 20 mm intermediate mix asphalt for a design traffic speed 10 km/h and design traffic volume of approximately 5x10^7 ESAs on a rural National Highway – Case Study 1 and Case Study 2

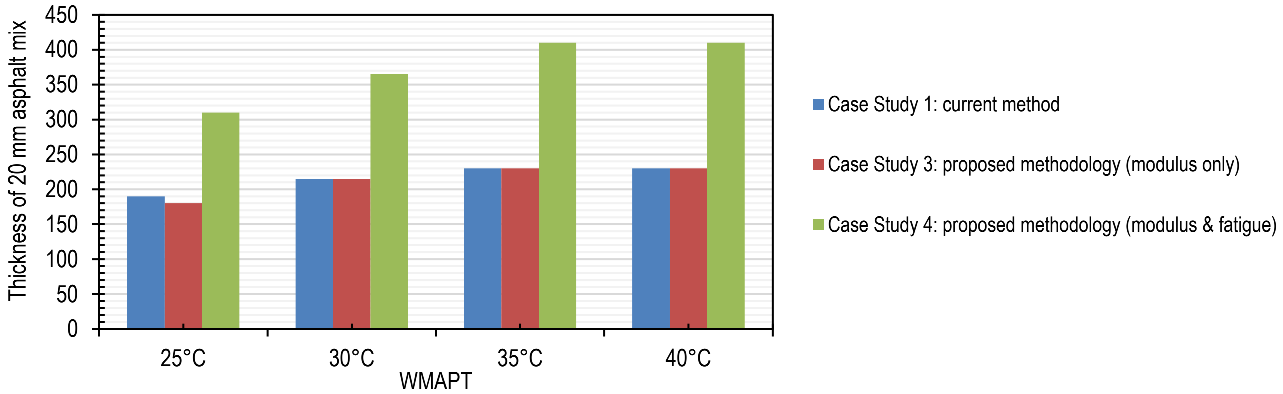


For a WMAPT of 30 °C, varying the presumptive ITT value within the range of typical values provided by AGPT Part 2 made a maximum 30 mm difference in the asphalt thickness design.

For a WMAPT of 40 °C, varying the presumptive ITT value did not affect the thickness design outcomes. This is partly due to the fact that for high temperatures and low design traffic speeds, the modulus values derived from ITT are close or lower than 1000 MPa, and based on the design methodology, the design modulus value is kept at a minimum of 1000 MPa.

The required thickness of the 20 mm intermediate mix asphalt for Case Study 1 compared to Case Study 3 and Case Study 4 is summarised in Figure 5.2.

Figure 5.2: Thickness of 20 mm intermediate mix asphalt for a design traffic speed 10 km/h and design traffic volume of approximately 5x10^7 ESAs on a rural National Highway – Case Study 1, Case Study 3 and Case Study 4



The results indicate that the use of flexural modulus values (Case Study 3) instead of design modulus derived from ITT (Case Study 1), did not alter the thickness design outcomes at WMAPTs of 30 °C or more.

The required asphalt thickness is considerably higher when the analysis is conducted using the flexural modulus and fatigue results presented in Section 4.5 (Case Study 4). This is because the laboratory derived fatigue model obtained, predicts significantly lower fatigue performance when compared to the Shell laboratory model.

However, a relationship between measured laboratory fatigue to an in-service, mix specific fatigue for the mix specific laboratory fatigue model derived in this study has not been explored. It has been assumed that the same reliability factor used in AGPT Part 2 (Austroads 2012) with the Shell equation applies. It is also noted that there is currently no evidence that AGPT Part 2 (Austroads 2012) is resulting in widespread under-design.

6 CONCLUSIONS

The objective of this project is to characterise the stiffness and fatigue behaviour of typical WA asphalt mixes in order to verify whether asphalt thickness design practices can be improved at high WMAPT.

Currently, four WA asphalt mixes have been tested as part of this project, namely:

- 20 mm intermediate mix with C320 binder;
- 14 mm intermediate mix with C320 binder;
- 20 mm intermediate mix with A35P grade polymer modified binder; and
- 14 mm intermediate mix with A35P grade polymer modified binder.

Findings from the test results indicate that:

- For the 20 mm Class 320 binder mix: the use of flexural modulus master curves resulted in lower design modulus at elevated temperatures compared to the values currently assigned in pavement design procedures, following the presumptive values in ERN9 and AGPT Part 2.
- For the 14 mm Class 320 binder mix: the use of flexural modulus master curves resulted in higher design modulus at 30 °C compared to AGPT Part 2 and ERN9 and approximately the same modulus values at 40 °C.
- The use of A35P polymer modified binder did not greatly influence the stiffness of the tested mixes, but it increased the fatigue life of those mixes.
- The Shell International Petroleum (1978) asphalt fatigue model predicts longer fatigue lives than the laboratory performance of the asphalt mixes tested, except for the 20 mm Class A35P binder mix when tested at 30 °C, which presented similar fatigue life compared to the Shell equation prediction.

The discrepancy between the presumptive modulus derived from ITT and the flexural modulus results obtained in this study may be due to a number of possible reasons, such as fundamental differences between indirect tensile modulus and flexural modulus and the fact that the presumptive values represent a range of mixes whereas the results presented in this report represent mix specific results. Test results for temperatures above 30 °C may be affected by non-linearity and creep of the specimens, and should be carefully considered.

The work conducted to-date did not examine laboratory-to-field shift factors, which are influenced by elements such as asphalt healing following rest periods and pavement temperature distribution.

Future work recommended include:

- exploring an interim design approach for the design of asphalt layers, informed by available literature and data, that accounts for the reduced fatigue damage expected to occur at elevated pavement temperatures
- exploring a methodology for laboratory testing of asphalt fatigue with rest periods and conducting testing with rest periods for the same mixes
- recommending a program aimed at improving laboratory-to-field shift functions.

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APPENDIX A MIX DESIGN SPECIFICATION

A.1 JM 53

Table A 1: JM 53 design target PSD

AS sieve size (mm)	Percentage passing by mass (%)	
	Minimum	Maximum
26.5	100	100
19.0	96	100
13.2	80	94
9.5	65	79
6.7	52	66
4.75	45	59
2.36	26	36
1.18	16	26
0.6	13	21
0.3	7	15
0.15	5	10
0.075	4	7

Binder range = 4.4% ± 0.3%

Marshall Properties

Marshall air voids = 3.5% – 5.5%
Voids in mineral aggregate = Not less than 14.0%
Stability = Not less than 8.0kN
Flow = 2.0–4.0 mm

Table A 2: Aggregates

Aggregate type	Aggregate source	Typical aggregate proportions (%)
20 mm granite	Hanson, Red Hill Quarry	14.0
14 mm granite	Hanson, Red Hill Quarry	14.0
10 mm granite	Hanson, Red Hill Quarry	16.0
5 mm granite	Hanson, Red Hill Quarry	24.5
2 mm granite	Hanson, Red Hill Quarry	22.0
Coarse quartz sand	Gingin quartz sand	8.0
Filler (hydrated lime)	Not applicable	1.5

A.2 JM 54

Table A 3: JM 54 design target PSD

AS sieve size (mm)	Percentage passing by mass (%)	
	Minimum	Maximum
26.5	100	100
19.0	100	100
13.2	92	100
9.5	80	94
6.7	65	79
4.75	50	64
2.36	28	38
1.18	21	31
0.6	16	24
0.3	8	16
0.15	5	10
0.075	4	7

Binder range = 4.8% ± 0.3%

Marshall Properties

Marshall air voids = 4.0% – 6.0%
Voids in mineral aggregate = Not less than 14.0%
Stability = Not less than 8.0kN
Flow = 2.0–4.0 mm

Table A 4: Aggregates

Aggregate type	Aggregate source	Typical aggregate proportions (%)
14 mm granite	Hanson, Red Hill Quarry	17.0
10 mm granite	Hanson, Red Hill Quarry	17.0
7 mm granite	Hanson, Red Hill Quarry	7.0
5 mm granite	Hanson, Red Hill Quarry	26.5
2 mm granite	Hanson, Red Hill Quarry	23.0
Coarse quartz sand	Gingin quartz sand	8.0
Filler (hydrated lime)	Not applicable	1.5

APPENDIX B SUMMARY OF AGE OF COMPACTED SLABS

B.1 Age of Slabs at Modulus Testing

Table B 1: Mix 1 (20 mm C320) Modulus summary at 10 Hz and various temperatures

Temperature °C	Beam ID	4453-1	4453-2	4453-3	4453-4	Average
	AV%	4.5	5.4	4.9	5	5.0
5	Modulus (@ 10Hz) MPa	20 184	18 312	18 423	19 235	19 039
	Age (days)	10	10	10	10	
10	Modulus (@ 10Hz) MPa	16 365	15 864	14 313	14 805	15 337
	Age (days)	2	2	2	2	
20	Modulus (@ 10Hz) MPa	9 084	8 243	8 050	8 078	8 364
	Age (days)	2	2	2	2	
30	Modulus (@ 10Hz) MPa	3 511	3 072	2 891	3 085	3 140
	Age (days)	2	2	2	2	
40	Modulus (@ 10Hz) MPa	941	850	858	915	891
	Age (days)	3	3	3	3	

Figure B 1: Mix 1 (20 mm C320) Modulus versus age of slab at testing

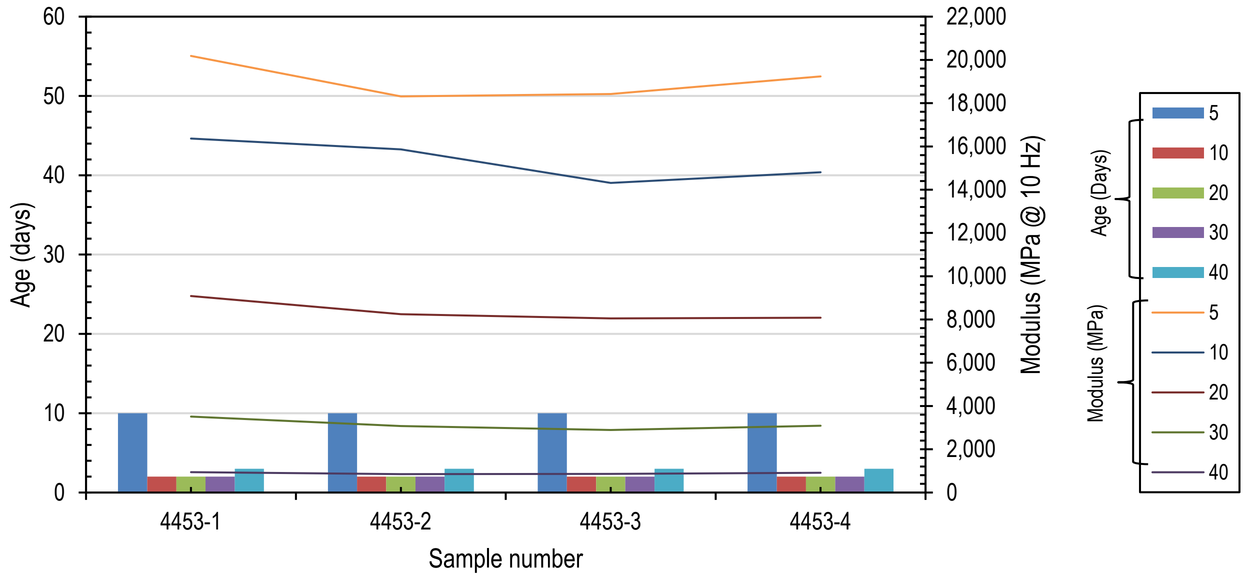


Table B 2: Mix 2 (14 mm C320) modulus summary at 10 Hz and various temperatures

Temperature °C	Beam ID	4873-1	4873-2	4873-3	4873-4	Average
	AV%	5.5	5.5	5.3	4.6	5.2
5	Modulus (@ 10Hz) MPa	20 074	19 401	19 564	18 249	19 322
	Age (days)	15	15	15	15	
10	Modulus (@ 10Hz) MPa	16 270	15 101	15 848	15 241	15 615
	Age (days)	16	16	16	16	
20	Modulus (@ 10Hz) MPa	9 410	8 733	9 284	8 862	9 072
	Age (days)	18	18	18	18	
30	Modulus (@ 10Hz) MPa	4 025	3 930	3 920	3 492	3 842
	Age (days)	21	21	21	21	
40	Modulus (@ 10Hz) MPa	1 374	1 262	1 520	1 359	1 379
	Age (days)	22	22	22	22	

Figure B 2: Mix 2 (14 mm C320) Modulus versus age of slab at testing

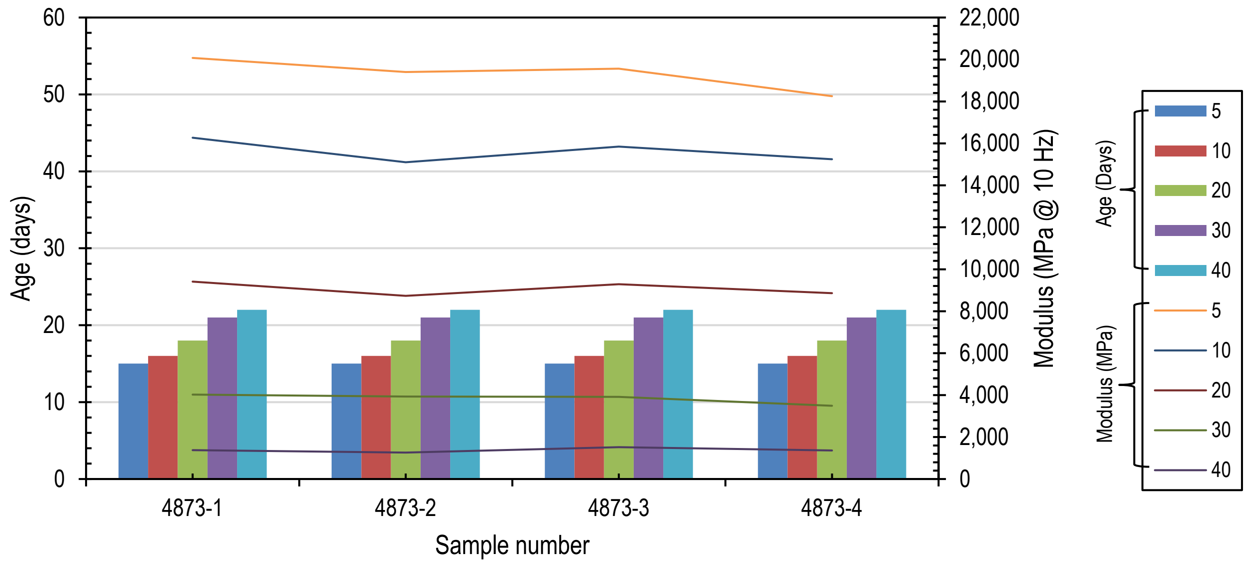


Table B 3: Mix 3 (20 mm A35P) modulus summary at 10 Hz and various temperatures

Temperature °C	Beam ID	5022-3	5022-4	5023-3	5027-1	Average
	AV%	4.8	5.1	5	5	5.0
5	Modulus (@ 10Hz) MPa	18 175	18 241	18 829	16 596	17 960
	Age (days)	21	20	16	11	
10	Modulus (@ 10Hz) MPa	14 459	13 998	14 782	14 458	14 424
	Age (days)	25	25	20	15	
20	Modulus (@ 10Hz) MPa	8 677	8 268	8 411	7 785	8 285
	Age (days)	27	27	22	17	
30	Modulus (@ 10Hz) MPa	4 096	3 914	3 882	3 711	3 901
	Age (days)	28	28	23	18	
40	Modulus (@ 10Hz) MPa	1 714	1 732	1 728	1 693	1 717
	Age (days)	32	32	28	23	

Figure B 3: Mix 3 (20 mm A35P) Modulus versus age of slab at testing

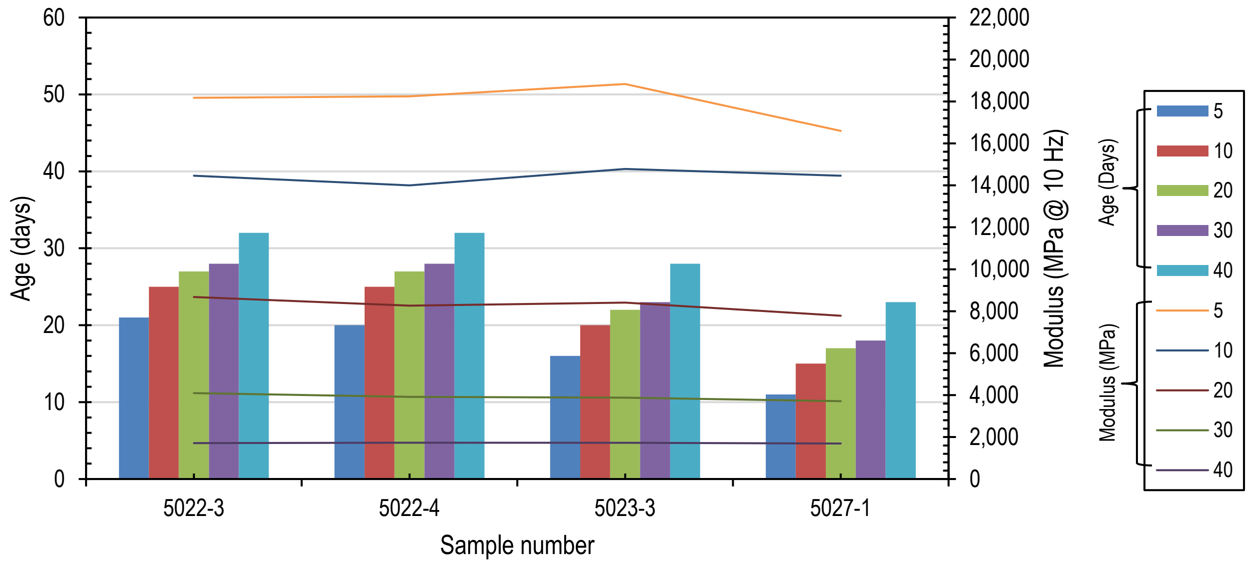
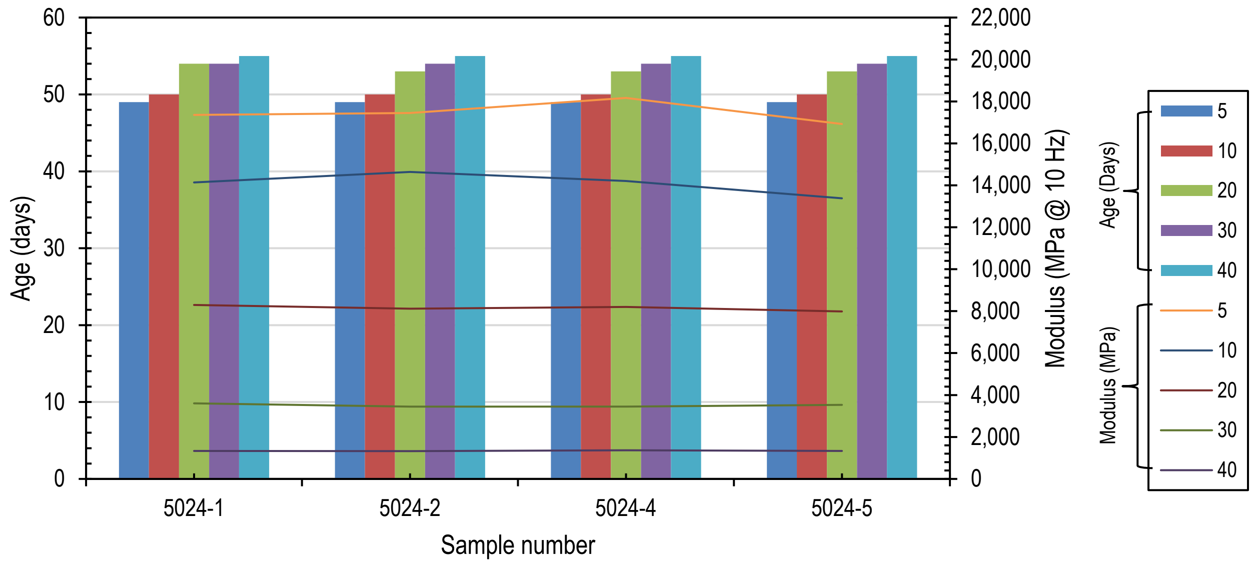


Table B 4: Mix 4 (14 mm A35P) modulus summary at 10 Hz and various temperatures

Temperature °C	Beam ID	5024-1	5024-2	5024-4	5024-5	Average
	AV%	4.6	4.9	4.5	4.7	4.7
5	Modulus (@ 10Hz) MPa	17 357	17 451	18 167	16 924	17 475
	Age (days)	49	49	49	49	
10	Modulus (@ 10Hz) MPa	14 136	14 638	14 209	13 380	14 091
	Age (days)	50	50	50	50	
20	Modulus (@ 10Hz) MPa	8 295	8 118	8 202	7 986	8 150
	Age (days)	54	53	53	53	
30	Modulus (@ 10Hz) MPa	3 604	3 449	3 448	3 532	3 508
	Age (days)	54	54	54	54	
40	Modulus (@ 10Hz) MPa	1 331	1 324	1 365	1 337	1 339
	Age (days)	55	55	55	55	

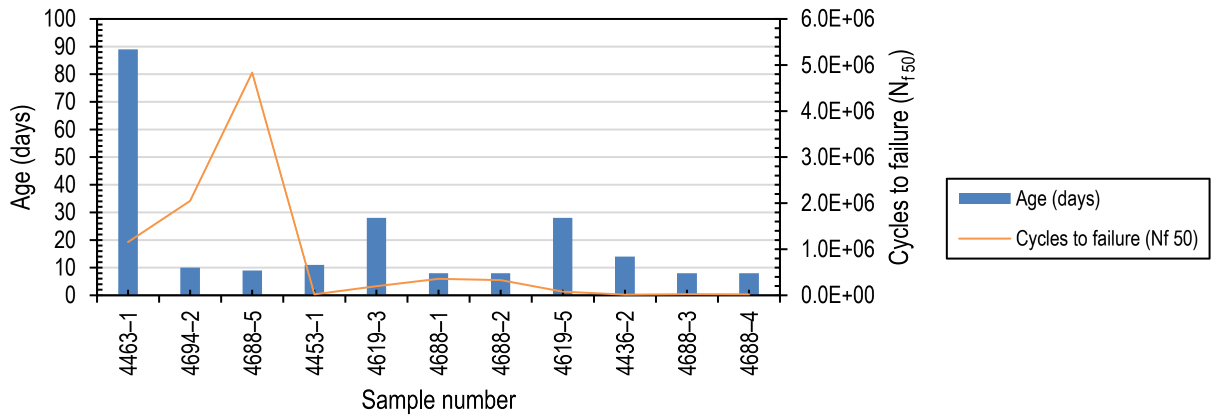
Figure B 4: Mix 4 (14 mm A35P) Modulus versus age of slab at testing



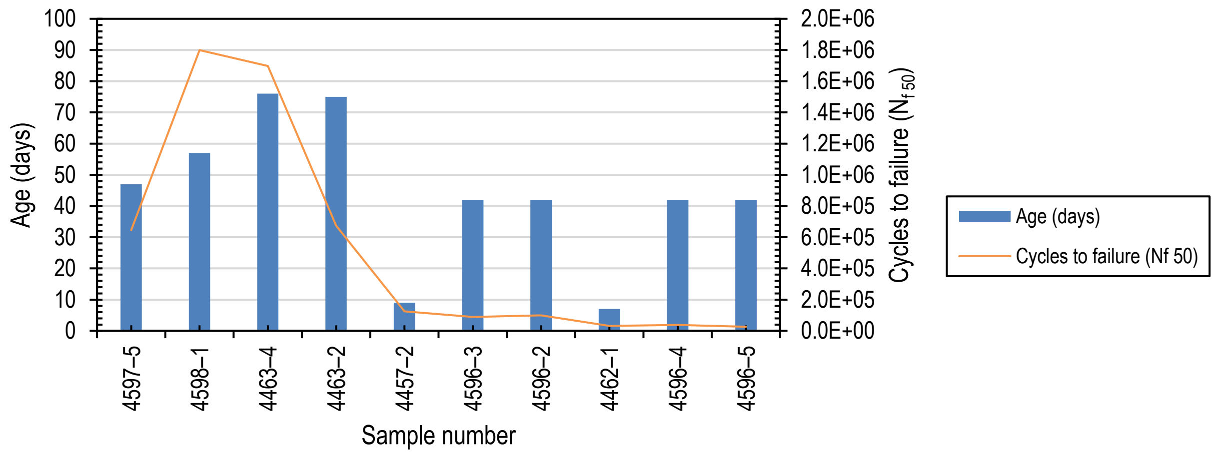
B.2 Age of Slabs at Fatigue Testing

Figure B 5: Mix 1 (20 mm C320) Initial modulus summary and various temperatures versus age of slabs

At 10 °C



At 20 °C



At 30 °C

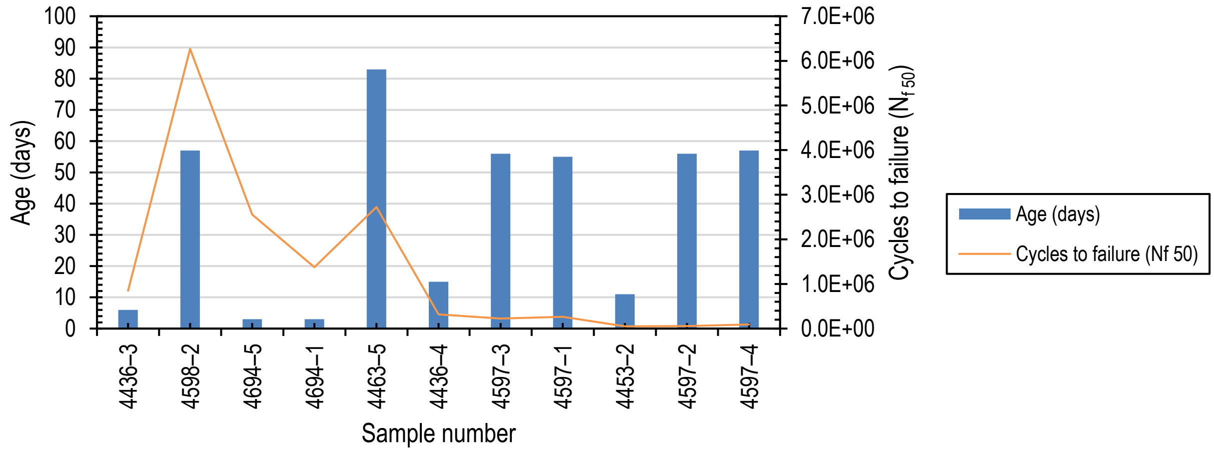
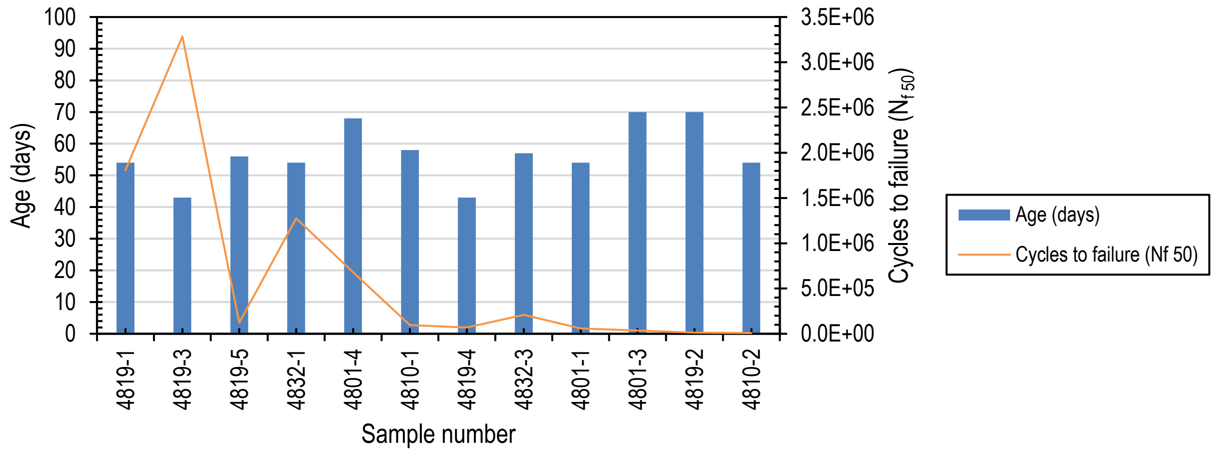
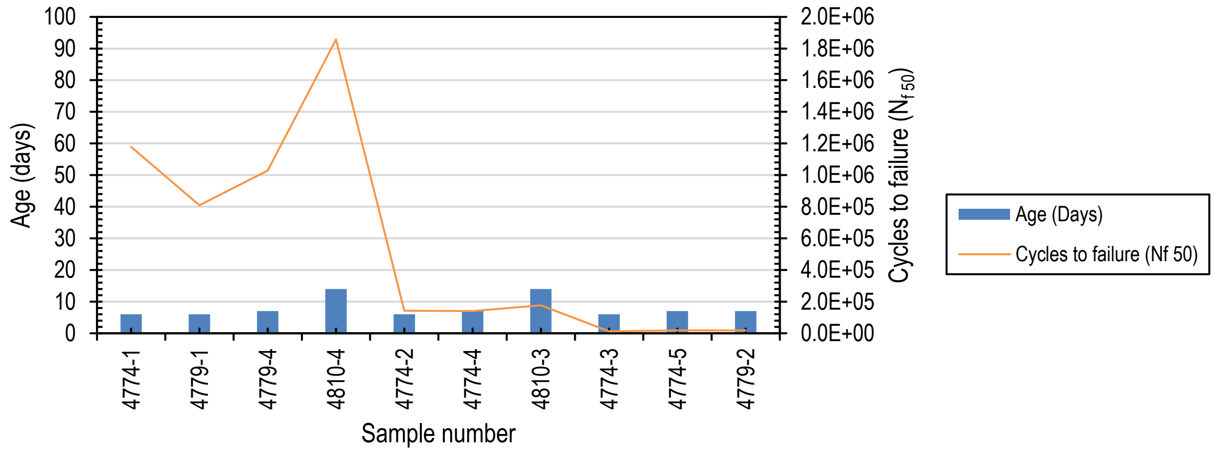


Figure B 6: Mix 2 (14 mm C320) Initial modulus summary and various temperatures versus age of slabs

At 10 °C



At 20 °C



At 30 °C

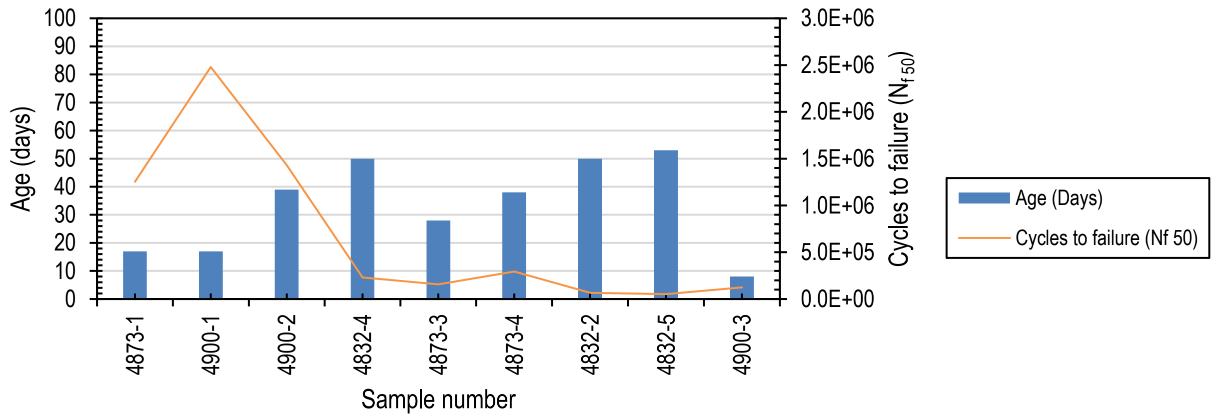
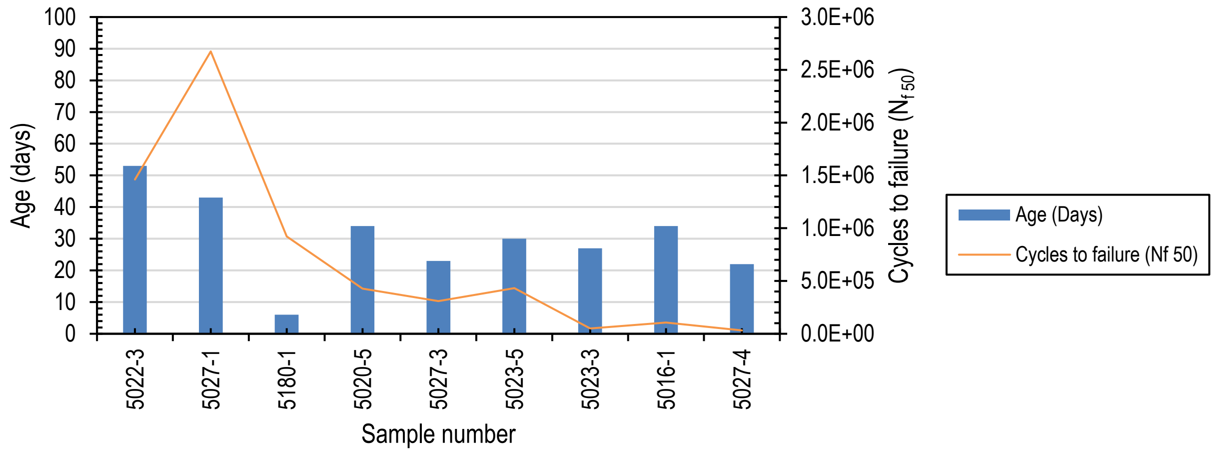
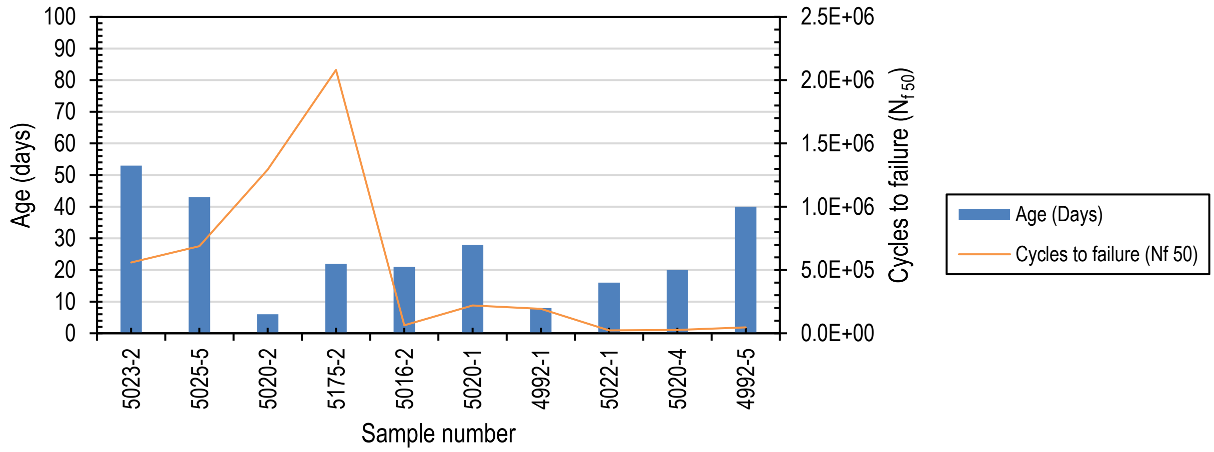


Figure B 7: Mix 3 (20 mm A35P) Initial modulus summary and various temperatures versus age of slabs

At 10 °C



At 20 °C



At 30 °C

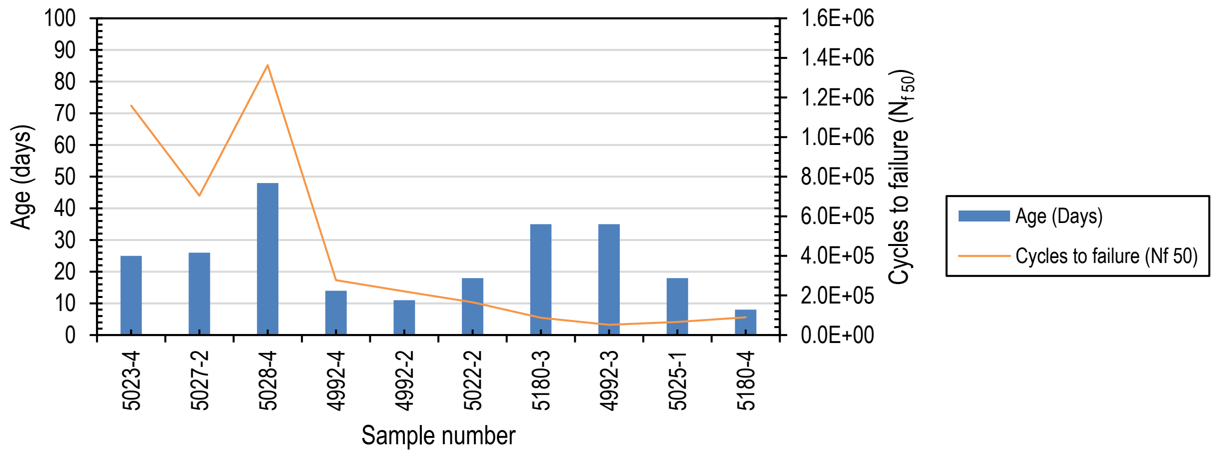
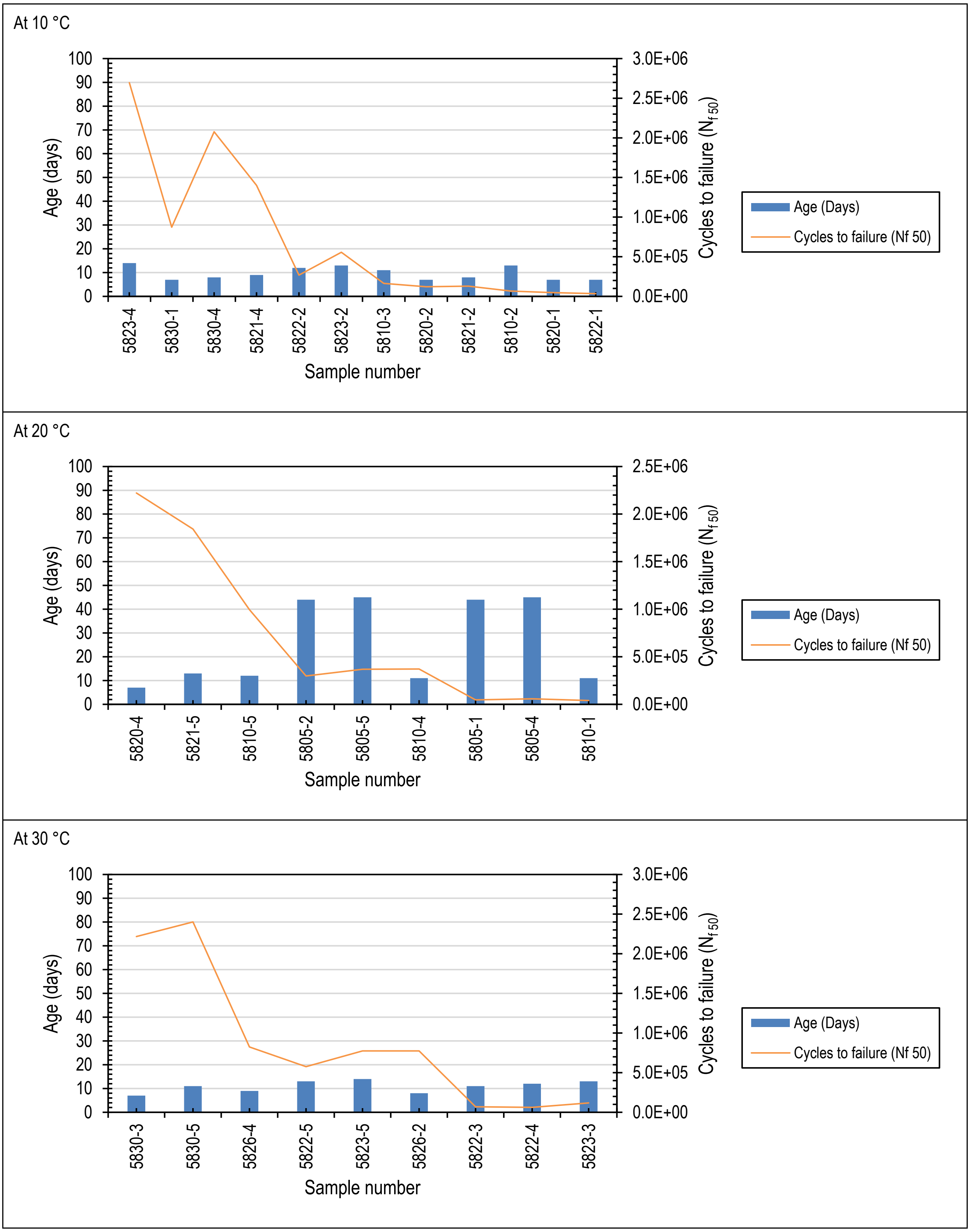


Figure B 8: Mix 4 (14 mm A35P) Initial modulus summary and various temperatures versus age of slabs



APPENDIX C MODULUS RESULTS

Table C 1: Flexural modulus results Mix 1 (20 mm C320)

Temperature (°C)	Frequency (Hz)	Flexural modulus for replicate specimens (MPa)				Statistics		
		4453–1	4453–2	4453–3	4453–4	Mean	STDEV	CoV (%)
5	0.1	11 056	9 017	10 262	10 892	10 307	925	8.98
	0.5	13 979	12 356	13 166	13 128	13 157	663	5.04
	1	15 496	13 878	14 571	14 502	14 612	667	4.56
	1	15 459	13 863	14 397	14 635	14 589	664	4.55
	3	17 942	16 121	16 455	16 780	16 825	792	4.71
	5	18 952	17 076	17 300	17 965	17 823	842	4.72
	10	20 184	18 312	18 423	19 235	19 039	867	4.56
	15	20 568	18 895	18 793	19 406	19 416	814	4.19
	20	20 666	19 028	18 824	19 576	19 524	825	4.23
10	0.1	8 072	6 971	5 735	6 739	6 879	959	13.94
	0.5	9 598	8 748	8 354	9 051	8 938	525	5.87
	1	11 141	10 444	9 692	10 263	10 385	597	5.75
	1	11 067	10 384	9 613	10 302	10 342	595	5.75
	3	13 495	12 712	11 923	12 338	12 617	668	5.30
	5	14 743	14 240	12 936	13 485	13 851	800	5.77
	10	16 365	15 864	14 313	14 805	15 337	943	6.15
	15	16 979	16 627	15 092	15 512	16 053	895	5.58
	20	17 019	17 267	15 424	15 714	16 356	922	5.64
20	0.1	2 356	2 040	2 075	2 145	2 154	142	6.57
	0.5	3 505	3 064	3 139	2 969	3 169	234	7.40
	1	4 627	3 994	4 076	3 944	4 160	316	7.59
	1	4 566	4 124	4 057	3 963	4 178	267	6.40
	3	6 594	5 917	5 775	5 728	6 004	402	6.69
	5	7 533	6 887	6 724	6 678	6 956	395	5.68
	10	9 084	8 243	8 050	8 078	8 364	488	5.83
	15	9 835	8 966	8 848	8 870	9 130	473	5.18
	20	10 127	9 280	9 091	9 147	9 411	484	5.14
30	0.1	626	528	479	478	528	70	13.18
	0.5	866	738	668	718	748	84	11.28
	1	1 158	1 056	920	984	1 030	102	9.92
	1	1 131	989	903	962	996	97	9.71
	3	2 106	1 852	1 726	1 784	1 867	167	8.97
	5	2 604	2 348	2 206	2 273	2 358	174	7.38
	10	3 511	3 072	2 891	3 085	3 140	263	8.37
	15	4 025	3 491	3 298	3 502	3 579	312	8.71
	20	4 307	3 781	3 538	3 783	3 852	324	8.42

Temperature (°C)	Frequency (Hz)	Flexural modulus for replicate specimens (MPa)				Statistics		
		4453–1	4453–2	4453–3	4453–4	Mean	STDEV	CoV (%)
40	0.1	200	164	165	187	179	18	9.82
	0.5	247	203	180	212	211	28	13.21
	1	266	245	231	250	248	14	5.82
	1	257	220	213	244	234	21	8.79
	3	469	419	424	458	443	25	5.59
	5	665	586	604	633	622	35	5.56
	10	941	850	858	915	891	44	4.95
	15	1 161	1 063	1 075	1 108	1 102	44	3.98
	20	1 323	1 212	1 237	1 282	1 264	49	3.89

Table C 2: Flexural modulus results Mix 2 (14 mm C320)

Temperature (°C)	Frequency (Hz)	Flexural modulus for replicate specimens (MPa)				Statistics		
		4873–1	4873–2	4873–3	4873–4	Mean	STDEV	CoV (%)
5	0.1	10 699	10 549	10 336	8 742	10 082	905	8.98
	0.5	13 994	12 952	13 428	11 862	13 059	905	6.93
	1	15 348	14 820	14 641	13 212	14 505	913	6.29
	3	17 556	17 087	16 955	15 642	16 810	820	4.88
	5	18 639	18 236	18 050	16 661	17 897	860	4.80
	10	20 074	19 401	19 564	18 249	19 322	771	3.99
	15	20 723	19 993	20 401	19 032	20 037	734	3.66
	20	20 931	20 152	20 741	19 108	20 233	820	4.05
10	0.1	7 087	6 402	6 743	6 943	6 794	297	4.37
	0.5	9 734	8 898	9 457	9 300	9 347	349	3.74
	1	11 148	10 196	10 780	10 574	10 675	398	3.73
	3	13 344	12 470	13 211	12 766	12 948	403	3.11
	5	14 648	13 655	14 453	13 778	14 134	491	3.47
	10	16 270	15 101	15 848	15 241	15 615	544	3.48
	15	17 048	15 769	16 705	16 072	16 399	583	3.55
	20	17 330	15 999	17 095	16 330	16 689	627	3.76
20	0.1	2 877	2 648	2 823	2 841	2 797	102	3.65
	0.5	4 059	3 763	3 899	3 912	3 908	121	3.10
	1	5 023	4 669	5 005	4 810	4 877	169	3.46
	3	6 891	6 505	6 919	6 627	6 736	202	3.00
	5	7 831	7 349	7 807	7 589	7 644	225	2.94
	10	9 410	8 733	9 284	8 862	9 072	326	3.59
	15	10 332	9 518	10 075	9 797	9 931	351	3.54
	20	10 762	9 825	10 727	10 161	10 369	455	4.39
30	0.1	818	717	770	650	739	72	9.76
	0.5	1 141	1 028	1 132	962	1 066	86	8.08
	1	1 568	1 399	1 487	1 311	1 441	111	7.70
	3	2 580	2 336	2 514	2 211	2 410	168	6.98
	5	3 158	2 994	3 147	2 747	3 012	192	6.36
	10	4 025	3 930	3 920	3 492	3 842	238	6.19
	15	4 547	4 343	4 415	3 920	4 306	271	6.29
	20	4 776	4 574	4 678	4 243	4 568	232	5.07
40	0.1	234	223	254	249	240	14	5.90
	0.5	289	290	318	290	297	14	4.78
	1	366	355	449	387	389	42	10.79
	3	715	665	821	722	731	65	8.94
	5	976	911	1 118	993	1 000	87	8.66
	10	1 374	1 262	1 520	1 359	1 379	106	7.72

Temperature (°C)	Frequency (Hz)	Flexural modulus for replicate specimens (MPa)				Statistics		
		4873–1	4873–2	4873–3	4873–4	Mean	STDEV	CoV (%)
	15	1 630	1 534	1 810	1 596	1 643	119	7.22
	20	1 879	1 780	2 153	1 924	1 934	158	8.16

Table C 3: Flexural modulus results Mix 3 (20 mm A35P)

Temperature (^oC)	Frequency (Hz)	Flexural modulus for replicate specimens (MPa)				Statistics		
		5022-3	5022-4	5023-3	5027-1	Mean	STDEV	CoV (%)
5	0.1	9 960	10 012	10 151	9 068	9 798	493	5.0
	0.5	12 710	12 769	13 125	11 731	12 584	597	4.7
	1	14 011	13 896	14 227	12 705	13 710	684	5.0
	3	15 950	15 907	16 394	14 516	15 692	814	5.2
	5	17 044	16 941	17 341	15 524	16 713	810	4.8
	10	18 175	18 241	18 829	16 596	17 960	956	5.3
	15	18 646	19 189	19 036	16 903	18 444	1 052	5.7
	20	18 842	19 346	19 158	17 088	18 609	1 035	5.6
10	0.1	7 119	6 729	6 720	6 783	6 838	190	2.8
	0.5	9 037	8 530	9 041	8 834	8 861	241	2.7
	1	10 191	9 697	10 193	10 057	10 035	234	2.3
	3	12 134	11 558	12 332	12 002	12 007	328	2.7
	5	13 090	12 790	13 340	13 009	13 057	227	1.7
	10	14 459	13 998	14 782	14 458	14 424	323	2.2
	15	15 075	14 755	15 481	15 118	15 107	297	2.0
	20	15 270	14 919	15 631	15 389	15 302	296	1.9
20	0.1	3 346	3 065	3 122	2 687	3 055	274	9.0
	0.5	4 246	4 093	4 008	3 891	4 060	149	3.7
	1	5 142	4 809	4 724	4 588	4 816	236	4.9
	3	6 610	6 264	6 310	5 964	6 287	264	4.2
	5	7 426	7 196	7 109	6 697	7 107	304	4.3
	10	8 677	8 268	8 411	7 785	8 285	374	4.5
	15	9 232	8 935	9 031	8 436	8 909	338	3.8
	20	9 518	9 226	9 269	8 779	9 198	308	3.3
30	0.1	1 444	1 190	1 145	1 043	1 206	170	14.1
	0.5	1 606	1 499	1 488	1 376	1 492	94	6.3
	1	2 025	1 871	1 859	1 726	1 870	122	6.5
	3	2 900	2 684	2 689	2 448	2 680	185	6.9
	5	3 344	3 206	3 142	2 997	3 172	144	4.5
	10	4 096	3 914	3 882	3 711	3 901	158	4.0
	15	4 550	4 389	4 324	4 165	4 357	159	3.7
	20	4 837	4 624	4 588	4 407	4 614	176	3.8
40	0.1	457	459	530	482	482	34	7.0
	0.5	574	607	657	618	614	34	5.6
	1	697	733	770	740	735	30	4.1
	3	1 095	1 095	1 105	1 069	1 091	15	1.4
	5	1 355	1 370	1 355	1 329	1 352	17	1.3
	10	1 714	1 732	1 728	1 693	1 717	18	1.0

Temperature (°C)	Frequency (Hz)	Flexural modulus for replicate specimens (MPa)				Statistics		
		5022-3	5022-4	5023-3	5027-1	Mean	STDEV	CoV (%)
	15	1 954	1 982	1 960	1 916	1 953	27	1.4
	20	2 179	2 167	2 169	2 086	2 150	43	2.0

Table C 4: Flexural modulus results Mix 4 (14 mm A35P)

Temperature (°C)	Frequency (Hz)	Flexural modulus for replicate specimens (MPa)				Statistics		
		5024-1	5024-2	5024-4	5024-5	Mean	STDEV	CoV (%)
5	0.1	9 405	9 031	9 476	9 349	9 315	196	2.1
	0.5	11 894	11 972	12 108	11 911	11 971	97	0.8
	1	13 028	13 181	13 383	13 131	13 181	149	1.1
	3	15 104	14 901	15 709	14 919	15 158	378	2.5
	5	16 140	15 866	16 789	15 865	16 165	436	2.7
	10	17 357	17 451	18 167	16 924	17 475	515	2.9
	15	18 116	17 784	19 057	17 639	18 149	637	3.5
	20	18 124	17 816	19 147	17 820	18 227	630	3.5
10	0.1	6 180	6 520	6 885	6 304	6 472	309	4.8
	0.5	8 639	9 040	8 971	8 029	8 670	462	5.3
	1	9 848	10 245	10 091	9 263	9 862	431	4.4
	3	11 804	12 186	11 792	11 017	11 700	491	4.2
	5	12 773	13 181	12 783	11 961	12 675	512	4.0
	10	14 136	14 638	14 209	13 380	14 091	523	3.7
	15	14 932	15 338	14 660	13 931	14 715	592	4.0
	20	15 173	15 556	14 881	14 296	14 977	531	3.5
20	0.1	3 062	3 225	3 013	2 918	3 055	128	4.2
	0.5	4 008	3 931	4 028	3 947	3 979	47	1.2
	1	4 777	4 652	4 802	4 700	4 733	69	1.5
	3	6 251	6 113	6 200	6 160	6 181	59	0.9
	5	7 113	6 959	7 032	6 841	6 986	115	1.7
	10	8 295	8 118	8 202	7 986	8 150	131	1.6
	15	8 856	8 708	8 829	8 588	8 745	123	1.4
	20	9 269	9 034	9 055	8 978	9 084	128	1.4
30	0.1	961	984	966	919	958	28	2.9
	0.5	1 305	1 321	1 326	1 334	1 322	12	0.9
	1	1 634	1 618	1 585	1 641	1 620	25	1.5
	3	2 380	2 310	2 326	2 378	2 349	36	1.5
	5	2 854	2 767	2 743	2 886	2 813	68	2.4
	10	3 604	3 449	3 448	3 532	3 508	75	2.1
	15	3 971	3 910	3 909	3 912	3 926	30	0.8
	20	4 221	4 061	4 128	4 142	4 138	66	1.6
40	0.1	348	371	365	343	357	13	3.7
	0.5	443	499	462	451	464	25	5.3
	1	534	561	552	539	547	12	2.2
	3	799	830	810	815	814	13	1.6
	5	996	1 013	1 019	1 008	1 009	10	1.0
	10	1 331	1 324	1 365	1 337	1 339	18	1.3

Temperature (^oC)	Frequency (Hz)	Flexural modulus for replicate specimens (MPa)				Statistics		
		5024-1	5024-2	5024-4	5024-5	Mean	STDEV	CoV (%)
	15	1 554	1 488	1 551	1 552	1 536	32	2.1
	20	1 751	1 707	1 762	1 763	1 746	26	1.5

APPENDIX D FATIGUE RESULTS

Table D 1: Fatigue results Mix 1 (20 mm C320)

Temperature (°C)	Sample #	Strain level (με)	$N_{f\,50}$	Air Void (%)	Age (days)	Initial Modulus (MPa)
10	4463–1	80	1 155 641	4.8	89	16 798
	4694–2	80	2 050 416	4.6	10	16 385
	4688–5	80	4 835 669	4.5	9	16 116
	4453–1	150	21 141	4.5	11	19 538
	4619–3	100	196 735	4.5	28	17 719
	4688–1	100	356 855	4.5	8	17 322
	4688–2	100	328 980	4.5	8	18 705
	4619–5	140	81 110	4.6	28	17 746
	4436–2	200	12 937	5.5	14	12 700
	4688–3	200	23 056	4.8	8	18 262
	4688–4	200	21 312	4.5	8	18 114
20	4597–5	100	645 529	5.5	47	9 113
	4598–1	90	1 799 364	4.5	57	10 222
	4463–4	90	1 697 499	4.5	76	8 958
	4463–2	90	674 076	4.5	75	9 912
	4457–2	150	124 302	4.5	9	10 463
	4596–3	150	88 462	5.3	42	8 586
	4596–2	150	99 627	5.2	42	10 759
	4462–1	200	32 015	5.5	7	6 457
	4596–4	200	37 216	5	42	8 413
	4596–5	200	26 083	4.9	42	9 074
30	4436–3	115	850 326	5.3	6	3 442
	4598–2	100	6 267 057	4.5	57	4 630
	4694–5	110	2 555 855	5.1	3	3 810
	4694–1	110	1 374 087	5.2	3	3 067
	4463–5	110	2 724 734	4.5	83	3 764
	4436–4	200	316 941	5.5	15	2 681
	4597–3	200	224 136	5	56	3 483
	4597–1	200	264 932	5.1	55	3 071
	4453–2	250	50 682	5.4	11	3 348
	4597–2	250	55 443	4.5	56	3 225
	4597–4	250	91 189	4.5	57	3 385

Low strain, medium strain, high strain.

Table D 2: Fatigue results Mix 2 (14mm C320)

Temperature (°C)	Sample #	Strain level (με)	$N_{f\,50}$	Air Void (%)	Age (days)	Initial Modulus (MPa)
10	4819-1	80	1 808 157	4.5	54	15 888
	4819-3	80	3 285 387	5.2	43	17 787
	4819-5	80	121 948	5.4	56	16 752
	4832-1	80	1 275 681	4.6	54	14 857
	4801-4	100	678 777	4.9	68	17 795
	4810-1	140	95 597	4.7	58	17 369
	4819-4	140	68 411	5.5	43	16 021
	4832-3	140	209 375	4.7	57	15 328
	4801-1	180	59 141	4.5	54	14 908
	4801-3	180	36 309	5.5	70	16 239
	4819-2	180	12 348	4.6	70	18 309
	4810-2	200	8 326	4.5	54	15 888
20	4774-1	100	1 178 248	5.0	6	10 802
	4779-1	100	809 295	5.4	6	8 611
	4779-4	100	1 028 241	5.2	7	8 449
	4810-4	100	1 857 457	5.0	14	7 815
	4774-2	150	142 125	4.8	6	8 814
	4774-4	150	140 544	4.9	7	8 502
	4810-3	150	177 065	5.0	14	10 503
	4774-3	250	12 476	5.3	6	8 213
	4774-5	250	18 106	4.7	7	7 924
	4779-2	250	18 446	4.9	7	10 136
30	4873-1	145	1 253 248	4.7	17	3 479
	4900-1	145	2 479 289	5.2	17	3 202
	4900-2	145	1 431 072	5.5	39	3 835
	4832-4	200	230 354	4.5	50	7 037
	4873-3	200	156 277	5.3	28	3 994
	4873-4	200	293 178	4.6	38	4 076
	4832-2	250	67 033	4.5	50	5 236
	4832-5	250	52 393	4.5	53	7 196
	4900-3	250	124 758	5.5	8	2 928

Low strain, medium strain, high strain.

Table D 3: Fatigue results Mix 3 (20 mm A35P)

Temperature (°C)	Sample #	Strain level (με)	$N_{f\,50}$	Air Void (%)	Age (days)	Initial Modulus (MPa)
10	5022-3	110	1 460 808	4.8	53	15 366
	5027-1	110	2 674 240	5	43	12 978
	5180-1	110	921 763	4.8	6	16 030
	5020-5	140	427 772	4.5	34	17 890
	5027-3	140	308 678	5.0	23	16 270
	5023-5	140	431 949	4.5	30	17 543
	5023-3	200	50 950	4.5	27	17 074
	5016-1	200	106 650	5.3	34	13 059
	5027-4	200	31 688	5.2	22	16 541
20	5023-2	140	559 615	4.8	53	15 366
	5025-5	140	688 130	5	43	12 978
	5020-2	140	1 292 860	4.8	6	16 030
	5175-2	140	2 081 060	4.5	22	8 379
	5016-2	200	63 207	4.5	21	8 982
	5020-1	200	219 460	5.3	28	7 997
	4992-1	200	193 030	4.5	8	9 703
	5022-1	280	22 859	4.5	16	9 069
	5020-4	280	25 895	4.7	20	7 173
	4992-5	280	47 130	5.1	40	8 229
30	5023-4	210	1 159 138	5.5	25	8 070
	5027-2	210	704 170	5.5	26	6 696
	5028-4	210	1 363 540	4.7	48	8 482
	4992-4	280	276 940	4.9	14	3 255
	4992-2	280	220 320	4.6	11	3 716
	5022-2	280	165 180	4.6	18	3 763
	5180-3	350	87 052	5.4	35	3 525
	4992-3	350	51 690	4.5	35	3 815
	5025-1	350	65 740	4.8	18	3 625
	5180-4	350	89 360	4.5	8	3 510

Low strain, medium strain, high strain.

Table D 4: Fatigue results Mix 4 (14 mm A35P)

Temperature (°C)	Sample #	Strain level (με)	$N_{f\,50}$	Air Void (%)	Age (days)	Initial Modulus (MPa)
10	5823-4	110	2 695 548	5.5	14	14 391
	5830-1	110	873 876	5.1	7	14 227
	5830-4	110	2 076 676	5.3	8	13 652
	5821-4	130	1 400 798	5.5	9	13 729
	5822-2	130	268 487	5.1	12	14 596
	5823-2	130	556 016	5.1	13	14 887
	5810-3	180	164 230	5.2	11	12 456
	5820-2	180	120 797	5.2	7	14 123
	5821-2	180	129 111	5.2	8	13 728
	5810-2	210	66 953	4.9	13	13 751
	5820-1	210	47 379	5.1	7	13 982
	5822-1	210	34 199	4.5	7	14 586
20	5820-4	132	2 221 730	4.6	7	8 207
	5821-5	132	1 842 406	5.1	13	7 816
	5810-5	140	995 078	5.2	12	8 859
	5805-2	180	298 382	5.2	44	7 879
	5805-5	180	369 298	4.5	45	8 386
	5810-4	180	371 481	5.1	11	7 831
	5805-1	250	47 248	5.0	44	8 261
	5805-4	250	57 804	4.7	45	7 831
	5810-1	250	40 163	5.0	11	7 603
30	5830-3	195	2 217 307	5.4	7	3 569
	5830-5	195	2 401 487	5.3	11	3 426
	5826-4	205	823 940	5.3	9	4 004
	5822-5	230	574 880	5.2	13	3 558
	5823-5	230	774 610	4.7	14	3 849
	5826-2	230	773 980	4.7	8	3 733
	5822-3	290	68 710	5.5	11	3 407
	5822-4	290	62 030	5.5	12	3 420
	5823-3	290	118 170	5.3	13	3 722

Low strain, medium strain, high strain.

APPENDIX E FATIGUE MODEL CONSIDERING THE AGE OF THE BEAMS

To ensure that the age of the specimens did not affect the asphalt fatigue test results, a model was created considering age of the beams as one of the variables. A coefficient for the age of the beams (c_6) was added to the 5-parameter model presented in the report (refer to Equation 8). The proposed 6-Parameter model is presented in Equation E.1:

$$\ln\left(N_{f(50)}\right) = c_1 \cdot ln^3(E) + c_2 \cdot ln^2(E) + c_3 \cdot \ln(E) + c_4 + c_5 \cdot \ln \varepsilon + c_6 \cdot \ln(age) \qquad \text{E.1}$$

where

$N_{f(50)}$ = number of load cycles to a 50% reduction in modulus

E = modulus of the asphalt (MPa)

ε = strain in µm/m (microstrain)

age = age of the beams

c_1, to c_5 = regression coefficients

A comparison between the coefficients obtained for the 5-Parameter model and the 6-Parameter model for the 20 mm mix with A35P PMB is presented in Table E 1.

Table E 1: Regression coefficients for 5-Parameter and 6-Parameter models

Model	Mix type	n	c_1	c_2	c_3	c_4	c_5	c_6	σ_y
5-Parameter	A35P 20 mm	29	0.423	–10.447	82.87	–168.4	–5.245	NA	0.41
6-Parameter	A35P 20 mm	29	0.423	–10.447	82.86	–168.4	–5.245	0.028	0.40

The coefficient related to the age of the beams is small and does not significantly improve the standard deviation of residuals (σ_y). Therefore, the age of the beams is not considered to influence the results.