FEASIBILITY OF RECLAIMED ASPHALT PAVEMENT AS AGGREGATE IN PORTLAND CEMENT CONCRETE PAVEMENTS
Phase II: Field Demonstration

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prepared by
Michael Berry
Kelsey Dalton
Flynn Murray

Civil Engineering Department
Western Transportation Institute
Montana State University- Bozeman

RESEARCH PROGRAMS

MDT

PHASE II: FEASIBILITY OF RECLAIMED ASPHALT PAVEMENT AS AGGREGATE IN PORTLAND CEMENT CONCRETE PAVEMENTS FIELD DEMONSTRATION

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Prepared by:

Michael Berry, PhD
Assistant Professor

Kelsey Dalton, MS
Graduate Research Assistant

and

Flynn Murray
Graduate Research Assistant

of the

Civil Engineering Department
Western Transportation Institute
College of Engineering
Montana State University – Bozeman

Prepared for:

Montana Department of Transportation
Research Programs
2701 Prospect Avenue
P.O. Box 201001
Helena, Montana 59620-1001

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16. Abstract

This research was focused on evaluating the feasibility of using minimally processed reclaimed asphalt pavement (RAP) as aggregate replacement in concrete pavements. An initial phase of research demonstrated that concretes with up to 50 percent of the fine aggregates and 100 percent of the coarse aggregates replaced with RAP were suitable for concrete pavement. However, the field performance of these mixes was not evaluated. Further, these initial mixes contained a significant proportion of cement in order to achieve the desired performance criteria, hindering the economic benefit of using this recycled material in concrete. Therefore, the research discussed herein focused on: (1) evaluating the field performance of RAP concrete through a field demonstration project, and (2) reducing the amount of cement required in the RAP concrete by including water-reducing admixtures. As part of the field demonstration project, two RAP-concrete slabs were cast on a roadway near Lewistown, MT, and monitored for damage, shrinkage, and curling over a two-year period. There were no logistical issues associated with the construction of the slabs, and no damage and only minor shrinkage/curling was observed. In regards to the mixture optimization, two modified mix designs with reduced cement contents were developed, and evaluated with a suite of mechanical and durability tests. Mechanical properties tested were compressive and tensile strength, elastic modulus, and shrinkage. Durability tests included abrasion, chloride permeability, freeze-thaw resistance, and scaling. These mixes performed well with respect to all tests, with the exception of the chloride permeability. Although both mixes performed well, the resulting mixes were lean and were difficult to get good consolidation. Moreover, the process for batching these mixes may be considered a hindrance, as it involved slump adjusting the mixes with the water-reducing admixture. This was required because the nature of the RAP aggregates made it difficult to adjust mixes for variations in moisture content.

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UNIT CONVERSIONS

Measurement	Metric	English
Length	1 cm	0.394 in
	1 m	3.281 ft
	1 km	0.621 mile
Area	1 cm^2	0.155 in^2
	1 m^2	1.196 yd^2
Volume	1 m^3	1.308 yd^3
	1 ml	0.034 oz
Force	1 N	0.225 lbf
	1 kN	0.225 kip
Stress	1 MPa	145 psi
	1 GPa	145 ksi
Unit Weight	1 kg/m^3	1.685 lbs/yd^3
Velocity	1 kph	0.621 mph

EXECUTIVE SUMMARY

This research investigated the feasibility of using reclaimed asphalt pavement (RAP) to replace virgin aggregates in concrete pavements. Specifically, this research considered using minimally processed RAP (i.e., simply fractionating into fine and coarse components with no washing or crushing) in this capacity for roadways in the state of Montana. This research was conducted in multiple phases.

The first phase of research used a statistical experimental design procedure (response surface methodology – RSM) to investigate mix proportioning in concrete mixtures containing RAP to achieve desired performance criteria. Based on the RSM models, two concretes were ultimately selected for further evaluation: a high RAP mix (HR) and a high strength mix (HS). These mixes were identical sans the RAP replacement rates; the HR mix, as the name implies, had a relatively large amount of RAP with 50 percent of the fines and 100 percent of the coarse aggregates replaced with RAP. The HS mix was designed to have a higher strength by using half of the RAP (25 percent of the fines were replaced and 50 percent of the coarse). These mixes were then evaluated through a suite of tests. These mixes performed well with respect to mechanical properties and durability, and were deemed suitable for applications in the state of Montana; however, the field application of these concretes was not evaluated, and the mixes contained a significant portion of cement, which hindered the economic benefits of using this recycled material in concrete.

The second phase of this research, discussed herein, was focused on: (1) the field application of RAP concretes, and (2) further optimizing the mixes in order to reduce the amount of required cement. The field application of RAP concrete was evaluated through a field demonstration project near Lewistown, MT, in which two RAP concrete test slabs (one HR slab and one HS slab) were placed on a roadway at the MSU/WTI Transcend Research Facility. The concretes in these slabs were batched, placed, and finished using conventional construction equipment, and no constructability issues were observed during their production. However, it is worth noting that the batching process was slightly complicated by the need to slump adjust mixes due to difficulties encountered in determining the water content of the RAP aggregates. Beyond this added requirement, this work indicates that constructability issues will not hinder the use of this concrete on larger pavement projects.

Once placed, the performance of these slabs was monitored for two years via site visits, and internal vibrating wire strain gauges. No observable damage (cracking or spalling) was observed on the test slabs during site visits over the two-year monitoring period. Further, the internal gauges revealed that the slabs did not experience excessive shrinkage or curling, although they did reveal that the HR slab experienced slightly more shrinkage and curling than the HS mix, which contained half as much RAP.

In regards to the mixture optimization, a total of 10 mixes with varying paste contents (and subsequently cement contents) and RAP replacement rates were carried out in the lab in an attempt to reduce the amount of cement in the mixes. Based on this investigation, two mixes were chosen for further evaluation. These mixes were identical to the HR and HS mixes from the original phase of research, with the exception of the paste content and the use of water-reducing admixtures. The modified HR (mHR) mix had a coarse RAP replacement rate of 100 percent and a fine RAP replacement rate of 50 percent, while the modified HS (mHS) mix contained half as much RAP with coarse and fine replacement rates of 50 and 25 percent, respectively. Both of these modified mixes contained approximately 6.5 sacks of cement per cubic yard, which is more consistent with conventional concrete pavements, and is significantly less than the 7.5 sacks used in the original HR and HS mixes.

These mixes were then evaluated with a suite of mechanical and durability tests. The mechanical tests performed were compressive and tensile strength, elastic moduli, and shrinkage, while the durability tests were abrasion, chloride permeability, freeze-thaw resistance, and scaling.

In regards to mechanical properties, the mHS mix met all MDT specification requirements for both compressive and flexural strengths, and had adequate elastic moduli. The mHR mix did not meet the compressive strength requirements, but did have adequate tensile capacity and elastic moduli. Further, neither mix exhibited excessive deformations associated with shrinkage. The amount of RAP had an obvious and significant negative impact on the mechanical properties. As was expected, the strength and stiffness of the concretes decreased with increasing RAP, and the deformations associated with shrinkage increased with increasing RAP content.

Both the mHR and mHS mixes demonstrated adequate durability for use in concrete pavements in Montana with the exception of chloride ion resistance, and the mHS mix generally performed better than the mHR mix. For the abrasion tests, both mixes lost very little mass and had wear depths less than 1.0 mm. Both concretes were rated as “High” for likelihood of chloride ion penetration. In regards to freeze-thaw resistance, the mHR and mHS mixes had durability factors of 98 and 99 respectively, after being exposed to 300 freeze-thaw cycles. For reference, a durability factor of 80 or more has been cited as being indicative of acceptable freeze-thaw resistance. For scaling resistance, both the mixes were rated as “moderately susceptible”.

Based on the results from this study, the following conclusions can be made:

1) RAP can be processed, and RAP concrete slabs can be batched/placed/finished with conventional concrete equipment, with no major logistical issues. Further, RAP concrete slabs will not see significant damage/shrinkage/curling throughout the first few years of use.

2) Suitable concrete mix designs containing a significant portion of RAP and conventional cement contents can be obtained by using commercially available water-reducing admixtures. The mHR and mHS mixes developed in this research contained

approximately 6.5 sacks of cement per cubic yard, and had adequate mechanical properties and durability (sans chloride permeability and compressive strength for the mHR mix) to be used in concrete pavements in the state of Montana. That being said, these mixes had significantly less paste, required the use of a water-reducing admixture to achieve the desired workability, and were difficult to consolidate. Further, the process for batching these mixes involved slump adjusting the mixes with the admixture, as the nature of the RAP aggregates made it difficult to adjust mixes for variations in moisture content. All of which, may hinder their use in real-world applications.

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1 INTRODUCTION

1.1 Background

Each year, the highway construction industry in the United States produces over 100 million tons of reclaimed asphalt pavement (RAP) through standard rehabilitation and construction of our nation's roads (Huang, Shu, & Li, 2005). Although this reusable material has been put to use in many applications (usually in the form of asphalt paving), a considerable portion of this material remains unused and is either stockpiled or landfilled. Therefore, alternative uses for this material are needed. One possible use for this material is the replacement of conventional aggregates in Portland cement concrete pavement (PCCP). The environmental benefit of using RAP as aggregate replacement in PCCP goes beyond an alternate use for RAP, it also reduces the need for conventional aggregates, and the environmental impacts associated with their creation and transportation.

Phase I of a research effort funded by the Montana Department of Transportation (MDT) focused on the optimization and characterization of RAP mix designs suitable for transportation applications (Berry, Kappes, & Kappes, 2015; Berry, Stephens, Bermel, Hagel, & Schroeder, 2013). As part of this research, two mix designs were developed using the Response Surface Methodology (RSM). The mechanical properties and durability of these mixes were then evaluated in the laboratory. These mixes performed well, and this phase of research demonstrated the feasibility of using RAP in concrete for transportation applications; however, the field performance of these concretes needs to be evaluated prior to their use in real-world applications. Additionally, there is a need to further optimize these mixes in order to reduce the amount of cement required in the mixtures, and subsequently improve their economy.

1.2 Objectives

The primary objectives of this research are to:

1. Evaluate the field performance of the RAP concretes developed in the Phase I research effort, including any potential constructability issues associated with their deployment.
2. Further optimize these mixes in order to reduce the cement content, and subsequently improve the economy of the mixes.
3. Evaluate the performance of the further optimized mixes through a suite of mechanical and durability tests.

1.3 Scope

The project objectives were realized through the following tasks:

- Three concrete test slabs were constructed on a roadway near Lewistown, MT: two RAP concrete slabs, and one control slab. These slabs were instrumented to evaluate the strains and subsequent curvatures associated with temperature gradients, and long-term shrinkage. Further, the slabs were instrumented with traffic counters to characterize the vehicle usage, and were visually inspected for potential damage every quarter, throughout the duration of the project.
- A total of 10 concrete mixtures with varying cement contents and RAP replacement rates were investigated. These mixes were then analyzed, and two mixes were chosen for further evaluation through a suite of mechanical and durability tests. The mechanical testing included compressive and tensile strengths, elastic moduli, and shrinkage, while the durability tests included abrasion, chloride permeability, freeze-thaw resistance, and scaling.

2 FIELD DEMONSTRATION – TEST SLABS

2.1 Test Section Layout

The field performance of RAP concrete was evaluated as part of a larger study on alternative concrete pavements. In this larger project, a total of seven experimental slabs and two smaller approach slabs were constructed on a roadway leading into the MSU/WTI Transcend research facility in Lewistown, MT. These slabs were cast over two days starting on November 1, 2012. The concretes evaluated in this research were as follows.

- High RAP (HR) concrete with fine and coarse RAP replacements of 100 and 50 percent, respectively.
- High Strength (HS) RAP concrete with fine and coarse RAP replacements of 50 and 25 percent, respectively.
- Hydraulic fly ash concrete with glass aggregates.
- Hydraulic fly ash concrete with conventional aggregates.
- A conventional concrete (for control purposes).

The 10-inch thick concrete slabs are 15 feet by 15 feet, and have a normal crown with a cross slope of 2 percent. The slabs are joined by 1.25-inch greased epoxy-coated steel dowels, and sit on 10 inches of base course. The layout and dimensions of the test slabs are provided in Figure 1, while the base course and formwork prior to placement of the concrete are shown in Figure 2.

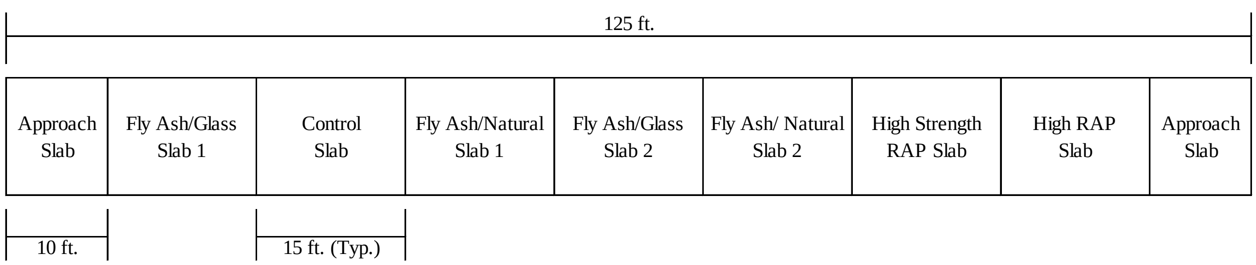


Figure 1: Layout of test slabs

Figure 2: Base course and formwork

2.2 RAP Concrete Mix Designs

The high RAP (HR) and high strength (HS) mix designs developed and evaluated in the Phase I research effort were used in the two RAP slabs. The HR mix had 50 percent of the virgin fine aggregates and 100 percent of the virgin coarse aggregates replaced with RAP. Whereas, the HS mix contained half the amount of RAP used in the HR mix, with fine and coarse replacement rates of 25 and 50 percent, respectively. A control slab was also placed using a conventional concrete pavement mix design. In particular, the mix design used for the control slab was identical to a mix used for an MDT project near Kalispell, MT: US 93 to Airport Road. The mixes were intended to have 1.5 inches of slump and 6 percent air entrainment. The mix parameters for the HR, HS, and control mixes are provided in Table 1; the mix proportions are provided in Table 2.

Table 1: Mix parameters for the HR, HS, and Control mixes

Mix Parameter	High RAP (HR)	High Strength (HS)	Control
w/c	0.385	0.385	0.45
Air Dosage Rate(mL/100#)	182.9	182.9	148.1
Paste Content	34.7%	34.7%	28.8%
Fly Ash Replacement Rate (by weight)	15%	15%	0%
Coarse to Fine Ratio	1.36	1.36	1.277
Fine RAP Replacement Rate (by vol.)	50%	25%	0%
Coarse RAP Replacement Rate (by vol.)	100%	50%	0%

Table 2: Mix proportions for HR, HS, and Control mixes (per cubic yard)

Item	High RAP (HR)	High Strength (HS)	Control
Water (lbs)	318.6	318.6	234.9
Portland Cement (lbs)	702.0	702.0	515.7
Fly Ash (lbs)	124.2	124.2	0.0
MicroAir (mL)	1282.5	1282.5	75.6
Combined Coarse RAP (lbs)	1347.3	672.3	0.0
Coarse Virgin (lbs)	0.0	739.8	1749.6
Fine RAP (lbs)	464.4	232.2	0.0
Fine Virgin (lbs)	523.8	788.4	1368.9

2.3 Slab Instrumentation

A total of 16 gauges were embedded in each of the two RAP slabs to monitor shrinkage and curling. The gauges were embedded 1.5 inches from the top and bottom of the slabs at the locations and orientations identified in Figure 3. The gauges were held in place prior to slab placement with rebar and light gauge wire as shown in Figure 4. In addition to these gauges, a pair of pneumatic traffic tubes was also installed to monitor vehicle volume, speed, and classification.

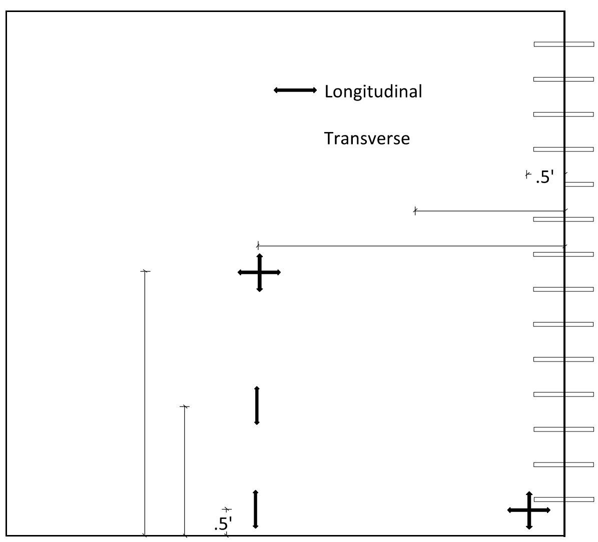


Figure 3: Vibrating wire strain gauge locations and orientations

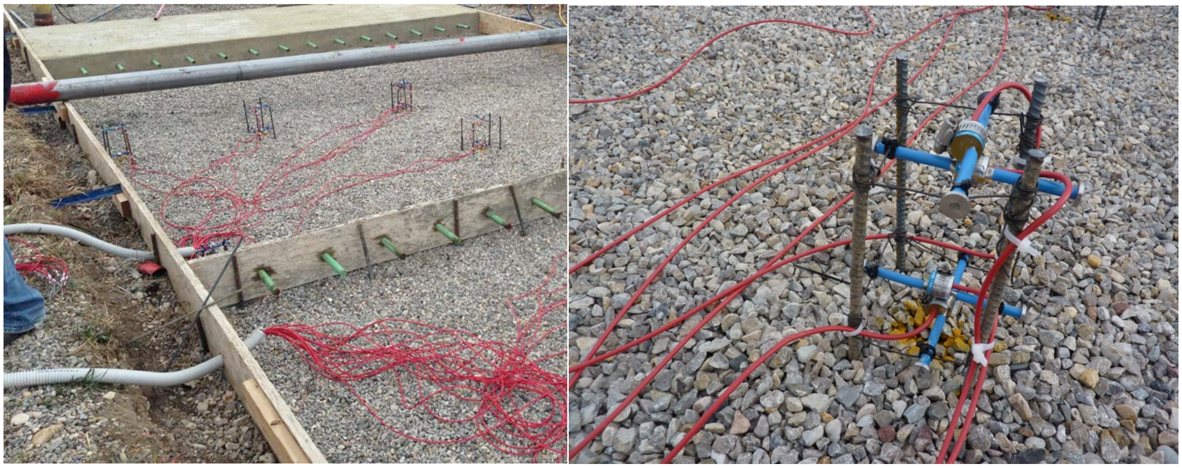


Figure 4: Strain gauges prior to slab placement

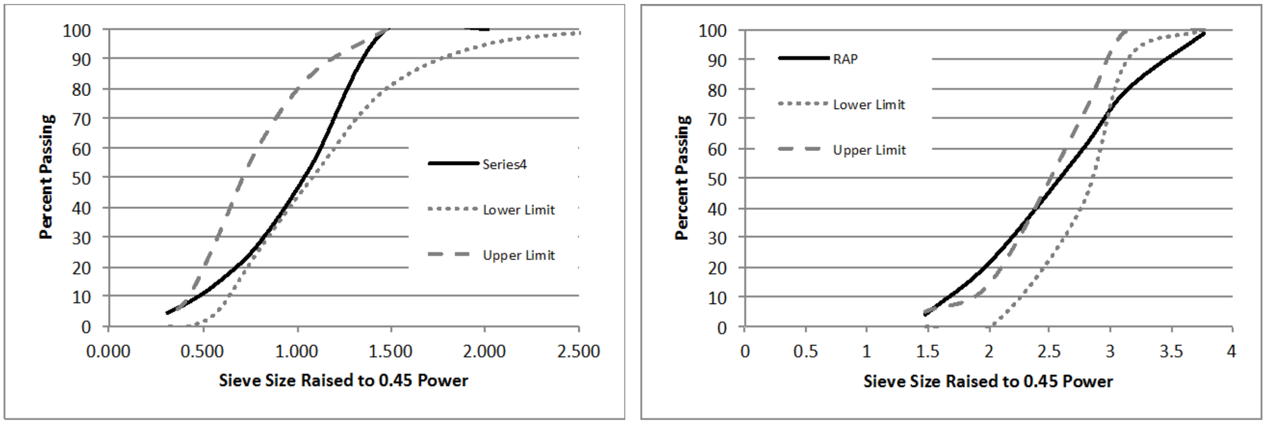
2.4 RAP Source and Processing

The RAP used in this research was freshly milled from a highway near Stanford, Montana and delivered to Casino Creek Concrete in Lewistown, Montana via four MDT belly-dump trucks. The RAP was processed into fine and coarse fractions with a triple-deck vibratory screener (Figure 5 and Figure 6). First, large particles were removed from the RAP by passing it through a ¾- inch sieve. The RAP was then fractionated on 0.375- and 0.125-inch sieves. The material caught on these two sieves were recombined and classified as the coarse aggregate. The material passing through the 0.125-inch sieve was classified as being the fine aggregate. Note that this process is slightly different from what was used in the Phase I research, in that the original work only used a 0.125-inch sieve. The additional sieve in this research was included due to concerns that -- at this scale -- the RAP would clog the smaller 0.125-inch sieve if some of the material was not removed prior to fractionating on this sieve. Processing the millings yielded approximately two parts coarse RAP aggregate to one part fine RAP aggregate. This 2:1 ratio was consistent with findings from Phase I research, and is approximately the replacement ratio used in both of the RAP mixes. The gradation curves for fine and coarse RAP fractions are provided in Figure 7, along with the corresponding limits for concrete aggregates specified by MDT for concrete aggregates. As can be seen in this figure, the fine RAP is within the specified limits, while the coarse RAP is slightly outside the limits required by the specifications.

The RAP material screened quickly and without complications; screening could be easily scaled-up and implemented with larger volumes seen in conventional pavement projects.

Figure 5: Overview of RAP processing

Figure 6: RAP processing



(a) (b)

Figure 7: Gradation curves for fine (a) and coarse (b) RAP fractions

2.5 Concrete Batching

The concrete was batched and delivered by Casino Creek Concrete in Lewistown, MT. A conventional rear-discharge concrete truck was used to mix and deliver the concrete. The batching of the concrete began with charging the truck with the RAP aggregates in the yard using a digital scale and conveyor, as seen in Figure 8. The truck was then charged with the air-entraining admixture (Micro-Air) and the mix water, and mixed for one minute prior to adding the portland cement, fly ash, and virgin aggregates. Once charged, the truck mixed at a medium-slow rate for approximately 15 minutes while in transit to the test site.

The mix water and aggregate proportions were not adjusted to account for aggregate moisture content because the nature of the RAP aggregate made it difficult to use conventional methods for accounting for moisture content. Instead, approximately 10 percent of the mix water was withheld from the mixes; the mixes were then slump adjusted (target slump of 1.5-inch) onsite by adding an appropriate portion of the withheld mix water.

Figure 8: Charging concrete truck with RAP aggregate

The control concrete was batched via a conventional procedure. The truck was charged first with the fine and coarse aggregates, and portland cement. These ingredients were then mixed for about 3 minutes, prior to adding the air-entraining admixture, water reducer, and mix water. The entire mixture was then mixed at a medium-slow rate for 15 minutes in transit to the test site.

2.6 Concrete Placement and Finishing

Prior to concrete placement the forms were lightly oiled, the base course was dampened, and the dowels were greased. The concrete was then placed in the forms and consolidated with a handheld concrete vibrator (Figure 9). Special care was taken in the region of the instrumentation to ensure that the gauges were not damaged and that the concrete was well consolidated. A 20-foot hydraulic roller screed was then used to level the concrete and homogenize the surface of the slabs (Figure 9). A magnesium bull float was then used to further finish the slabs, and once the slabs began to set, a broom finish was applied to the surface. A curing compound was then applied to the surface of the slabs. Finally, the slabs were blanketed, as the overnight temperatures were expected to drop below freezing. The blankets were removed from the slabs approximately 48 hours after they were cast. The finished slabs are shown in Figure 10.

Figure 9: Placing and finishing a RAP slab

Figure 10: Finished test slabs

2.7 Concrete Properties

While each slab was being cast, a representative sample of concrete was collected to test slump and air content, and to cast compression and tension test specimens. The plastic and mechanical properties of the concretes were tested in accordance with the relevant ASTM specifications. Table 3 provides the measured air content, slump, 28-day compressive and tensile strength, and one-year compressive strength. Also included in this table are the MDT specifications for concrete pavements. Referring to this table, all three of the concretes had air contents slightly less than the target air content of 6 percent, with the HR, HS, and control mixes having respective air contents of 4.8, 5.0, and 4.6 percent. In regard to slump, both of the RAP mixes had slumps within the limits specified by MDT, with the HR and HS mixes having slumps of 2.25 and 1.75 inches, respectively. The control mix had a slump of 3.5 inches, which is outside of the MDT specification. In regard to compression strength, all three mixes met MDT specifications at 28 days. As expected, the control mix had a compressive strength significantly stronger than the RAP mixes (51 percent stronger than the HS mix at 28 days), and the HS mix was significantly stronger than the HR mix (32 percent at 28 days). In regard to tensile strength, the HR mix was the only concrete that did not meet the rupture strength requirement of 500 psi. The rupture strength of the HS mix (670 psi) is close to the rupture strength observed for the control mix (697 psi); this, despite having significantly less compressive strength. Note that the strengths observed for the RAP concretes are nearly identical to the strengths observed for these respective mixes when they were prepared in the laboratory.

Table 3: Measured Concrete Properties of Test Slabs

Parameter	High RAP (HR)	High Strength (HS)	Control	**MDT Specifications**
Air Content (%)	4.80	5.00	4.60	5 to 7
Slump (in)	2.25	1.75	3.50	0.75 to 2.25
28-day: f'c (psi)	3001	3960	6000	3000
28-day: MOR (psi)	431	670	697	500
365-day: f'c (psi)	3505	4676	6119	-

2.8 Summary

In this task, two 15-foot by 15-foot by 10-inch thick RAP concrete slabs were constructed on a roadway leading into the Transcend research facility in Lewistown, MT. The RAP was freshly milled from a nearby highway and processed without incident with a conventional vibratory screener. The concrete was batched, placed, and finished using standard construction equipment. No constructability issues (beyond slump adjusting the concrete) were observed during the production of these slabs, indicating that constructability issues will not hinder the use of this concrete on a larger pavement project. The field performance of these slabs was then monitored through field observations, traffic tubes, and the embedded strain gauges, discussed in the following chapter.

3 EVALUATION OF TEST SLABS

The test slabs discussed in the previous section were monitored for approximately two years after casting to evaluate their performance. This chapter discusses the results of this evaluation. First, the amount/classification of traffic on the slabs is discussed, followed by the results of the field observations. Finally, the observed shrinkage and curvature of the slabs are presented and discussed.

3.1 Traffic

The amount and classification of traffic on the test slabs were monitored with traffic counters over a 6-month period (July-December, 2013). The amount of traffic, the FHWA vehicle classification (Appendix B), and the speed of this traffic are provided in Table 4. As can be seen in this figure, 1177 vehicles drove over the test slabs over this time period, with a majority of this traffic being class 2 and class 3 vehicles (passenger cars and pickup trucks, respectively). This amounts to approximately seven cars per day. A majority of this traffic was travelling at speeds less than 30 mph.

Table 4: Recorded Traffic on Slabs

Speed Range (MPH)	1	2	3	4	5	6	7	Speed Totals	
0-5	0	3	0	0	1	0	0	4	0.3%
5-10	0	48	5	2	0	0	1	56	4.8%
10-15	0	155	44	3	0	1	0	203	17.2%
15 - 20	0	144	41	1	0	1	0	187	15.9%
20 - 25	0	162	131	1	0	0	0	294	25.0%
25 - 30	0	207	79	0	0	1	0	287	24.4%
30 - 35	0	63	47	0	0	0	0	110	9.3%
35 - 40	0	9	9	0	0	0	0	18	1.5%
40 - 45	0	9	5	0	0	0	0	14	1.2%
45 - 50	0	2	2	0	0	0	0	4	0.3%
Vehicle Totals	0	802	363	7	1	3	1	1177	
	0.0%	68.1%	30.8%	0.6%	0.1%	0.3%	0.1%		

3.2 Visual Inspection

The slabs were visually inspected for damage (cracks/spalling) once per quarter over the duration of the project. Figures 11 - 13 show the HS, HR, and Control slab at two years after the casting. No cracks or spalling was observed on these test slabs over the duration of the project.

Figure 11: HS Slab at Two Years

Figure 12: HR Slab at Two Years

Figure 13: Control Slab at Two Years

3.3 Strains and Curling

As discussed previously, a total of 16 gauges were embedded in each of the two RAP slabs to monitor shrinkage in the concrete and subsequent curling. These 16 gauges consisted of two gauges (top and bottom) at eight distinct locations, with half of the gauges oriented in the longitudinal direction (parallel to the direction of travel) and half oriented in the transverse direction (perpendicular to the direction of travel). The gauges were embedded 1.5 inches from the top and bottom of the slabs at the locations and orientations identified in Figure 14. The labeling scheme for the gauges is also identified in this figure. Strains and temperatures were recorded for each gauge at two-hour increments.

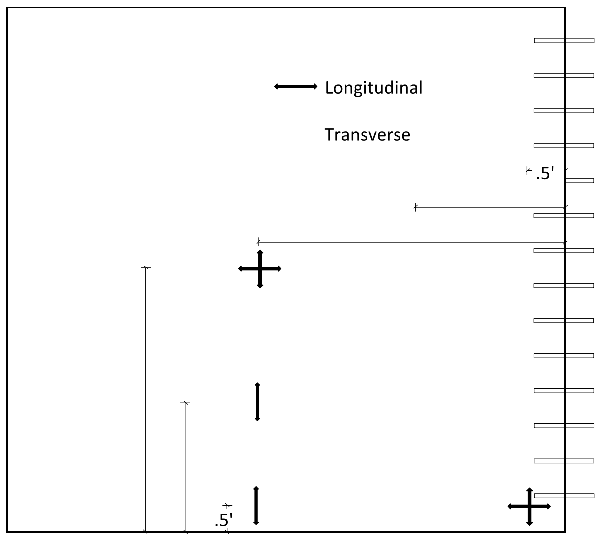


Figure 14: Vibrating wire strain gauge locations and orientations

Prior to evaluating the strains and curvatures, the recorded strains were adjusted to account for the effects that temperature has on the vibrating wire gauges. Specifically, the actual microstrain ($\mu\epsilon_{actual}$) was calculated from the recorded microstrain ($\mu\epsilon_{recorded}$) with the following equation.

$$\mu\epsilon_{actual} = \mu\epsilon_{recorded} + \Delta T \propto \quad \text{(Eq 1)}$$

where ΔT is the change in temperature in $°F$ from the initial recording, and $\propto$ is the coefficient of linear thermal expansion of the steel wire, 6.7 $\mu\epsilon/°F$.

The approximate curvatures in the slabs(ϕ) were calculated at each location from the adjusted strains at the top (ϵ_{top}) and bottom (ϵ_{bottom}) of the slabs with the following equation.

$$\phi = \frac{(\epsilon_{top} - \epsilon_{bottom})}{d} \quad \text{(Eq 2)}$$

where d, is the vertical distance between the gauges (7 inches). For reference, a negative strain indicates a shortening of the gauge, and positive strain indicates lengthening. Further, a positive curvature indicates that the slab is curving down, while a negative curvature indicates curving up.

The 28-day average strains at the top and bottom, and the approximate curvatures are provided in Appendix A for each location for both slabs. The 28-day average strains/curvatures were used rather than the actual recorded values at 2-hour increments because the slabs experienced increases/decreases in strains/curvatures throughout the day/night due to temperature differentials, and these diurnal cycles had a tendency to cloud the long-term effects. The diurnal effects can be observed in Figure 15, where the top/bottom strains and curvatures for a typical gauge are plotted over several days in April. Also included in this plot are the recorded temperatures at the top and bottom of the slab and the difference between these temperatures over the same time period. As can be observed in this figure, and as expected, the slabs increased in length as the temperature increased, and decreased in length as the temperature decreased. Moreover, the top of the slab experienced greater swings in temperature than the bottom of the slab, and thus expanded and contracted more than the bottom. Further, the temperature and expansion/contraction of the bottom of the slab is slightly delayed relative to the top, due to the thermal resistivity of the concrete. The curvature of the slab varied throughout the day/night and had peak values coincident with the peak differences in temperature between the top and bottom of the slab, as would be expected. Specifically, the slabs were observed to have more upward curvature when the temperature at the top of the slab was less than that at the bottom, and the slab was observed to have less upward curvature when the temperature at the top exceeded that at the bottom.

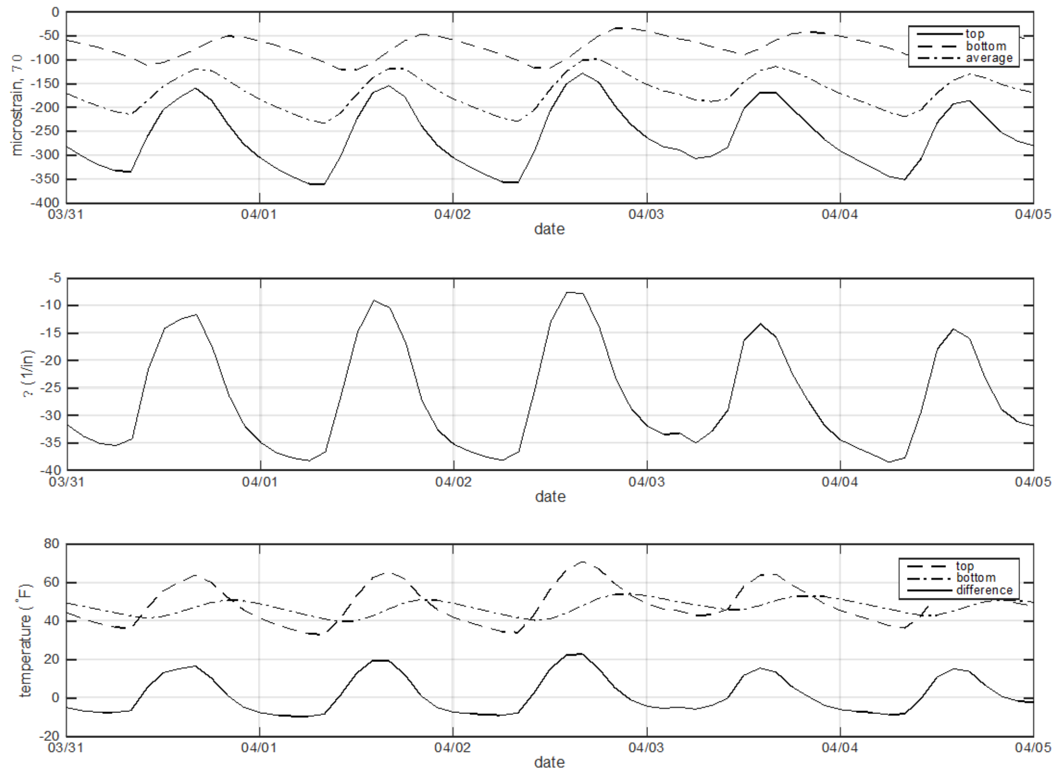


Figure 15: Diurnal Cycles in HS Slab at L1

To evaluate the effects of RAP content on the average shrinkage in the slabs, the average of the top and bottom strains at each location in the longitudinal and transverse direction are plotted for both slabs over the approximately 2.5 years of the project in Figure 16 and Figure 17, respectively. As can be observed in these figures, overall both slabs have decreased in length over the duration of the project, with all recorded strains being less than zero. In addition, observable in these figures are seasonal effects; the slabs had a tendency to shrink less during the hotter months, and shrink more during the colder months. In regards to the effect of RAP content on shrinkage, there does not appear to be a systematic difference in observed strains between the HS and HR slabs in the longitudinal direction. However, in the transverse direction, the HS slab experienced less shrinkage than the HR slab at three of the four locations.

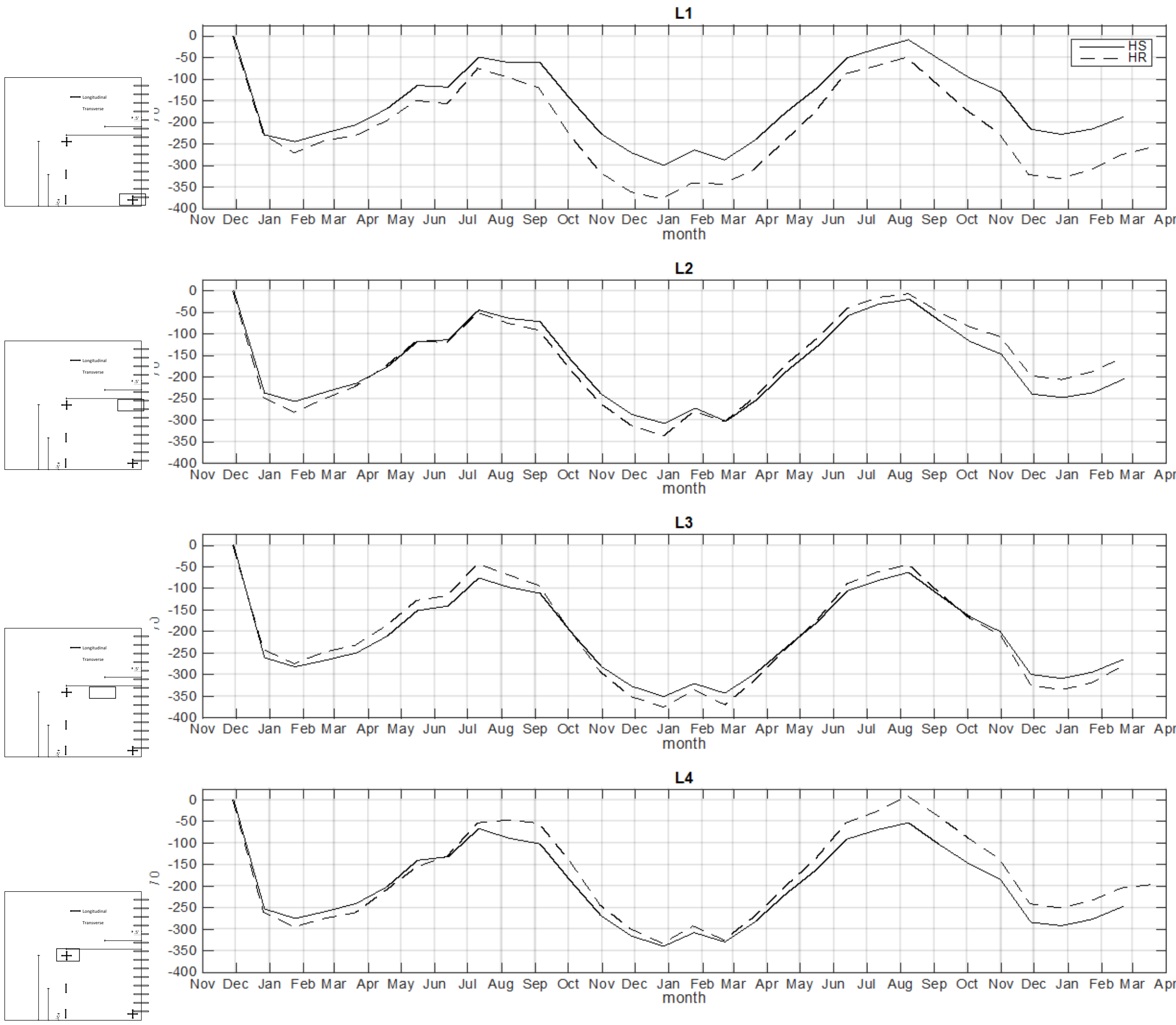


Figure 16: Average Strains in Longitudinal Direction for HS and HR Slabs

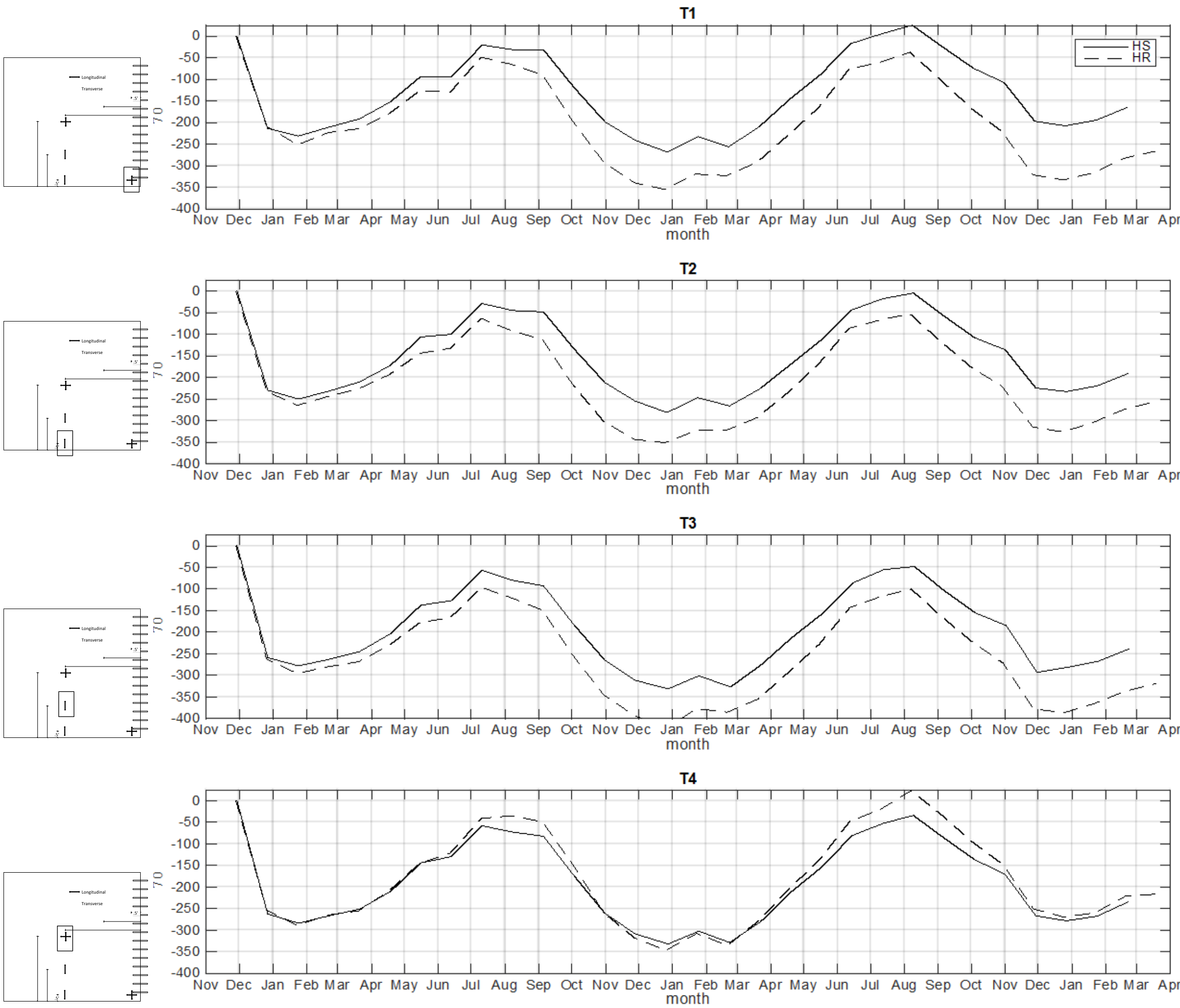


Figure 17: Average Strains in Transverse Direction for HS and HR Slabs

Similar to what was done for average strains; the observed curvatures in the slabs at each location in the longitudinal and transverse direction are shown in Figure 18 and Figure 19, respectively. Overall, with the exception of the center of the slab (L4 and T4), both slabs have increasing upward curvature (-), as may be expected due the increased shrinkage observed in the exposed tops of the slabs. Further, this curvature is observed to vary with the seasons, with less curvature during the hotter months, and more during the colder months. With respect to the effect of RAP content, unlike what was observed for average strain, the HS slab had less upward curvature than the HR slab in both directions at all locations with the exception of the middle (L4 and T4). This indicates that increased RAP content has a negative impact on the potential for slab curling.

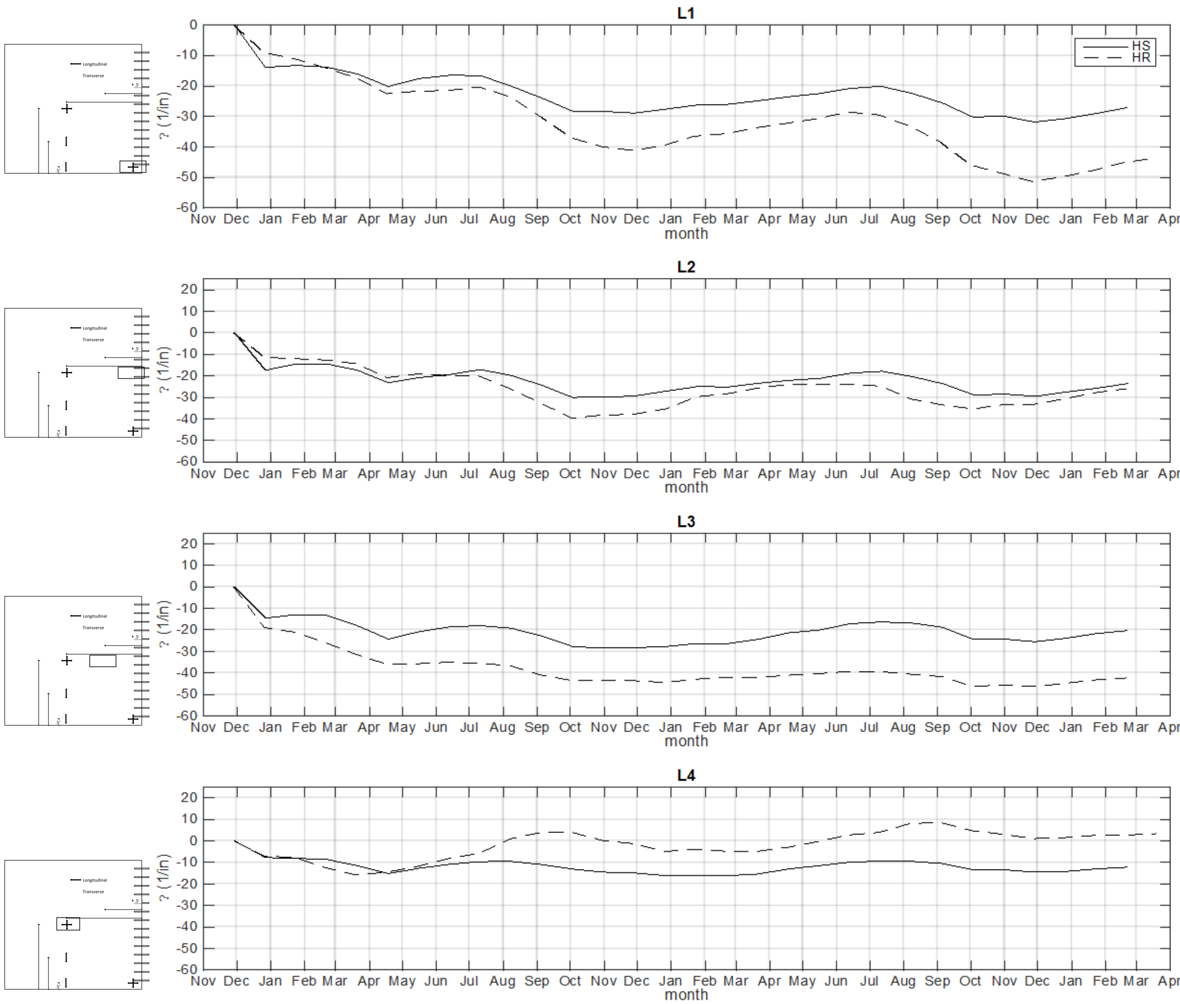


Figure 18: Longitudinal Curvatures for HS and HR Slabs

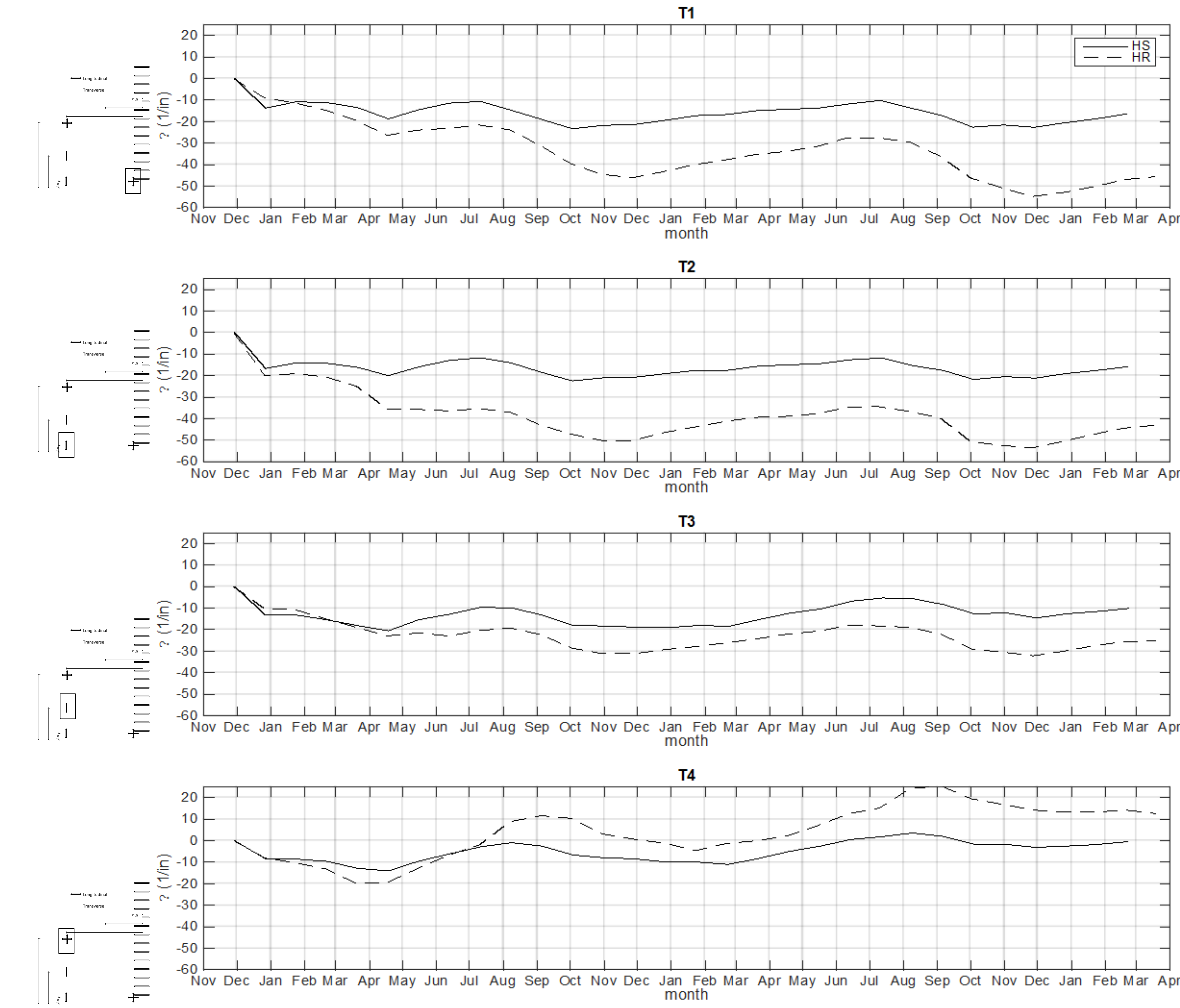


Figure 19: Transverse Curvatures for HS and HR Slabs

It should be noted, that although shrinkage and curling were observed in the strain gauge data, these deformations were not excessive, and were not observable in visual inspections.

4 FURTHER OPTIMIZATION OF MIXES

The RAP mixes developed in the Phase I research were further optimized in this research to reduce the amount of cement included in the mixes, while maintaining performance. This chapter documents the findings from this study.

The original mixes from the Phase I investigation contained a significant amount of cement per cubic yard. Specifically, both the HR and HS mixes contained approximately 7.5 sacks (704 pounds) per cubic yard, while a standard pavement mix may contain 6-6.5 sacks of cement. In order to reduce the amount of cement, mixtures with reduced paste contents were investigated in this research. In these mixes, a commercially available water-reducing admixture was used to make up for the decrease in workability caused by the reduced paste. Further, it was expected that the reduced paste contents may have a negative effect on strength, and therefore mixes with varying RAP replacement rates were also investigated.

A total of 10 mixes with varying paste contents (and subsequently less cement) and varying RAP contents were investigated in this research, and are summarized in Table 5. It should be noted, that all other mix parameters in these mixes were the same as those used in the original study, including the ratio of coarse to fine RAP, which was 2 for all mixes. The 28-day compressive strengths for each of the mixes are plotted in Figure 20 versus the course RAP replacement rate. As can be observed in this table and figure, 28-day compressive strength generally decreased with decreasing paste content and increasing RAP replacement rate. It should be noted that although 6-sack mixes were workable and had adequate strength, a significant amount of water reducer was required, and the reduced paste content of 28 percent resulted in a very lean mix, which may cause problems with forming and finishing.

Table 5: Mix Designs and Results

Mix Name	Mix Parameters					Results				
	Cement (sacks/yd^3)	PC (%)	Coarse (%)	Fine (%)	Glenium (mL/lb of cm)	Slump (in.)	Air (%)	7-Day (psi)	28-Day (psi)	Rupture (psi)
MDT Specifications	-	-	-	-	-	1.5±0.75	6±1	2000	3000	500
6-50-25	6	0.280	50	25	1.08	0.75	5.5	2761	3224	799
6-62-31	6	0.280	62.5	31.25	2.17	1.5	5	2356	2988	473
6-75-37	6	0.280	75	37.5	2.17	1	5.5	2224	2939	-
6-100-50	6	0.280	100	50	2.17	1.5	6.4	1882	2075	521
6.5-50-25	6.5	0.302	50	25	1.01	1.25	3.5	2948	3764	668
6.5-62-31	6.5	0.302	62.5	31.25	2.01	1.5	5	2880	3427	603
6.5-75-37	6.5	0.302	75	37.5	1.61	1	4	2625	2851	542
6.5-100-50	6.5	0.302	100	25	1.41	1.25	4.8	2016	2168	584
7.5-50-25	7.5	0.347	50	25	0.00	0.75	5	2927	4089	-
7.5-100-50	7.5	0.347	100	50	0.00	2	4.9	2283	2656	580

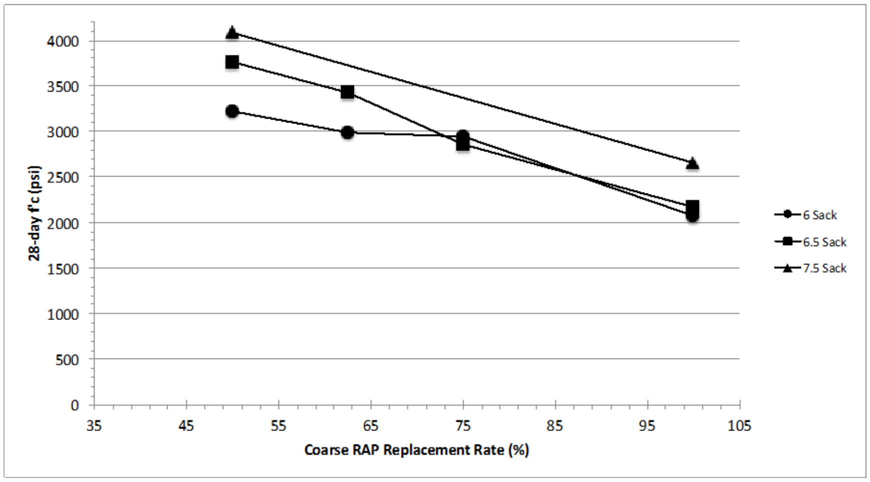


Figure 20: Compressive Strength vs. Time for Optimization Study

Based on the results of this investigation and the fact that 6.5 sack mixes are commonly used in standard concrete pavements, two 6.5-sack mixes were chosen for further evaluation: (1) the 6.5-sack mix with 100 and 50 percent coarse and fine RAP replacement rates, and (2) the 6.5-sack mix with 50 and 25 percent coarse and fine RAP replacement rates. These two mixes are the same mixes that we evaluated in the Phase I effort, but with 6.5-sacks of cement per cubic yard instead of 7.5. Note that the 6.5-sack mix with replacement rates of 100 and 50 percent does not meet MDT minimum compressive strength requirements at 28 days, but does meet the tensile strength requirements. Further, this reduced compressive strength may not be indicative of poor pavement performance. For this research, the modified mix with 100/50 replacement rates will be denoted mHR, while the modified higher strength mix will be denoted mHS. The mix parameters and proportions for these mixes are provided in Table 6.

Table 6: Mix Parameters and Proportions for Select Mixes for Further Evaluation

	Parameters				Proportions								
Mix Name	PC (%)	Coarse (%)	Fine (%)	Glenium (mL/lb of cm)	Water (lbs)	Cement (lbs)	Fly Ash (lbs)	MicroAir (mL)	Glenium (mL)	Coarse RAP (lbs)	Coarse Virgin (lbs)	Fine RAP (lbs)	Fine Virgin (lbs)
mHR	0.30	50	25	1.01	277	610	108	1116	639	1454	0	502	567
mHS	0.30	100	25	1.41	277	610	108	1116	0	727	798	251	851

It should be noted, that although the developed mixes met the desired slump/air requirements, the mixes were fairly lean with respect to paste, and were difficult to consolidate. Moreover, the process for batching these mixes may be considered a hindrance, as it involved slump adjusting the mixes with the water-reducing admixture. This was required, because the nature of the RAP aggregates made it difficult to adjust mixes for variations in moisture content.

These mixes were then evaluated with a suite of mechanical and durability tests, which will be discussed in the following chapter.

5 MECHANICAL PROPERTIES OF OPTIMIZED MIXES

The two concrete mixtures developed in the previous chapter (mHR and mHS) were evaluated with a full suite of mechanical and durability tests to assess their potential for use as concrete pavements. This chapter reports on the results of the mechanical tests, while the following chapter reports the results of the durability tests. A summary of the mechanical properties tested in this research is provided in Table 7. It should be noted that multiple batches of both concrete mixtures were required to complete all of these tests, and although some variation was observed between mixes, this variation was not substantial.

Table 7: Mechanical Properties

Material Property	ASTM Test Method
Compressive Strength	C39
Elastic Modulus	C469
Modulus of Rupture	C78
Shrinkage	C512

5.1 Unconfined Compressive Strength, f'_c

An often cited and important property of hardened concrete is its unconfined compressive strength, which can also be indicative of many other material properties. Figure 21 shows the compressive strength profiles as a function of time for the mHR and mHS concretes over one year. These strengths were determined in accordance with ASTM C39, and were calculated as the averages of three 4-by-8 inch test cylinders. As can be seen in the figure, all concretes continued to gain strength over time. However, the rate of strength gain decreased with time, with both concretes reaching around 90 percent of their one-year capacity at 120 days. Also, it can be seen that the amount of RAP significantly affects the concrete compressive strength, with the mHR mix only obtaining 72 percent of the capacity of the mHS mix at one year (2,720 versus 3,770 psi). This tendency is to be expected and has been well documented by previous research efforts.

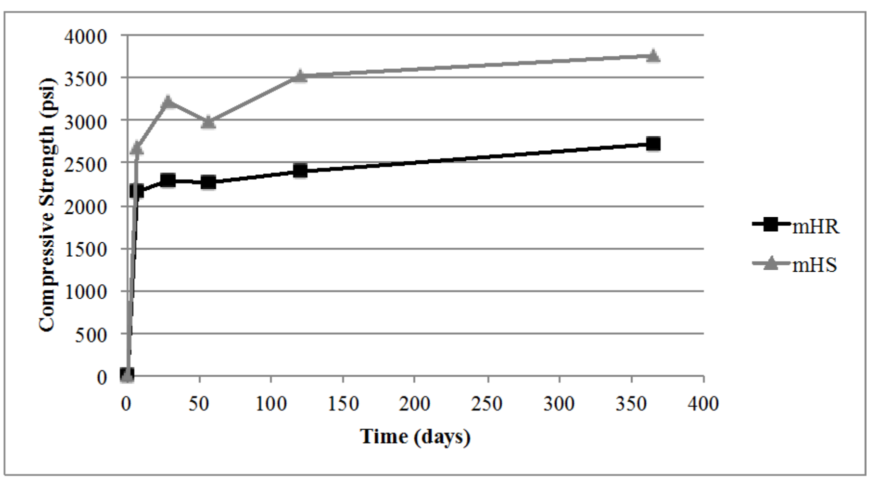


Figure 21: Unconfined Compressive Strength vs. Time for mHR and mHS Mixes

5.2 Elastic Modulus, E_c

The elastic modulus of each concrete was determined as the average of three tests on 4-by-8 inch cylinders, tested in accordance to ASTM C469. The results for each concrete are provided in Table 8 and Figure 22. Also included in this table, for comparison, are the predicted values of the modulus according to ACI 318: $E_c = {w_c}^{1.5} 33\sqrt{f'_c}$. In this equation, E_c is the elastic modulus in psi, w_c is the unit weight of the concrete in pcf, and f'_c is the compressive strength of the concrete in psi.

Generally speaking, the elastic modulus of each concrete increased with time, as one would expect with increasing compressive strength. It is difficult to make comparisons between the elastic moduli of the different mixtures and hence isolate the effect of increasing RAP replacement rate since their compressive strengths varied significantly. However, the effect of including RAP can be isolated by comparing the measured and predicted moduli for each concrete. The ratio of measured-to-predicted moduli for each concrete is plotted in Figure 23. Referring to Figure 23, the ratio of measured-to-predicted moduli for the mHS mix was higher than the mHR mix on all days, indicating that the amount of RAP affects the stiffness of the concrete and the applicability of this ACI prediction. It should also be noted that this ratio is less than 1.0 on all days for both the mHS and mHR mixes, indicating that concrete containing RAP is less stiff than conventional concrete, as one might expect.

Table 8: Elastic Modulus for mHR and mHS Mixes

Mix	Age (days)	$f'c$ (psi)	E_{Meas} (ksi)	E_{Pred} (ksi)	$\frac{E_{Meas}}{E_{Pred}}$
mHR	7	1791	1759	2412	0.73
	28	2201	2041	2674	0.76
	56	2311	2028	2735	0.74
	120	2566	2024	2886	0.70
	365	2809	2228	3013	0.74
mHS	7	2513	2630	2858	0.92
	28	3315	2709	3281	0.83
	56	3091	2719	3167	0.86
	120	3303	2867	3274	0.88
	365	3756	2810	3492	0.80

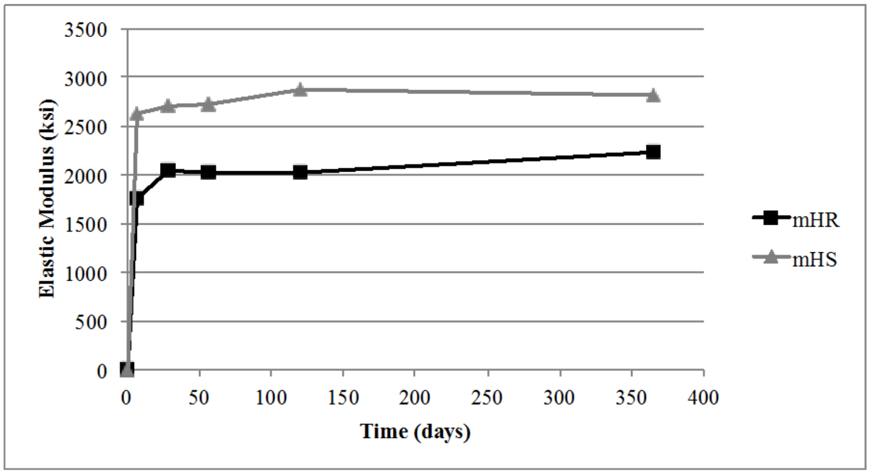


Figure 22: Elastic Modulus for the mHR and mHS Mixes

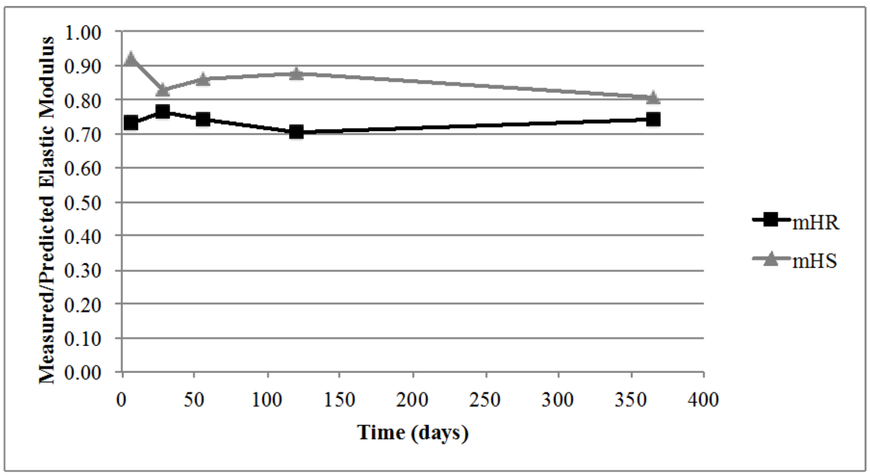


Figure 23: Measured/Predicted Elastic Modulus for mHR and mHS Mixes

5.3 Modulus of Rupture, f_r

Modulus of rupture was calculated as the average of two 20-by-6-by-6 inch prisms tested according to ASTM C78. The measured data for both concrete mixes is provided in Table 9 and Figure 24. Table 9 also includes the strengths predicted by the ACI equation for modulus of rupture: $f_r = 7.5\sqrt{f'_c}$ (f_r and f'_c in psi). The modulus of rupture was also measured at 28 days for the control specimen, and this result is provided in Table 9.

As can be seen in this data, the mHS mix had a higher tensile capacity than the mHR mix at every stage. However, this is somewhat expected considering the increased compressive strength of the mHS concrete and the strong relationship between compressive and tensile strengths. In comparison to the predicted rupture strengths, both concrete mixtures had rupture strengths greater than the estimated values at every time stage. The ratios of measured-to-predicted rupture strengths are plotted versus time for both concretes in Figure 25. As can be observed in this figure, this ratio is fairly close for both the mHS and mHR mixes, indicating that the inclusion of RAP does not significantly affect the tensile capacity of the concrete beyond its effect on compressive strength.

Table 9: Modulus of Rupture for mHR and mHS Mixes

Mix	Age (days)	$f'c$	$f_{r\,Meas}$	$f_{r\,Pred}$	$\frac{f_{r\,Meas}}{f_{r\,Pred}}$
mHR	7	2157	442.5	348.3	1.27
	28	2283	562.5	358.4	1.57
	56	2280	462.5	358.1	1.29
	120	2393	507.5	366.9	1.38
	365	2723	545	391.4	1.39
mHS	7	2683	505	388.5	1.30
	28	3220	682.5	425.6	1.60
	56	2977	617.5	409.2	1.51
	120	3533	557.5	445.8	1.25
	365	3770	570	460.5	1.24

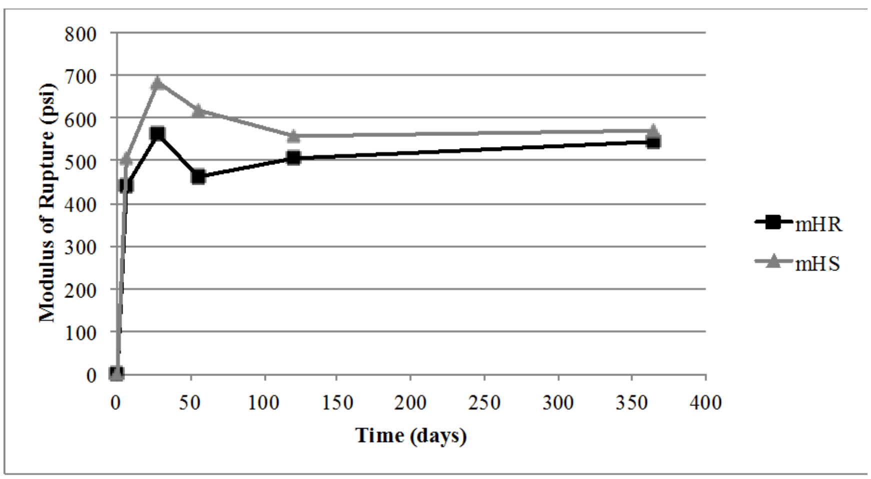


Figure 24: Modulus of Rupture for mHR and mHS Mixes

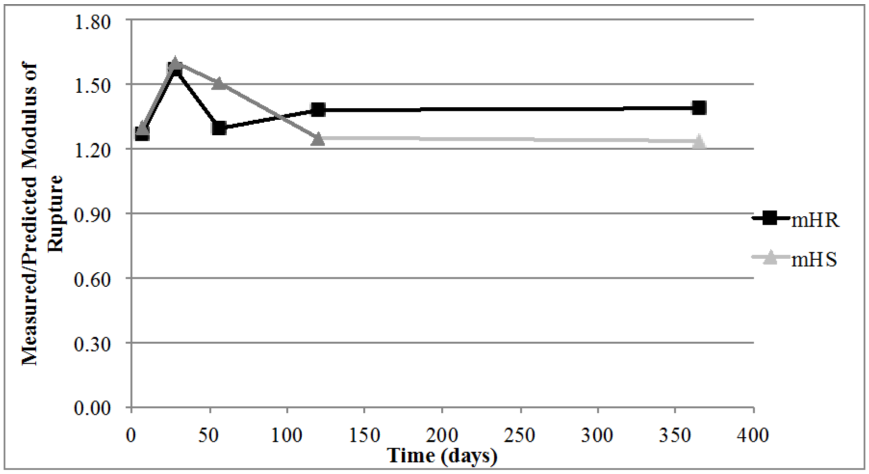


Figure 25: Measured/Predicted Modulus of Rupture for mHR and mHS Mixes

5.4 Shrinkage

Shrinkage strains were measured in substantial accordance with the procedures outlined in ASTM C512. Three 6-by-12 inch cylinders were cast from both the mHR and mHS mix designs. All six cylinders were then moist cured for 28 days. Each cylinder was equipped with two vibrating wire strain gauges to monitor deflections (Geokon Model 4000). Once cured, the cylinders were then placed in the laboratory at room temperature, where they remained for the duration of the test.

The measured shrinkage strains over 6 months are provided in Figure 26. As can be seen in these figures, both mixes continued to shrink over time, with the rate of shrinkage decreasing with time. The mHR mix experienced more shrinkage than the mHS mix at every time step.

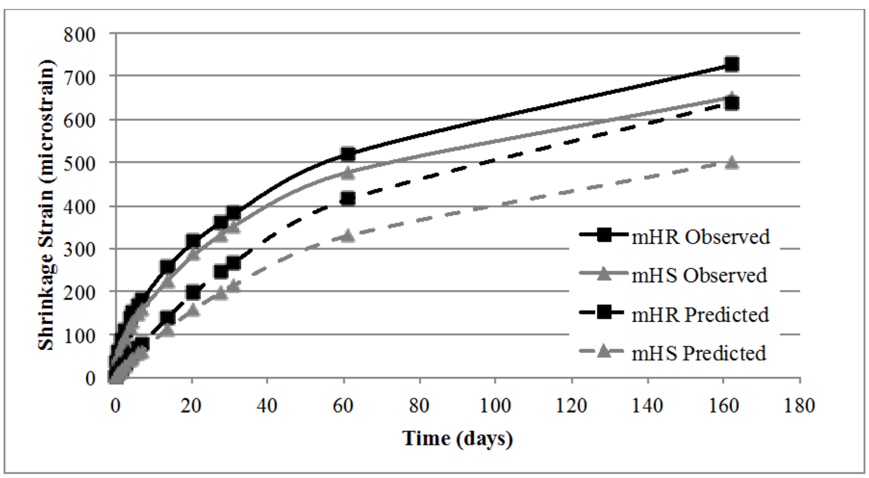


Figure 26: Shrinkage Strain vs. Time for mHR and mHS Mixes

To serve as a point of comparison, the shrinkage strain was estimated using the methodology presented in the *AASHTO LRFD Bridge Design Specifications (Section 5.4.2.3) (2010)* and included in the figure above. The AASHTO approximation for shrinkage strain (ε_{sh}) was calculated with the following equation.

$$\varepsilon_{sh}(t) = k_{vs} k_{hs} k_f k_{td} 0.48 * 10^{-3}$$

in which:

t = maturity of concrete (*days*)

k_{vs}= factor for the effect of the volume-to-surface ratio $= 1.45 - 0.13\left(\frac{V}{S}\right) \geq 1.0$

k_{hs} = humidity factor for shrinkage $= 2.00 - 0.014H$

k_f = factor for the effect of concrete strength $= \frac{5}{1+f_{ci}'}$

k_{td} = time development factor $= \frac{t}{61-4f_{ci}'+t}$ (f_{ci}' in ksi)

where:

H = relative humidity (%)

f_{ci}' = specified compressive strength at the time of loading (ksi)

V/S is the volume-to-surface ratio (in.)

The relative humidity in the lab was H = 25%.

For comparison, the ratios of measured-to-calculated shrinkage strains are plotted versus time in Figure 27 for both concretes. As can be observed in this figure, the AASHTO methodology predicted the long-term shrinkage strains fairly accurately. The ratio of measured-to-calculated strains decreased from fairly large values the first couple of weeks, to values at one year of around 1.13 and 1.29 for the mHR and mHS mixes, respectively. Shrinkage is a function, in part, of the compressive strength; thus, one would expect higher-than-predicted shrinkage from the less resilient RAP mixes. By extension, it is not surprising that the mHR mix had larger shrinkage strains than the mHS mix, and that the measured-to-calculated ratios would be higher for this concrete.

With respect to the applicability of the AASHTO methodology for predicting shrinkage, the trends observed in Figure 27 indicate that this methodology is not very accurate at early ages. This finding may be contributed to the possible delay in curing associated with the inclusion of 15 percent fly ash. However, this method proved to be adequate at predicting long-term shrinkage.

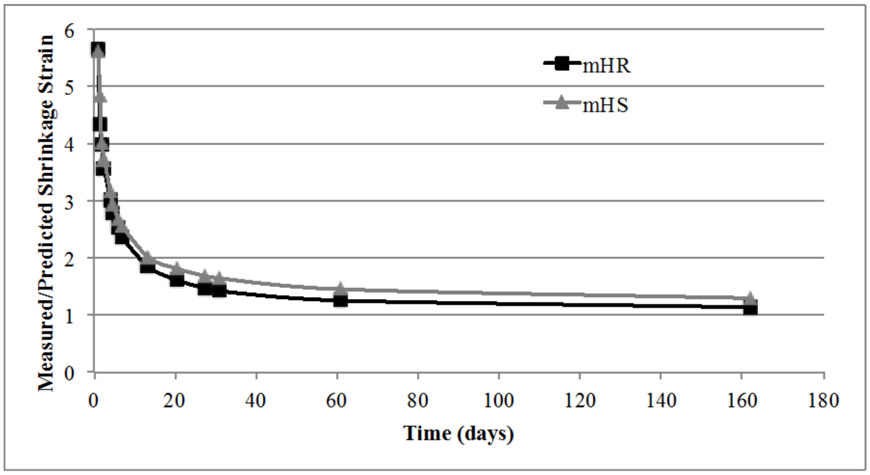


Figure 27: Measured/Calculated Shrinkage Strains for mHR and mHS Mixes

6 DURABILITY OF OPTIMIZED MIXES

The durability properties of PCCP are of particular interest in Montana, due to the harsh climatic conditions. In this research, several durability properties were evaluated for both the mHR and mHS mixes to determine the suitability of RAP aggregates in PCCP in Montana. The durability properties that were tested are listed in Table 10. The results of these tests are discussed in this chapter.

Table 10: Durability Properties

Durability Property	ASTM Test Method
Abrasion	C944
Chloride Permeability	C1202
Freeze-Thaw	C666
Scaling	C672

6.1 Abrasion

The abrasion properties of the mHR and mHS mix designs were determined according to ASTM C944. Three samples from each mix were abraded using a 22-pound load applied to a 3¼-inch rotating cutter. The cutter was rotated at approximately 200 rpm for a duration of 2 minutes. The resulting average change in mass for each of the two mix designs is reported in Table 11, and both concretes had wear depths less than 1.0 mm. For reference, concretes with wear depths of less than 1.0 mm meet FHWA standards for Grade 2 high performance structural concrete (Goodspeed, Vanikar, & Cook, 2013). Both sets of samples performed well and warranted a further investigation using a doubled load (44 pounds). Again, there was very little weight loss and wear depth for either sample.

Table 11: Abrasion tests results

	Concrete	Weight Loss	
Mix	Strength (psi)	22 Pound (g)	44 Pound (g)
mHR	2283	8.7	4.97
mHS	3220	2.93	1.77

Referring to this table, the fraction of RAP appeared to influence the abrasion resistance of the concrete, as the mHR mix lost more mass than the mHS mix. This finding might be expected, as abrasion resistance is, in part, a function of compression strength, which was higher in the mHS mix. However, both mixes performed well.

6.2 Chloride Permeability

ASTM C1202 was used to determine the chloride permeability resistance of the RAP concretes. Three specimens were tested from each mix and the average values of chloride ion penetrability are reported in Table 12.

Table 12: Chloride permeability results

Mix	Age at Test (days)	Avg. Adj. Charge Passed (coulombs)	Chloride Ion Penetrability
mHR	56	5840	High
mHS	57	6398	High

Following ASTM C1202, these results correlate with "High" likelihood of chloride ion penetration issues for both experimental mixes. The mHS concrete had a slightly higher average adjusted charge passed; however, both mixes exhibited quite a bit of variability (mHR 5153 and 6527 coulombs; mHS 5605 and 7189 coulombs). These mixes may have had poor chloride penetration performance due to the lower paste contents used in this phase of research, as the HR and HS mixes from the Phase I research performed well in this regard.

6.3 Freeze-Thaw

A primary mechanism of physical deterioration for unprotected concrete is prolonged exposure to cycles of freezing and thawing in the presence of moisture. This damage, which can occur at both a microscopic and macroscopic level, accumulates over time, eventually contributing to the failure of the concrete. The freezing-and-thawing resistance of the RAP concrete was quantified according to ASTM C666. This test method consists of subjecting concrete specimens to multiple freezing-and-thawing cycles while fully saturated. Weight loss and change in dynamic modulus are monitored as a function of accumulated freezing-and-thawing cycles. As may be obvious, the degree of damage sustained by the concrete due to microcracking and macrocracking under freezing-and-thawing action is reflected by its attendant loss of weight and stiffness, where material stiffness can be nondestructively measured in terms of dynamic modulus. The relative dynamic moduli were calculated from fundamental transverse frequency measurements (ASTM C215). The durability factor, DF, is used as one of the indicators of performance. The durability factor is defined as: $DF = PN/M$, where P is the relative dynamic modulus, and N and M, in this case, are the total number of cycles at which the exposure is to be terminated (300).

Three 3-by-4-by-16 inch rectangular prisms were cast from both the mHR and mHS mixtures. The specimens were exposed to several freeze-thaw cycles per day for 300 cycles. The results from this test are reported in Table 13. The relative dynamic moduli for both mixes are plotted in Figure 28 as a function of cycles.

Table 13: Freeze-thaw durability results

Mix	Number of Cycles	Avg. Mass Change (%)	Avg. Durability Factor
mHR	300	-0.01	98.0
mHS	300	-0.01	99.5

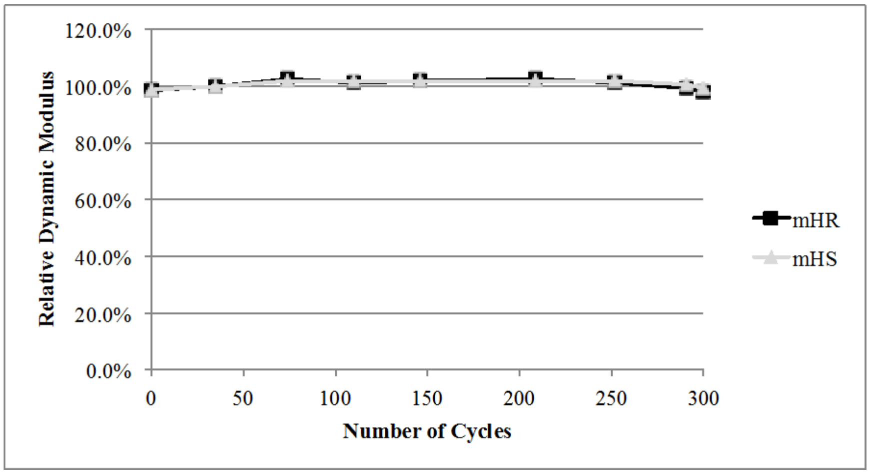


Figure 28: Relative Dynamic Modulus vs. Cycles

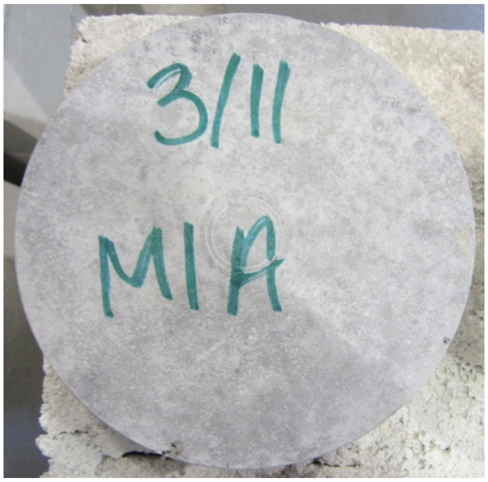
For the mHR mix, at 300 cycles the average durability factor was 98, while the HS mix maintained an average durability factor of 99.5. For reference, a value of 100 corresponds to no loss of stiffness, with decreasing values corresponding to increasing deterioration; a relative dynamic modulus of 80% or greater after 300 cycles is often assumed to indicate good freezing-and-thawing resistance. With respect to the average mass change, there was no observable mass loss for either concrete mixture. The mHR mix had a slightly smaller durability factor than the mHS mix, indicating that the RAP has a minimal effect on the freeze-thaw resistance of the concrete.

6.4 Scaling

The resistance to scaling resulting from deicing chemicals was determined following the methods outlined in ASTM C672. One 6-by-12 inch cylinder was tested from both mHR and mHS concretes. The specimens were immersed in a 0.04 g/ml solution of CaCl for 25 freeze-thaw cycles and a visual evaluation of the scaling was conducted every 5 cycles. The numerical rating applied at each evaluation step was taken from ASTM; it ranges from zero, or “no scaling”, up to 5, which corresponds to “severe scaling” (where coarse aggregate is visible over the entire surface). The condition of each specimen is presented in Table 14, while the initial and final conditions of the cylinders are shown in Figure 29.

Table 14: Scaling Surface Condition

	Surface Condition	
Day	**mHR**	**mHS**
1	0	0
5	2.3	3.0
10	3.0	3.0
15	3.3	3.3
20	3.3	3.3
25	3.3	3.7



mHR: Day 1

mHR: Cycle 25

mHS: Day 1

mHS: Cycle 25

Figure 29: Scaling Surface Conditions

Both the mHR and mHS concretes were “moderately susceptible” to scaling, and the amount of RAP replacement did not appear to affect the damaging effects of deicers.

7 SUMMARY AND CONCLUSIONS

This research investigated the feasibility of using reclaimed asphalt pavement (RAP) to replace virgin aggregates in concrete pavements. Specifically, this research considered using minimally processed RAP (i.e., simply fractionating into fine and coarse components with no washing or crushing) in this capacity for roadways in the state of Montana. This research was conducted in multiple phases.

The first phase of research used a statistical experimental design procedure (response surface methodology – RSM) to investigate mix proportioning in concrete mixtures containing RAP to achieve desired performance criteria. Based on the RSM models, two concretes were ultimately selected for further evaluation: a high RAP mix (HR) and a high strength mix (HS). These mixes were identical sans the RAP replacement rates; the HR mix, as the name implies, had a relatively large amount of RAP with 50 percent of the fines and 100 percent of the coarse aggregates replaced with RAP. The HS mix was designed to have a higher strength by using half of the RAP (25 percent of the fines were replaced and 50 percent of the coarse). These mixes were then evaluated through a suite of tests. These mixes performed well with respect to mechanical properties and durability, and were deemed suitable for applications in the state of Montana; however, the field application of these concretes was not evaluated, and the mixes contained a significant portion of cement, which hindered the economic benefits of using this recycled material in concrete.

The second phase of this research, discussed herein, was focused on: (1) the field application of RAP concretes, and (2) further optimizing the mixes in order to reduce the amount of required cement. The field application of RAP concrete was evaluated through a field demonstration project near Lewistown, MT, in which two RAP concrete test slabs (one HR slab and one HS slab) were placed on a roadway at the MSU/WTI Transcend Research Facility. The concretes in these slabs were batched, placed, and finished using conventional construction equipment, and no constructability issues were observed during their production. However, it is worth noting that the batching process was slightly complicated by the need to slump adjust mixes due to difficulties encountered in determining the water content of the RAP aggregates. Beyond this added requirement, this work indicates that constructability issues will not hinder the use of this concrete on larger pavement projects.

Once placed, the performance of these slabs was monitored for two years via site visits, and internal vibrating wire strain gauges. No observable damage (cracking or spalling) was observed on the test slabs during site visits over the two-year monitoring period. Further, the internal gauges revealed that the slabs did not experience excessive shrinkage or curling, although they did reveal that the HR slab experienced slightly more shrinkage and curling than the HS mix, which contained half as much RAP.

In regards to the mixture optimization, a total of 10 mixes with varying paste contents (and subsequently cement contents) and RAP replacement rates were carried out in the lab in an attempt to reduce the amount of cement in the mixes. Based on this investigation, two mixes were chosen for further evaluation. These mixes were identical to the HR and HS mixes from the original phase of research, with the exception of the paste content and the use of water-reducing admixtures. The modified HR (mHR) mix had a coarse RAP replacement rate of 100 percent and a fine RAP replacement rate of 50 percent, while the modified HS (mHS) mix contained half as much RAP with coarse and fine replacement rates of 50 and 25 percent, respectively. Both of these modified mixes contained approximately 6.5 sacks of cement per cubic yard, which is more consistent with conventional concrete pavements, and is significantly less than the 7.5 sacks used in the original HR and HS mixes.

These mixes were then evaluated with a suite of mechanical and durability tests. The mechanical tests performed were compressive and tensile strength, elastic moduli, and shrinkage, while the durability tests were abrasion, chloride permeability, freeze-thaw resistance, and scaling.

In regards to mechanical properties, the mHS mix met all MDT specification requirements for both compressive and flexural strengths, and had adequate elastic moduli. The mHR mix did not meet the compressive strength requirements, but did have adequate tensile capacity and elastic moduli. Further, neither mix exhibited excessive deformations associated with shrinkage. The amount of RAP had an obvious and significant negative impact on the mechanical properties. As was expected, the strength and stiffness of the concretes decreased with increasing RAP, and the deformations associated with shrinkage increased with increasing RAP content.

Both the mHR and mHS mixes demonstrated adequate durability for use in concrete pavements in Montana with the exception of chloride ion resistance, and the mHS mix generally performed better than the mHR mix. For the abrasion tests, both mixes lost very little mass and had wear depths less than 1.0 mm. Both concretes were rated as “High” for likelihood of chloride ion penetration. In regards to freeze-thaw resistance, the mHR and mHS mixes had durability factors of 98 and 99 respectively, after being exposed to 300 freeze-thaw cycles. For reference, a durability factor of 80 or more has been cited as being indicative of acceptable freeze-thaw resistance. For scaling resistance, both the mixes were rated as “moderately susceptible”.

Based on the results from this study, the following conclusions can be made:

1) RAP can be processed, and RAP concrete slabs can be batched/placed/finished with conventional concrete equipment, with no major logistical issues. Further, RAP concrete slabs will not see significant damage/shrinkage/curling throughout the first few years of use.

2) Suitable concrete mix designs containing a significant portion of RAP and conventional cement contents can be obtained by using commercially available water-reducing admixtures. The mHR and mHS mixes developed in this research contained

approximately 6.5 sacks of cement per cubic yard, and had adequate mechanical properties and durability (sans chloride permeability and compressive strength for the mHR mix) to be used in concrete pavements in the state of Montana. That being said, these mixes had significantly less paste, required the use of a water-reducing admixture to achieve the desired workability, and were difficult to consolidate. Further, the process for batching these mixes involved slump adjusting the mixes with the admixture, as the nature of the RAP aggregates made it difficult to adjust mixes for variations in moisture content. All of which, may hinder their use in real-world applications.

8 REFERENCES

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APPENDIX A: STRAIN DATA

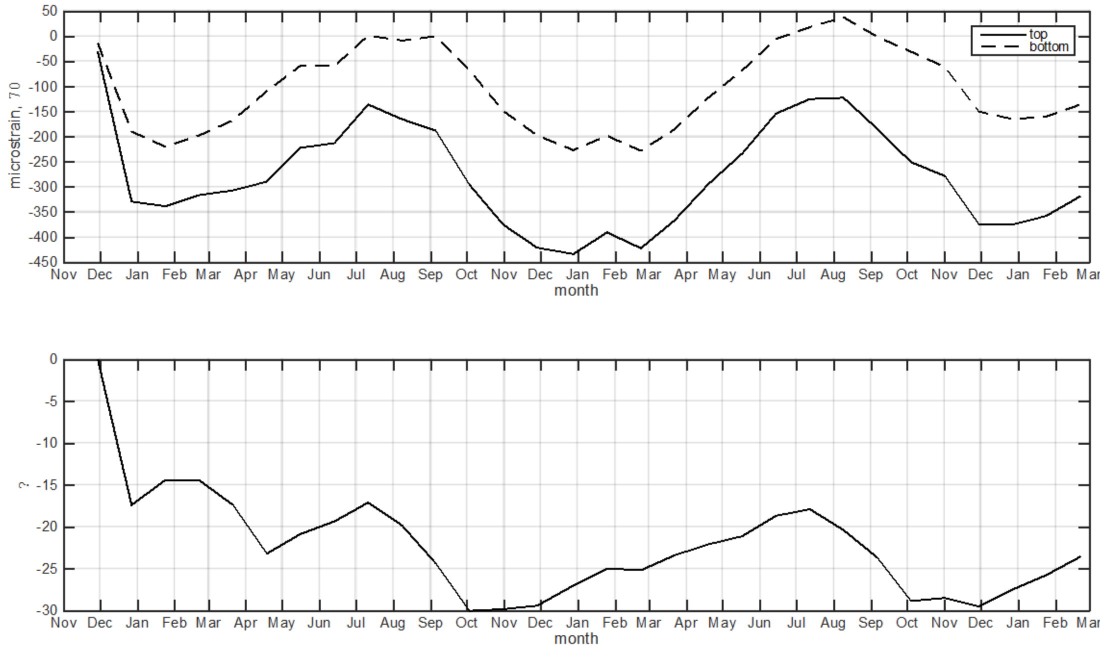

Figure 30: Strains and Curvatures for Slab 1 - L1

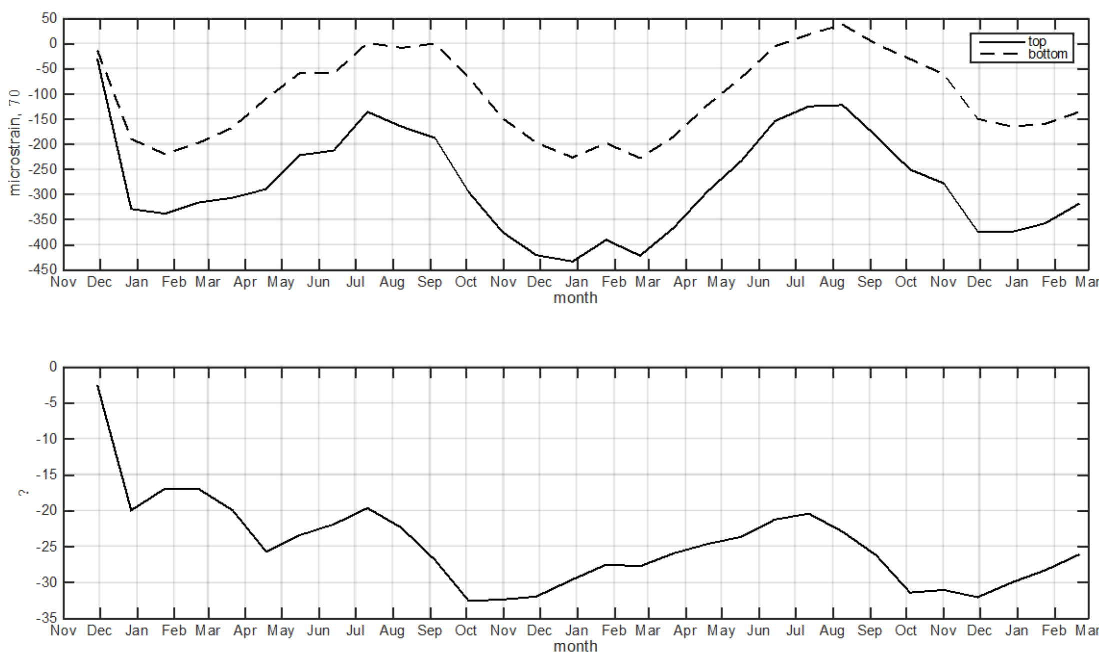

Figure 31: Strains and Curvatures for Slab 1 - L2

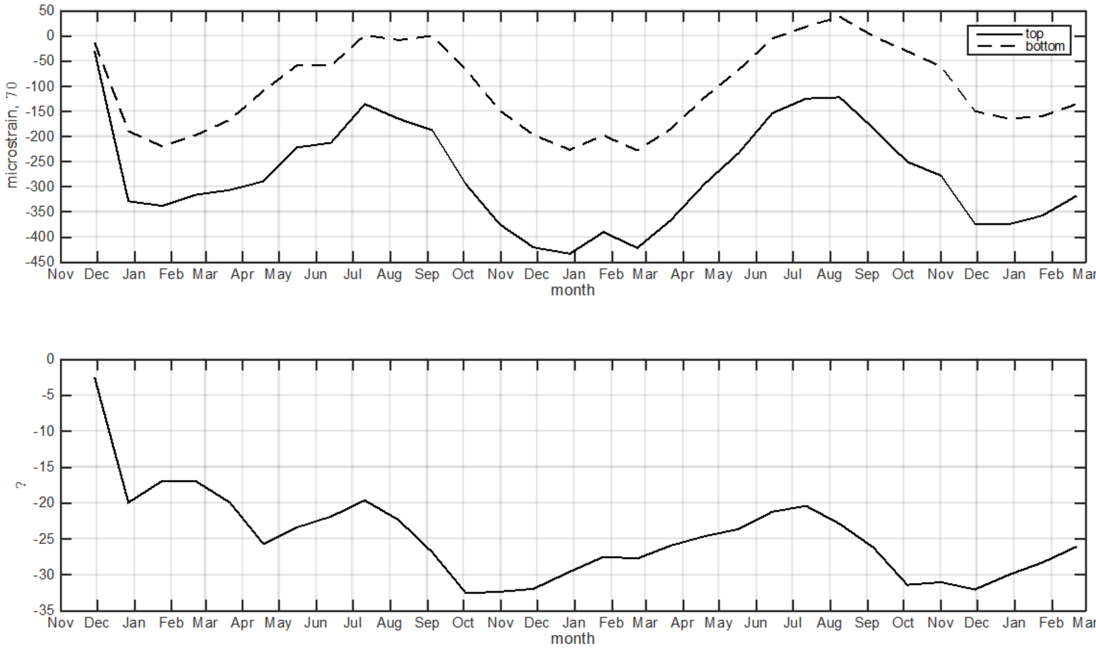

Figure 32: Strains and Curvatures for Slab 1 - L3

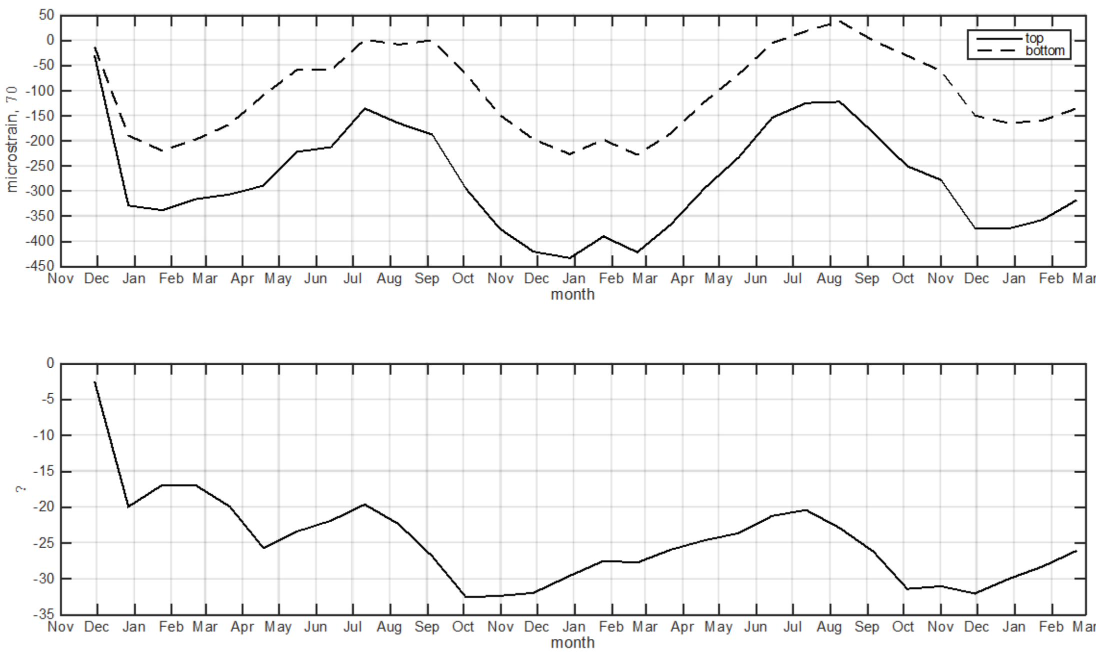

Figure 33: Strains and Curvatures for Slab 1 - L4

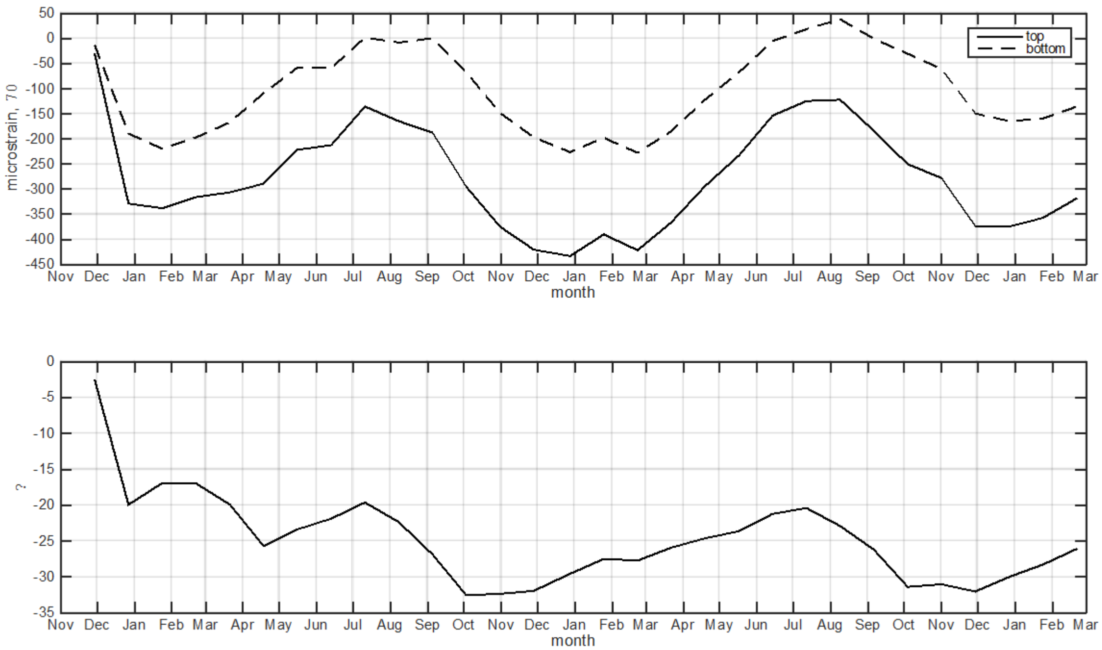

Figure 34: Strains and Curvatures for Slab 1 - T1

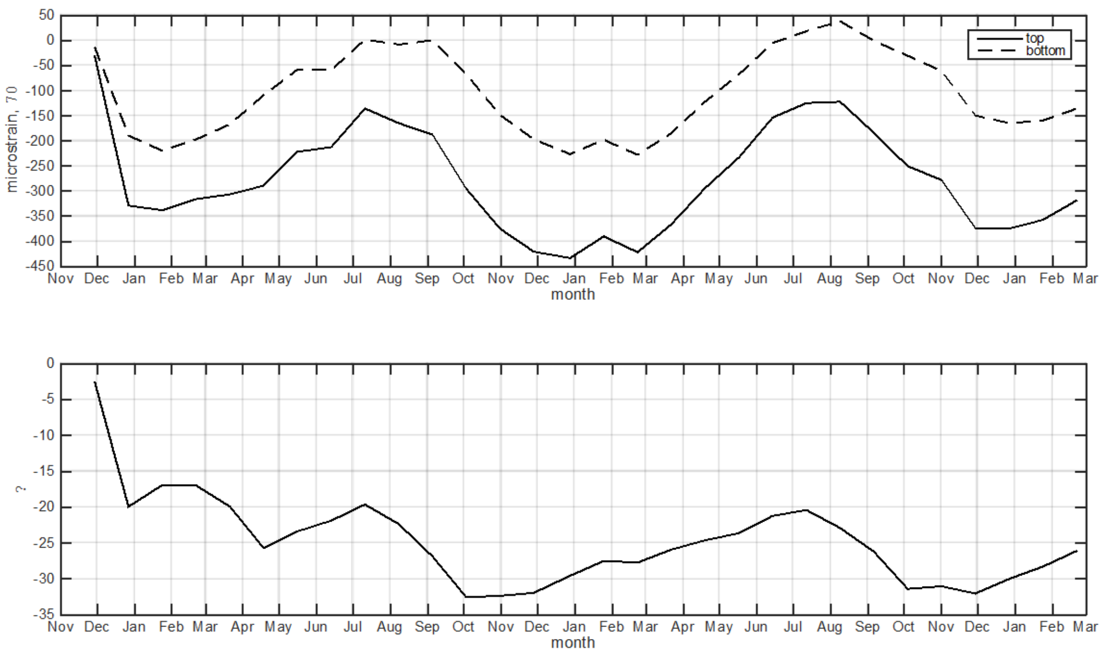

Figure 35: Strains and Curvatures for Slab 1 - T2

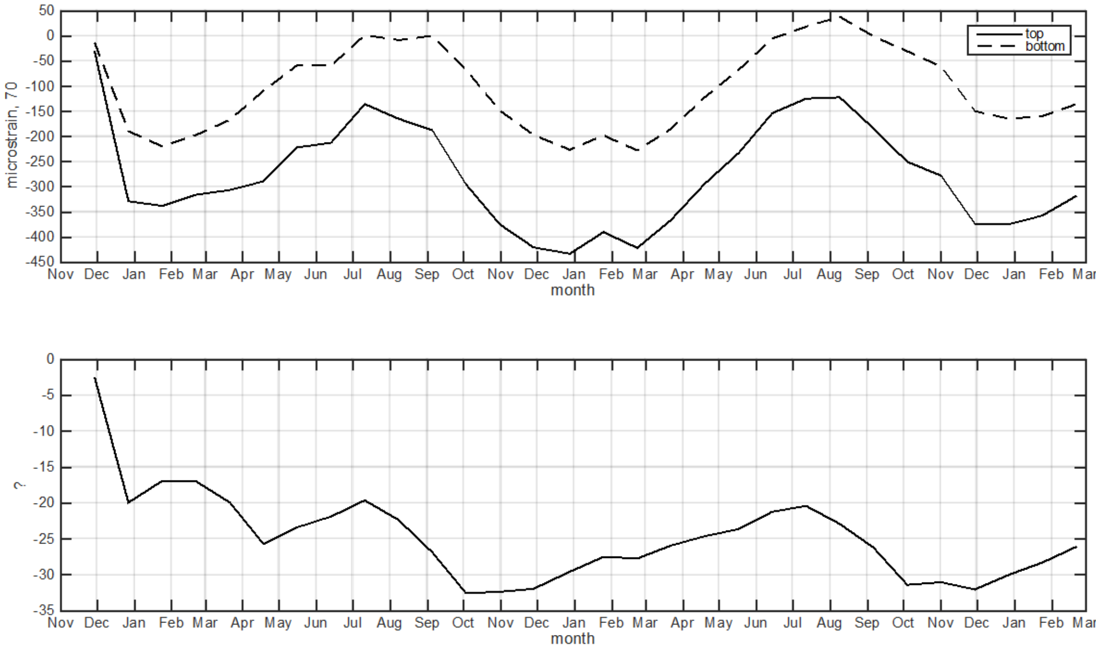

Figure 36: Strains and Curvatures for Slab 1 - T3

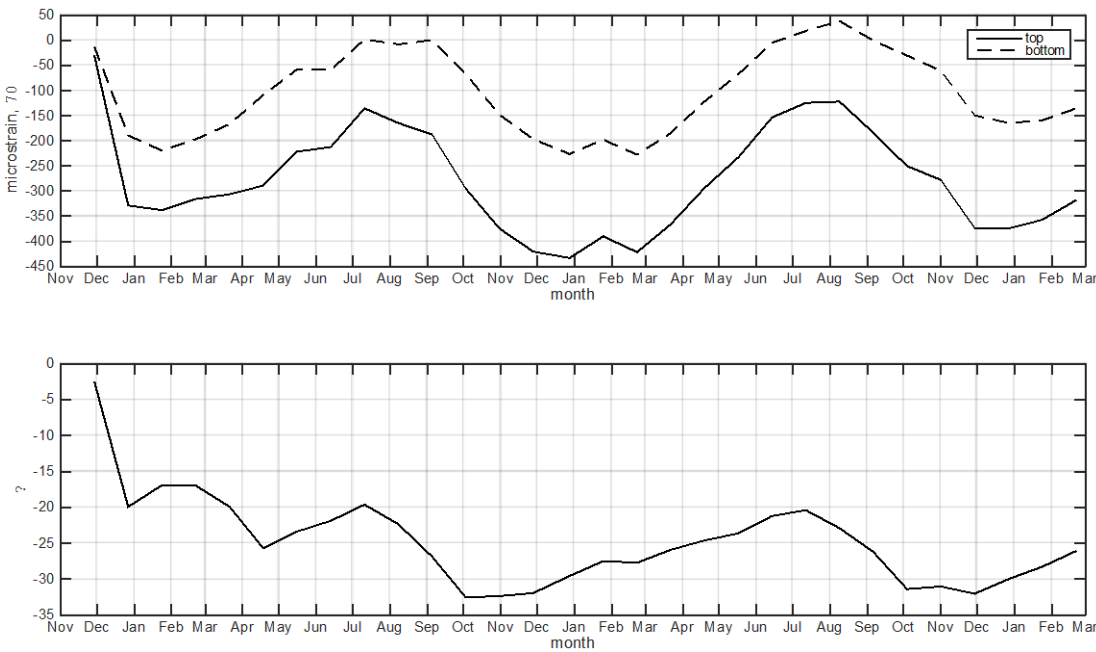

Figure 37: Strains and Curvatures for Slab 1 - T4

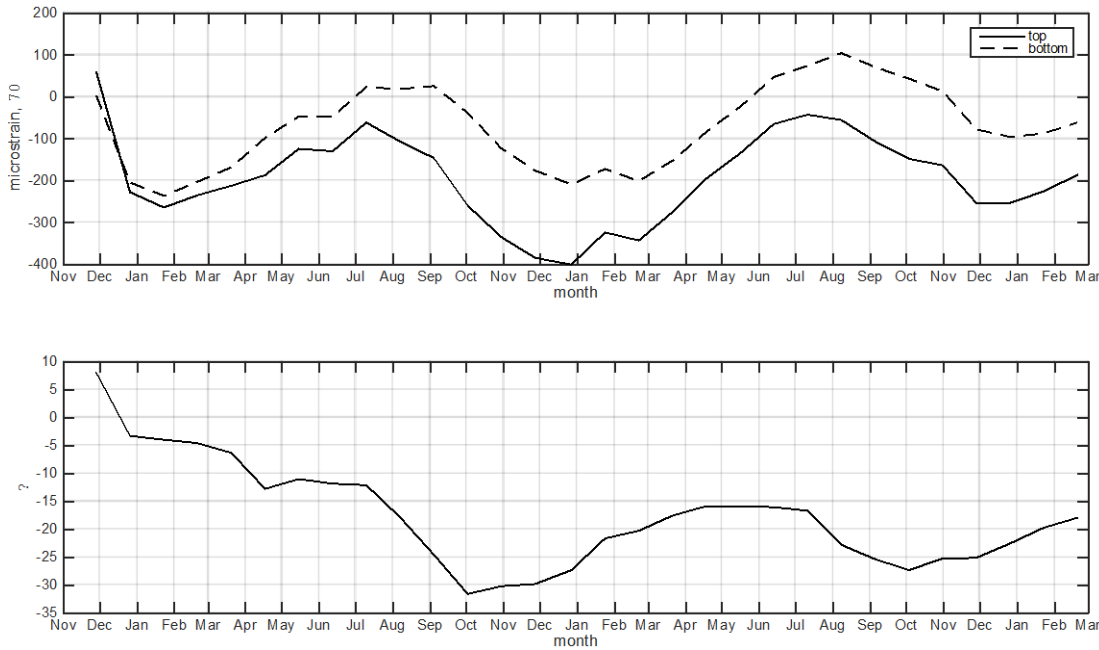

Figure 38: Strains and Curvatures for Slab 2 - L1

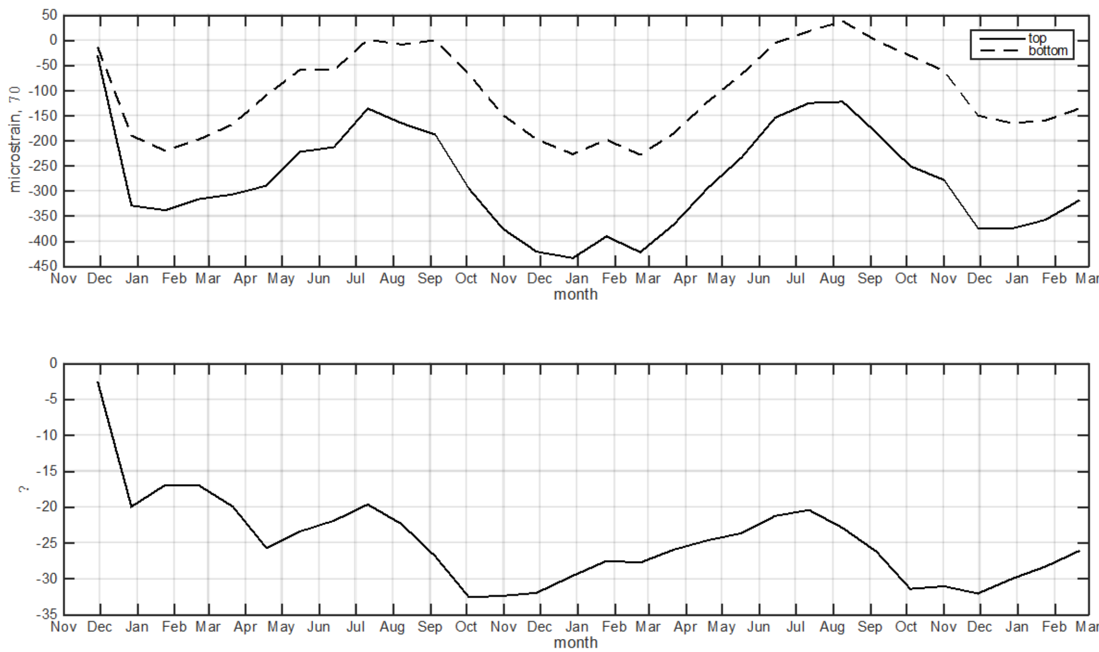

Figure 39: Strains and Curvatures for Slab 2 - L2

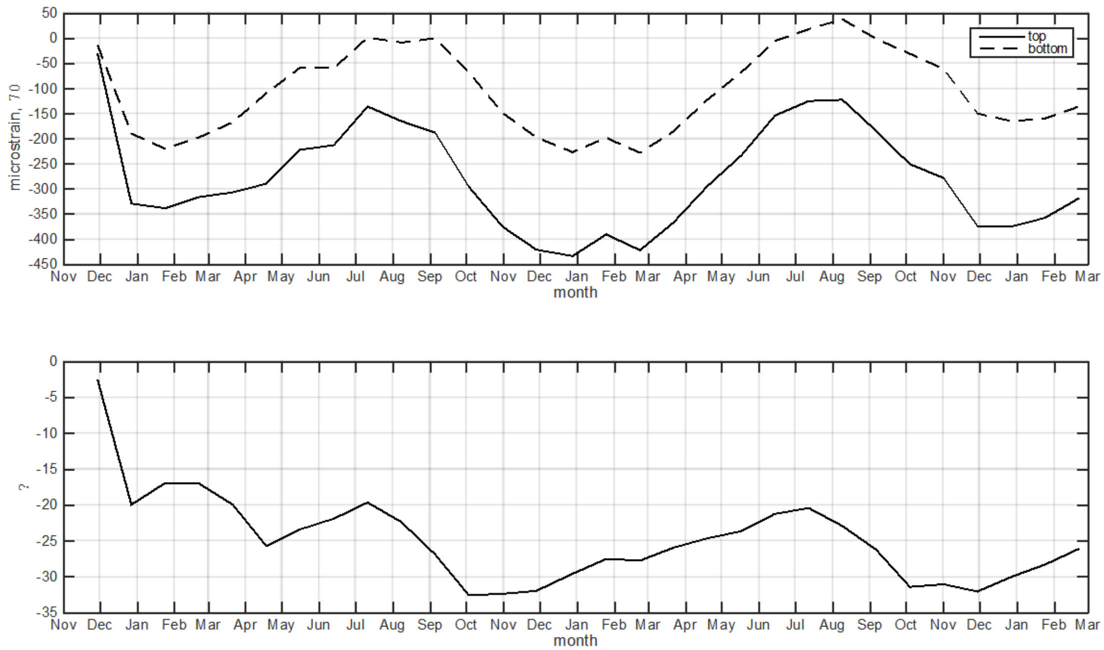

Figure 40: Strains and Curvatures for Slab 2 - L3

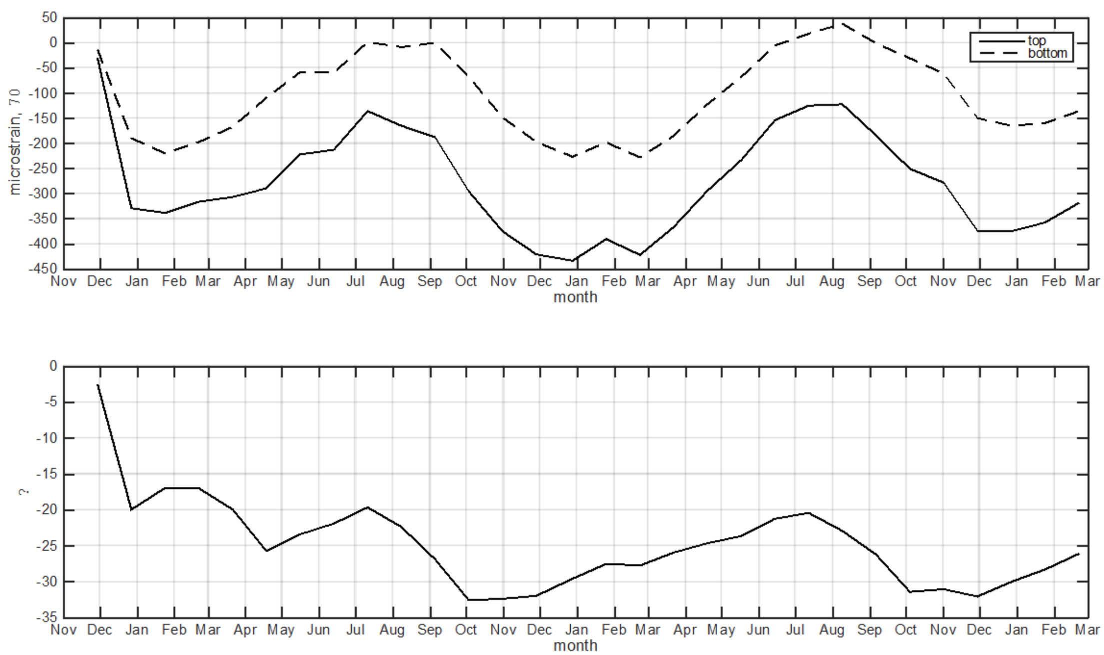

Figure 41: Strains and Curvatures for Slab 2 - L4

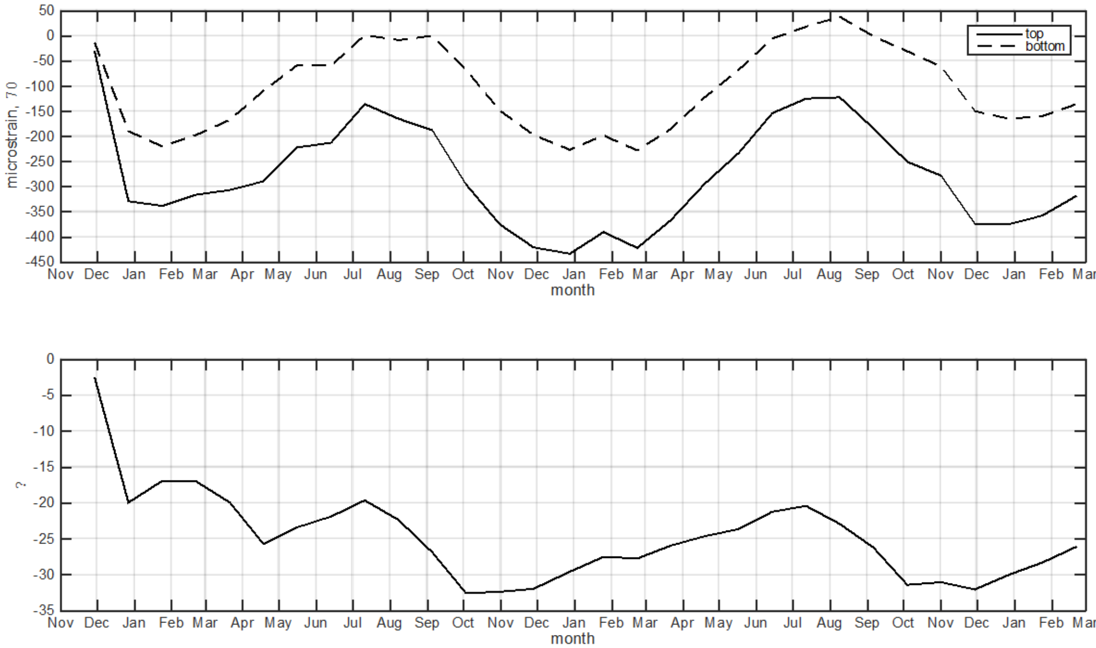

Figure 42: Strains and Curvatures for Slab 2 - T1

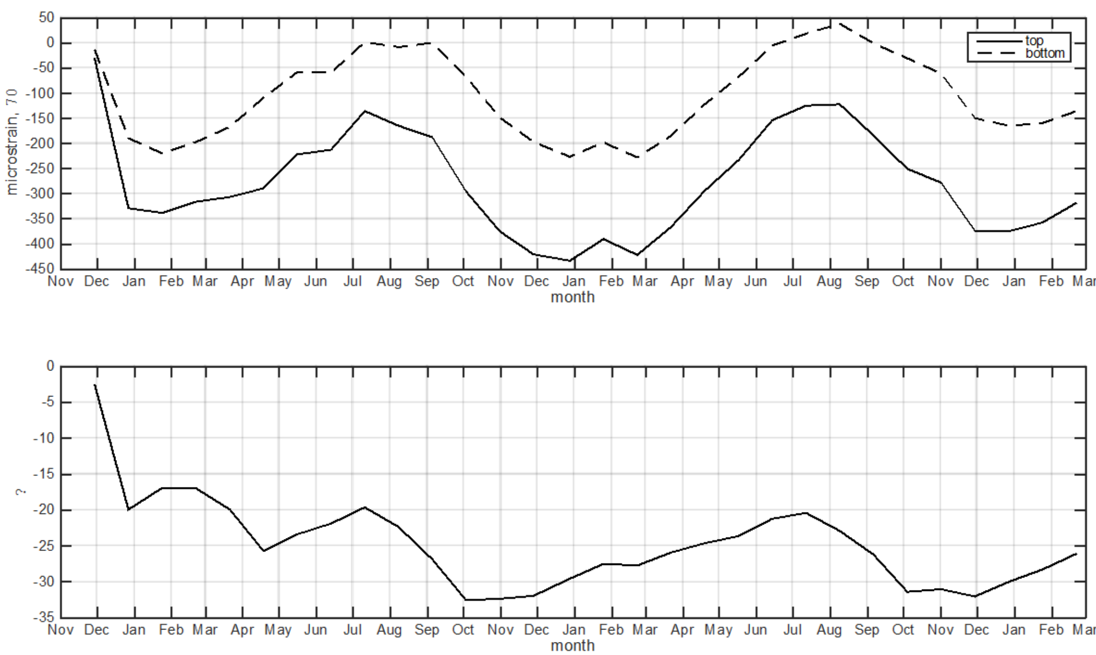

Figure 43: Strains and Curvatures for Slab 2 - T2

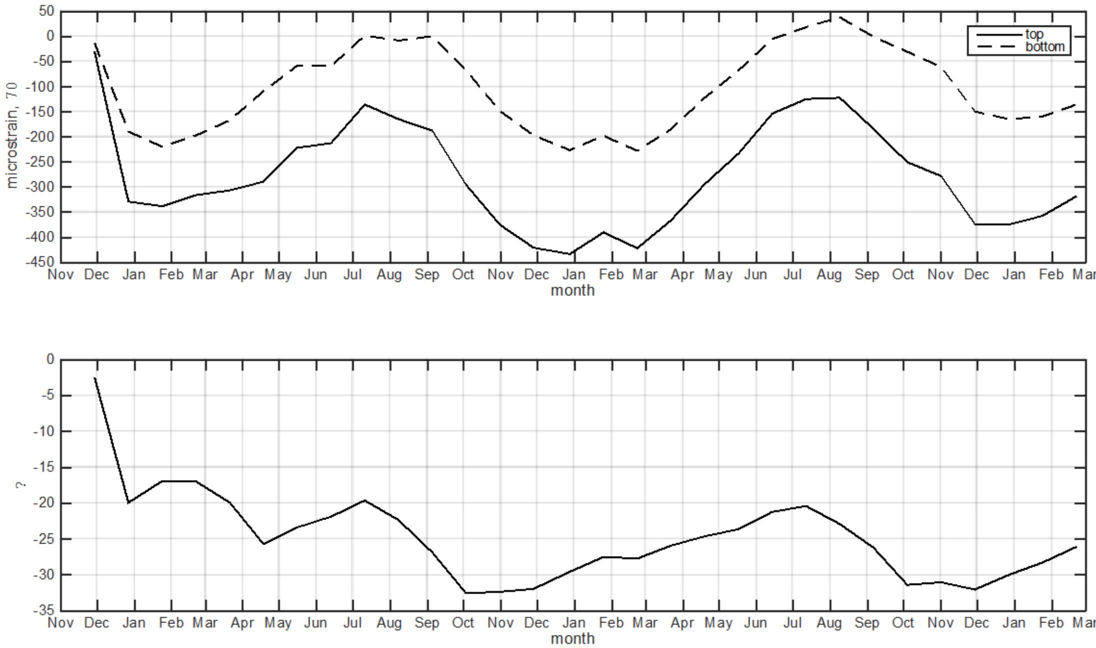

Figure 44: Strains and Curvatures for Slab 2 - T3

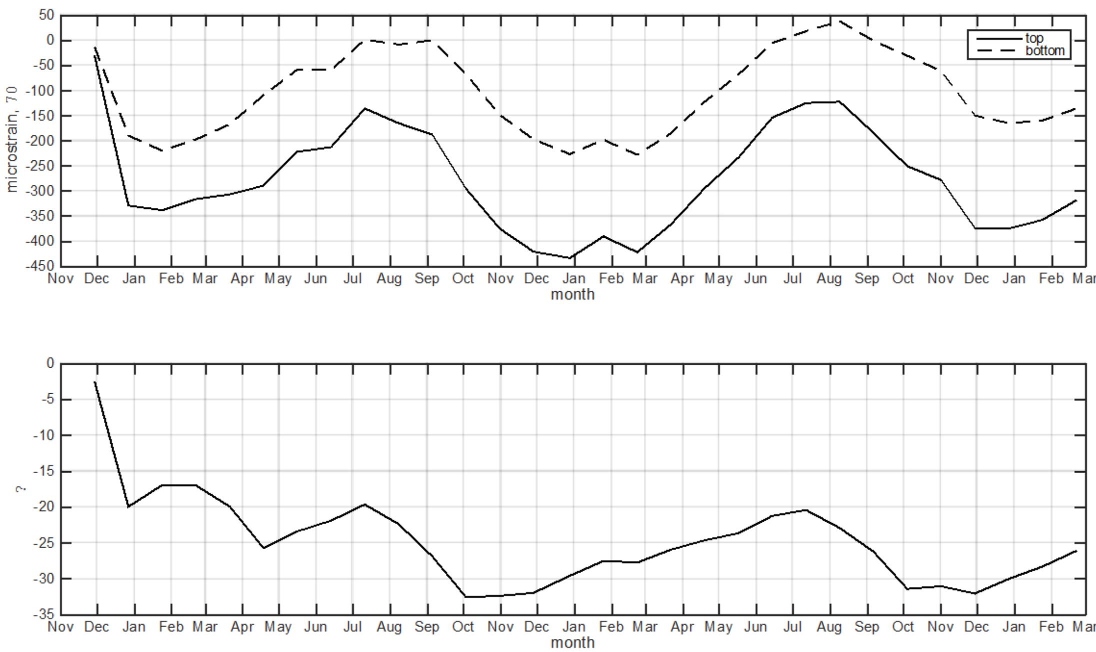

Figure 45: Strains and Curvatures for Slab 2 - T4

APPENDIX B: FHWA VEHICLE CLASSIFICATIONS

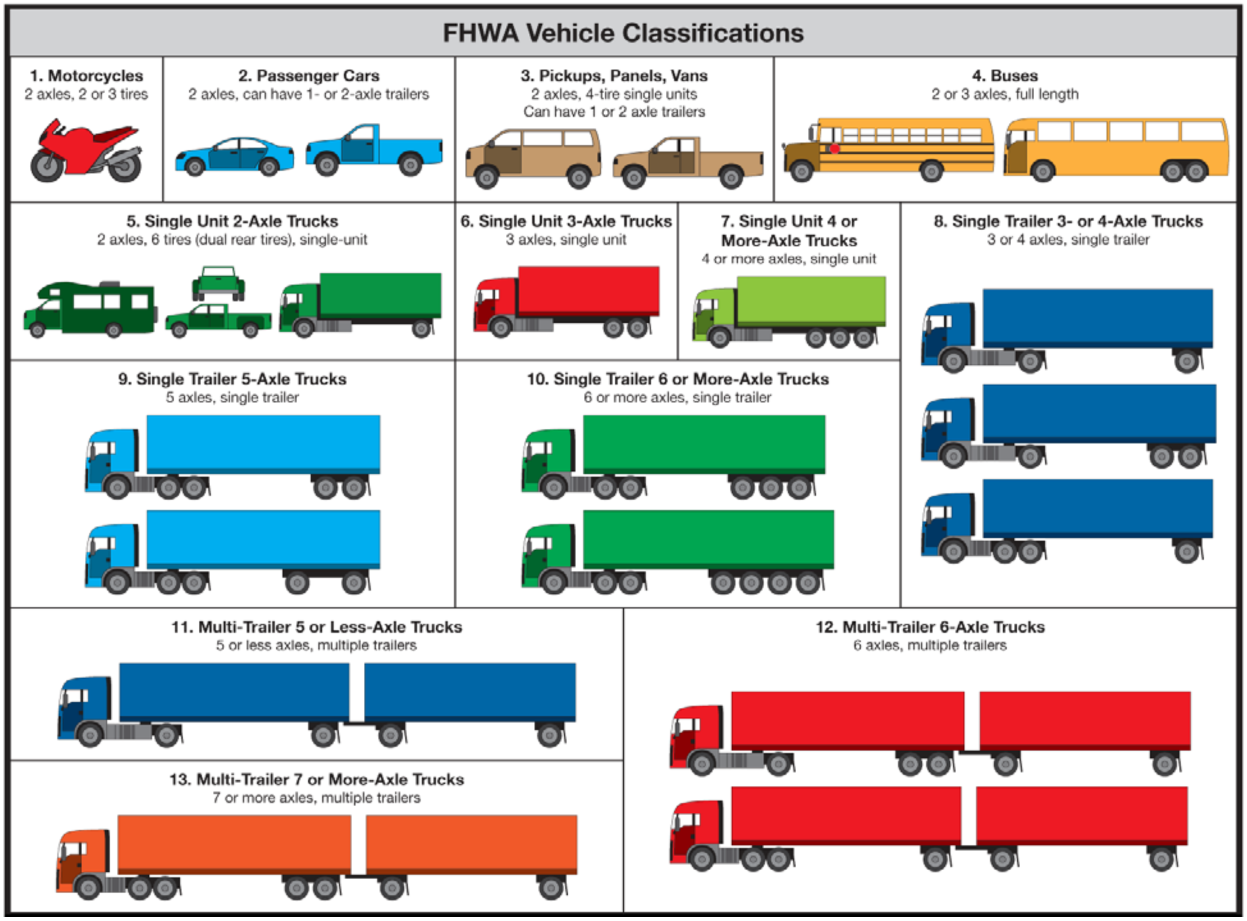

Figure 46: FHWA Vehicle Classifications
http://onlinemanuals.txdot.gov/txdotmanuals/tri/images/FHWA_Classification_Chart_FINAL.png