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Mechanical Properties of High Strength Portland Cement Concrete with Recycled Asphalt Pavement as a Percent Replacement of Coarse Aggregate

By

Andrew Joseph Fellows

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Committee Approval

To the Graduate Faculty:

The members of the committee appointed to examine the thesis of ANDREW JOSEPH FELLOWS find it satisfactory and recommend that it be accepted.

____

Dr. Andrew Sorensen
Major Advisor

____

Dr. James Mahar
Committee Member

____

Dr. Shannon Kobs-Nawotniak
Graduate Faculty Representative

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Abstract

Finding constructive uses for construction waste byproducts contributes to green engineering principles. One such plentiful material is recycled asphalt pavement (RAP). This thesis looks at the mechanical viability of including RAP in a high strength concrete mix. The mechanical behaviors studied are: freeze-thaw durability, chloride ion penetration, bond strength, ductility, strain-rate, coefficient of thermal expansion and modulus of elasticity. The tests conducted follow ASTM and AASHTO standards where possible. A few variations to the standards are made to accommodate the limitations of the Idaho State University Laboratory. In each of the tests conducted the RAP mix performed as well or better than the control mix, except for the bond strength and strain rate tests; where the testing procedure is modified. These results show that the inclusion of RAP coarse aggregate in a high strength mix is a viable solution to achieve a "green" alternative to normal concrete mixes.

Chapter 1
Introduction

1.1 Background and Motivation

Each time a road paved with asphalt is replaced, the old asphalt must be removed. With the number of construction projects that take place each year, the disposal of the old asphalt is an increasing problem. Research has been carried out to find a better use for the asphalt waste instead of filling up the landfills. Additionally, there is currently a focus on green engineering, and finding a use for the asphalt pavement is a way to achieve greener construction methods. One of the uses of this material, or Recycled Asphalt Pavement (RAP) is for the replacement of coarse and fine aggregate in pavement and concrete mixtures. Portland Cement Concrete (PCC) is a primary building material in construction projects. PCC is used in bridges, parking garages, foundations, buildings, and many other construction applications. Concrete has a high compressive strength and is very durable. However, in order to achieve a green construction material using RAP in PCC, the mechanical behavior of these mixtures must perform as well or better than traditional PCC mixes.

While RAP is a good alternative to coarse aggregate in non-structural pavement, there is a desire to be able to use RAP in structural applications. Previous studies done on the compressive strength of concrete with RAP, have found that a reduction of compressive strength takes place with the addition of RAP (Hassan et al., 2000; Huang et al., 2008; Okafor, 2010). These studies show that it is possible to use RAP as a coarse aggregate, but with a loss to compressive strength that is too great to be used in any kind of structural application (traditionally lower than 4000 psi).

Therefore, in order to achieve a high enough compressive strength to be useful in structural applications, a high-strength concrete (HSC) mix needs to be studied. In studies done in the past, Limbachiya and others tested RAP in a HSC mix (2000). The results of this study show potential for a RAP concrete mix with a high enough compressive strength to be used in structural applications. A study conducted by Capson and Sorensen show that the compressive strength of concrete with RAP can be reached with the use of a HSC mix (2013). The results of Capson and Sorensen show that a HSC mix used with RAP the compressive strengths can reach in excess of 4000 psi. Capson and Sorensen test 25 to 50% RAP for coarse aggregate replacement with all RAP percentages achieving a compressive strength over 4000 psi.

In addition to compressive strength, there needs to be testing on the mechanical properties of RAP aggregate as a coarse aggregate in a PCC mix in order to be used in a structural concrete mix. To ensure the applicability of RAP concrete, durability, bond strength, toughness, strain rate of crushing and coefficient of thermal expansion need to be studied.

This thesis studies the mechanical properties that makes RAP concrete viable for structural applications. Specifically:

- Durability of RAP concrete is important to ensure the concrete can resist weathering action.
- Bond strength is important to ensure that there will be no slippage between the concrete and the steel reinforcements.
- Toughness is the amount of energy required per unit volume to rupture the concrete.
- Strain rate of crushing needs to be evaluated as a limiting strain of .003 in./in. which is utilized by The American Concrete Institute (ACI) ACI-318 in design calculations.

If RAP concrete can meet the given requirements for the mechanical properties listed above, it can be considered a good alternative for traditional concrete mixes.

1.2 Continuation of past research

This thesis is a continuation of a past thesis project done by Tara Capson. Capson studied the compressive and tensile strength of RAP concrete. Capson wanted to eliminate the variations in compressive strength due to inconsistent RAP. Capson concluded that harvest locations have a direct connection to compressive strength of RAP as a coarse aggregate replacement. Capson also determined that replacing the coarse aggregate with the same size RAP aggregate also helped reduce the variations in compressive strength. With the use of a high strength concrete mix and sieving the RAP it is possible to use RAP concrete in a structural application. However, before RAP can be used in structural applications the mechanical and durability properties need to be understood. The purpose of this thesis is to understand the mechanical and durability properties of RAP concrete.

1.3 Problem Definition and Scope

This research examines the mechanical behavior of Recycled Asphalt Pavement (RAP) as a percent of coarse aggregate replacement in high strength concrete mixes. Using RAP concrete as a structural concrete will require different tests to ensure that the concrete can handle the multiple loading conditions. The questions this study seeks to answer are:

- Can RAP concrete improve ductility over traditional concrete mixes?
- How does RAP concrete perform under different strain rates of loading?
- Does RAP concrete follow ACI equations for Modulus of Elasticity?
- Can the RAP concrete improve durability under freezing and thawing conditions?
- What is the likelihood of corrosion and the corrosion rate of RAP concrete due to chloride ion penetration?
- Does the bond strength of RAP concrete improve over traditional concrete mixes?
- How is the coefficient of thermal expansion affected by RAP aggregate?

1.4 Research Objective

The objective of this study is to find the mechanical properties of RAP concrete under applied conditions and loadings. More specifically the objectives are to:

- Determine the strain-rate of crushing of RAP concrete.
- Determine the ductility of RAP concrete beam under a flexural load.
- Determine the bond strength of RAP concrete to steel reinforcement.
- Determine the durability of RAP concrete under rapid freeze thaw conditions.
- Determine the chloride ion penetration of RAP concrete
- Determine the coefficient of thermal expansion of RAP concrete
- Verify the ACI modulus of elasticity equations
- Compare results to traditional concrete mixes

1.5 Research Tasks & Methodology

In order to meet the objectives, a series of test and experiments are designed to give results that will describe the mechanical performance of RAP concrete. The research tasks and methodology are discussed in the following section. The methodology follows American Society for Testing and Materials (ASTM) and American Association of State Highway and Transportation Officials (AASHTO) standards with very little deviation due only to experimental limitations as stated.

1.5.1 Stain-Rate of Crushing

Strain-rate of crushing is tested by casting concrete cylinders in accordance with ASTM C192 (ASTM, 2007). The concrete cylinders are cured in a lime water bath in accordance with ASTM C511 (ASTM, 2013). The concrete cylinders are crushed in a compression testing machine at the Idaho State University structures laboratory. The concrete cylinders with the different RAP coarse aggregate have strain gauges placed on them to measure both lateral and vertical strain. The test methodology is discussed in Chapter 3, Section 9, and the results are discussed in Chapter 5, Section 4.

1.5.2 Ductility

To measure the ductility of the concrete, concrete beams with varying RAP percentages are cast. The samples are cured in a lime water bath with accordance to ASTM C511 (ASTM, 2013). Once the samples are cast ASTM C1018 is followed. It should be noted that ASTM C1018 was withdrawn in 2006, however it is still a good mechanical description of concrete (ASTM, 2006). The test methodology is discussed in Chapter 3, Section 7, and the results are discussed in Chapter 5, Section 3.

1.5.3 Bond Strength

Bond strength is tested with a push-through test. The test consists of RAP concrete samples with steel rebar exposed. ASTM C900 will be followed to measure the pullout strength of the RAP hardened concrete (ASTM, 2006) . A compression machine is used to provide the force to push the steel rebar through the samples. The test methodology is discussed in Chapter 3, Section 6, and the results are discussed in Chapter 5, Section 2.

1.5.4 Freeze-Thaw Durability

Freeze-thaw durability is followed in accordance with ASTM C666 (ASTM, 2008). Concrete cylinder samples are cast with coarse RAP at the specified percentages. The concrete cylinders are placed in a CARON freeze-thaw chamber. ASTM C666 specifies that each cylinder is subjected to 300 freeze-thaw cycles, or until the cylinders fail to maintain 60% of the initial modulus (ASTM, 2008). The modulus is tested with an E meter and with accordance with ASTM C215. The test methodology is discussed in Chapter 3, Section 4, and the results are discussed in Chapter 4, Section 1.

1.5.5 Chloride Ion Penetration

Chloride ion penetration is tested in accordance with the American Association of State and Highway Transportation Officials (AASHTO) TP-95-11 (AASHTO 2011). Sample are cast and cured in a lime water bath. The chloride penetration is tested using a proceq resipod. The test methodology is discussed in Chapter 3, Section 5, and the results are discussed in Chapter 4, Section 2.

1.5.6 Coefficient of Thermal Expansion

The CTE is tested using the guidelines from the Portland Cement Concrete Pavement (PCCP) research, which is conducted by the Federal Highway Administration (FHWA, 2011). RAP concrete is tested for its length change using linear strain conversion transducers (LSCT) and the change in temperature is controlled by a CARON freeze-thaw chamber. The test methodology is discussed in Chapter 3, Section 9, and the results are discussed in Chapter 5, Section 5.

1.5.7 Modulus of Elasticity

ACI equations are used to calculate the MOE of concrete in design. The applicability of these equations to a RAP concrete are verified by testing. The test methodology is discussed in Chapter 3, Section 10, and the results are discussed in Chapter 5, Section 6.

1.6 Thesis Overview

This thesis consists of 6 chapters. Chapter 1 is the Introduction, followed by a literary review of past research that is relevant to this study in Chapter 2. Chapter 3 discusses the testing methodology. Chapter 4 and Chapter 5 discuss the results of each test. Chapter 6 is a summary of results and future work. A bibliography is included at the end of this thesis.

Chapter 2
Literature Review

2.1 Introduction

Recycled Asphalt Pavement (RAP) has been studied for use in concrete since 1997. Most of these studies look at the feasibility of RAP concrete as an alternative to normal Portland cement concrete (PCC). However, the majority of these studies do not look at the possibility of RAP in concrete being used in structural applications. In the previous studies on RAP as an aggregate replacement, the researchers use RAP as a coarse and a fine aggregate. This thesis studies the effects of high strength PCC with RAP coarse aggregate.

2.2 RAP in Road Design

The compressive strength for road design does not have to meet the same standards as that of structural design. Having a decrease of 50% in compressive strength of RAP concrete can still meet the standards for road and pavement deign. A study done by Mathias and others (2011) concludes that RAP could be a solution to utilizing old asphalt. The Illinois Center of Transportation did a study on the use of RAP as a substitute of coarse aggregate in road design. With the addition of RAP for pavement deigns meeting the required Illinois Department of Transportation (IDOT) of 3500 psi, it is feasible to use RAP as a pavement aggregate. While the use of RAP is adequate in pavement design, there still needs to be studies on the feasibility of RAP as a concrete aggregate in a structural application.

2.2.1 Modelling of Mechanical Properties of Cement Concrete Incorporating Reclaimed Asphalt Pavement

Mathias and others (2011) test the viability of using RAP as an aggregate in cement concrete for the solution of the inability to landfill old asphalt pavement. Mathias and others use different amounts of

RAP replacement, and test for compressive strength, tensile splitting, and modulus of elasticity. The conclusions of the study show:

- The compressive strength decreases with the addition of RAP.
- The tensile splitting test results show a decrease in strength with the addition of RAP to the mix.
- The modulus of elasticity experiences a decrease with the addition of RAP.
- RAP concrete mixes can be used in road and pavement deign.

2.2.2 Fractionated Reclaimed Asphalt Pavement (FRAP) as a Coarse Aggregate Replacement in a Ternary Blended Concrete Pavement

Illinois Center of Transportation performed an extensive literature review for this study, a few of the more relavent studies are chosen and summarized in this thesis, for a complete literature review, refer the Illinois Center of Transportation report (Brand, A., Roesler, J., Al-Qadi, I., Shangguan, P. "Fractionated Reclaimed Asphalt Pavement (FRAP) as a Coarse Aggregate Replacement in a Ternary Blended Concrete Pavement" Illinois Center of Transportation Research Report, 2012)

In a study done by the Illinois Center of Transportation (2012), the viability of using RAP as a coarse aggregate in concrete pavement is studied. The compressive strength, split tensile strength, flexural strength, modulus of elasticity, dynamic modulus, shrinkage, and freeze-thaw durability. The Illinois Center of Transportation evaluated concrete with different amounts of RAP replacement of coarse aggregate. The amount of RAP replacement was 20%, 35% and 50% mix. The conclusion of this study indicate that the slump increase, and the unit weight decreases. The strength parameters show that the addition of RAP at any percent results in decrease in compression, flexural and split tension. The freeze-thaw test shows that the inclusion of RAP may reduce the durability, but it still meets the requirement at 300 cycles. The results show that up to a 35% RAP meets the required 3500 psi while the 50% RAP falls just short of that requirement by 0.3%. The conclusions of this study are:

- Compressive and tensile strength decrease with the addition of RAP.
- Slump increase with the addition of RAP.
- Flexural strength decrease with the addition of RAP.
- RAP may reduce the durability over a normal concrete mix.

2.3 100% RAP Replacement

In a study done by Okafor (2010), 100% RAP coarse aggregate replacement is compared to 100% RAP fine aggregate in PCC. This study does not look at different percent replacements of RAP. Another study done by Huang and others (2008) also looks at 100% coarse and fine RAP aggregate replacement. Hassan and others (2000) also looks at RAP for both fine and coarse aggregate in a concrete mix.

2.3.1 Performance of Recycled Asphalt Pavement as Coarse Aggregate in Concrete

Okafor (2010) conducted studies comparing 100% coarse RAP to 100% virgin gravel aggregate. The results of the studies show concrete with coarse RAP to be more durable then virgin coarse aggregate. The study done by Okafor looks at six different mixes with different water/cement ratios and mix proportions are made up using 100% RAP coarse aggregate. The RAP concrete is subjected to different tests, including compressive and flexural tests. The results of this study are summarized as follows:

- RAP aggregate has a lower specific gravity and water absorption that the natural aggregate.
- RAP concrete is less workable than natural gravel aggregate.
- Concrete with RAP as coarse aggregate are found to be lower in compressive and flexural strength than concrete with natural aggregate.
- The strength of the RAP concrete is dependent on the bond strength of the asphalt-mortar coating on the concrete.

- RAP is feasible to use in concrete in a low to middle strength applications.

2.3.2 Laboratory investigation of Portland cement concrete containing recycled asphalt pavements

Huang and others (2005) conducted a study where they test four different mix designs of Portland cement concrete with 0% RAP replacement and 100% RAP replacement. The four different mixes are

1. Control no RAP coarse, no RAP fine aggregate
2. No RAP fine, 100% RAP coarse aggregate
3. 100% RAP fine, no RAP coarse aggregate
4. 100% RAP fine and 100% RAP coarse aggregate

The same water to cement ratio is used for each mix design, and the RAP is laboratory-made. The compressive strength and split tensile strength are tested and from the results of the tensile test a toughness index (TI) is calculated.

The compressive test is carried out on 4x8 in cylinders at 3, 7, 28 days at 25°C with 3 cylinder per mix design for each day tested for a total of 36 cylinders. The compressive test is carried out following the ASTM C39 standard. An MTS machine is used to conduct the split tensile strength. The split tensile testing is done at 3, 7, 14 and 28-days at 25 °C, with a load rate of 1.0 MPa. The TI is a parameter that describes the toughness in the post-peak region. The TI is calculated from the indirect tensile test results.

The results of the study show a decrease in compressive strength and split tensile strength. The biggest drop in compressive strength is with both fine and course RAP (mix 4 above) with a 72% decrease in compressive strength at 28 days, and a 68% decrease at both 3 and 7 days. The concrete with the best results is the 100% coarse RAP and 0% fine RAP aggregate with a decrease in compressive strength of 41% at 28 days, 32% decrease at 7 days and 26% decrease at 3 days. The split tensile test shows a decrease in tensile strength that is significant in the mixtures that contain either both fine and course

RAP and only fine RAP. The mixture with coarse RAP shows a slight decrease in tensile strength with only a 5% decrease in strength at 28 days, but for 3, 7 and 14 days there is a 18%, 11% and 20% decrease in tensile strength respectively. The toughness increases with the addition of RAP; however, the concrete with only fine RAP saw toughness close to that of the control mixture.

The conclusions of this study show that:

- That concrete with only coarse RAP has the least amount of reduction in compressive strength and tensile strength when compared to the control.
- Generally, the higher the RAP percent in the concrete, the lower the strength and higher the toughness.
- Concrete made with RAP has a much higher toughness than concrete with natural aggregate.

2.3.3 The use of reclaimed asphalt pavement (RAP) aggregates in concrete

Hassan and others (2000) presents a laboratory study of PCC with RAP to substitute natural aggregate. The study is done with concrete mixes made with various combinations of natural and reclaimed aggregates. The different mixes of concrete aggregate utilized in this study are:

1. Control mixture using natural sand and natural gravel.
2. Mixture using RAP for fine and coarse aggregate.
3. Mixture using RAP for coarse and natural sand for fine aggregate.
4. Mixture using RAP for coarse and natural sand where 30% of the Portland cement is substituted with fly ash (FA)

All of the mixtures have the same water/cement ratio. Each of the mixtures are tested for: Compressive strength, Flexural strength and toughness, Porosity, and Permeability.

The compressive strength is tested at 3, 7, and 28 days. 100 mm cubes are cast for the compression test. The flexural strength and toughness is carried out on concrete prisms of size 100 x 100 x 500 mm. The prisms are simply supported and symmetrically loaded with two-point loading. The loading increases gradually until failure at mid-span along with the deflection being measured.

The results of the compression test show that the concrete with no RAP performs the best, with more than double the compressive strength over the RAP concrete. The results show no improvement with the FA added. There is a 63% reduction in strength from the control to the concrete with RAP as coarse aggregate. The flexural strength testing shows similar results; there is about a 35% reduction in flexural strength. The slope of the load deflection curve also shows that the concrete containing RAP has a lower modulus of elasticity.

The conclusions of the study are:

- The use of RAP reduces the strength properties of concrete.
- RAP concrete can be used in low-strength and high ductility applications.

2.3.4 Results of 100% RAP

While using 100% RAP for fine aggregate, coarse aggregate or both show potential for road and pavement design. The reduction in compressive strength does not achieve the required strength for structural applications. However, Okafor (2010) concludes that RAP concrete can be used for middle to low strength concrete. Furthermore, Huang and others (2008) conclude that a 100% coarse RAP mixture shows less strength reduction and a significant increase in toughness. Hassan and others (2000) suggest that RAP concrete be used in non-structural applications. The three studies all concur that the inclusion of 100% fine and/or coarse RAP in concrete significantly lowers the strength of the concrete. For a structural application of RAP concrete, there needs to be more research on different percentages of RAP replacement. Using different percentages of RAP may reduce the reduction of compressive strength of

RAP concrete. The aforementioned studies show that a fine RAP aggregate increases the loss of strength in the concrete, rendering it ineffective in structural concrete applications.

2.4 RAP at Different replacement percentages

While RAP concrete has shown to be effective for roadway and pavement design, there needs to be more research done to achieve a high enough compressive strength so it can be used in structural applications. Using 100% RAP has too large of a decrease in compressive strength to be used in structural applications. Studies have been conducted that look at different percentages of RAP to try to lower the reduction of compressive strength. If the percentage of RAP is reduced it can lead to a higher compressive strength than using 100% RAP. In a study done by Huang and others (2006) different percentages of RAP aggregate is studied. In this study both fine and coarse aggregate are looked at, ranging from 10% to 100% for both coarse and fine aggregate. In a study done by Hossiney and others (2010) different percentages of RAP are examined. Three different mixes are studied: 10, 20 and 40 percent, with both coarse and fine RAP aggregate tested. In a study by Al-Oraimi and others (2007) PCC with 25, 50, 75 and 100% RAP is examined. This study looks only at coarse RAP aggregate. Bilodeau and others (2010) look at RAP concrete with steel fibers with a fixed amount of RAP, at 0, 40 and 80 percent replacement. Delwar and others (1997) study the use of RAP with different percentages, ranging from 25% to 100%.

2.4.1 Mechanical properties of concrete containing recycled asphalt pavements

In the study done by Huang and others (2006), the mechanical properties of concrete containing RAP are studied. Compressive strength and split tensile tests are used to assess the mechanical properties of concrete at 28-days of curing. The cement used is a Type I Portland cement. Two types of RAP are used; a coarse and a fine aggregate. A total of 17 concrete mixtures are prepared in this study. The ratio of water to cement remains the same for all the mixes tested. For each of the mixes there are three 6x12 inch cylinder specimens, tested for the compressive strength and elastic modulus, and six circular plate specimens are cut from the cylinders for the determination of the indirect tensile strength and TI. The compression test and modulus of elasticity are carried out in accordance with ASTM C39 and C469, respectively. The tests are performed at a curing time of 28 days and at 25°C. A split tensile strength test is performed on the specimens at 28 days with a loading rate of 0.01in/min. The TI is calculated from the indirect tensile test results.

The results of the study show that the compression strength decreases with the addition of RAP. The compressive strength with coarse aggregate decreases more than that of the concrete with fine RAP replacement. The decrease is approximately 75% from the control which contains no RAP to 100% coarse RAP replacement. There is less of a decrease when the RAP replaces the fine aggregate, with approximately 50% decrease in compressive strength from the control batch and the 100% replacement. With about 20% RAP replacement the decrease in compressive strength for coarse replacement is about 37%. RAP concrete with both coarse and fine RAP showed a greater decrease in compressive strength then that of only coarse or fine RAP replacement. The split tensile test results are similar to that of the compressive strength. The coarse RAP replacement has less of a decrease in the split tensile test than that of the compression test. The fine aggregate RAP replacement is very similar to the compressive results. The elastic modulus steadily declines with the addition of RAP, meaning that the “stiffness” of the concrete decreases with the added RAP. The TI is increased with the addition of RAP replacement. Fine RAP has a much higher increase at 100% RAP replacement then that of 100% RAP coarse replacement. The slump of the RAP concrete increases over concrete with no RAP, slump of higher amounts of RAP decrease dramatically. The concrete mixes with fine RAP replacement at 100% show a slump of almost 0.

The results of this study are:

- Concrete with RAP shows a systematic reduction in compressive and tensile strength and modulus of elasticity, regardless of coarse and fine RAP aggregate replacement.

- The higher the RAP amount, the lower the compressive strength, tensile strength, modulus of elasticity and higher the toughness.
- Concrete with fine RAP show a relatively small reduction in strength and significant increase in toughness.
- Concrete made with RAP has a much higher energy-absorbing toughness then concrete without RAP.

2.4.2 Concrete Containing RAP for Use in Concrete Pavement

In the study done by Hossiney and others (2010) two different types of aggregate are compared, RAP and virgin natural stone coarse aggregate. The RAP is separated into a coarse and fine aggregate using a #4 sieve. The RAP is collected at an asphalt plant in Gainesville, FL. For the virgin aggregate, a porous limestone and silica sand is used for the coarse and fine aggregates respectively. The different specimens are separated into different mixes containing both mixes have RAP replacement of fine and coarse aggregate. The different mixes that are tested are RAP-1 and RAP-2. RAP-1 has RAP percent replacements of 0, 10, 20, and 40%, while RAP-2 has RAP replacements of 0, 20, and 40%. RAP-1 is coarser while RAP-2 has a lower water to cement ratio and a higher fine RAP replacement. The slump, compressive strength, modulus of elasticity, coefficient of thermal expansion, splitting tensile, and flexural strength are tested according to ASTM standards. The hardened concrete tests are performed at 14, 28 and 90 days.

The results of the test show that the slump of the control concrete for RAP-1 is 108 mm. The slump increases when more RAP replacement is introduced. The slump increases to 134, 158, and 178 mm for 10, 20, and 40% replacement, respectively. For RAP-2, which has a higher RAP replacement of fine aggregate, the slump increases with more RAP added. With a higher water to cement ratio (W/C) the slump difference is less from the control to the highest RAP replacement percent. For the highest W/C ratio, with the highest RAP of fine aggregate, the slump remains the same.

The test results for the compressive strength show that for RAP-1 the compressive strength for 40% replacement experience a 55% decrease in strength at 14 days. That decrease remains the same for 28 days and 90 days. The 20% RAP replacement experiences a decrease of 41%, 32%, and 35% for the 14, 28 and 90 days respectively. The results of the flexural test and the splitting test saw less of a decrease in strength for all curing days. The decrease in compressive strength is higher for the RAP-2, which contains a higher percentage of fine RAP. The decrease is approximately 50% for 28 days with a 40% replacement. The flexural and splitting tensile strength decrease as RAP is introduced. The MOE decreases with the addition of RAP. The decrease is higher for the mixtures with more fine aggregate. The conclusions of the study are:

- Compressive strength, splitting tensile strength, and flexural strength decreased with the addition of RAP.
- MOE decreases with the addition of RAP.
- The coefficient of thermal expansion does not appear to be affected by RAP content.

2.4.3 Recycling of Reclaimed Asphalt Pavement in Portland Cement Concrete

In a study done by Al-Oraimi and others (2007), reclaimed asphalt pavement (RAP) is used in PCC as aggregate replacement. This study uses a coarse aggregate replacement at 25, 50, 75 and 100% by weight. The fresh concrete is tested for slump according to ASTM C143-98. The hardened concrete is tested for MOE, compressive strength and flexural strength. There are two different types of control mixes: Mix 30 and Mix 50. The mixes are designed to have a 28 day compressive strength of 33 MPa and 50 MPa, respectively. The control mixes have a ratio of 1: 1.9: 2.9: 0.5 and 1: 1.7: 2.5: 0.45 for cement to fine aggregate to coarse aggregate to water, for Mix 30 and Mix 50, respectively.

There are twelve 100 mm cubes, three 150 mm cubes, three 150 by 300 mm cylinders, and three 100 by 100 by 500 mm prisms cast for each mix. The 100 mm cubes are tested for compression at 7, 14, 28, and 90 days of curing. The cylinders are tested for MOE and compressive strength after 28 days according to

ASTM C469-94 and ASTM C873, respectively. The prisms are tested for flexural strength at 28 days in accordance with ASTM C78.

The results of this study show that the compressive strength in both Mix 30 and Mix 50 decrease in strength with the increased RAP replacement. Mix 50 has higher control strength, but when RAP is added to the mix, the compressive strength fell at a faster rate than Mix 30. At 100% RAP there is a 58% reduction in strength in both mixes. The results of the flexural strength show a reduction in strength of 33% for Mix 30, while Mix 50 saw a reduction of 29% for the 100% RAP. The MOE tests are compared to the values that can be expected from ACI 318-83. The results show a decrease in MOE as the RAP percentage is increased. The slump experiences a significant decrease with the addition of RAP. With 25% RAP the slump decreases by about 40% for Mix 30, and only decreases about 20% for Mix 50.

The conclusions of the study are:

- The slump decreases with the increase in RAP.
- Compressive and flexural strength also decrease with the addition of RAP.
- The relationship between flexural strength, elastic modulus and compressive strength for the RAP mixes agree with that for normal PCC.
- The results indicate the viability of RAP as an aggregate in non-structural concrete applications.

2.4.4 Laboratory and in situ investigations of steel fiber reinforced compacted concrete containing reclaimed asphalt pavement

A study done by Bilodeau and others (2010) looks at steel fiber reinforced concrete containing RAP. The steel fibers used are 6 cm long with a diameter of 0.75 mm. The three different mix designs with a fixed amount of RAP are used; mixes of 0, 40 and 80% by weight of the aggregate. The mixes are referred to as F0%, F40% and F80%, respectively. The RAP is sieved to ensure that the aggregate is properly sized. F0% has a water to cement ratio(W/C) of 0.508 while F40% and F80% have a W/C ratio of 0.516.

The study looks at compressive strength, modulus of elasticity and tensile splitting strength. To perform the compressive modulus and the tensile splitting test, the same specimens are used. The compression strength is tested at 28 days using a 10 cm by 20 cm cylinder. The modulus of elasticy and the tensile splitting strength test use a 16 cm by 32 cm cylinders and are tested at 28, 63 and 360 days.

The results of this study show there is a decrease in the compressive strength. F0% has an average compressive strength of 32 Mpa, while F40% has a compressive strength of about 17.5 Mpa That is a decrease of about 45%. F0% has a much higher standard deviation then F40%. F80% has a 63% decrease from F0%. The tensile spitting test results also show a decrease in strength with the addition of RAP. F40% has about a 15% decrease at 28 days. The decrease expands at 63 days, than decreases at 360 days. While the F80% mix shows the same behavior but at a reduced strength, of about 40%.

The conclusions of this test are:

- The higher the RAP content, the higher the decrease in strength and modulus of elasticity.
- With an increase in cement content, the strength and the modulus of the specimen are both increased.
- MOE decreases with the addition of RAP.

2.4.5 Use of Reclaimed Asphalt Pavement as an Aggregate in Portland Cement Concrete

Delwar and others (1997) investigate a number of different mixtures with varying percent replacements of coarse and fine aggregate with RAP (0%, 25%, 50%, 75%, and 100%) and two water-cement (w/c) ratios (0.4 and 0.5). The authors test the concrete with RAP replacement for:

1. Unit weight, air voids and slump of fresh concrete.
2. 7- and 28-day compressive strength.
3. Examine the stress-strain behavior of the RAP concrete.

The authors state a concern that the aggregates from RAP have the potential to be contaminated by a variety of different materials. There have been studies on contamination problems in RAP, and this study concluded that it should not be a problem for concrete used in pavements, retaining walls, bridges, and others, unless it is contaminated by chlorides or sulfates. The studies conclude that:

- Concrete made with virgin aggregate is stronger than concrete with any percentage of RAP.
- For any combination of RAP and virgin aggregate, higher w/c ratio yields a concrete with a lower compressive strength, thus more cement needs to be added.
- RAP concrete enhances the ductility and elastic behavior of the concrete.

2.4.6 Results of RAP at Different Percentages

In studies (Huang et al., 200; Hossiney et al., 2010; Al-Oraimi et al., 2009; Bilodeau et al., 2010; Delwar et al. 1997) done on RAP concrete with different percentages, it has been found that the lower the percent RAP the higher compressive strength. The addition of fine RAP aggregate has a greater reduction in compressive strength as well as flexural strength, than the addition of coarse RAP aggregate. However, for RAP to be used in structural applications the reduction of compressive strength needs to be minimal while also saving enough coarse aggregate to achieve a greener concrete. RAP concrete needs to meet workability standards in order to be used as a structural concrete. Some of the studies stated above also look at slump. While Hossiney and others (2010) conclude the slump of the concrete increases when more RAP is introduced to the concrete, Al-Oraimi and others (2009) found that the slump decreases with the addition of RAP aggregate. With conflicting data, slump tests needs to be studied further. While RAP concrete performs better at lower aggregate percentages, but still not achieving a desired compressive strength, therefor, a high-strength concrete (HSC) needs to be tested. A HSC containing RAP coarse aggregate can possibly achieve enough strength to be used in a structural application.

2.5 High Strength concrete containing RAP aggregate

In order for RAP aggregate to be used in structural concrete applications, a compressive strength 4000 psi or higher needs to be achieved. It has been found that having lower than 50% RAP has produced the least amount of reduction in compressive strength compared to concrete with natural aggregate. It should be noted that having a lower percent replacement RAP aggregate still reduces the compressive strength below the desired strength. In order to use RAP concrete as a structural concrete, a HSC mix with RAP coarse aggregate may achieve the desired compressive strength for structural applications. There has not been much research on HCC containing RAP coarse aggregate. Limbachiya and others (2000) studies the effect of RAP in HSC at different curing days and at different RAP percentages. Capson and Sorensen (2013), Capson (2014) look at HSC containing RAP at different percentages.

2.5.1 Use of recycled concrete aggregate in high-strength concrete

In a study done by Limbachiya and others (2000), recycled concrete aggregate (RCA) was looked at in high-strength concrete mixes. Limbachiya and others (2000) study the compressive strength and freeze-thaw resistance of RCA in a high-strength concrete mix. In order to determine the compression strength, a ceiling strength is established. Standard strength testing is done on 100 mm cubes cured at 7, 28, 60 and 90 days. The results show that up to 30% coarse RCA has no effect on the ceiling strength. To design an RCA with the same strength of a PCC, the water cement ratio (w/c) is changed. After equal performance of RCA and PCC is accomplished, the study moves to engineering properties. The durability study that is of interest to this thesis is the freeze-thaw durability. In the Limbachiya (2000) study ASTM C666 procedure A is used. For both RCA and PCC the freeze thaw specimens reach 300 cycles before the dynamic modulus has a 40% reduction.

The results show that RCA concrete has durability factors that achieve above 95% for concrete with up to 100% RCA replacement. The conclusions that can be drawn from this study are that the test results with a RCA up to 30% have no effect on the ceiling strength of the concrete, but above 30% had an increase in the reduction of compressive strength. The w/c ratio can be adjusted to add compressive strength to the RCA concrete. Coarse aggregate can be used in a HSC to achieve desirable compressive strength, flexural strength and modulus of elasticity. RCA concrete performs well under freeze-thaw conditions, therefore showing good freeze-thaw durability potential.

The conclusions of the study done by Limbachiya and others(2000) are:

- RAP above 30% replacement have a great reduction in strength.
- Changing the W/C ratio has shown to increase to the compressive strength of RAP concrete.
- RAP concrete is more durable under freeze-thaw conditions than normal PCC.

2.5.2 Recycled Asphalt Pavement as Coarse Aggregate Replacement in High Strength Concrete Mixes

In studies done by Capson (2014) and by Capson and Sorensen (2013), the compressive and tensile strength of a HSC with RAP as a percent of coarse aggregate replacement is researched. Capson and Sorensen look at three objectives:

1. Determine the variability in the compressive strength between RAP concrete with RAP gradated to match the replaced coarse aggregate versus RAP that is not gradated.
2. Determine the variability in compressive strength of gradated RAP based on different RAP harvest locations.
3. Determine the variability of RAP concrete with different percentages of RAP for coarse aggregate replacement.

Using a 35% RAP replacement, Capson and Sorensen choose two different harvest locations to compare gradated and non-gradated RAP. To study the second objective, five different harvest locations are chosen throughout the State of Idaho. The RAP is sieved and gradated in the same manner for all five of the chosen harvest locations. Specimens are cast using the 35% RAP mix design and testing is done on all five locations. For each harvest location, topographical data is collected, including: temperature, traffic count, road type (highway vs. interstate) elevation, and population. To find the ideal RAP percentage to use in HSC, Capson and Sorensen use RAP percentages of 25, 30, 35, 40, 45, and 50%. Concrete cylinders are cast and tested in both tension and compression for each RAP percentage.

The results of the studies show that by replacing the coarse aggregate with RAP coarse aggregate of the same grain size, it is possible to eliminate some of the variability in compressive strength. The tests for the two studies are inconclusive as to whether gradating increase or decreases the compressive strength; however, gradating the RAP decreases the standard deviation yielding less of a variation in compressive strength. The results of the compressive test for different harvest locations show traffic counts have an effect on the compressive strength of the RAP concrete. As the traffic of the road being used is increases the compressive strength of the RAP mix is decreases. However, the temperature, elevation, annual precipitation, and snow pack do not appear to have an effect on the compressive strength. The type of road has an impact on the compressive strength. State highway RAP yields a higher compressive strength than interstate RAP concrete. Capson and Sorensen determined that using RAP up to 50% replacement can still achieve a 4500 psi compressive strength using a HSC mix.

The conclusions of this study are:

- Sieving the RAP into the appropriate gradation size directly affects the strength of the concrete.
- RAP must replace the appropriate percentages to match the normal coarse aggregate.
- Traffic count of RAP harvest locations affects the compressive strength.

The Results of Capson and Sorensen study show the potential of RAP to be used in a high-strength concrete. However, the mechanical properties of RAP concrete needs to be understood before RAP can be used.

2.5.3 High Strength concrete containing RAP aggregate Results

HS concrete containing RAP as a coarse aggregate has a possible application for structural concrete. Both of the above studies concluded that it is possible to achieve a high enough compressive strength to be used in a structural applications. For the possible structural applications of RAP concrete, more tests need to be done, besides compressive strength, for RAP to be used in structural grade concrete.

2.6 Mechanical Properties of RAP concrete

Mechanical properties need to be evaluated in order to achieve the desired application for RAP concrete. Li (2008) examines the mechanical behavior of RAP concrete. The mechanical properties that Li investigates are compressive strength, flexural strength, modulus of elasticity, bond strength of concrete to steel reinforcement and fracture energy. In a study done by Kenai and others (2002). mechanical properties and durability are looked at.

2.6.1 Recycling and reuse of waste concrete in China Part I. Material behavior of recycled aggregate concrete

Li (2008) looks at the mechanical behavior of concrete with recycled coarse aggregate. Li looks at freeze-thaw durability, compressive strength, flexural strength, modulus of elasticity, bond strength between recycled aggregate concrete and steel rebar and fracture energy.

The results from the study shows that with 200 freeze-thaw cycles the compressive strength decrease about 25% and reduces faster with RCA. With 100% RAC replacement concrete the decrease in compressive strength is about 12-25%. With about 20% RCA replacement the decrease in compressive strength is negligible. The bond strength between the RAP and the steel rebar is carried out following Chinese standard. In the test, 3 RCA replacement ratios of 0, 50%, 100% are used. Plain and deformed bars are used with a diameter of 10 mm. The results of the test show with the plane bar, that the RCA replacement ratios of 50% and 100% have a decrease of 12 and 6% respectively. The conclusions of this study are:

- RCA has nearly no influence on freeze-thaw resistance.
- Compressive and tensile strength decrease when RCA is added to the concrete mix.
- MOE decreases when RCA is added.

2.6.2 Mechanical Properties and Durability of Concrete Made with Coarse and Fine Recycled Aggregates

In this study done by Kanai and others (2002), the performance of concrete made with coarse and fine recycled aggregate (RA) are reported. The percentages range between 25, 50, 75 and 100% of either coarse, fine or coarse and fine aggregate. The compressive and flexural strength are compared to concrete with natural aggregate. The recycled aggregate is made in the lab, consisting of small slabs of concrete crushed at 28 days.

The specimens used in this study consist of cubic specimens of 100 mm x 10 mm x 10 mm, and 70 mm x70 mm x 280 mm for the flexural test. The mix design is constructed to have a constant slump of 70 mm. To achieve a constant slump the water to cement (w/c) ratio is different for each mix.

The results of this study show that the compressive strength decreases with added recycled aggregate. The results indicate that the compressive strength of the concrete with coarse RA at 28 days is about 10 to 20% of the concrete with natural aggregate. The concrete with fine RA has a decrease of about 10-30% in compressive strength. A decrease in compressive strength of 35% for the concrete that contains both fine and coarse aggregate is found. The flexural test results shows a decrease of about 20% at 28 days and about 70% at 90 days. The conclusions of this study are:

- Compressive and tensile strength decease with the increase of RAP to the concrete mix.
- MOE decreases with the addition of RAP to the concrete mix.

2.7 Results of RAP Studies and Future Related Work

RAP can be an adequate replacement for coarse aggregate in pavement design. The reduction in strength of the RAP concrete for pavement meets the required strength if the percent of RAP is below 50% replacement. However, the reduction in strength is too high to be used in a structural application; therefore, RAP needs to be studied more if a structural application can be considered. The studies have shown that the inclusion of both fine and coarse has dramatically reduced the compressive strength. It has also been found that the RAP needs to be held to 50% or below to keep the reduction of compressive strength minimal. The use of high-strength concrete with RAP coarse aggregate has been studied and shows that it has potential to be used as a structural concrete. There needs to be further testing to ensure the safety of RAP aggregate in a structural concrete. Concrete with RAP as a coarse aggregate replacement with a percent replacement of 25, 30, 35, 40, 45 and 50% are tested in this thesis. In order for RAP concrete to be used as a structural concrete the mechanical properties must be determined.

Chapter 3
Methodology

3.1 Introduction

This thesis studies the effect of Recycled Asphalt Pavement (RAP) as a coarse aggregate replacement in Portland cement concrete mix. This chapter discusses the methodology that is used for each test performed in this thesis, ASTM and AASHTO standards are used for each respective test. Each test is discusses in separate sections as follows:

- Section 3.4 Freeze-Thaw
- Section 3.5 Chloride Penetration
- Section 3.6 Bond Strength
- Section 3.7 Ductility Test
- Section 3.8 Strain Rate of Crushing
- Section 3.9 Coefficient of Thermal Expansion
- Section 3.10 Modulus of Elasticity

The result of these tests is discussed in Chapters 4 and 5. Chapter 4 discusses the results of the long term durability tests (Sections 3.4 and 3.5), while Chapter 5 discusses the results of the mechanical behavior test (Sections 3.6-3.10).

3.2 Mix Design

The mix design used in this design is based on a high strength mix that is used by a local cement batch plant; Pocatello Ready Mix. The gradation of RAP that is used in each mix is found by sieving the coarse aggregate supplied by Pocatello Ready Mix to find the size distribution of the normal aggregate. Once the size of the normal aggregate is found, it is replaced by gradation and weight with RAP aggregate. A super plasticizer is added to the mix design to make the concrete more workable.

The mix design for each of the different RAP percentages are shown in Table 3.1

Table 3.1: Mix Design

	Control	25% RAP	35% RAP	35% RAP	40% RAP	45% RAP	50% RAP
Cement (lbs)	8.3	8.3	8.3	8.3	8.3	8.3	8.3
Fly Ash (lbs)	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Fine Agg (lbs)	15.4	15.4	15.4	15.4	15.4	15.4	15.4
Course Agg (lbs)	23.3	17.5	16.3	15.1	14.0	12.8	11.6
RAP (lbs)	0.0	5.8	7.0	8.1	9.3	10.5	11.6
Super Plasticizer (oz)	0.7	0.7	0.7	0.7	0.7	0.7	0.7
Water (lbs)	3.6	3.6	3.6	3.6	3.6	3.6	3.6
w/c ratio	0.4	0.4	0.4	0.4	0.4	0.4	0.4

As shown in Table 3.1, each mix design has the same water to cement ratio (w/c). Therefore, the only difference between each mix is the amount of RAP that replaces the coarse aggregate.

The RAP that is used in this study is taken from a milled stockpile at the Idaho Transportation Department District 5 office in Pocatello, Idaho. The RAP is brought to the Concrete Lab at Idaho State University and laid in a pan to dry. After the RAP is dried, it is sieved to separate it into the proper gradation. The sizes of RAP aggregate that is used in the mix design are, 3/4” 5/8” 1/2” 3/8” and what is left in the pan. The RAP is sieved according to ASTM C136 (ASTM, 2006). The RAP is sieved for 5 minutes in a mechanical shaker to ensure proper size distribution. In order to best replicate normal aggregate, the RAP is distributed in the same percent by weight of the normal aggregate, shown in Table 3.2.

Table 3.2: RAP replacement percent by weight of coarse aggregate

Sieve Size	% weight of coarse aggregate
3/4"	10
5/8"	21
1/2"	22
3/8"	28
Pan	19

Once the RAP is separated by size, the amount of each aggregate size is calculated by running the normal coarse aggregate though the same size sieves, and the percent replacement by weight is found. This determines the amount of each grain size of normal coarse aggregate such that the RAP can match in grain size and distribution.

3.3 Casting

Casting for the all tests conducted in this thesis follows the appropriate and relative ASTM and AASHTO standards. The ASTM standards that are followed are: ASTM C33 Specifications for Concrete Aggregate (ASTM, 2013), ASTM C125 Terminology Relating to Concrete and Concrete Aggregate (ASTM, 2013), ASTM C136 Test Method for Sieve Analysis of Fine and Coarse Aggregate (ASTM, 2006), ASTM C192 Practice for Making and Curing Concrete Test Specimens in the Laboratory (ASTM, 2014), ASTM C617 Practice for Capping Cylindrical Concrete Specimens (ASTM, 2012). Casting of the concrete specimens is done in the Concrete Laboratory at Idaho State University. All concrete mixing is done according to ASTM standards and mixed with a Kobalt portable concrete mixer shown in Figure 3.1

Figure 3.1: Kobalt portable drum cement mixer

3.4 Freeze-Thaw

Freeze-thaw tests the durability of concrete when subjected to rapid freezing and thawing. Using RAP as a coarse aggregate replacement, it is important to understand the effects of freezing and thawing on the concrete. Specimens are produced in a 4x8 cylinder with the different RAP replacement percentages discussed in Section 3.2 There are five (5) samples of each percent replacement that are cast, for a total of 35 samples.

The standard used in this study is ASTM C666 Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing (ASTM, 2008) gives two different procedures for testing freeze-thaw durability. The procedure used in this study is Procedure A: the rapid freezing and thawing in water method. The freezing and thawing of the concrete specimens is done by lowering the temperature from 40 to 0° F (4 to -18° C) and then raising the temperature from 0 to 40°F (-18 to 4°C) in approximately 4 hours, with not less than 25% of the time in the thawing stage. The Freeze-thaw profile is shown in Figure 3.2.

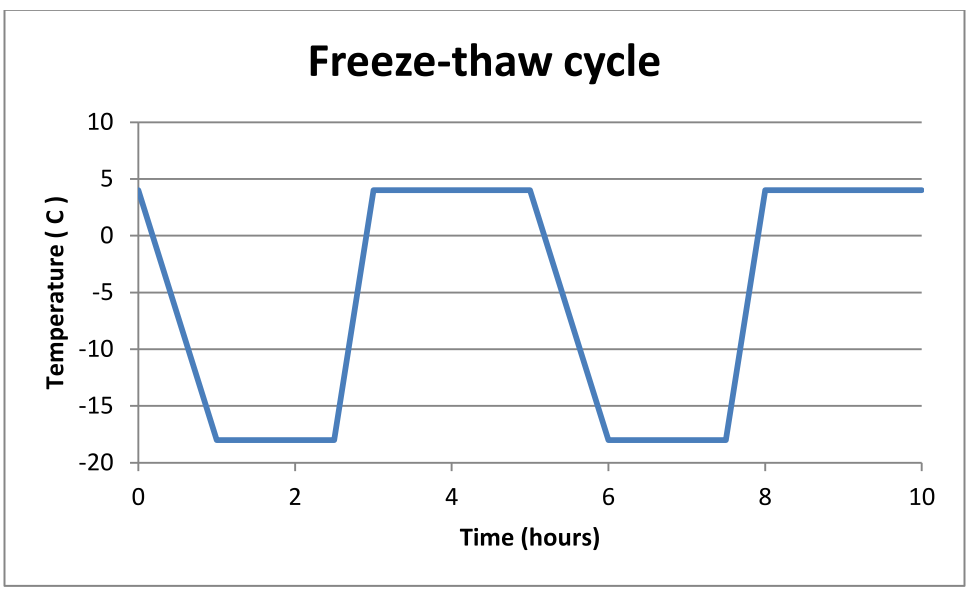

Figure 3.2: Temperature profile for freeze-thaw test.

According to ASTM C666 the specimens must be completely surrounded by not less than 1/32 of an inch but not more then 1/8 of an inch of water at all times during the freeze-thaw cycles. To achieve this, class 100 pipes are used with a 4 in diameter. A picture of the samples in the freeze thaw chamber is shown in Figure 3.3

After the cylinders are cast according to ASTM C192 (ASTM, 2007), they are capped and let to cure for approximately 24 hours. The concrete cylinders are cured in accordance with ASTM C511 (ASTM, 2013) and ASTM C192 (ASTM, 2007). The cylinders are cured in a water bath at the Idaho Transportation Department District 5 Lab. The water bath is set at a temperature of 22° C. After 28 days in the water bath, the samples are placed in the freeze-thaw chamber at 6^{o} C. so that the thaw temperature when the initial fundamental frequencies are found.

Figure 3.3: Samples in freeze-thaw chamber

Degradation of the specimens is determined by monitoring the fundamental frequency using a V-E 400 emodumeter (E-meter) as shown in Figure 3.4. The specimens must first be brought to a temperature within -2°F and +4°F of the target thaw temperature to test for the fundamental frequency

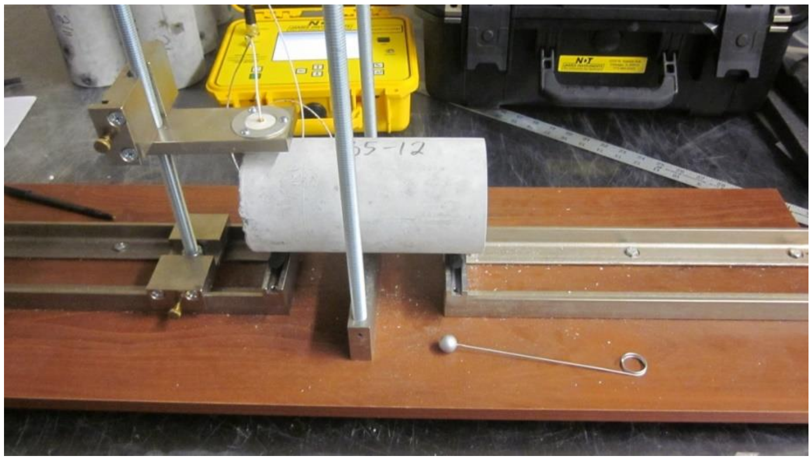

Figure 3.4: Emodumeter

In order to obtain the elastic constant of the specimen, the user inputs data (length, diameter and mass) in the E-meter. The accelerometer is placed at one end of the specimen. Then tap the specimen using a hardened steel ball, the instrument will trigger and a signal will appear on the screen. The system will obtain the fundamental frequency. For complete methodology on the E-meter refer to the owner's manual. (V-E-400 Emodumeter Operator's Manual, revised February 2013).

The fundamental frequency is used to find the dynamic modulus of elasticity using Equation 3.1 from ASTM C666:

$$P_c = \left(\frac{n_1^2}{n^2}\right) * 100 \qquad (3.1)$$

Where:

P_c = The relative dynamic modulus

n_1= fundamental transverse frequency at c cycles of freezing and thawing

n = fundamental transverse frequency at 0 cycles of freezing and thawing

Once the initial dynamic modulus is known, the specimens are placed in the freeze-thaw chamber (Figure 3.3) to start the freeze-thaw test cycles. ASTM C666 standard requires that the dynamic modulus is tested no more than every 36 cycles, and terminate the test after 300 cycles. The mass is measured every 36 cycles in addition to the dynamic modulus. After the mass and the dynamic modulus are measured, the containers that hold each specimen are cleaned and new water is added. This process is repeated for 300 cycles or until the dynamic modulus reaches 60% of its initial reading. When the specimens reach 300 cycles or the dynamic modulus falls below 60% of the initial dynamic modulus the durability factor (DF) is calculated using Equation 3.2 from ASTM C666:

$$DF = \frac{P*N}{M} \tag{3.2}$$

Where:

P = the relative dynamic modulus of elasticity at N cycles

N = number of cycles at which P reaches the specified minimum value for discontinuing the test or the specified number of cycles at which the exposure is to be terminated, whichever is less

M = specified number of cycles at which the exposure is to be terminated

If the specimen fails to maintain a minimum of 60% of its initial dynamic modulus they are considered to have failed the test and no longer need to be subjected to freezing and thawing. Results of this test are shown in Section 4.1 of this thesis.

3.5 Chloride Penetration

Chloride penetration is tested following AASHTO standard TP 95-11 (AASHTO 2011) Surface Resistivity Indication of Concrete's Ability to Resist Chloride ion Penetration 2011. 4" x 8" Concrete cylinders with a diameter of 4 inches are cast in the Idaho State University Concrete Laboratory. There are 21 total samples cast with 3 samples for each percent RAP. The samples are soaked in a lime bath shown in Figure 3.5 until testing according to ASTM C192 Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory (ASTM, 2007).

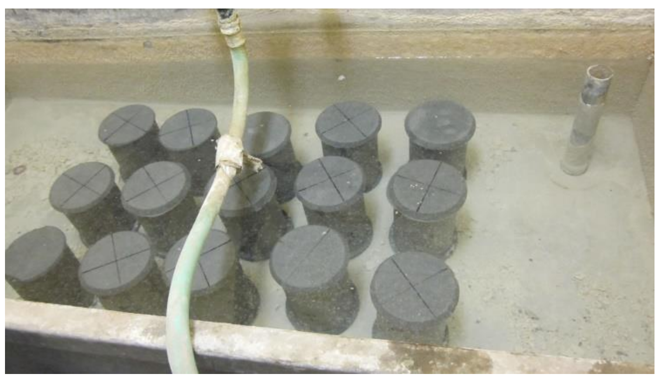

Figure 3.5: Chloride penetration samples in lime bath

Once the samples are ready to test, a Proceq resipod 38mm device, specifically manufactured for this test, is used to calculate the chloride penetration, as shown in Figure 3.6.

Figure 3.6: Proceq Resipod 38mm

Two measurements are taken at 90^{o} from each other (shown in Figure 3.7), for a total of 8 measurements for each specimen.

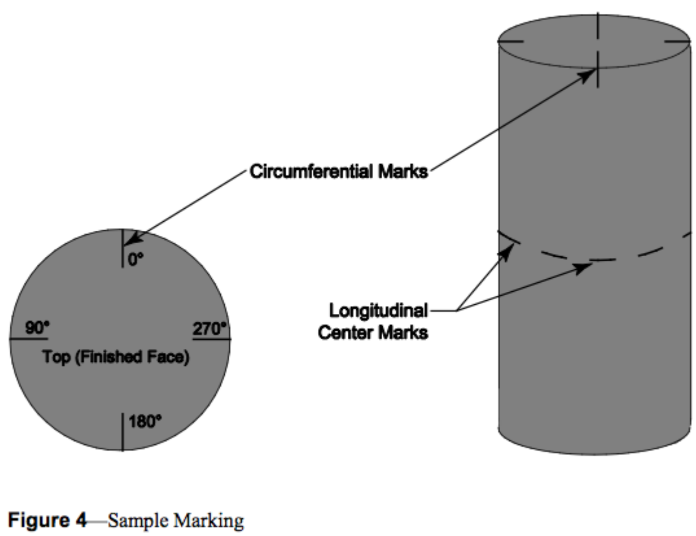

Figure 3.7: Sample Markings (AASHTO, 2014)

Each set consist of three samples for each RAP percent. Since the samples are cured in a lime bath, AASHTO TP 95-11 requires a 10% increase of the reading from the resipod due to known effects of the resistivity by the curing conditions is added to the final value due to the lime bath. Results are discussed in Section 4.2.

3.6 Bond Strength

The bond strength is tested using a push through test. This test follows ASTM C900 but with a few variations made due to equipment limitations of the Concrete Laboratory at Idaho State University. The steel rebar is cast all the way through the specimen as opposed to embedded in the specimen, as shown in Figure 3.8.

Figure 3.8: Set-up for bond strength test

ASTM C900 requires a pull-out test, in which the steel rebar is pulled out of the specimen; the test that is performed in this study is a push though-test which test bond strength but under a different failure mechanism. The bond strength is tested on four samples percentages of 0, 30, 40, 50 percent replacement RAP. The specimen for each percent has dimensions of 12 x 6 x 4 inch. A number three rebar is placed in the specimen with approximately ¾ inch exposed from the top and 1.5 inch out of the bottom. The control samples are made with a 4000 psi mix design provided by the Wyoming Department of Transportation (WYDOT) as shown in Table 3.3.

Table 3.3: WYDOT mix design

WYDOT mix design	Weight (lbs)
Type II cement	21.2
Fly Ash Type F	5.3
Coarse aggregate	77.2
Fine aggregate	46.3
Water	11.2
w/c ratio	0.53

The push-through test is conducted in the Concrete Laboratory at Idaho State University. A Gilson MC-300 compression machine with a 300,000 lb load capacity is used to provide the force to push the steel rebar though the concrete specimen. The force required to push the rebar through the concrete is measured. The force is then compared to the control sample (0% RAP) to observe any lost bond strength. The use of a push through test does not accurately give the bond strength of the concrete, but does give a relative strength that can be compared to the control mix. The results of this test are discussed in Section 5.1 of this thesis.

3.7 Ductility

Ductility measures the amount of deformation in a material prior to rupture. The samples that are used for the ductility test are RAP mixes with 0, 25, 30, 35, 40% coarse aggregate replacement. The samples are prepared in the Concrete Laboratory at Idaho State University with dimensions of 4 x 4 x 24 inches. The samples are cured for 28 days before de-molding, and tested at a 33 day curing period. The test is conducted at 33 days due to timing issues of the test.

The ductility of the beam is measured instead of the curvature due to the ease of measuring deflection. The most crucial parameter in measuring ductility is the maximum deformation that the beam experiences before the beam ruptures. To measure ductility, a ductility factor μ, is calculated as shown in Equation 3.3 (pam et al. 2001).

$$\mu = \frac{\Delta_{Max}}{\Delta_{Yield}} \tag{3.3}$$

Where:

Δ_{max} is the maximum deformation

Δ_{yield} is the deformation at the yield point

Finding the yield point can be difficult since a well-defined yield point may not occur on the load-deflection curve. From past studies, it is found that taking the deflection at 75% of the ultimate load gives a good indication of the yield point. The maximum deflection at failure is taken at the point when the beam ruptures. Since there is no steel reinforcement in the samples, the beam will crack and then break into two pieces without warning. This creates a problem because the linear strain conversion transducers (LSCT) still reads deflection as the beam is breaking thus Δ_{max} will be taken when the load has fallen to 90% of the peak loading. The test set up consists of an LSCT that measures the deflection at the center of the beam. Pictured in Figure 3.9.

Figure 3.9: Set-up for ductility test

A steel bracket is glued to the side of the beam so the LSCT can measure deflection without possible damage due to the fallen beam, as shown in Figure 3.10.

Figure 3.10: LSCT set-up for ductility test

Three point loading is used to provide a uniform moment over the center span of the beam. The beam will be turned on its side in relation to casing according to ASTM C78 “Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)” (ASTM, 2010). The compression force is supplied by a Gilson MC-300m machine. The force is measured using a transducer techniques 300k load cell with a capacity of 300,000 pounds. The load cell and the LSCT are connected to a computer taking 5 readings a second. Data collection software is used to record the data. The data is collected and then imported to excel, where the results are plotted to determine Δ_{max} and Δ_{yield}. The results from the RAP specimens are compared to the control specimens with the specimen with a higher ductility index is considered to be more ductile. Results of this testing are discussed in Section 5.2 of this thesis.

3.8 Strain-Rate

The stain-rate of crushing test verifies the limitation in the ACI equations of a .003 in/in for a compression controlled design, as given in ACI 318-11 (ACI, 2011). 4” x 8” samples are used in this test.

Strain rate of crushing is calculated by the stress-strain diagram. Two strain gages with a 120Ω resistance are attached to the concrete cylinders in both the horizontal and vertical direction. This measures the strain in both the vertical and the horizontal direction. Micro-measurments CEA-06-250UW-120 strain gages are used and a 200 bond kit is used to prep and attach the strain gages. Wire is attached to the strain gage and then connected to strain smart software, as shown in Figure 3.11.

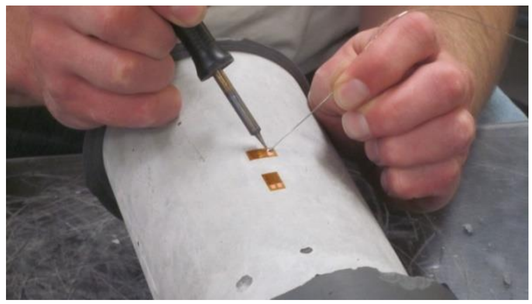

Figure 3.11: Strain gage attachment

The strain gage is connected to software that scans at a rate of 10 scans per second. A load cell with a 300,000 lb capacity is used to record the load applied to the concrete cylinder. A Gilson MC-300 compression machine is used to provide the force. A picture of the test set up is shown in Figure 3.12.

Figure 3.12: Test set-up for strain rate test

The load cell is placed on top of the cylinder with a plate underneath it to provide uniform loading to the concrete cylinder. The load cell is connected to a computer with strain smart data acquisition software. The load cell reads at a rate of 5 scans per second. In order for the data from the load cell and the strain gage to match up, every other data point from the strain gages has to be deleted. Once the data is recorded, it is exported to excel and then a load vs. strain graph is made. The results are discussed in Section 5.3 of this thesis.

3.9 Coefficient of Thermal Expansion

The coefficient of thermal expansion (CTE) is a measure of contraction and expansion of a material as the temperature changes. CTE is measured as microstrains per unit temperature change. CTE of Portland cement concrete (PCC) is in the range of 8 to 12 microstrains/°C . The CTE varies because of the nature of concrete, the CTE will change if a different aggregate is used. Since concrete is about 70% aggregate, CTE is greatly influenced by what aggregate is used.

CTE is tested following a previous test done by the Portland Cement Concrete Pavements (PCCP) research (FHWA, 2011). This test determines the CTE of a concrete cylinder. A total of 21 samples are cast with 4” x 8” dimensions. The samples are placed in a freeze-thaw chamber located in the Concrete Laboratory at Idaho State University. The temperature range is 10°C to 50°C, as shown in Figure 3.13.

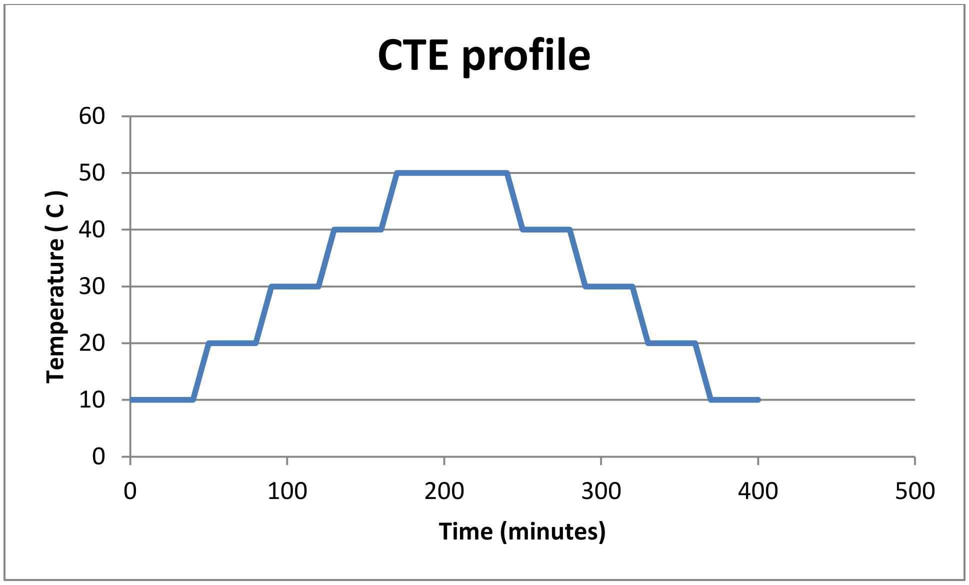

Figure 3.13: Temperature profile for CTE test.

As shown in Figure 3.13, the test starts at 10^{o} C then ramps up to 20^{o}C and then soaks for 30 minutes. A reading is taken at 20^{o}C then the temperature is ramped to 30^{0}C. This process is repeated to 50^{o}C then from 50^{o}C to 10^{o}C with reading taken every 10^{0}C.

The length change is measured by linear strain conversion transducers (LSCT). A picture of the test set up is shown in Figure 3.14 and Figure 3.15.

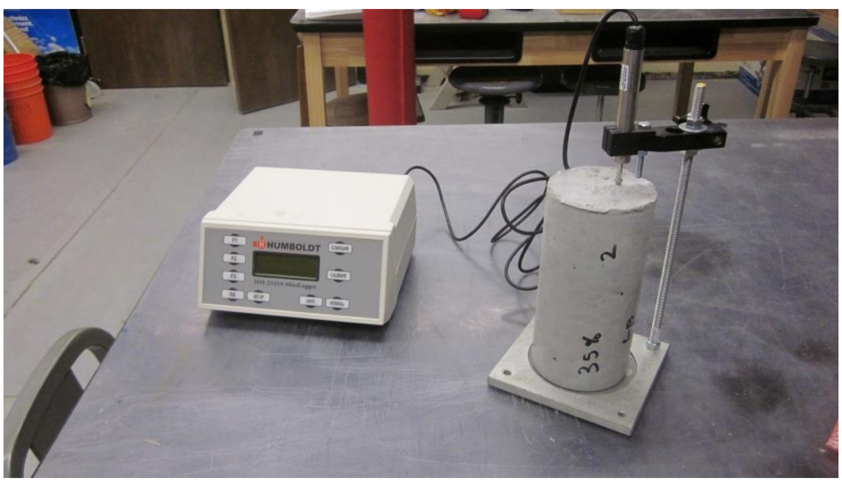

Figure 3.14: CTE test set up

The LSCT is connected to a mini logger that measures the change in length. The mini logger is then connected to a computer so the data can be imported.

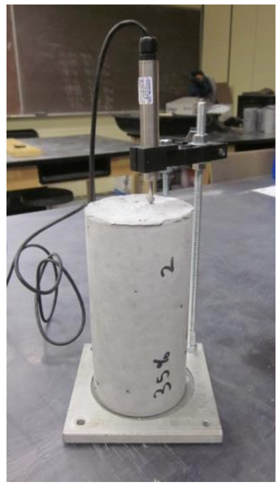

Figure 3.15: CTE test set up

Each sample is placed in the freeze-thaw chamber and set to 10°C for about 2 hours to ensure that the entire specimen is at a uniform temperature, the reading on the mini logger after 2 hours in the initial displacement. The temperature of the specimen is raised by 10°C over a 40 minute period and the displacement is recorded. This is repeated until the specimen reaches 50°C. once at 50°C the specimen is allowed to sit for 80 minutes. This process is then repeated from 50° to 10°C. This process is repeated for all 21 samples. Once the data is collected, Equation 3.4 as given by the FHWA, is used to calculate the CTE.

$$CTE=(\Delta L/L_o)/\Delta T \qquad (3.4)$$

Where:

ΔL= change in length

L_o = initial measured length of specimen

ΔT = Change in temperature

Once the data is collected, it is imported into excel and analyzed. The results are discussed in detail in Section 5.4 of this thesis.

3.10 Modulus of Elasticity

The modulus of elasticity (MOE) of concrete is time dependent. The ACI code determines MOE as a function of compressive strength. Many deflection ACI equations require the use of MOE. To measure the MOE an Emodumeter (E-Meter) is used. The samples that will be tested are twenty one 4" x 8" cylinders. Taking the dimension of the cylinders and inputting them in the E-meter and using a hardened steel hammer to strike the cylinder. The E-meter will trigger and yield a fundamental frequency. The E-meter can calculate the MOE using the fundamental frequency. The cylinders are then tested in compressive and the MOE is calculated using the ACI equations for MOE as given in Equation 3.5.

$$E_c = 33 * w_c^{1.5}\sqrt{f_c'} \quad \text{(psi)} \qquad (3.5)$$

Where:

w_c = is the density of concrete in lb/ft^3 and for 90 < w_c < 155 lb/ft^3

f_c' is the compressive strength of the concrete.

Equation 3.5 is applicable for f_c' up to 6000 psi. The mix that is being tested is a high-strength mix with a compressive strength of up to 12,000 psi. With the inclusion of RAP the compressive strength decreases below 6,000 psi. The ACI equation for concrete with a compressive strength of 6,000-12,000 psi is shown in Equation 3.6.

$$E_C = 57{,}000 * \sqrt{f_c'} \quad \text{(psi)} \qquad (3.6)$$

Where:

f_c' is the compressive strength of the concrete

The MOE provided from the E-meter is compared to the results from the ACI equations. Results are discussed in further detail in Section 5.5 of this thesis.

Chapter 4
Durability Testing Results

4.1 Introduction

This chapter present and discusses the results of the long term durability testing. Two tests are conducted for long term durability, freeze-thaw durability and chloride penetration. Section 4.2 presents the freeze-thaw durability results, and Section 4.3 consists of the chloride penetration results.

4.2 Freeze-Thaw

Freeze-thaw durability is the measure of the durability of concrete when exposed to rapid freezing and thawing. To measure the durability of concrete, a relative dynamic modulus of elasticity and a durability factor (DF) is calculated. For this test, there are 35 samples, 5 samples for each RAP percentage. The RAP percentages that are tested are 0, 25, 30, 35, 40, 45 and 50%. The methodology for this test can be found in Section 3.4 of this thesis.

The transverse frequency for each specimen is measured in order to calculate the dynamic modulus and the DF. A full list of results can be found in the Appendix A. After the transverse frequency is found the dynamic modulus is calculated. The results of the dynamic modulus are presented in Table 4.1. Since all RAP samples passed the required 300 freeze-thaw cycles that are required by ASTM C666, the final dynamic modulus of elasticity is the same as the DF.

Table 4.1: DF results for all samples

	Control		25%			30%			35%		
# of	Average	Std. D	Average	Std. D		Average	Std. D		Average	Std. D	
Cycles	Dynamic		Dynamic		Percent	Dynamic		Percent	Dynamic		Percent
0	Modulus		Modulus		different	Modulus		different	Modulus		different
36	105	0.7	105	3.1	0%	108	3.5	2%	103	3.9	-2%
72	107	0.7	106	5.0	-1%	107	8.2	0%	102	6.8	-5%
108	106	2.3	107	4.5	1%	107	7.9	1%	102	5.1	-3%
144	98	12.4	105	3.4	6%	107	8.2	8%	102	5.9	3%
180	88	13.8	105	4.0	16%	104	7.3	15%	102	6.7	14%
216	77	11.6	100	7.2	23%	99	6.1	22%	102	5.4	24%
252	62	7.7	87	12.7	29%	91	7.2	33%	101	4.3	39%
288	55	5.4	84	13.2	35%	86	7.5	36%	96	4.7	43%
300	55	5.4	84	12.7	35%	86	7.5	36%	95	4.3	43%
			40%			**45%**			**50%**		
			Average	**Std. D**		**Average**	**Std. D**		**Average**	**Std. D**	
			Dynamic		**Percent**	**Dynamic**		**Percent**	**Dynamic**		**Percent**
			Modulus		**different**			**different**			**different**
			101	1.8	-5%	106	1.6	1%	103	4.3	-3%
			102	5.0	-5%	108	1.3	1%	102	4.4	-5%
			101	5.3	-5%	107	2.6	1%	102	4.2	-4%
			101	2.0	3%	108	2.9	9%	102	6.2	4%
			100	5.9	12%	104	3.5	15%	100	7.6	11%
			99	3.8	22%	93	5.9	17%	87	12.9	11%
			97	4.1	37%	79	10.9	22%	78	12.7	21%
			94	4.3	41%	73	10.0	25%	68	10.5	20%
			94	4.3	41%	73	10.0	25%	66	11.6	17%

The results show that the control samples have the lowest DF, with a DF of 52 at 300 cycles. The samples that have the highest DF are those with 35 and 40% RAP, with an average DF of 95 and 94, respectively. The samples with 50% RAP have the highest standard deviation of 11.6 at 300 cycles. The dynamic modulus results are plotted in Figure 4.1.

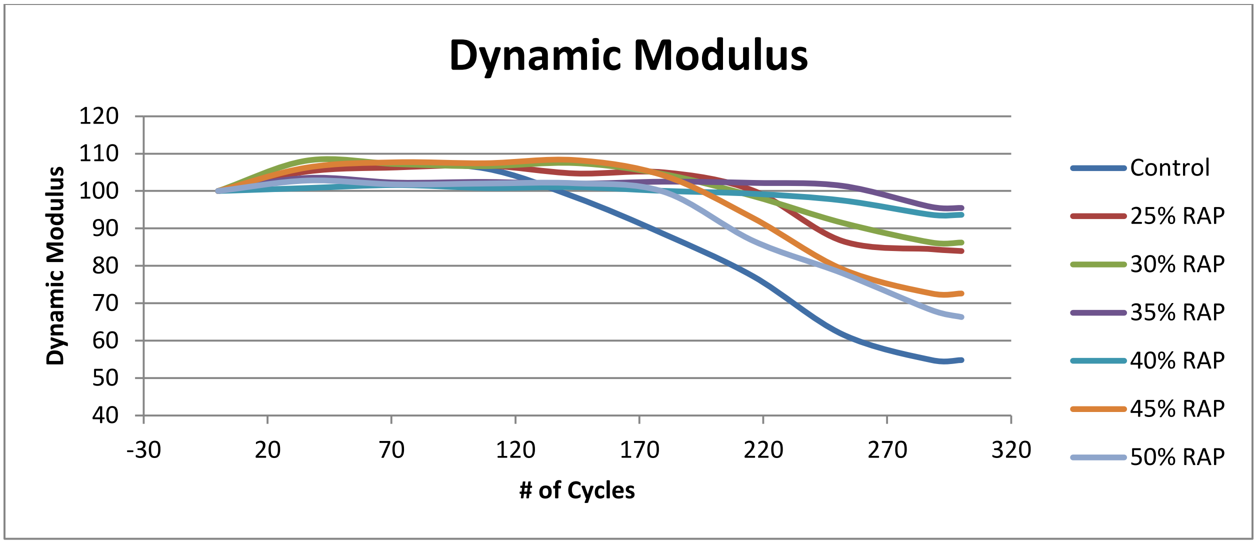

Figure 4.1: Graph of dynamic modulus of all sets

As shown in Figure 4.1, the control samples have the lowest durability factor. This is expected due to the nature of the high strength concrete; without RAP aggregate the concrete is less ductile and is more susceptible to freezing and thawing. The samples that performed the best are the 35 and 40% RAP. The 25, 45 and 50% have a high standard deviation, leading to a lower DF. The higher standard deviation is due to the RAP aggregate. When normal aggregate is sieved, you know the nature of the aggregate. However, when RAP aggregate is sieved the aggregate has a tar coating. The tar coating provides a larger aggregate with a smaller rock diameter. Not knowing the actual rock aggregate diameter leads to a larger standard deviation. However, the RAP also provides some resistance to freeze thaw cycles. The RAP aggregate has a soft asphalt coating on it, this allows it to expand and contract without experiencing stress due to the freezing conditions, leading to more durability. A high strength concrete without RAP has no room to expand and contract, leading to a less durable concrete. The control samples also started to become less durable earlier than any of the other samples, meaning that the concrete started to breakdown sooner than the concrete with RAP. The samples with the lowest standard deviation is also the samples that have the highest DF, the standard deviation is 4.3 for both 35 and 40% RAP. Figure 4.2 shows a bar chart for the DF for all sets.

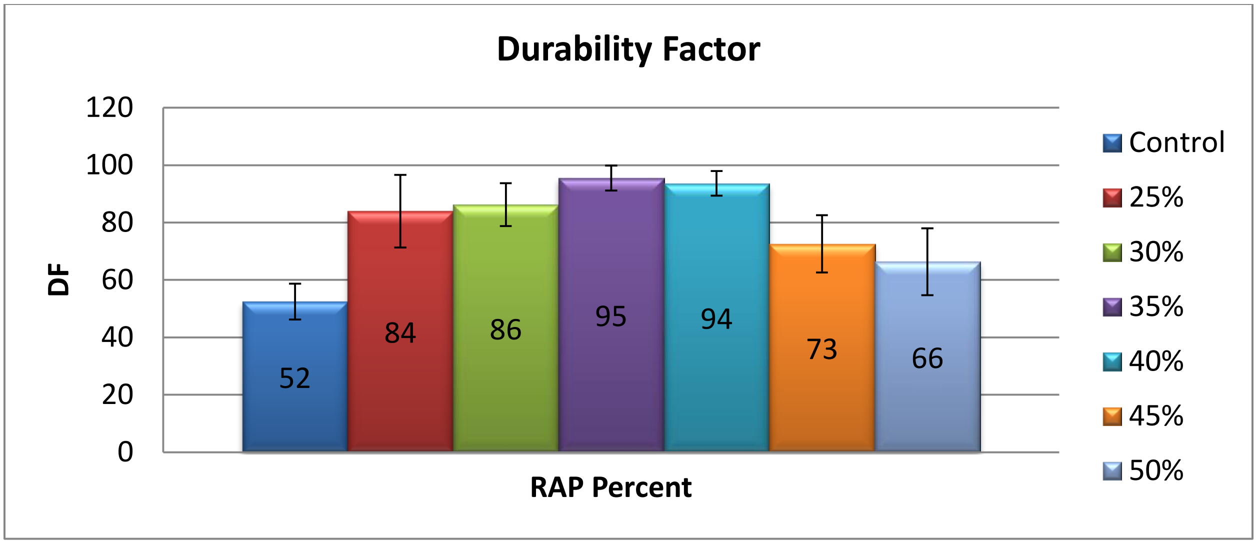

Figure 4.2: DF of all sets

As shown in Figure 4.2, the DF increases from the control with zero RAP to 35% RAP replacement, then the DF decreases from 35 to 50% RAP replacement. This is due to the nature of the RAP aggregate. There is a big increase in DF once RAP is introduced to the concrete mix. However, when RAP reaches 45% replacement, the DF starts to decrease. The cause of the decrease in DF in specimens above a 40% RAP replacement is not known. However, this behavior corresponds to results from Capson's study where RAP mixes with more than 40% RAP provide unstable results.

The mass is recorded before the start of the freeze-thaw cycles and then again after the 300 freeze-thaw cycles. The results of the mass loss are shown in Table 4.2.

Table 4.2: Mass loss after 300 freeze-thaw cycles

	Control	25%	30%	35%	40%	45%	50%
Initial Mass (g)	3923	3902	3893	3892	3850	3878	3832
Final Mass (g)	3947	3918	3914	3913	3873	3899	3851
Mass Loss (g)	-24	-16	-21	-21	-23	-21	-19
Percent Decrease	-1%	0%	-1%	-1%	-1%	-1%	0%
Percent Difference		33%	13%	13%	4%	13%	21%

All the samples gained an average of about 20 grams of mass after the freeze-thaw cycles, this can be due to the samples absorbing water during the cycles. One of the 25% RAP samples lost a large peice of concrete after 108 freeze-thaw cycles, shown in Figure 4.3.

Figure 4.3: 25% RAP after 108 freeze-thaw cycles

Figure 4.3 shows an approximate ½ of an inch radius piece of concrete missing on the bottom half of the cylinder, this piece of concrete broke off during the freeze-thaw cycles. This sample is the only sample that lost any significant mass during testing. Excluding this sample in the average of mass loss, the entire set of samples are in the range of 20 grams gained. There is no other visual evidence of mass loss from the freeze-thaw cycles on any other specimen.

The compressive strength of the concrete cylinders are tested after the 300 freeze-thaw cycles, the samples are stabilized (brought to room temperature) before the compressive test, and then compared to the 28 day strength. The 28 day compressive strength is from an ACI equation shown in Equation 4.1 below.

$$f_c'(t) = f_c'(28) * \frac{t}{4+0.85t} \tag{4.1}$$

Where:

$f_c'(t)$ = the compressive strength at time t

$f_c'(28)$ = 28 day compressive strength

t = time in days

Using the above equation, the results of the freeze-thaw compressive strength results are compared to the results using Equation 4.1, the results are shown in Table 4.3.

Table 4.3: Results of freeze thaw compression test

	Control	**25%**	**30%**	**35%**	**40%**	**45%**	**50%**
Compressive Strength (psi)	5586	4859	4313	4620	4085	3542	3272
Standard Deviation (psi)	1172	414	498	761	639	606	255
Computed 28 day compressive strength	10713	6881	5495	5654	4914	4763	4361
Standard Deviation (psi)	133	821	766	13	452	145	121
Reduction in strength (%)	48%	29%	22%	18%	17%	26%	25%

The samples that experienced the greatest reduction in compressive strength are the control samples with a 48% reduction in strength. The samples that experienced the least amount of reduction are the samples with 40% RAP. All the samples with RAP experienced less of a reduction then the control samples. The reduction in compressive strength after 300 freeze-thaw cycles for the control can be due to the fact that only one of the five control samples passed the freeze-thaw test. Leaving the control samples less durable then the control providing a reduction in compressive strength.

A correlation can be drawn between the DF and the compressive strength after 300 freeze thaw cycles. The samples that have a higher DF also have a lower reduction in compressive strength for RAP concrete. The control has the lowest DF but not the highest reduction in compressive strength. This can be due to the high strength mix and lack of RAP in the concrete. The three samples that have the highest reduction in compressive strength also have the lowest DF. The conclusion can be drawn that with a lower DF after freezing and thawing there will be a greater reduction in compressive strength. This is expected due to the breakdown of the concrete during the freeze-thaw cycles. The samples that performed the best (under freeze thaw conditions) are the mixes with 35 and 40% RAP replacement. While the mixes with 30 and 40% RAP carried the most load after the freeze-thaw test, the mix with 40% RAP replacement is the optimal mix of freeze-thaw resistance and strength after being subjected to 300 freeze-thaw cycles.

The summary of the results are:

- No reduction in durability with the addition of RAP aggregate in a high strength concrete mix.
- There is a direct correlation between DF and compressive strength after 300 freeze-thaw cycles
- The lower the DF of the concrete, the greater reduction in compressive strength for RAP concrete.
- There is no mass loss after 300 freeze-thaw cycles. However, there is a mass gain due to soaking in water.

4.3 Chloride Penetration

The chloride ion penetration test is important to determine the corrosion rate of concrete. A control sample as well as 25, 30, 35, 40, 45 and 50% RAP are tested for chloride ion penetration. Three samples for each percent group for a total of 21 samples are tested. A Proceq resipod meter is used to test for chloride ion penetration. The methodology is discussed in greater detail in Section 3.5 of this thesis. The results of the chloride penetration test are presented in Table 4.4

Table 4.4: Chloride Penetration Results

Surface Resistivity Recordings (kΩ-cm)											
Sample Identification	**0°**	**90°**	**180°**	**270°**	**0°**	**90°**	**180°**	**270°**	**Average**	**STD DEV**	**Chloride Penetration**
0% (1)	28.8	33.2	31.6	32.2	29.1	33.1	31.1	34.0	31.64	1.90	
0% (3)	37.8	37.0	35.2	35.0	35.6	38.2	36.7	35.3	36.35	1.25	
0% (4)	35.7	35.1	29.0	33.1	35.2	36.8	35.0	35.1	34.38	2.40	**37.53**
								STD DEV	**2.37**		**Very Low**
								Set Average:	**34.12**		
25% (1)	35.3	33.5	31.1	29.1	34.7	33.0	31.6	29.7	32.25	2.26	
25% (2)	32.4	32.7	28.5	32.7	32.3	31.9	30.4	31.6	31.56	1.45	
25% (5)	31.5	31.6	29.7	31.8	30.3	32.7	30.1	32.1	31.23	1.06	**34.85**
								STD DEV	**0.52**		**Very Low**
								Set Average:	**31.68**		
30% (1)	30.6	28.9	33.9	30.5	30.3	28.3	33.5	30.7	30.84	1.97	
30% (2)	27.6	26.7	27.1	27.9	29.0	26.6	26.9	27.4	27.40	0.79	
30% (3)	30.4	30.2	27.3	26.2	32.2	30.3	28.4	27.5	29.06	2.02	**32.01**
								STD DEV	**1.72**		**Very Low**
								Set Average:	**29.10**		
35% (1)	34.5	35.7	37.5	33.7	34.7	35.0	34.3	35.0	35.05	1.15	
35% (3)	32.3	35.8	35.8	35.4	34.7	37.9	35.7	35.7	35.41	1.55	
35% (4)	34.2	33.2	33.1	31.9	34.0	35.0	32.4	32.3	33.26	1.07	**38.03**
								STD DEV	**1.15**		**Very Low**
								Set Average:	**34.58**		
40% (1)	32.2	32.7	38.0	36.9	32.9	31.6	36.6	35.4	34.54	2.47	
40% (2)	31.5	31.7	31.3	32.5	33.0	31.4	31.4	32.5	31.91	0.65	
40% (3)	31.9	33.3	31.8	35.6	31.7	33.4	31.7	34.8	33.03	1.52	**36.47**
								STD DEV	**1.32**		**Very Low**
								Set Average:	**33.16**		
45% (1)	35	33.2	34.3	34	36	33.4	34.3	33.2	34.18	0.97	
45% (3)	31.8	32.5	32.8	32.4	35.6	32.9	32.6	33.2	32.98	1.14	
45% (4)	32	30.6	31.9	31.9	32.5	31	32.6	32.5	31.88	0.73	**36.31**
								STD DEV	**1.15**		**Very Low**
								Set Average:	**33.01**		
50% (2)	33.5	34.4	35.8	35.4	33.3	35.6	36.1	35.4	34.94	1.07	
50% (4)	30.9	37.3	35.0	31.8	31.0	37.7	36.8	31.7	34.03	2.98	
50% (5)	30.3	28.8	28.5	31.3	29.7	27.4	26.6	31.2	29.23	1.71	**36.00**
								STD DEV	**3.07**		**Very Low**
								Set Average:	**32.73**		

Table 4.4 shows that concrete with RAP added has a slight increase in corrosion rate. However, the corrosion rate for all samples is practically unchanged. All the samples fall in the range of moderate risk of corrosion and have a low corrosion rate. Figure 4.4 shows the comparison of the chloride Ion penetration results.

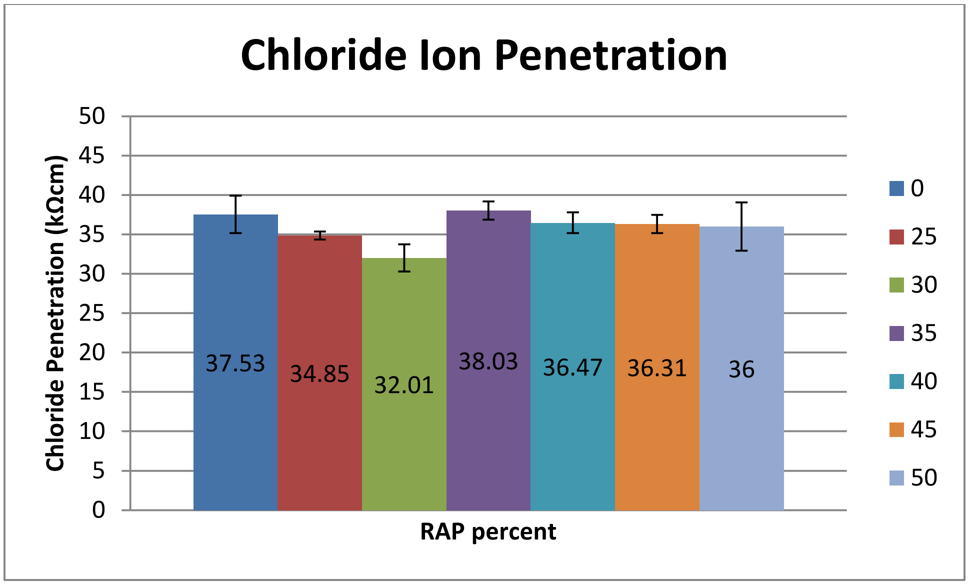

Figure 4.4: Chloride Penetrating Results

The chloride ion penetration results are all well above the 20 kΩ cm cut off for low corrosion rate and well within the 10 to 50 kΩ cm for moderate risk of corrosion (Operating Instructions resipod pg 11). The results of the chloride ion penetration test yield a smaller value for the RAP concrete for the risk and rate of corrosion than that of the concrete with no RAP aggregate. Therefore, concrete with RAP added does not change the risk of damage to the reinforcing steel in the concrete.

The results of this test show no significant change in chloride Ion penetration. All of the RAP percentages fall within the range of moderate risk of corrosion and well above the 20 kΩ cm cut off for low corrosion rate. Concrete with RAP added does not affect the corrosion risk or the corrosion rate.

Chapter 5
Mechanical Behavior Testing Results

5.1 Introduction

This chapter discusses the results of the mechanical behavior of RAP concrete. The first section includes the results of the bond strength test, followed by ductility, strain rate, coefficient of thermal expansion and modulus of elasticity.

5.2 Bond Strength

Bond strength is the measure of the concrete bond to the steel reinforcement. For concrete to be used in structural applications the concrete needs to adhere to steel reinforcement without slippage. The methodology is discussed in Section 3.6 of this thesis.

The bond strength is measured on a RAP concrete mix with 30, 40 and 50% RAP replacement of coarse aggregate, and is compared to a control sample that is a 4000 psi mix as discussed in the methodology Section 3.6. The max axial force that it takes for each #3 steel rebar to pass through the concrete specimen is measured. Table 5.1 shows the average axial load that each specimen was able to experience prior to de-bonding. Table 5.1 also shows the 33 day strength of each mix.

Table 5.1: Compressive and axial load at de-bonding

Percent RAP	f'c (psi)	f'c (psi) percent difference	Average Load (lb)	Std Deviation (lb)	Percent Difference From Control
Control	6673		19595	2063	-
30	5291	21%	11890	454	39%
40	4867	34%	11643	1843	41%
50	4511	44%	11753	494	40%

Table 5.1 shows that the average loads the specimens experience are in the 11,000 pound range. Bond strength is a function of the tensile strength of the concrete, since RAP concrete has a lower tensile strength then concrete with no RAP, it is expected to have a lower bond strength. The control sample for this test is a 4000 psi rated concrete, not a high strength concrete mix like the RAP concrete. This is done because a high strength mix without RAP aggregate provides a higher tensile strength; this will give a comparison that is not accurate. Using a high strength concrete mix with RAP aggregate will achieve a rating of 4000 psi, so it is desired to compare the high strength concrete mix with RAP to a concrete mix of 4000 psi. The compressive strength at the time of testing of the control has an average of 6871 psi. Since testing took place at 33 days, Equation 5.1 is used to get a 28 day strength (ACI 318-11).

$$f_c'(t) = f_c'(28) * \frac{t}{4+0.85t} \tag{5.1}$$

Where:

f'c is compressive strength

t is in days

The failure modes for the push-through test are all the same. Figure 5.1 shows the typical failure mode of an RAP concrete specimen.

Figure 5.1: Failure mode of 30-1 bottom

Figure 5.1 shows cracking along the bottom of the specimen. The concrete also flaked off around the rebar. This is due to the force that is applied to the rebar. The slippage of the rebar contributes to the de-bonding of the RAP concrete to the steel rebar. From Figure 5.1 you can see the concrete that has flaked off; this explains why the control has a higher bond strength as it takes more force to crack or chip the control concrete. Figure 5.2 shows the failure mode of a control sample.

Figure 5.2: Failure mode of the control

With a higher compressive strength, the fracture around the rebar is minimal. This leads to a higher force required to push the rebar through the concrete, leading to higher bond strength. This test shows that the bond strength of concrete with RAP added is not affected by the percent of RAP added. However, this test does show that the bond strength is dramatically decreased when RAP is introduced. The reduction in bond strength is due to the initial addition of RAP to the concrete. The reduction is bond strength can be due to the higher compressive strength of the control mix. However, the RAP aggregate may lead to a reduction in bond strength due to the tar coating around the aggregate. The tar coating may not provide enough “grip” on the steel rebar, leading to a reduction in bond strength.

The results of this test show that the bond strength decreases with the addition of RAP. Once RAP is introduced into the concrete, the bond strength does not have a significant decrease with the amount of RAP present in the concrete. It should be noted that these test deviated from the specified ASTM C-900 standard. As such, a standard pull-out test following ASTM C-900 needs to be conducted with a control sample that has the same compressive strength as the samples with RAP present.

5.3 Ductility

Ductility is the measure of deflection or deformation a material experiences before failure. A ductility index is used to measure the ductility which is measured by taking the ratio of the max deflection to yield deflection. The max deflection is taken as 90% of the ultimate strength (when the strength drops 10% after max loading) and the yield deflection is taken at 75% of the ultimate strength. This is done because there is no well-defined yield point. Furthermore, the max deflection is difficult to measure as the plain concrete beam fails abruptly without warning. The methodology used in this test is discussed in Section 3.7 of this thesis.

Ductility is measured on seven different RAP replacements, control, 25, 30, 35, 40, 45, and 50%, with three samples for each RAP percent replacement. The deflection is measured on all the samples.

The deflection increased for the 25% and the 30% compared to the control mix. Figures 5.3 A-I show the deflection v. time curve for the control, 25% and 30% samples.

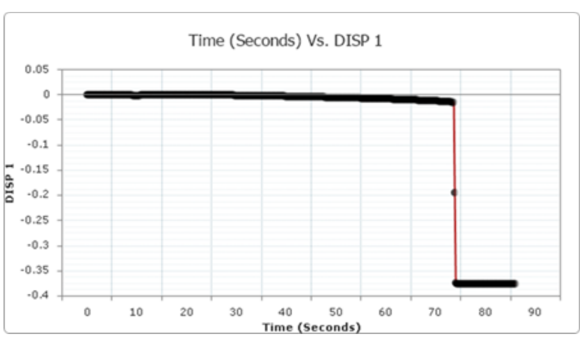

Figure 5.3A: Deflection vs time Control-1

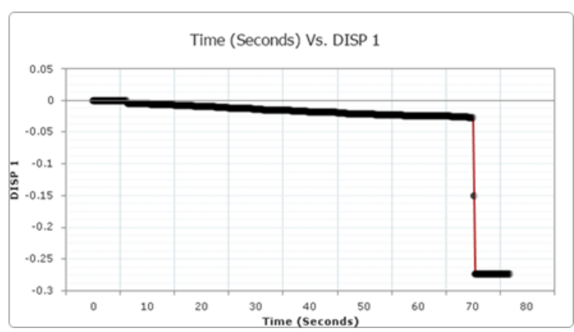

Figure 5.3B: Deflection vs time Control-2

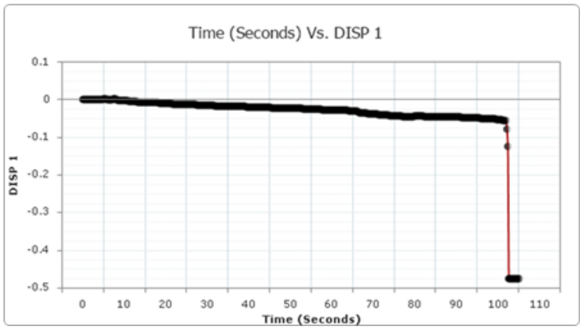

Figure 5.3C: Deflection vs time Control-3

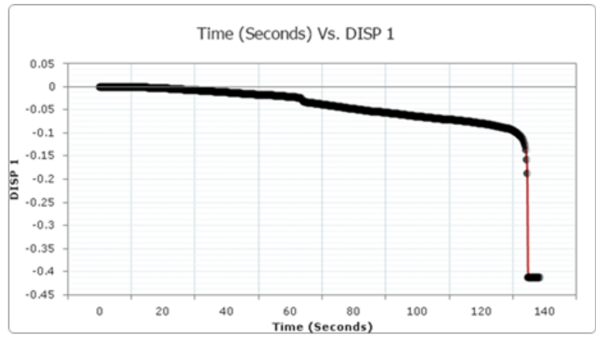

Figure 5.3D: Deflection vs time 25%-1

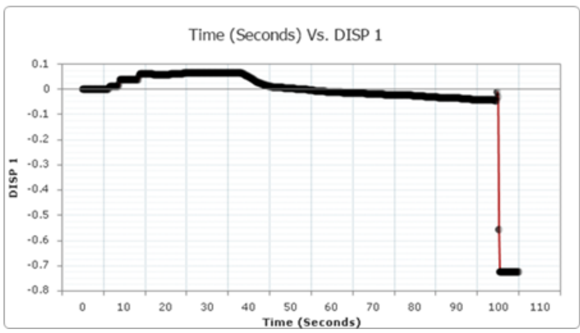

Figure 5.3E: Deflection vs time 25%-2

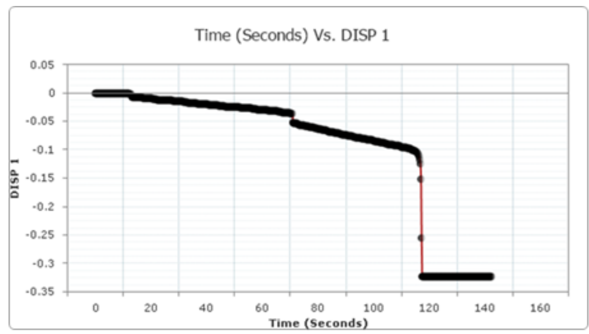

Figure 5.3F: Deflection vs time 25%-3

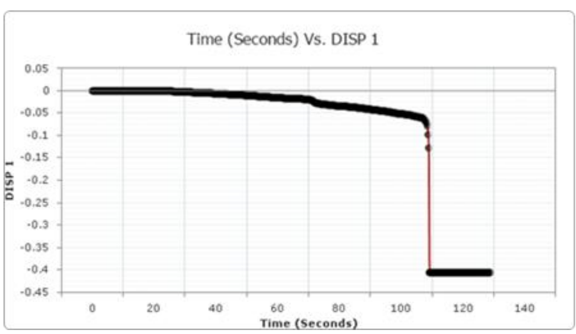

Figure 5.3G: Deflection vs time 30%-1

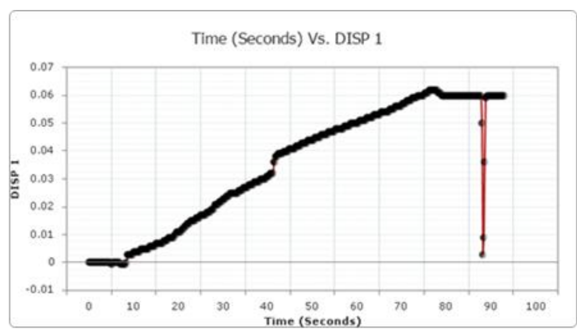

Figure 5.3H: Deflection vs time 30%-2

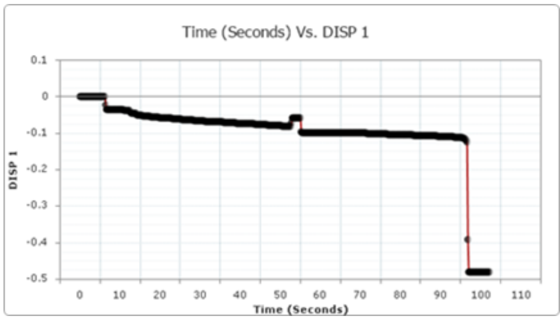

Figure 5.3I: Deflection vs time 30%-3

The deflection of the beams experiences an increase from the control. Table 5.2 shows the average deflection of each mix compared to the control.

Table 5.2: Average deflection vs. control

Sample	1	2	3	Average (in)	std dev	% difference from Control
RAP%	Deflection (in)	Deflection (in)	Deflection (in)	Inches		
0	0.016	0.027	0.08	0.041	0.028	
25	0.099	0.112	0.1	0.104	0.006	60%
30	0.075	0.06	0.081	0.072	0.009	43%

Table 5.2 shows that there is an increase in total deflection of the concrete beam. The control samples averaged a deflection of 0.041 inches, the 25% RAP has a deflection of 0.116 inches and the 30% has a deflection of 0.072 inches. This demonstrates that with the addition of RAP to the concrete the deflection of the beam increases. When the RAP is increased to 35, 40 and 45% the deflection also increases. This is shown in Figure 5.4

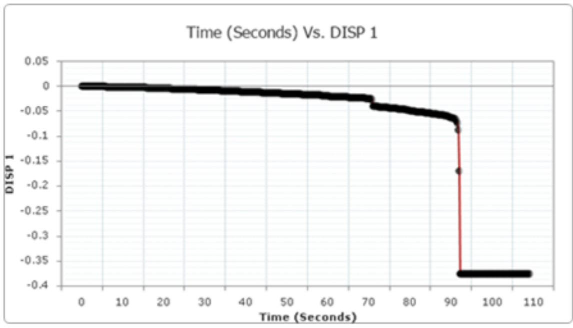

Figure 5.4A: Deflection vs time 35%-1

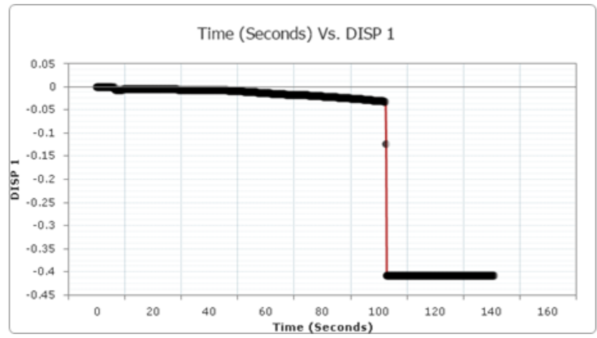

Figure 5.4B: Deflection vs time 35%-2

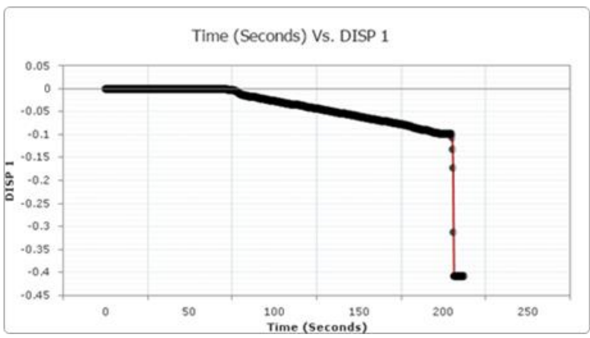

Figure 5.4C: Deflection vs time 35%-3

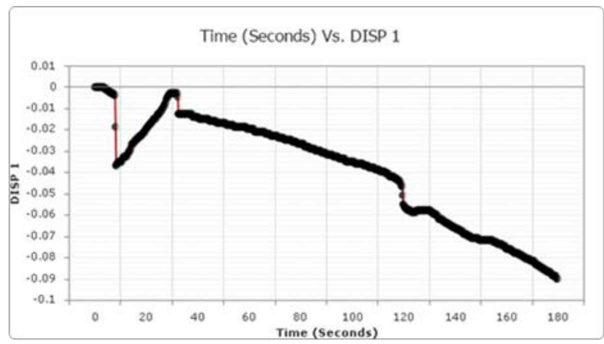

Figure 5.4D: Deflection vs time 40%-1

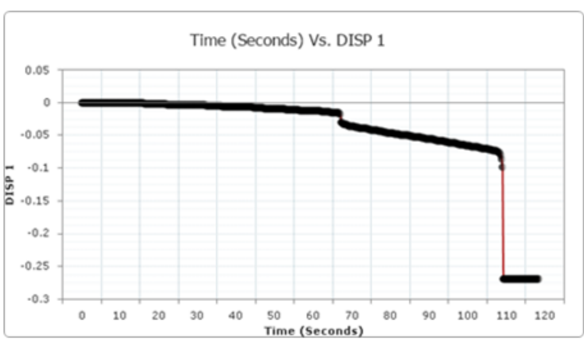

Figure 5.4E: Deflection vs time 40%-2

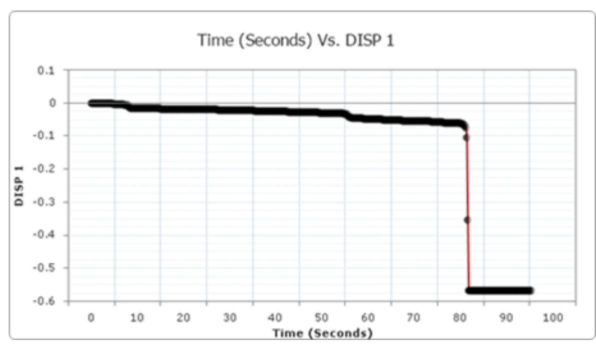

Figure 5.4F: Deflection vs time 40%-3

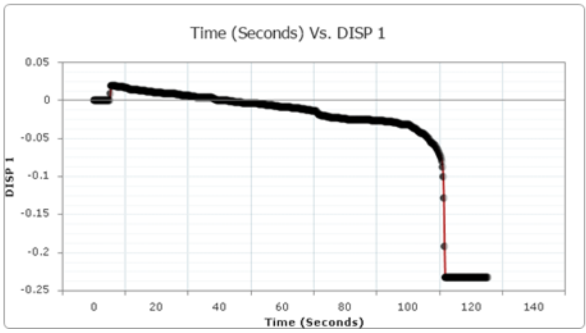

Figure 5.4G: Deflection vs time 45%-1

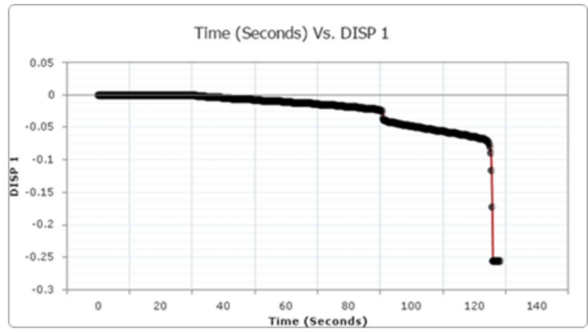

Figure 5.4H: Deflection vs time 45%-2

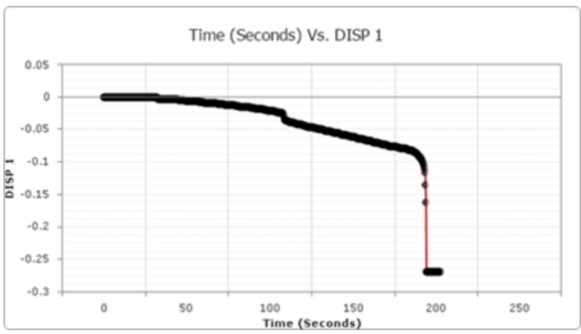

Figure 5.4I: Deflection vs time 45%-3

The above figures show the deflection of the beams with 35, 40 and 45% RAP coarse aggregate percent replacement. The average deflection of the concrete increased as shown in Table 5.3.

Table 5.3: Average total deflection in inches

Sample	1	2	3	Average	St. dev	% difference from Control
RAP %	Deflection (in)	Deflection (in)	Deflection (in)	inches		
35%	0.066	0.032	0.1	0.066	0.028	38%
40%	0.09	0.078	0.077	0.082	0.006	50%
45%	0.1	0.075	0.11	0.095	0.015	57%

Table 5.3 shows with the increase of RAP, the total deflection of the beam prior to rupture is also increased. The total deflection from 35 to 40% RAP increases by about 20% while the deflection of the

RAP from 40 to 45% experienced an increase of about 14%. Figure 5.5 show the displacement v. time curve for 50% RAP.

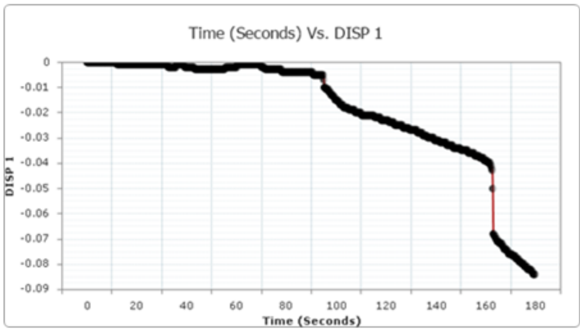

Figure 5.5A: Deflection vs time 50%-1

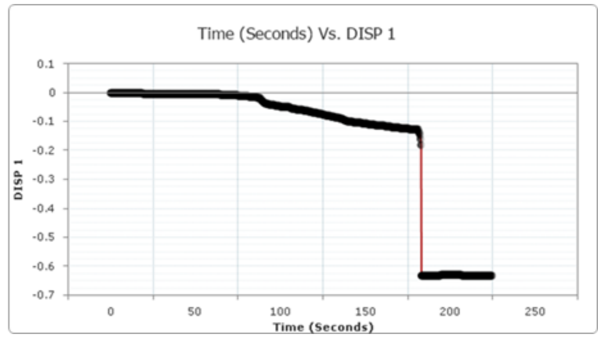

Figure 5.5B: Deflection vs time 50%-2

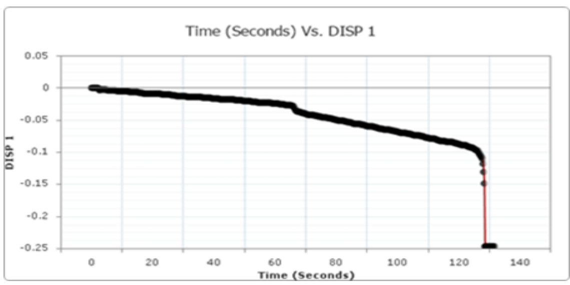

Figure 5.5C: Deflection vs time 50%-3

Table 5.4 shows the deflection of the mix with 50% RAP.

Table 5.4: Total deflection in Inches

Sample	1	2	3	Average	St. dev	% difference from Control
RAP %	Deflection (in)	Deflection (in)	Deflection (in)	inches		
50%	0.04	0.126	0.099	0.088	0.036	54%

Table 5.4 shows that there is an increase in deflection for the 50% RAP mix. This total deflection is not as much as the 45% but is significantly higher than the control mix. The ductility index (μ) is calculated using Equation 3.3 in Section 3.7 for each mix and is shown in Table 5.5.

Table 5.5: Ductility index results

	Control	25%	30%	35%	40%	45%	50%
μ1	2	1.6	2.1	2.2	1.9	1.6	2.4
μ2	1.3	1.2	1.2	1.4	2.9	2.6	3.3
μ3	1.5	2	1.1	2.3	1.4	2.3	1.7
μ average	1.6	1.6	1.5	2.0	2.1	2.2	2.5
st. dev	0.36	0.41	0.55	0.49	0.76	0.51	0.80
Percent Difference from control		0%	-6%	18%	23%	26%	35%

Table 5.5 shows that with the addition of RAP as a coarse aggregate replacement in a high strength concrete mix increases the ductility. The control, 25, and 30% RAP experience no increase in ductility. This can be due to the fact that most of the coarse aggregate in the high strength concrete mix is normal aggregate, thus providing a stiff concrete. With only 25 or 30% replacement of coarse aggregate the RAP does not provide enough aggregate to increase ductility.

The overall trend of the ductility index and total deflection shows an increase as the RAP percent increases. Figure 5.6 shows the average increase in deflection for each mix.

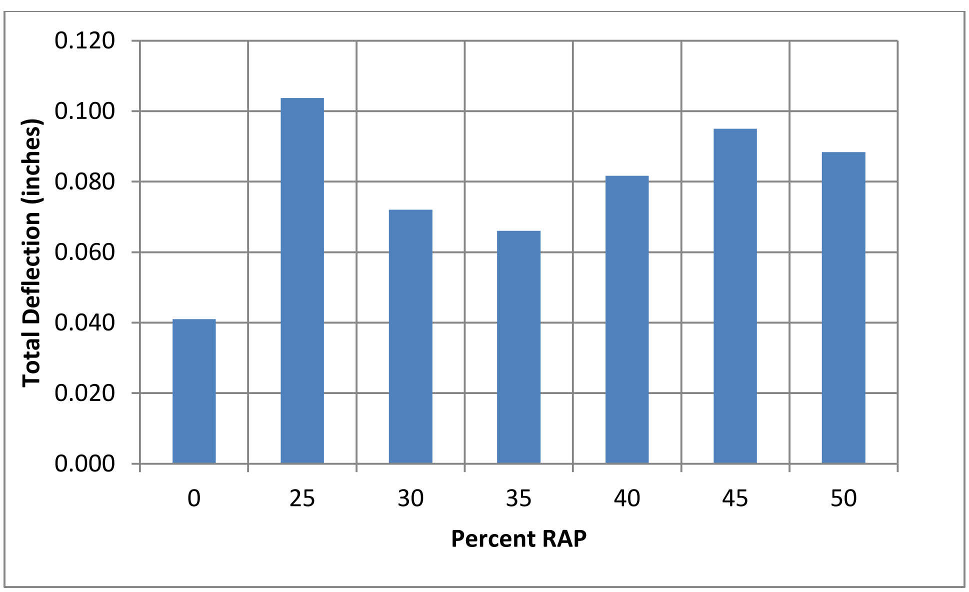

Figure 5.6: Total average deflection for each mix

Figure5.6 shows that the overall increase of total deflection of the beams is increased with the addition of RAP. The sample with 25% RAP shows the highest total deflection, while 35% shows the lowest total deflection of the mixes with RAP aggregate. All of the total deflections are higher than the control samples.

Figure 5.7 shows all of the calculated ductility indexes for each mix.

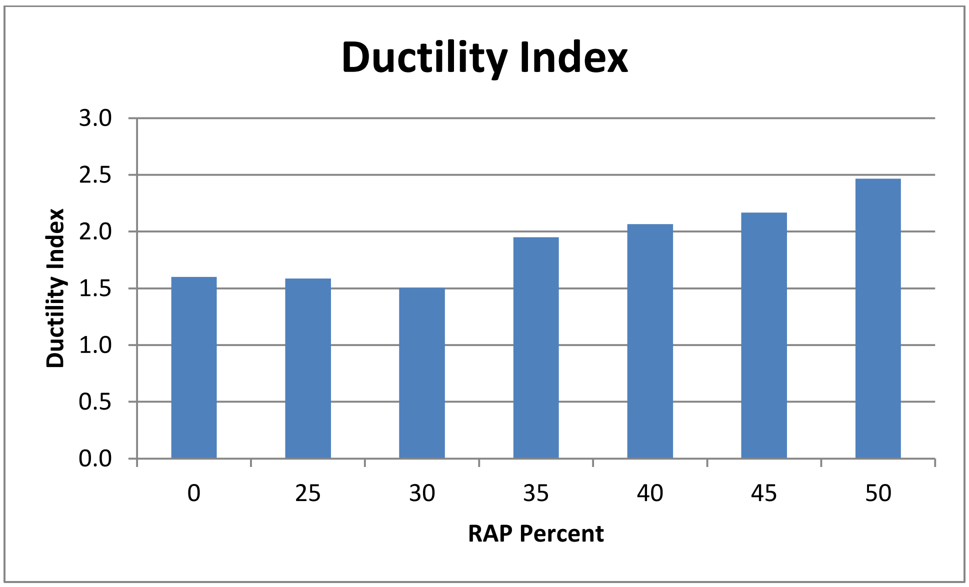

Figure 5.7: Ductility index vs RAP

Figure5.7 shows an increase of ductility index as the RAP increases in the beam. The highest ductility index occurs with the 50% RAP coarse aggregate percent replacement.

The results vary from percent to percent; however, this can be due to the fact that an increase of only 5% does not give enough RAP to increase the ductility of the concrete from one set of samples to another set of samples. However, each of the results are higher than the control except the ductility index for the 30% RAP mix. The increase in ductility of the RAP concrete mixes is due to the nature of the RAP aggregate. RAP aggregate has a soft tar coating that provides more flexible aggregate. This leads to a higher ductility but a lower strength.

The results of the ductility test show that as the RAP percent increases from the control, so does the ductility index and total deflection. There is a steady increase of the ductility index as the RAP increases in the mix design, with 50% giving the highest ductility index. The total deflection also experiences an increase from the control as the RAP is increased. The concrete mix with 25% saw the highest increase in total deflection, all of the total deflection is higher than the control mix.

As RAP is introduced to the mix, the ductility index and total deflection is increased with any percent RAP replacement, giving a more desirable concrete to use in structural applications.

5.4 Strain Rate

ACI-318 code design equations assume a concrete strain limit of 0.003 in./in. when using the design equations. Limiting the strain to 0.003 in./in. ensures the concrete is tension controlled. When the concrete is tension controlled, the behavior is fully ductile, giving warning of failure by deflection and cracking. The methodology for this test is discussed in Section 3.8 of this thesis.

The results of this test are inconclusive. Table 5.5 shows which specimens were able to be completed and a map of the results.

Table5.6: List of figures for strain rate

Load Rate (lb/sec)	600		500		400	
RAP Percent	Sample #	Figure	Sample #	Fig	Sample #	Fig
0	2	5.13	1	No Data	9	5.24
0	5	No Data	4	No Data	11	No Data
0	10	No Data	6	No Data	12	5.25
25	2	No Data	1	5.18	9	No Data
25	3	No Data	5	No Data	12	No Data
25	14	No Data	7	No Data	15	No Data
30	3	No Data	2	No Data	1	No Data
30	6	No Data	7	No Data	5	No Data
30	10	No Data	15	No Data	8	5.26
35	5	No Data	4	No Data	2	No Data
35	9	5.14	11	5.19	8	No Data
35	13	No Data	15	No Data	16	5.27
40	2	5.15	3	5.20	8	5.28
40	5	No Data	4	5.21	10	No Data
40	12	No Data	13	5.22	15	No Data
45	1	No Data	5	No Data	10	No Data
45	3	5.16	9	No Data	14	5.29
45	8	No Data	12	No Data	144	No Data
50	7	5.17	5	No Data	3	No Data
50	10	No Data	8	5.23	6	No Data
50	11	No Data	14	No Data	13	No Data

The stresses v. strain curves for the load rate of 600 lbs/second are presented in Figure 5.8-5.12.

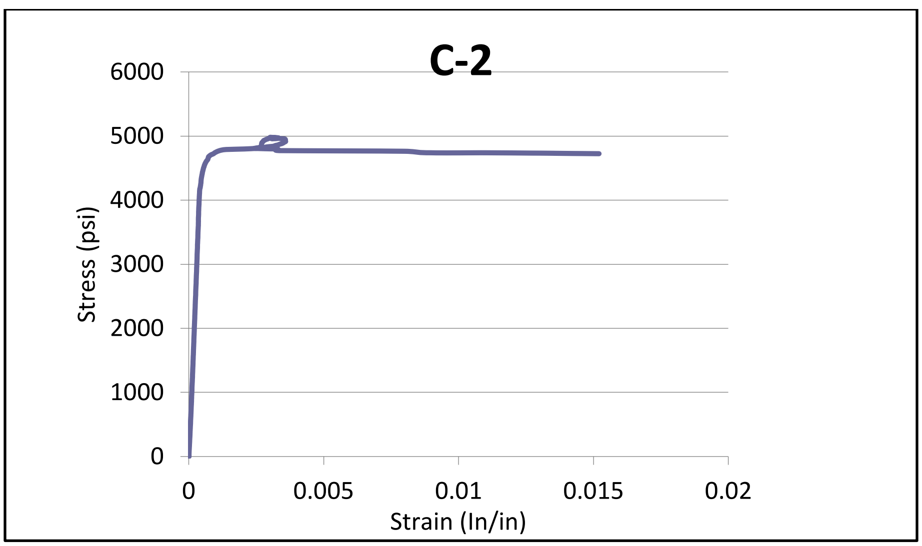

Figure 5.8: Stress v strain control-2

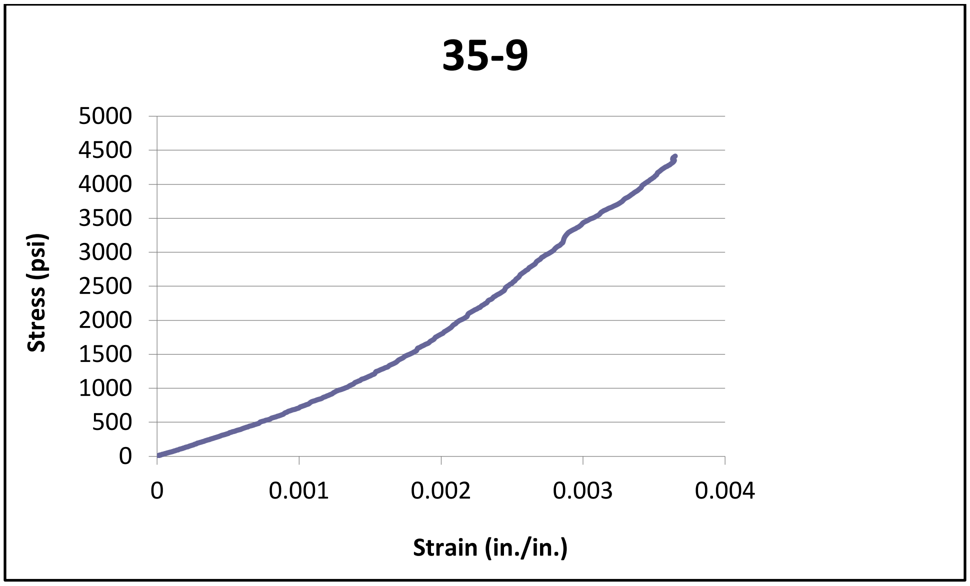

Figure 5.9: Stress v strain 35-9

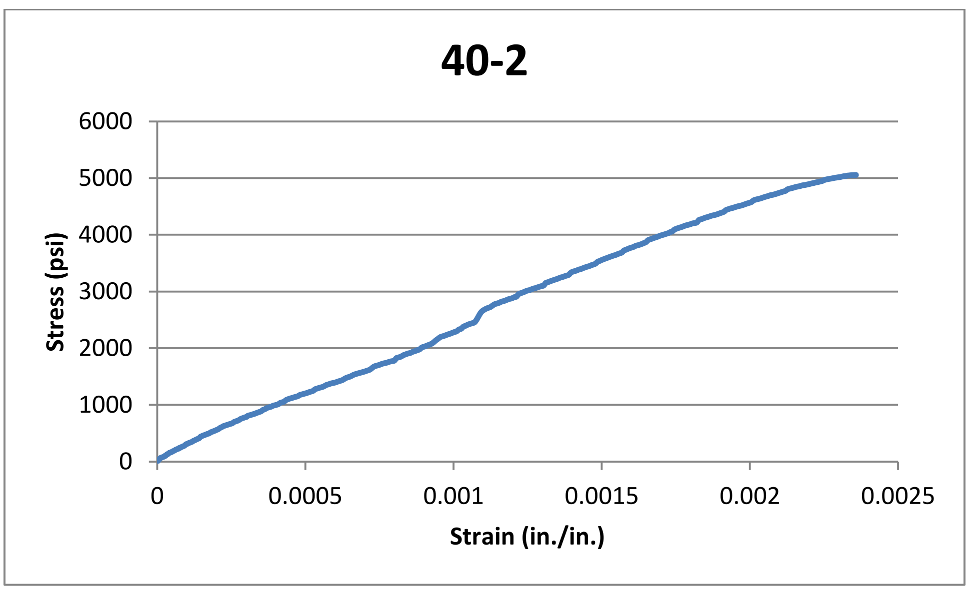

Figure 5.10: Stress v strain 40-2

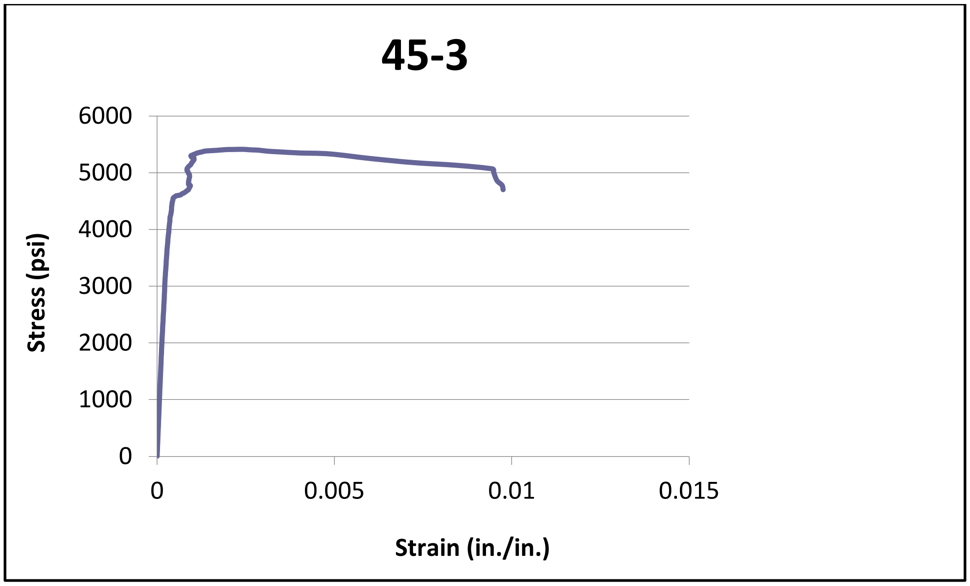

Figure 5.11: Stress v strain 45-3

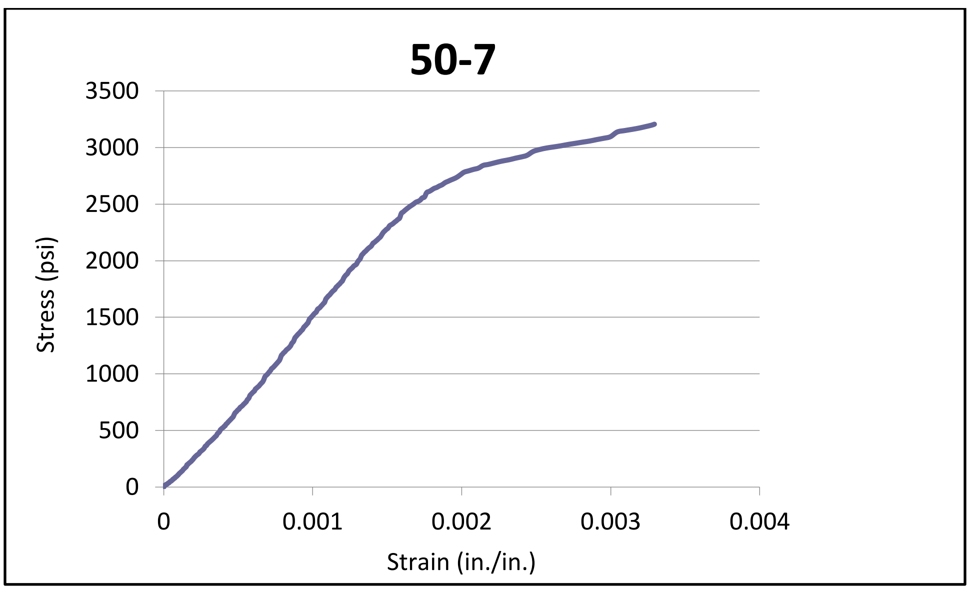

Figure 5.12: Stress v strain 50-7

The five samples that are shown above are the only samples that gave reliable data for the load rate of 600 lb/second. The strain gages that are attached to the RAP concrete cylinders gave false readings due to the cracking of the cylinders. The cracks went through the strain gages giving data that is unreliable.

Figure 5.8 shows control sample #2. The strain at failure is well above 0.003 in./in. The graph shows a "loop" in the stress strain curve at around 0.003 in./in. This can be due to a false reading of the strain gage. Figure 5.10 shows a sample with 40% RAP. The sample breaks before the strain reaches 0.003 in./in. This shows that either the sample didn't pass the assumed 0.003 in./in. or that the cracking of the samples corrupted the strain gages before it can reach 0.003 in./in. The samples that passed the test are C-2, 35-9, 45-3, 50-7.

The results of the strain rate test for the load rate of 500 lbs/second are presented in Figure 5.13-5.18.

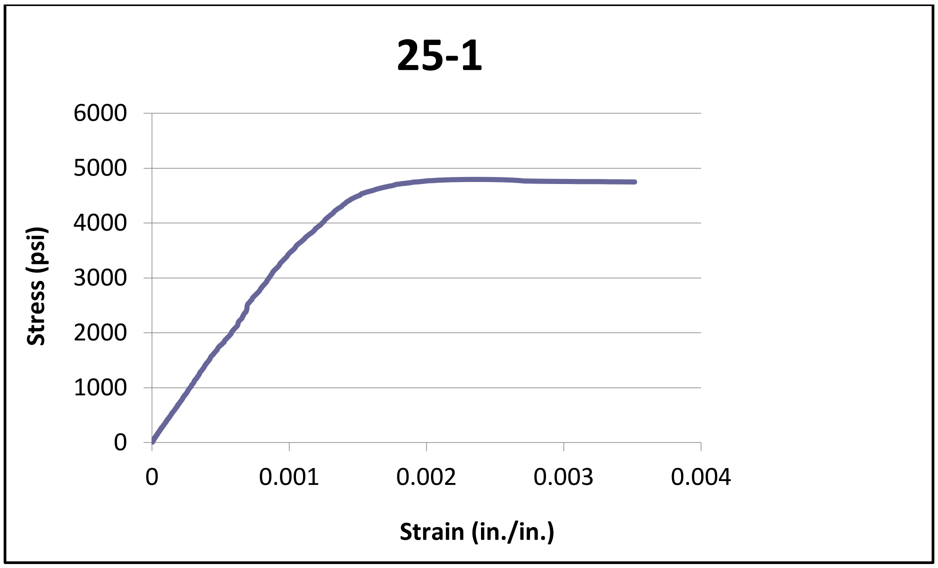

Figure 5.13: Stress v strain 25-1

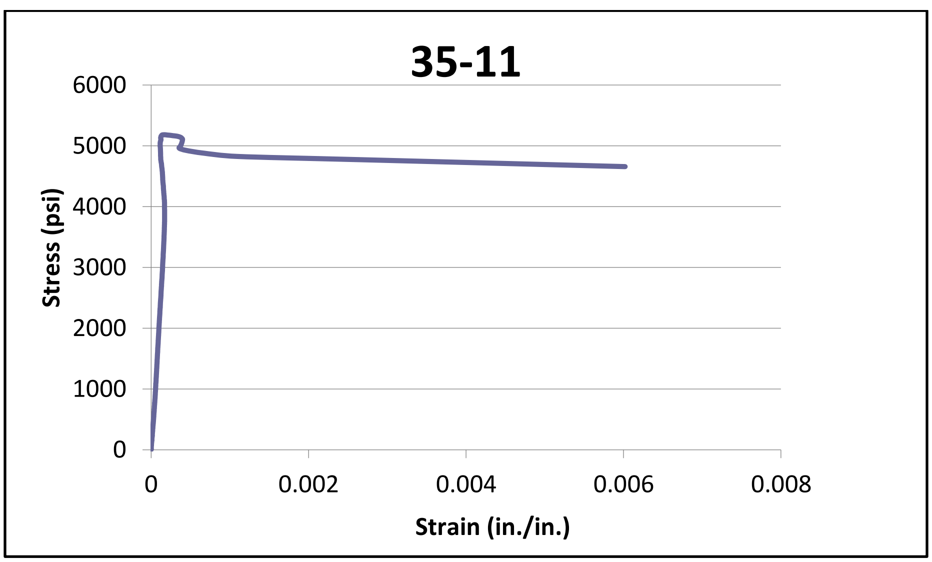

Figure 5.14: Stress v strain 35-11

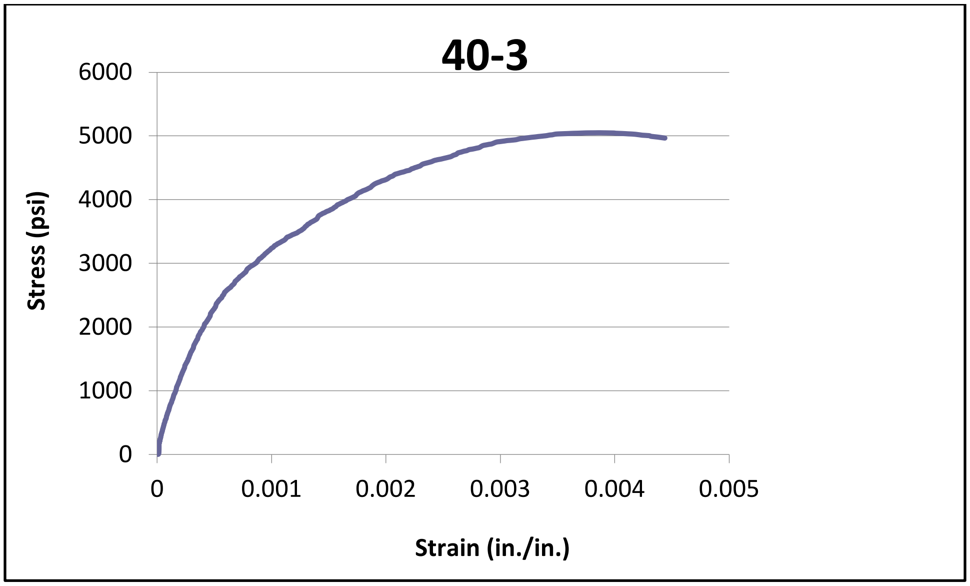

Figure 5.15: Stress v strain 40-3

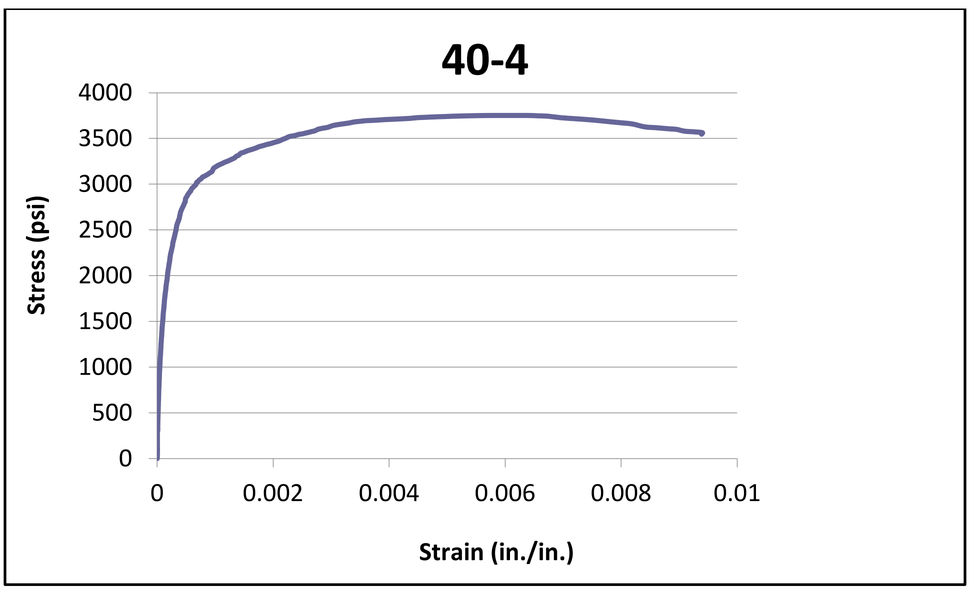

Figure 5.16: Stress v strain 40-4

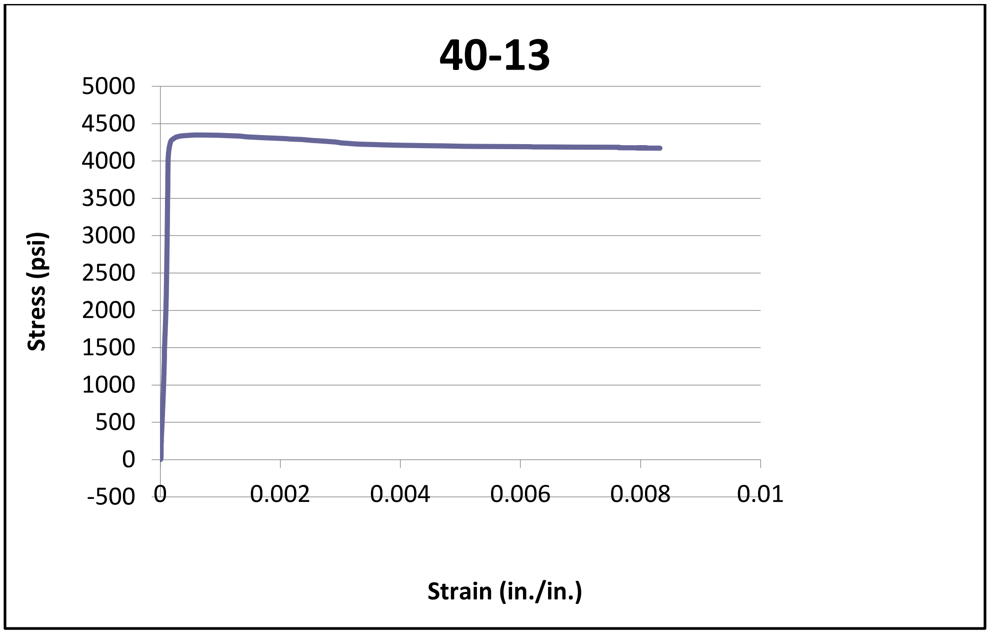

Figure 5.17: Stress v strain 40-13

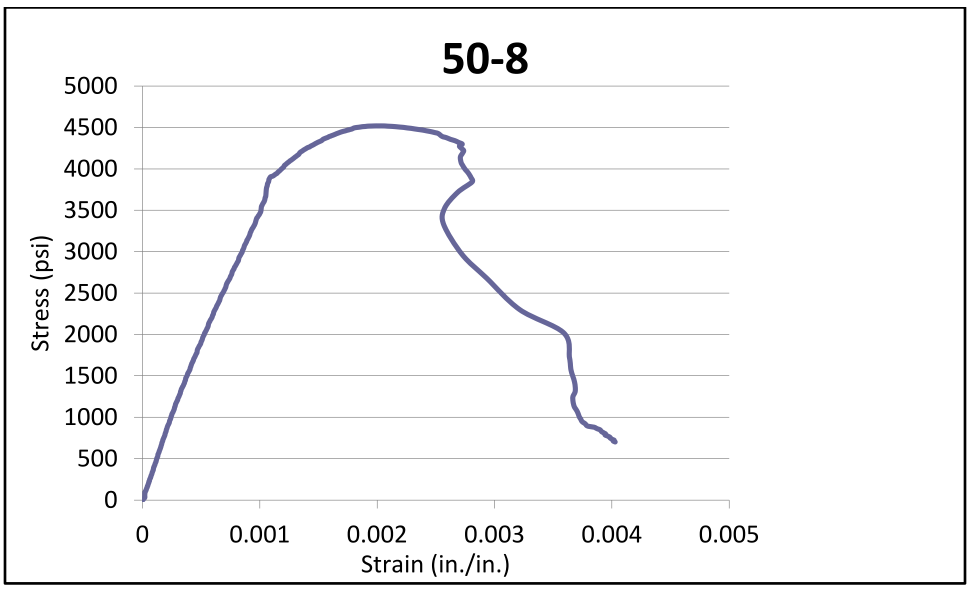

Figure 5.18: Stress v strain 50-8

The graphs shown above are the only samples that exceeded the strain rate of 0.003 in./in. as required by ACI-318 (ACI, 2011). Each of the graphs above show that the strain exceeds the 0.003 in./in. before rupture. Figure 5.23 shows the sample with 50% RAP, the stress strain curve performs as expected until the strain starts to go down in value. This can be due to cracking at the strain gage causing the strain gage to give false data. The other samples that are presented above show that the stress strain curves behaved as expected.

The load rate of 400lbs/sec is shown in Figures 5.19-5.24.

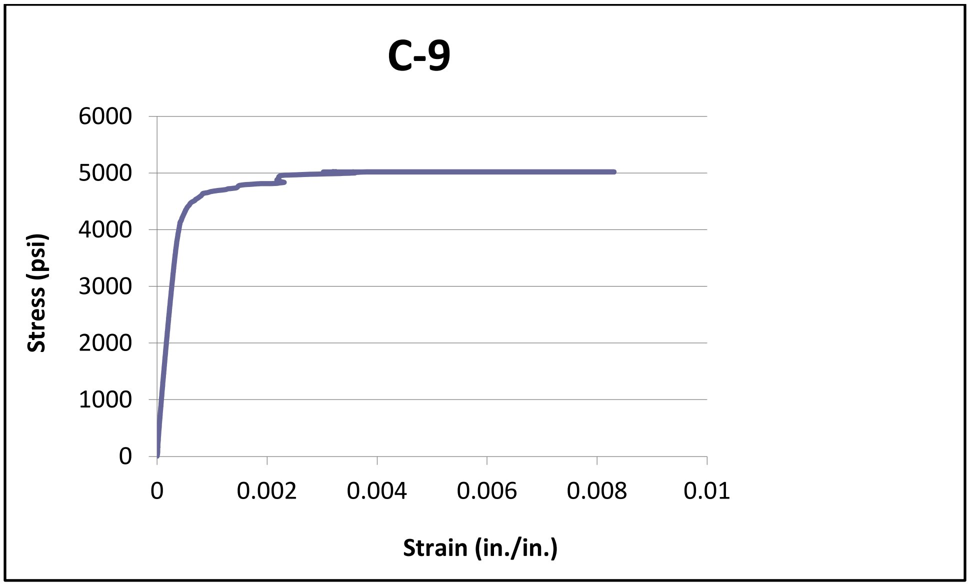

Figure 5.19: Stress v strain C-9

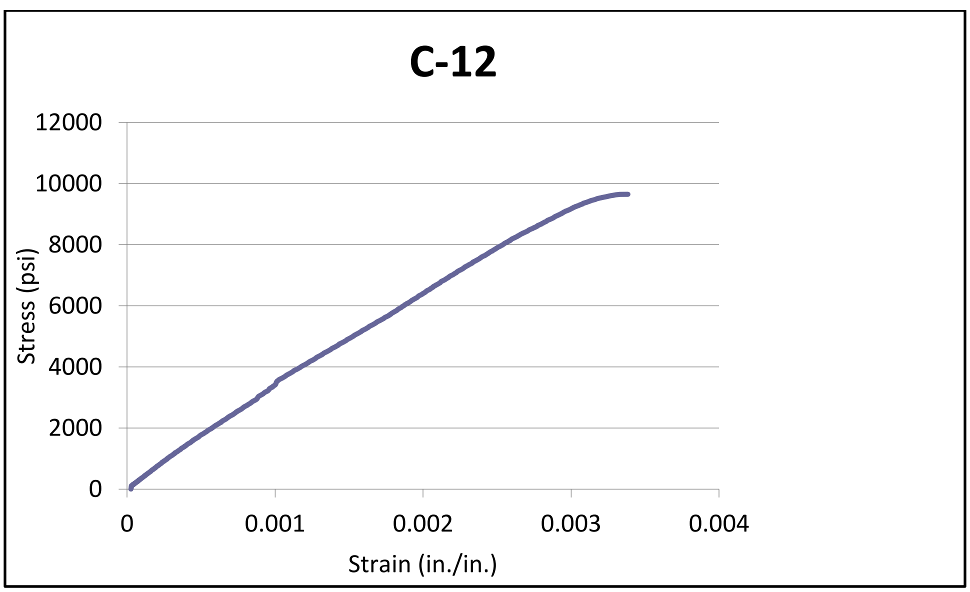

Figure 5.20: Stress v strain C-12

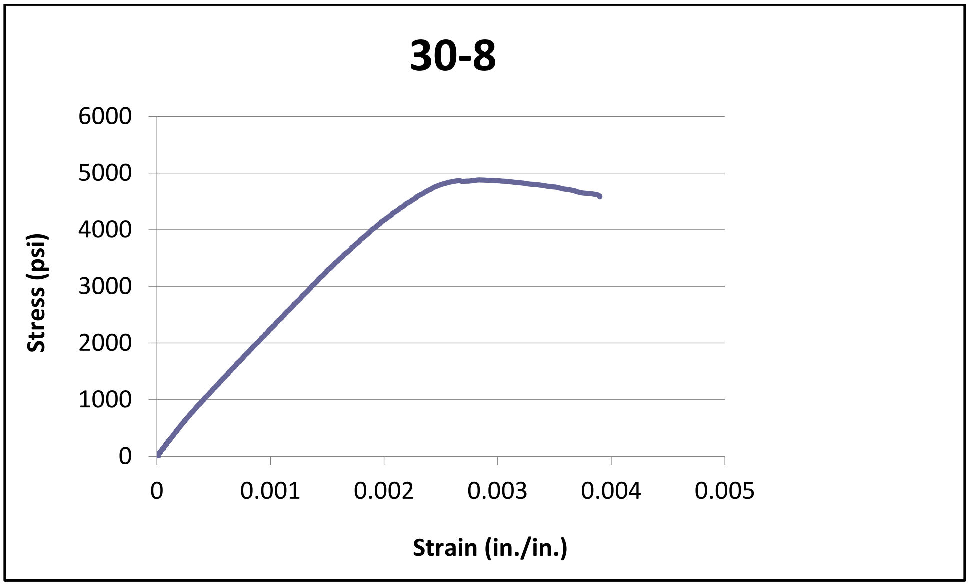

Figure 5.21: Stress v strain 30-8

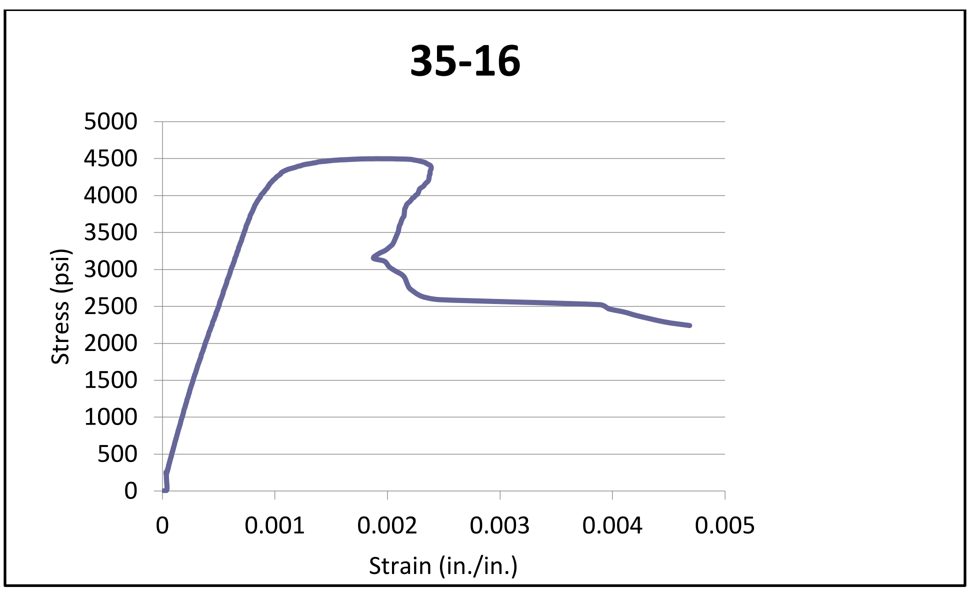

Figure 5.22: Stress v strain 35-16

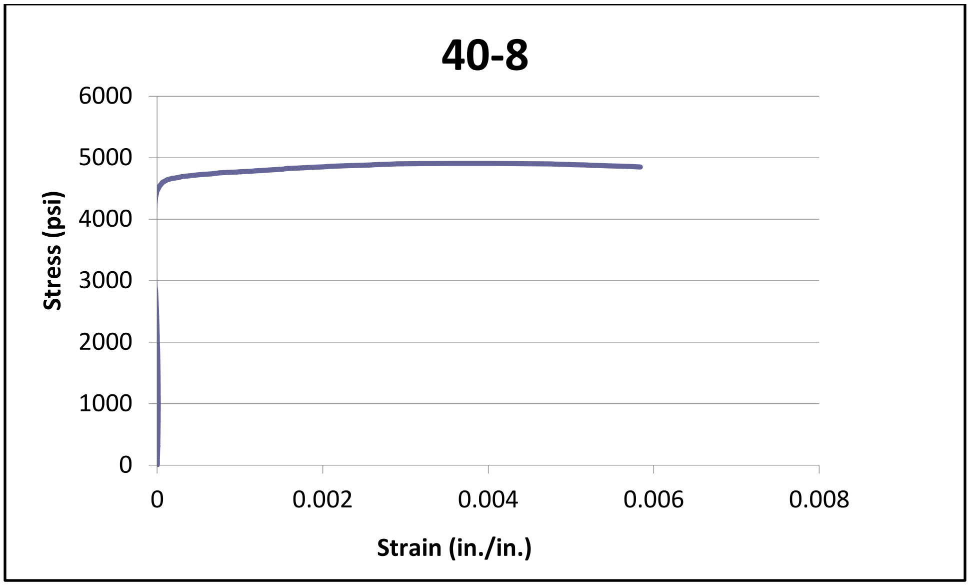

Figure 5.23: Stress v strain 40-8

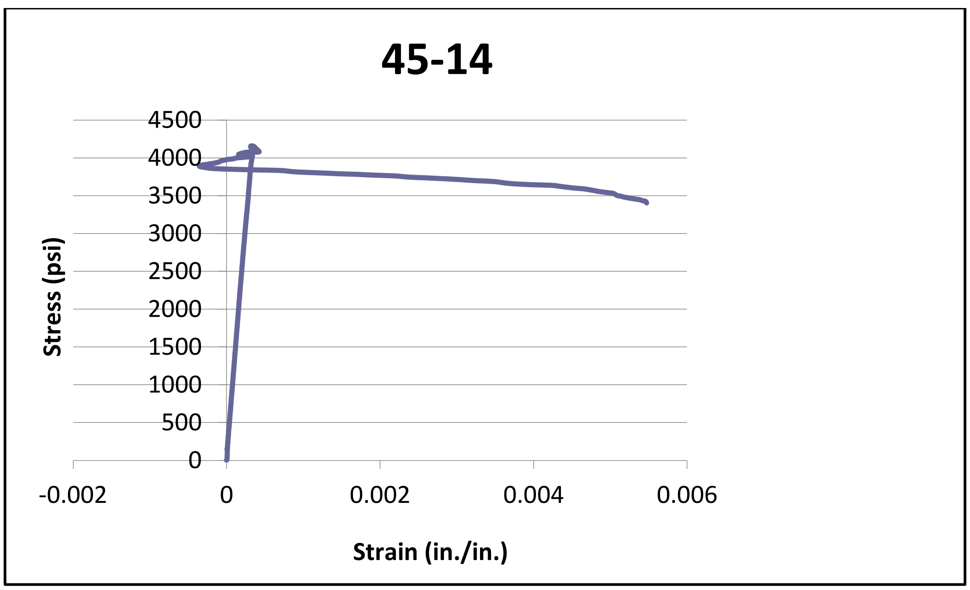

Figure 5.24: Stress v strain 45-14

The figures shown above are the samples that exceeded the 0.003 in./in. strain limit. Figure 5.22 shows that the strain reached the minimum of 0.003 in./in.; however, the data show that the strain decreased before it reached 0.003 in./in., Figure 5.23 has very low strain readings as the stress is increasing, this can be due to a corrupted strain gage providing false readings. Figure 5.24 shows an increase in strain.

However, once the stress reaches its peak, the strain becomes negative. This can be due to cracking of the samples at the strain gage giving invalid data. The rest of the samples did not have reliable data.

The majority of the data for the strain rate test is unreliable. The load cell either stopped recording data, so a stress strain curve could not be constructed. Or the strain gages gave data that is not reasonable. This could be due to cracking of the concrete samples through the strain gages. Furthermore, by not having all the control samples reach the 0.003 in./in. strain, gives an indication of an invalid test, since the control should all reach 0.003 in./in. of strain for all load rates**.**

The results for the strain rate of loading indicate that when the load cell and the strain gages gave data that is reliable, the samples meet the ACI-318 code requirement of 0.003 in./in. The samples that are presented in section 5.3.1 shows a normal stress strain curve. The data shows that RAP concrete meets the ACI-318 code for stain rate, thus providing a reliable alternate to normal coarse aggregate. However, since the majority of the samples did not provide reliable data, a new strain rate test is recommended to verify the results from this study.

5.5 Coefficient of Thermal Expansion

Coefficient of Thermal Expansion (CTE) is a test to determine the amount of expansion the concrete will experience under differential temperatures. The complete methodology is discussed in Section 3.9 of this thesis.

The results of the CTE test are found in Table 5.7-5.13 shown below

Table 5.7: CTE results for 25% RAP

25% RAP				
	Sample 1	Sample 2	Sample 3	
Deg. (^{0}C)	Displ. (in.)	Displ. (in)	Displ. (in.)	
10	0	0	0	
20	0.001	0	0	
30	0.001	0	0.001	
40	0.001	0	0.001	
50	0.002	0.001	0.003	
40	0.002	0.002	0.003	
30	0.002	0.002	0.003	
20	0.001	0.001	0.001	
10	0.001	0	0.001	
				Av. CTE
CTE A	6.40767E-06	3.17818E-06	9.55845E-06	6.38143E-06
CTE B	3.20383E-06	6.35636E-06	6.3723E-06	5.31083E-06

Table 5.8: CTE results for 30% RAP

30% RAP				
	Sample 1	Sample 2	Sample 3	
Deg. (^{0}C)	Displ. (in.)	Displ. (in)	Displ. (in.)	
10	0	0	0	
20	0	0	0	
30	0	0	0	
40	0.001	0	0.001	
50	0.002	0.001	0.002	
40	0.002	0.002	0.002	
30	0.002	0.002	0.001	
20	0.001	0.001	0.001	
10	0	0.001	0	
				Av. CTE
CTE A	6.36591E-06	3.17024E-06	6.33416E-06	5.29011E-06
CTE B	6.36591E-06	3.17024E-06	6.33416E-06	5.29011E-06

Table 5.9: CTE results for 35% RAP

35% RAP				
	Sample 1	Sample 2	Sample 3	
Deg. (^{0}C)	Displ. (in.)	Displ. (in)	Displ. (in.)	
10	0	0	0	
20	0	0	0.001	
30	0.001	0.001	0.001	
40	0.001	0.002	0.001	
50	0.003	0.003	0.002	
40	0.003	0.003	0.002	
30	0.002	0.002	0.002	
20	0.001	0.002	0.001	
10	0.001	0	0	
				Av. CTE
CTE A	9.45409E-06	9.49651E-06	6.32785E-06	8.42615E-06
CTE B	6.30273E-06	9.49651E-06	6.32785E-06	7.3757E-06

Table 5.10: CTE results for 40% RAP

40% RAP				
	Sample 1	Sample 2	Sample 3	
Deg. (^{0}C)	Displ. (in.)	Displ. (in)	Displ. (in.)	
10	0	0	0	
20	0.001	0	0	
30	0.001	0	0.001	
40	0.001	0.001	0.001	
50	0.002	0.001	0.003	
40	0.002	0.002	0.003	
30	0.002	0.001	0.002	
20	0.001	0.001	0.001	
10	0	0	0	
				Av. CTE
CTE A	6.39476E-06	3.16708E-06	9.60666E-06	6.3895E-06
CTE B	6.39476E-06	6.33416E-06	9.60666E-06	7.44519E-06

Table 5.11: CTE results for 45% RAP

45% RAP				
	Sample 1	Sample 2	Sample 3	
Deg. (^{0}C)	Displ. (in.)	Displ. (in)	Displ. (in.)	
10	0	0	0	
20	0.001	0	0	
30	0.001	0	0.001	
40	0.001	0	0.001	
50	0.002	0.001	0.002	
40	0.002	0.001	0.002	
30	0.002	0.001	0.002	
20	0.002	0	0.001	
10	0	0	0	
				Av. CTE
CTE A	6.34049E-06	3.19095E-06	6.31841E-06	5.28328E-06
CTE B	6.34049E-06	3.19095E-06	6.31841E-06	5.28328E-06

Table 5.12: CTE results for 50% RAP

50% RAP				
	Sample 1	Sample 2	Sample 3	
Deg. (^{0}C)	Displ. (in.)	Displ. (in)	Displ. (in.)	
10	0	0	0	
20	0	0	0	
30	0	0	0.001	
40	0.001	0.001	0.001	
50	0.001	0.003	0.002	
40	0.002	0.002	0.002	
30	0.001	0.002	0.002	
20	0.001	0.001	0.001	
10	0	0	0	
				Av. CTE
CTE A	3.19095E-06	9.71443E-06	6.40121E-06	6.43553E-06
CTE B	6.38191E-06	9.71443E-06	6.40121E-06	7.49918E-06

Table 5.13: CTE results for Control

Control 0% RAP				
	Sample 1	Sample 2	Sample 3	
Deg. (^{0}C)	Displ. (in.)	Displ. (in)	Displ. (in.)	
10	0	0	0.001	
20	0	0	0.001	
30	0.001	0	0.001	
40	0.001	0.001	0.002	
50	0.001	0.002	0.003	
40	0.001	0.002	0.003	
30	0.001	0.002	0.002	
20	0	0.002	0.002	
10	0	0.001	0.001	
				Av. CTE
CTE A	3.18296E-06	6.41738E-06	6.4109E-06	5.33708E-06
CTE B	3.18296E-06	3.20869E-06	6.4109E-06	4.26752E-06

The results show an increase in CTE with the addition of RAP. To better compare the results, Table 5.7 to Table 5.13 are shown as a bar chart in Figure 5.25.

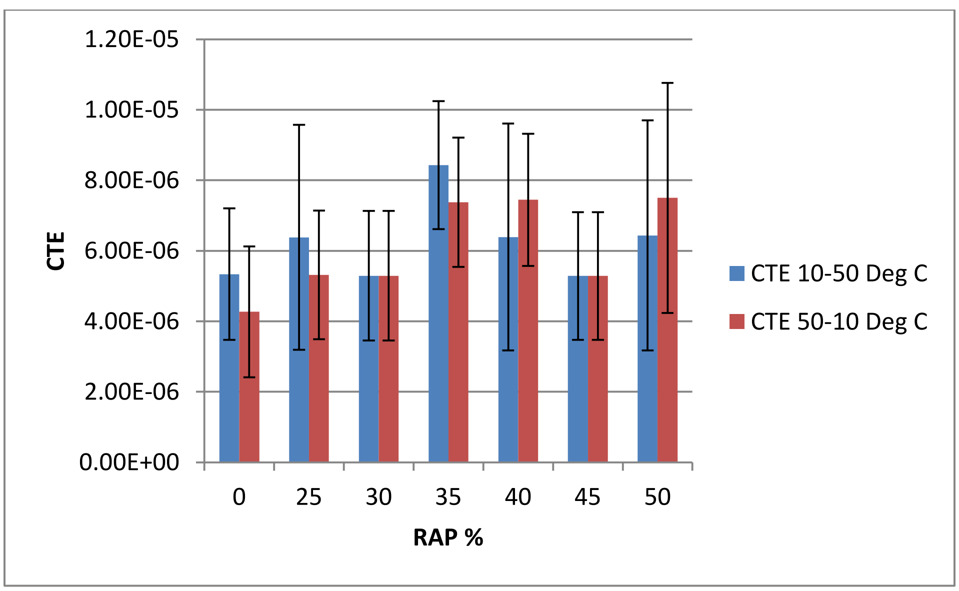

Figure 5.25: Summary of CTE results

The range of CTE is 4.3x10^{-6}/°C to 8.4x10^{-6}/°C indicate a slight increase of the CTE as RAP is introduced into Portland Cement Concrete (PCC). There is no increase in CTE from the control, 30 and 45% RAP. Once RAP is introduced into PCC the CTE does not significantly increase. With a slight increase of CTE the RAP will expand more as the temperature will change. The wide range of CTE results can be possible if the core temperature of the specimens being tested do not reach the required temperature. The results can be due to the nature of the RAP aggregate. The tar that coats the aggregate expands more than normal aggregate that is used in the control mix.

The addition of RAP into PCC causes the CTE to increase. However, the CTE did not increase from the control to the 30% and 45% RAP. RAP aggregate expands more than normal aggregate. The biggest jump in CTE is with the 35% rap which is almost double that of the control. With A higher CTE, the concrete will experience cracking and reduce durability. The Federal Highway Administration conducted a study ("Thermal Coefficient of Portland Cement Concrete" 2011) the results of the CTE test give a range of CTE for concrete of 7.4-13x10^{-6}/^{o}C, the results of the CTE test conducted for this study yield results similar to the results of the study conducted by the Federal Highway Administration. The results of the CTE for this study yield a range of 5.9-8.4x10^{-6}/^{o}C for RAP concrete. The results of the CTE test show that the range of the CTE for RAP concrete is lower than the range of the CTE for concrete conducted by the Federal Highway Administration.

5.6 Modulus of Elasticity

Modulus of elasticity (MOE) is calculated by using ACI equations for design purposes. Since MOE is a function of the compressive strength of the concrete, ACI uses different equations to calculate MOE depending on the compressive strength of the concrete. The complete methodology is discussed in Section 3.10 of this thesis.

The results of the MOE test are presented in Table 5.13

Table 5.14: MOE results vs E equation up to 6000 psi

RAP	E (E-meter) psi	Std. Dev (psi)	E (eq 6000 psi) psi	Std. Dev (psi)	% difference
0	6691074	65401	6252165	47671	6.56%
25	5675810	180182	5002149	359576	11.87%
30	5409908	199394	4466465	388771	17.44%
35	5540442	237437	4542017	6043	18.02%
40	5308381	394812	4229992	240987	20.31%
45	5211689	323123	4168567	77384	20.02%
50	4945787	100485	3988933	68028	19.35%

Table 5.14 is the results of the MOE using the E-meter compared to Equation 3.5 in Section 3.10.

Table 5.15: MOE results vs E equation 6000-12000 psi

RAP	E (E-meter) psi	Std. Dev (psi)	E (eq over 6000 psi) psi	Std. Dev (psi)	% difference
0	6691074	65401	6649767	50703	0.62%
25	5675810	180182	5320257	382442	6.26%
30	5409908	199394	4750506	413495	12.19%
35	5540442	237437	4830863	6427	12.81%
40	5308381	394812	4498995	256312	15.25%
45	5211689	323123	4433664	82305	14.93%
50	4945787	100485	4242606	72354	14.22%

Table 5.15 is the results of the MOE using the E-meter compared to Equation 3.6 in Section 3.10.

These results show a discrepancy between the ACI equation and the reading from the E-meter. The control experiences only a .62% change in MOE from the ACI equation. The sample with 25% RAP have a 6.26% decrease in RAP the biggest change is the samples with 40% RAP, which has a 15.25% decrease. The equation that is the best fit is the equation for over 6000 psi compressive strength. This can be due to the fact that a high strength concrete is used but the nature of the RAP aggregate decreases the compressive strength. Figure 5.26 shows the results of all three MOE readings.

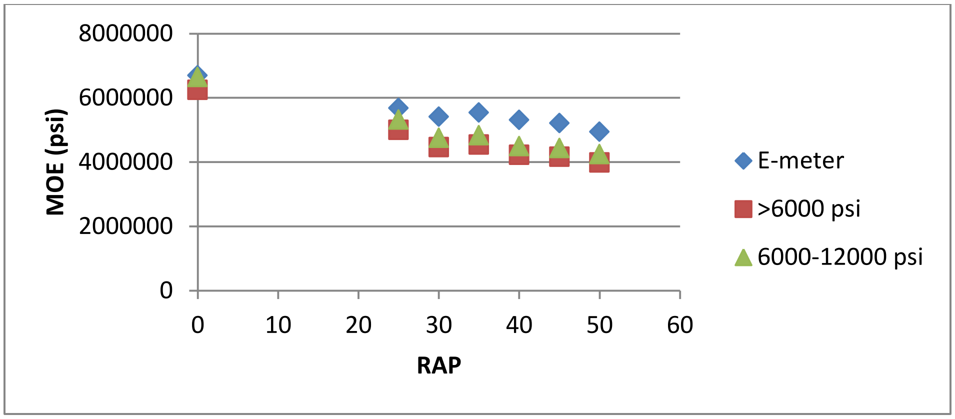

Figure 5.26: Results of all three MOE readings

The results show a decrease in MOE from the E-meter. The MOE has a steady decline in MOE as the RAP is increased in the concrete. This is expected due to the fact that MOE is a function of compressive strength, and as RAP increases the compressive strength decreases.

The results show a decrease in MOE as the RAP increases. There is a small discrepancy from the E-meter to the ACI equation, with the biggest difference with the 40% RAP which has a 15% decrease in MOE. MOE is a function of the compressive strength of the concrete. With the addition of RAP to the concrete mix the compressive strength is shown to decrease, thus yielding a lower MOE. The E-meter provids a MOE reading that is greater then what Equations 5.5 and 5.6 yields, giving a more conservative value of MOE when using design equations. Using Equation 5.6 yields the more accurate results, Equation 5.6 is for concretes with a compressive strength between 6000-12000 psi. Equation 5.6 is more accurate due to compressive strength of the high-strength concrete mix with 0% RAP with a 28 day compressive strength around 10000 psi. Using the Equation provided for 6000-12000 psi gives a better estimate of the MOE.

Chapter 6
Summary of Results and Implications

6.1 Introduction

This thesis studies the long term durability and mechanical behavior of concrete with recycled asphalt pavement (RAP) as a percentage of coarse aggregate replacement. The RAP percent replacement ranges from 25-50% for each test. The long term durability tests are:

- Freeze-thaw durability
- Chloride ion penetration

The following tests are conducted to test the mechanical behavior of RAP concrete:

- Bond strength
- Ductility
- Strain Rate
- Coefficient of thermal expansion (CTE)
- Modulus of elasticity (MOE)

This chapter summarizes the test results for each and discusses the implications of those results.

6.2 Freeze-thaw Durability

Freeze-thaw durability tests the durability of RAP concrete when subjected to rapid freezing and thawing. The durability is expressed by a durability factor (DF), the DF is calculated by taking the transverse frequency of each sample after no more than 36 freeze-thaw cycles. If the samples maintain at least 60% of its initial transverse frequency then it is considered to pass the freeze-thaw test. During the test, after each 36 cycle increment, the dynamic modulus of elasticity is found. If the sample passes the 300 cycles without dropping to 60% of initial transverse frequency, the DF is the same as the dynamic frequency at 300 cycles, and is said to pass with accordance with ASTM C666 (ASTM, 2008). The higher the DF the more durable the sample is. After the 300 cycles are concluded, a compressive strength test is conducted to compare the strength of the RAP concrete after freeze-thaw to that at 28 days of curing.

6.2.1 Summary of Freeze-thaw Results

There are 35 samples that are tested for freeze-thaw durability, five samples for each RAP percent, the RAP percent ranges from 25-50%. The results show that the control samples (0% RAP) has the lowest DF after 300 freeze-thaw cycles. The control samples start to drop dynamic modulus faster than any of the other samples. The samples that preformed the best are the RAP concrete with 35% and 40% replacement. The DF of the samples with 35% and 40% RAP are 95 and 94, respectively. The 25% and 30% RAP have a DF of 84 and 86, respectively. The RAP concrete with 45% and 50% has a DF of 73 and 66, respectively, while the control sample has the lowest DF of 55. The results show as the RAP increases so does the durability, however if too much RAP is used the DF starts to decrease.

The compressive strength of the RAP concrete is tested after the 300 freeze-thaw cycles. The results of this test show that the compressive strength of the RAP concrete is greatest for the samples with 45% and 50% RAP, with a 26 and 27% decrease from their 28 days strength, respectively. The sample with the lowest reduction in strength is the samples with 40% RAP, which saw a 16% reduction in 28 day strength. The control samples saw an average of 21% reduction in strength. The samples have a correlation between DF and compressive strength; the lower the DF the lower the loss of compressive strength. This hold true for all sample expect the control, 45 and 50% RAP.

6.2.2 Implication of Results

The results show that using RAP in concrete gives better durability then a high strength concrete mix with normal aggregate. However, with too much RAP the durability decreases. The compressive strength of the concrete is not affected by the freeze-thaw when compared to the control samples. As expected a reduction in strength occurs when the concrete is subjected to freeze-thaw conditions. However, the reduction in strength is less for the concrete with RAP up to 40% then that of the concrete with no RAP. The results indicate that it is viable to use RAP concrete as a coarse aggregate replacement in freeze-thaw conditions without losing strength as long as the RAP does not exceed 40% replacement.

6.3 Chloride Ion Penetration

Chloride ion penetration tests the likelihood of corrosion and the corrosion rate of concrete due to chloride ions. This test is conducted with a 4-pin wenner prode array, and measures the current through the concrete. The resultant potential difference is measured between the inner pins. The current used and resultant potential along with the affected sample area are used to calculate the resistivity of the concrete. This test is important if RAP concrete is to be used as a structural concrete, chloride ion penetration affects the steel rebar in the concrete.

6.3.1 Summary of Chloride Ion Penetration Results

The chloride penetration results show a slight decrease in resistivity but all samples still fall in the low corrosion rate and are within the range of moderate rate of corrosion. There is no significant different between any of the RAP concrete and the control.

6.3.2 Implications of Results

The results show no variation of corrosion rate and risk of corrosion of chloride ions with the addition of RAP concrete. The conclusion of this test is that RAP of up to 50% can be used is structural applications without increasing the risk of corrosion or the corrosion rate compared to a normal high strength concrete mix.

6.4 Bond Strength

Bond strength is the ability of concrete to adhere to steel rebar without slippage. This is important in structural application to ensure that the load that the concrete is carrying can get transferred to the steel rebar. To test bond strength, a push through test is carried out. A #3 rebar is cast into the samples with the bar extruding out of the sample. A compression machine is used to apply an axial force to the rebar, and the applied force that is required to dislodge the rebar is measured. The bond strength is directly related to the tensile strength, so a different control mix is used in order to get a better comparison of RAP concrete to a 4000 psi mix.

6.4.1 Summary of Bond Strength Results

There is a reduction of bond strength when RAP is introduced; however the control mix that is used has an average 28 day strength of 6673 psi which is significantly higher than the 28 day strength of the RAP mixes, which are 5291 psi, 4867 psi and 4511 psi for 30%, 40% and 50% RAP respectively. There is an average of a 40% decrease of bond strength when RAP is added. When RAP is added, there is no reduction in strength between the different amounts of RAP in the concrete, with only a 2% difference between the highest and lowest bond strength.

6.4.2 Implication of Results

The results show a reduction in bond strength when RAP is added to the concrete. However, once RAP is added there is no adequate change in bond strength. This can be due to the fact that the control has a higher compressive strength then the RAP concrete. The conclusion of this test shows no difference in bond strength once RAP is added. It is recommended that this test be conducted again with a control with the same compressive strength as the RAP mixes to verify the results.

6.5 Ductility

Ductility is a measure of a materials ability to deform or deflect without rupture. It is beneficial for RAP concrete that is being used in structural application to have the same or more ductility as normal Portland cement concrete. To measure the ductility, a ductility factor (DF) is used. The ductility factor is a ratio of the deflection at max loading to the deflection at yield. The ductility factors are compared to the control to see if RAP concrete has a greater ductility.

6.5.1 Ductility Results

The results of the ductility test show that with the addition of RAP as a coarse aggregate replacement, the DF increases. The highest DF is the specimen with 50% RAP, with a DF of 2.5, while the control sample has a DF of 1.6. The DF holds at 1.6 for the control, and 25% RAP but drops to 1.5 for the 30% RAP mix. As the RAP increases above 30% replacement, the DF also increases. The samples with 35%, 40%, 45% and 50% have DFs of 2.0, 2.1, 2.2 and 2.5 respectively. These results are expected due to the nature of the RAP. RAP has an asphalt coating that make it more flexible than normal coarse aggregate. With a more flexible aggregate, the concrete can be expected to withstand more deflection before rupture then a normal concrete mix. With the addition of RAP in concrete as a replacement for normal coarse aggregate, the concrete as a whole becomes more ductile.

6.5.2 Implications of Results

The results show an increase in ductility as RAP is increased. The ductility of RAP concrete increases as the percentages of RAP increases. Adding RAP to the concrete as a coarse aggregate replacement is beneficial, as it increases the ductility therefore making it a more desirable concrete. RAP from 35% to 50% showed the most potential to use in the high strength concrete.

6.6 Strain Rate of Loading

Strain rate is used in ACI-318 design code equations, and is set at 0.003 in/in for a tension controlled design. If the design is tension controlled, the concrete will act ductile giving adequate notice of failure in the mode of deflections and cracking prior to failure.

6.6.1 Strain Rate of Loading Results

The results of the strain rate of loading test show that the samples that were able to complete the test met the ACI-318 code for strain rate of 0.003 in./in. However, 47 out of the 63 samples that are tested did not give data that is reliable to use. The cracking of the concrete interrupted the reading of the strain gages, giving data that is inconclusive. The 17 samples that gave reliable data shows that RAP concrete in any percent that is tested (25%, 30%, 35%, 40%, 45% and 50%) meets the ACI-318 code requirement of 0.003 in./in. of strain at crushing.

6.6.2 Implication of Results

The results show that using RAP concrete meets the ACI-318 code for strain rate of 0.003 ion./in. making it a desirable replacement for normal Portland concrete mixes. The samples that are able to complete the test gave a good indication that RAP does not affect the required strain rate.

6.7 Coefficient of Thermal Expansion

Coefficient of thermal expansion (CTE) is a measure of how much a material changes in length due to temperature change. CTE is important in structural applications because if a material expands or contracts too much it creates stresses on the concrete, these stresses can lead to cracking and eventually failure.

This test is carried out on three samples of each RAP percent, the RAP percentages that are tested are 25, 30, 35, 40, 45, 50 and control. A linear strain conversion transducers (LSCT) is used to measure the change in length, a freeze-thaw chamber is used to control the temperature change. The samples are brought to a temperature of 10^{o}C and then raised to 50^{o}C at 10^{o}C increments, with length measurements taking at every 10^{o}C increments. The results are put into an equation and the CTE is then calculated.

6.7.1 Results of Coefficient of Thermal Expansion

The results of the CTE test show a slight increase of CTE as RAP is introduced as a coarse aggregate replacement. The control samples have a CTE of 5.3x10^{-6} /^{o}C from 10-50^{o}C and 4.3x10^{-6} /^{o}C from 50-10^{o}C. The samples with the biggest CTE are the samples with 35% RAP, with a CTE of 8.4x10^{-6}/^{o}C and 7.4x10^{-6}/^{o}C from 10-50^{o}C and 50-10^{o}C, respectively. There is no change in the CTE between the control samples and the samples with 30 and 45% RAP. The samples with 40 and 50% RAP have the same CTE of 6.4x10^{-6}/^{o}C and 7.4x10^{-6}/^{o}C from 10-50^{o}C and 50-10^{o}C, respectively. All of the samples fall within the range of common concrete CTE’s.

6.7.2 Implication of Results

The results show a slight increases in CTE as the RAP is increased. With a higher CTE the concrete can expand more and that can be critical in structural applications. For concrete with steel reinforcements a higher CTE with create stresses around the steel rebar. Having the RAP concretes CTE in the range of CTE for concrete makes RAP a viable alternative to coarse aggregate.

6.8 Modulus of Elasticity

In design modulus of elasticity (MOE) is calculated using ACI equations which are a function of compressive strength. ACI uses different equations based on different compressive strengths. This test uses an Emodumeter (E-Meter) to calculate the MOE, then the results are compared to the ACI equations. This test is done on three samples from each RAP percentage. The RAP percentages that are used are 25, 30, 35, 40, 45 and 50%.

6.8.1 Modulus of Elasticity

The results show a decrease in MOE when RAP is added, this is expected due to the loss in compressive strength in the concrete when RAP is added. The results also show that the E-meter recorded a higher MOE then the ACI equations. The biggest discrepancy is the samples with 40% RAP with a 15% difference from E-meter and ACI equations, whereas the control has a percent difference of .62%.

6.8.2 Implication of Results

The results show that the ACI equations are used for concrete with normal aggregate. With a max of 15% difference between the ACI equation and the E-meter, The ACI equation to determine MOE is acceptable for concrete with RAP.

6.9 Complete Summary of Results

In order to evaluate the overall performance of RAP concrete, the mechanical properties and the long term durability results are tabulated in Table 6.2. The results of each test are assigned quantitative score of either a high pass (HP), a pass (P), a fail (F), or a low fail (LF). Each score is weighted as shown in Table 6.1.

Table 6.1: Weighted value for each score

HP=	1
P=	0.7
F=	0.5
LF=	0.3

Table 6.2 shows the results of the test and the total value for each RAP percent tested. It should be noted that the strain rate test results are excluded because of their inconclusive results. Additionally, the bond strength test is excluded from this table because the test is not performed on all RAP percent and different test is recommended because of lab restrictions.

Table 6.2: Weighted results for each test

	RAP percent					
Test	**25%**	**30%**	**35%**	**40%**	**45%**	**50%**
Freeze-thaw	P	HP	HP	P	P	P
F-T strength	P	P	HP	HP	P	P
Chloride ion penetration	P	P	P	P	P	P
Ductility	P	F	HP	HP	HP	HP
Coefficient of thermal expansion	P	P	F	F	P	F
Modulus of elasticity	P	P	P	P	P	P
Total value	4.2	4.3	4.7	4.6	4.5	4.3

From Table 6.2 it is shown that the RAP percent that preformed the best under all test is the concrete with 35% with a score of 4.7 making it the optimal concrete mix with RAP as a coarse aggregate replacement. Using a concrete with 35% RAP coarse aggregate replacement will achieve a green construction material for use in structural applications, that performs as well or better than traditional concrete mixes

6.10 Future Work

Although RAP is a viable option to replace normal coarse aggregate in a high strength concrete mix, additional studies are warranted. An in situ test strip of RAP concrete would be beneficial as it will show how the RAP concrete withstands actual loading applications. The strain rate of loading test needs to be retested due to the invalid results. Having the strain rate test verify ACI-318 code will ensure a tension controlled concrete. A pull-out test should be conducted and compared to a control with the same 28 day compressive strength. Conducting a pull-out test can verify the bond strength results that are presented in this thesis.

The compressive strength of RAP concrete is reduced by a great amount, the reduction is caused by the RAP coarse aggregate failing during loading. RAP aggregate is not as strong as normal aggregate. If the size of RAP replacement was reduced from a max size of 3/4th inch to 5/8th inch, this would reduce the 3/4th inch RAP that has more asphalt coasting causing it to be weaker. This would decrease the ductility but should also increase the compressive strength.

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Appendix A
Freeze-thaw data

A.1 Freeze-thaw Durability

Appendix A shows the raw data of the dynamic modulus testing during the freeze-thaw (F-T) cycles. The Data is shown in Figure A.1-A.35.

Table A.1 F-T Data for sample C-8

C-8							
control					**dynamic**		**durability**
Cycles	**Mass (g)**	**Length (cm)**	**Diameter (cm)**	**Frequency (Hz)**	**modulus**		**factor**
0	3941	19.79	9.7	6250	100		
36	3980	-	-	6406	105		
72	3982	-	-	6484	108	good	
108	3986	-	-	6445	106	good	
144	3993	-	-	6289	101	good	
180	3993	-	-	5280	71	good	
216	3995	-	-	5117	67	good	
252	3994	-	-	4805	59	no good	
288	3997	-	-	4297	47	no good	0.68752
300	3997			4297	47	no good	47.26837504

Table A.2: F-T Data for sample C-3

C-3							
control					**dynamic**		**durability**
Cycles	**Mass (g)**	**Length (cm)**	**Diameter (cm)**	**Frequency (Hz)**	**modulus**		**factor**
0	3946	19.95	9.68	6367	100		
36	3985	-	-	6523	105	good	
72	3986	-	-	6602	108	good	
108	3990	-	-	6484	104	good	
144	3998	-	-	5625	78	good	
180	4001	-	-	5703	80	good	
216	4002	-	-	5352	71	good	
252	4002	-	-	5430	73	good	
288	4004	-	-	5000	62	good	0.785299199
300	4004			5000	62	good	61.66948319

Table A.3: F-T Data for sample C-15

C-15							
control					**dynamic**		**durability**
Cycles	**Mass (g)**	**Length (cm)**	**Diameter (cm)**	**Frequency (Hz)**	**modulus**		**factor**
0	3890	19.81	9.83	6289	100		
36	3925	-	-	6484	106	good	
72	3926	-	-	6484	106	good	
108	3932	-	-	6523	108	good	
144	3937	-	-	6523	108	good	
180	3942	-	-	6211	98	good	
216	3945	-	-	5703	82	good	
252	3947	-	-	5000	63	good	
288	3948	-	-	4609	54	no good	0.73286691
300	3947			4609	54	no good	53.70939085

Table A.4: F-T data for sample C-14

C-14							
control					**dynamic**		**durability**
Cycles	**Mass (g)**	**Length (cm)**	**Diameter (cm)**	**Frequency (Hz)**	**modulus**		**factor**
0	3910	20.08	9.69	6367	100		
36	3957	-	-	6563	106	good	
72	3958	-	-	6602	108	good	
108	3958	-	-	6641	109	good	
144	3962	-	-	6641	109	good	
180	3968	-	-	6563	106	good	
216	3973	-	-	6211	95	good	
252	3975	-	-	4570	52	no good	
288	3977	-	-	4648	53	no good	0.730014135
300	3997			4648	53	no good	53.29206379

Table A.5: F-T data for sample C-7

C-7							
control					**dynamic**		**durability**
Cycles	**Mass (g)**	**Length (cm)**	**Diameter (cm)**	**Frequency (Hz)**	**modulus**		**factor**
0	3926	20.05	9.6	6409	100		
36	3963	-	-	6563	105	good	
72	3965	-	-	6602	106	good	
108	3970	-	-	6523	104	good	
144	3977	-	-	6289	96	good	
180	3979	-	-	5977	87	good	
216	3981	-	-	5391	71	good	
252	3982	-	-	5039	62	good	
288	3983	-	-	4883	58	no good	0.761897332
300	3983			4883	58	no good	58.04875443

Table A.6: F-T data for sample 25-8

25-8							
25% RAP					**dynamic**		**durability**
Cycles	**Mass (g)**	**Length (cm)**	**Diameter (cm)**	**Frequency (Hz)**	**modulus**		**factor**
0	3871	19.82	9.66	6055			
36	3905	-	-	6289	108	good	
72	3905	-	-	6406	112	good	
108	3908	-	-	6406	112	good	
144	3914	-	-	6328	109	good	
180	3916	-	-	6367	111	good	
216	3919	-	-	6367	111	good	
252	3921	-	-	6250	107	good	
288	3922	-	-	6094	101	good	
300	3922			6094	101	good	101.2923402

Table A.7: F-T data for sample 25-13

25-13							
25% RAP					**dynamic**		**durability**
Cycles	**Mass (g)**	**Length (cm)**	**Diameter (cm)**	**Frequency (Hz)**	**modulus**		**factor**
0	3929	19.98	9.65	6094			
36	3963	-	-	6289	107	good	
72	3964	-	-	6367	109	good	
108	3967	-	-	6328	108	good	
144	3974	-	-	6289	107	good	
180	3977	-	-	6172	103	good	
216	3979	-	-	6016	97	good	
252	3978	-	-	5508	82	good	
288	3980	-	-	5273	75	good	
300	3980			5273	75	good	74.87048439

Table A.8: F-T data for sample 25-11

25-11							
25% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3892	19.93	9.7	6055			
36	3934	-	-	6250	107	good	
72	3934	-	-	6250	107	good	
108	3932	-	-	6211	105	good	
144	3943	-	-	6172	104	good	
180	3947	-	-	6250	107	good	
216	3951	-	-	5781	91	good	
252	3954	-	-	5547	84	good	
288	3953	-	-	5195	74	good	
300	3953			5195	74	good	73.61101546

Table A.9 F-T data for sample 25-6

25-6							
25% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3904	19.98	9.66	6172			
36	3939	-	-	6172	100	good	
72	3941	-	-	6133	99	good	
108	3944	-	-	6172	100	good	
144	3948	-	-	6172	100	good	
180	3956	-	-	6172	100	good	
216	3953	-	-	6133	99	good	
252	3951	-	-	5820	89	good	
288	3953	-	-	5391	76	good	
300	3952			5391	76	good	76.29337528

Table A.10: F-T data for sample 25-7

25-7							
25% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3914	20.1	9.65	6094			
36	3952	-	-	6250	105	good	
72	3953	-	-	6250	105	good	
108	3853	-	-	6367	109	good	
144	3941	-	-	6211	104	good	
180	3941	-	-	6250	105	good	
216	3941	-	-	6172	103	good	
252	3940	-	-	5172	72	good	
288	3938	-	-	5977	96	good	
300	3936			5898	94	good	93.67088785

Table A.11: F-T data for sample 30-14

30-14							
30% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3889	20.05	9.7	5898			
36	3921	-	-	6094	107	good	
72	3922	-	-	6172	110	good	
108	3925	-	-	6172	110	good	
144	3930	-	-	6172	110	good	
180	3934	-	-	6055	105	good	
216	3936	-	-	5822	97	good	
252	3935	-	-	5703	93	good	
288	3938	-	-	5273	80	good	
300	3938			5273	80	good	79.92929939

Table A.12: F-T data for sample 30-13

30-13							
30% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3900	20	9.7	5898			
36	3936	-	-	6211	111	good	
72	3937	-	-	6211	111	good	
108	3938	-	-	6211	111	good	
144	3945	-	-	6211	111	good	
180	3948	-	-	6172	110	good	
216	3951	-	-	5898	100	good	
252	3952	-	-	5547	88	good	
288	3953	-	-	5391	84	good	
300	3954			5391	84	good	83.54666668

Table A.13: F-T data for sample 30-9

30-9							
30% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3885	19.95	9.65	5938			
36	3923	-	-	6211	109.4063859	good	
72	3924	-	-	6133	93.74205226	good	
108	3929	-	-	6133	93.74205226	good	
144	3933	-	-	6133	93.74205226	good	
180	3936	-	-	6133	93.74205226	good	
216	3933	-	-	6172	92.5611095	good	
252	3934	-	-	6172	92.5611095	good	
288	3938	-	-	6094	94.94574067	good	
300	3938			6094	94.94574067	good	94.94574067

Table A.14: F-T data for sample 30-12

30-12							
30% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3877	19.99	9.65	5820			
36	3911	-	-	6133	111	good	
72	3914	-	-	6250	115	good	
108	3916	-	-	6211	114	good	
144	3910	-	-	6250	115	good	
180	3926	-	-	6172	112	good	
216	3928	-	-	6055	108	good	
252	3928	-	-	5859	101	good	
288	3930	-	-	5625	93	good	
300	3930			5625	93	good	93.41122861

Table A.15: F-T data for sample 30-11

30-11							
30% RAP					**dynamic**		**durability**
Cycles	**Mass (g)**	**Length (cm)**	**Diameter (cm)**	**Frequency (Hz)**	**modulus**		**factor**
0	3914	20	9.73	6055			
36	3949	-	-	6133	103	good	
72	3950	-	-	6250	107	good	
108	3946	-	-	6211	105	good	
144	3959	-	-	6289	108	good	
180	3962	-	-	6094	101	good	
216	3966	-	-	5898	95	good	
252	3968	-	-	5469	82	good	
288	3968	-	-	5391	79	good	
300	3968			5391	79	good	79.27027571

Table A.16: F-T data for sample 35-10

35-10							
35% RAP					**dynamic**		**durability**
Cycles	**Mass (g)**	**Length (cm)**	**Diameter (cm)**	**Frequency (Hz)**	**modulus**		**factor**
0	3881	20.06	9.7	6094			
36	3894	-	-	6094	100	good	
72	3915	-	-	6172	103	good	
108	3916	-	-	6133	101	good	
144	3924	-	-	6094	100	good	
180	3926	-	-	6172	103	good	
216	3929	-	-	6133	101	good	
252	3930	-	-	6055	99	good	
288	3932	-	-	5938	95	good	
300	3932			5938	95	good	94.94574067

Table A.17: F-T data for sample 35-14

35-14							
35% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3890	21	9.63	5859			
36	3902	-	-	6133	110	good	
72	3929	-	-	6172	111	good	
108	3929	-	-	6133	110	good	
144	3936	-	-	6133	110	good	
180	3939	-	-	6172	111	good	
216	3942	-	-	6094	108	good	
252	3940	-	-	6055	107	good	
288	3945	-	-	5820	99	good	
300	3945			5820	99	good	98.6731456

Table A.18: F-T data for sample 35-3

35-3							
35% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3910	20.15	9.61	5898			
36	3949	-	-	6016	104	good	
72	3951	-	-	6133	92	good	
108	3950	-	-	6016	96	good	
144	3957	-	-	6094	94	good	
180	3960	-	-	6133	92	good	
216	3962	-	-	6094	94	good	
252	3964	-	-	5977	97	good	
288	3965	-	-	6055	95	good	
300	3965			6055	95	good	94.88143436

Table A.19: F-T data for sample 35-12

35-12							
35% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3901	20	9.7	6133			
36	3935	-	-	6250	104	good	
72	3936	-	-	6289	105	good	
108	3941	-	-	6289	105	good	
144	3945	-	-	6289	105	good	
180	3947	-	-	6289	105	good	
216	3950	-	-	6250	104	good	
252	3951	-	-	6094	99	good	
288	3953	-	-	5781	89	good	
300	3953			5781	89	good	88.85052726

Table A.20: F-T data for sample 35-7or9

35-7or9							
35% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3879	19.91	9.63	6055			
36	3913	-	-	6055	100	good	
72	3914	-	-	6055	100	good	
108	3919	-	-	6055	100	good	
144	3921	-	-	6094	101	good	
180	3925	-	-	6094	101	good	
216	3926	-	-	6172	104	good	
252	3926	-	-	6211	105	good	
288	3927	-	-	6094	101	good	
300	3928			6055	100	good	100

Table A.21: F-T data for sample 40-14

40-14							
40% RAP					**dynamic**		**durability**
Cycles	**Mass (g)**	**Length (cm)**	**Diameter (cm)**	**Frequency (Hz)**	**modulus**		**factor**
0	3857	19.83	9.69	6094			
36	3893	-	-	6094	100	good	
72	3893	-	-	6289	107	good	
108	3898	-	-	6289	107	good	
144	3902	-	-	6133	101	good	
180	3906	-	-	6289	107	good	
216	3908	-	-	6055	99	good	
252	3909	-	-	5859	92	good	
288	3911	-	-	6094	100	good	
300	3911			6094	100	good	100

Table A.22: F-T data for sample 40-10

40-10							
40% RAP					**dynamic**		**durability**
Cycles	**Mass (g)**	**Length (cm)**	**Diameter (cm)**	**Frequency (Hz)**	**modulus**		**factor**
0	3865	20.05	9.694	5938			
36	3898	-	-	6055	104	good	
72	3899	-	-	6133	107	good	
108	3903	-	-	6094	105	good	
144	3905	-	-	6055	104	good	
180	3910	-	-	6055	104	good	
216	3913	-	-	6094	105	good	
252	3912	-	-	6016	103	good	
288	3917	-	-	5703	92	good	
300	3917			5703	92	good	92.24149999

Table A.23: F-T data for sample 40-9

40-9							
40% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3816	19.86	9.69	5898			
36	3858	-	-	5898	100	good	
72	3859	-	-	5742	95	good	
108	3864	-	-	5703	93	good	
144	3869	-	-	5859	99	good	
180	3873	-	-	5625	91	good	
216	3872	-	-	5742	95	good	
252	3873	-	-	5742	95	good	
288	3874	-	-	5547	88	good	
300	3874			5547	88	good	88.45182445

Table A.24: F-T data for sample 40-1

40-1							
40% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3840	19.7	9.69	6211			
36	3883	-	-	6211	100	good	
72	3884	-	-	6211	100	good	
108	3889	-	-	6172	99	good	
144	3894	-	-	6250	101	good	
180	3897	-	-	6211	100	good	
216	3898	-	-	6172	99	good	
252	3899	-	-	6133	98	good	
288	3900	-	-	6055	95	good	
300	3901			6055	95	good	95.03973933

Table A.25: F-T data for sample 40-7

40-7							
40% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3870	19.95	9.68	6094			
36	3899	-	-	6094	100	good	
72	3899	-	-	6094	100	good	
108	3905	-	-	6094	100	good	
144	3909	-	-	6094	100	good	
180	3912	-	-	6055	99	good	
216	3914	-	-	6055	99	good	
252	3913	-	-	6094	100	good	
288	3917	-	-	5859	92	good	
300	3918			5859	92	good	92.4362027

Table A.26: F-T data for sample 45-15

45-15							
45% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3921	20.1	9.7	5898			
36	3960	-	-	6133	108	good	
72	3959	-	-	6133	108	good	
108	3967	-	-	6172	110	good	
144	3970	-	-	6211	111	good	
180	3974	-	-	6133	108	good	
216	3976	-	-	5938	101	good	
252	3976	-	-	5742	95	good	
288	3979	-	-	5391	84	good	
300	3979			5391	84	good	83.54666668

Table A.27: F-T data for sample 45-2

45-2							
45% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3866	20.03	9.7	5703			
36	3902	-	-	5898	107	good	
72	3902	-	-	5938	108	good	
108	3908	-	-	5938	108	good	
144	3912	-	-	5977	110	good	
180	3914	-	-	5898	107	good	
216	3919	-	-	5508	93	good	
252	3919	-	-	4961	76	good	
288	3921	-	-	4961	76	good	
300	3921			4961	76	good	75.67138989

Table A.28: F-T data for sample 45-7

45-7							
45% RAP					modulus		factor
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)			
0	3830	19.9	9.68	5938			
36	3864	-	-	6055	104	good	
72	3864	-	-	6094	105	good	
108	3864	-	-	6094	105	good	
144	3874	-	-	6055	104	good	
180	3878	-	-	5977	101	good	
216	3882	-	-	5469	85	good	
252	3879	-	-	4961	70	good	
288	3886	-	-	4609	60	good	
300	3886			4609	60	good	60.24666757

Table A.29: F-T data for sample 45-6

45-6							
45% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3887	20.47	9.7	5703			
36	3917	-	-	5898	107	good	
72	3918	-	-	5938	108	good	
108	3926	-	-	5977	110	good	
144	3930	-	-	5977	110	good	
180	3930	-	-	5820	104	good	
216	3933	-	-	5508	93	good	
252	3931	-	-	5273	85	good	
288	3934	-	-	5078	79	good	
300	3935			5078	79	good	79.28273982

Table A.30: F-T data for sample 45-11

45-11							
45% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3885	20	9.68	5898			
36	3923	-	-	6055	105	good	
72	3925	-	-	6133	108	good	
108	3926	-	-	6016	104	good	
144	3934	-	-	6094	107	good	
180	3939	-	-	5898	100	good	
216	3941	-	-	5625	91	good	
252	3940	-	-	4922	70	good	
288	3944	-	-	4727	64	good	
300	3944			4727	64	good	64.23351204

Table A.31: F-T data for sample 50-15

50-15							
50% RAP					dynamic modulus		durability factor
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)			
0	3812	19.84	9.66	6133			
36	3845	-	-	6133	100	good	
72	3846	-	-	6016	96	good	
108	3844	-	-	6055	97	good	
144	3856	-	-	6016	96	good	
180	3862	-	-	5977	95	good	
216	3866	-	-	5195	72	good	
252	3865	-	-	4844	62	good	
288	3869	-	-	4609	56	good	
300	3869			4609	56	good	56.4764626

Table A.32: F-T data for sample 50-2

50-2							
50% RAP					dynamic modulus		durability factor
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)			
0	3846	19.9	9.7	6016			
36	3877	-	-	6016	100	good	
72	3878	-	-	6016	100	good	
108	3888	-	-	5977	99	good	
144	3892	-	-	5898	96	good	
180	3895	-	-	5703	90	good	
216	3897	-	-	5156	73	good	
252	3896	-	-	4922	67	good	
288	3898	-	-	4766	63	good	
300	3899			4766	63	good	62.76137083

Table A.33: F-T data for sample 50-12

50-12							
50% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3829	19.61	9.68	6055			
36	3860	-	-	6055	100	good	
72	3861	-	-	6094	100	good	
108	3869	-	-	6094	101	good	
144	3872	-	-	6055	101	good	
180	3878	-	-	5898	100	good	
216	3881	-	-	5691	95	good	
252	3881	-	-	4844	88	good	
288	3884	-	-	4531	64	good	
300	3885			4609	56	good	55.99636418

Table A.34: F-T data for sample 50-4

50-4							
50% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3827	19.92	9.68	5703			
36	3858	-	-	5977	110	good	
72	3859	-	-	5898	107	good	
108	3869	-	-	5898	107	good	
144	3872	-	-	5977	110	good	
180	3876	-	-	5938	108	good	
216	3878	-	-	5586	96	good	
252	3878	-	-	5156	82	good	
288	3880	-	-	4922	74	good	
300	3880			4883	73	good	73.31058843

Table A.35: F-T data for sample 50-3

50-3							
50% RAP					dynamic		durability
Cycles	Mass (g)	Length (cm)	Diameter (cm)	Frequency (Hz)	modulus		factor
0	3845	20.14	9.69	5703			
36	3877	-	-	5820	104	good	
72	3878	-	-	5859	106	good	
108	3887	-	-	5859	106	good	
144	3891	-	-	5898	107	good	
180	3891	-	-	5859	106	good	
216	3898	-	-	5625	97	good	
252	3899	-	-	5430	91	good	
288	3901	-	-	5195	83	good	
300	3902			5195	83	good	82.97826705

A.2 Freeze-Thaw Loading Data

Table A.36: F-T Loading data for 25% RAP

25%

Sample	8	13	11	7	6
Initial Mass(g)	3871	3929	3892	3914	3904
Final Mass (g)	3893	3950	3922	3904	3920
Mass loss (g)	-22	-21	-30	10	-16
D1 (in)	4.003	4.016	4.004	3.999	4
D2 (in)	4.006	4.012	4.006	4.001	4.025
D3 (in)	3.993	3.999	3.998	4.001	4.011
Average (in)	4.000667	4.009	4.002667	4.000333	4.012
Area (in^2)	12.57056	12.62298	12.58313	12.56847	12.64188
Load (lb)	68190	61390	59500	62790	54170
PSI	5424.579	4863.351	4728.553	4995.837	4284.963

Table A.37: F-T Loading data for 30% RAP

30%					
Sample	**13**	**14**	**12**	**9**	**11**
Initial Mass(g)	3900	3889	3877	3885	3914
Final Mass (g)	3922	3906	3900	3908	3934
Mass loss (g)	-22	-17	-23	-23	-20
D1 (in)	4.001	4.007	4.016	4.005	4.005
D2 (in)	4.01	4.021	4.019	4.009	4.008
D3 (in)	4.019	3.995	4.028	3.983	4.037
Average (in)	4.01	4.007667	4.021	3.999	4.016667
Area (in^2)	12.62928	12.61459	12.69866	12.56009	12.67131
Load (lb)	53930	54970	45940	53990	63650
PSI	4270.235	4357.653	3617.703	4298.537	5023.159

Table A.38: F-T Loading data for 35%

35%					
Sample	**7or9**	**12**	**14**	**3**	**10**
Initial Mass(g)	3879	3901	3890	3910	3881
Final Mass (g)	3897	3921	3914	3934	3899
Mass loss (g)	-18	-20	-24	-24	-18
D1 (in)	4.011	4.013	4.015	4.006	4.011
D2 (in)	4.007	4.023	4.016	4.022	4.016
D3 (in)	4.002	4.022	3.993	4.019	4.014
Average (in)	4.006667	4.019333	4.008	4.015667	4.013667
Area (in^2)	12.60829	12.68814	12.61669	12.665	12.65239
Load (lb)	42610	65470	56900	60260	66970
PSI	3379.522	5159.937	4509.9	4757.994	5293.072

Table A.39: F-T loading data for 40%

40%					
Sample	**7**	**1**	**9**	**10**	**14**
Initial Mass(g)	3870	3840	3816	3865	3857
Final Mass (g)	3886	3871	3840	3886	3880
Mass loss (g)	-16	-31	-24	-21	-23
D1 (in)	4.008	4.002	4.007	3.998	4.004
D2 (in)	4.009	4.009	3.997	3.996	4.014
D3 (in)	4.006	4.006	4.006	3.984	4.014
Average (in)	4.007667	4.005667	4.003333	3.992667	4.010667
Area (in^2)	12.61459	12.602	12.58732	12.52034	12.63348
Load (lb)	41660	63120	48240	54620	49480
PSI	3302.526	5008.729	3832.427	4362.503	3916.577

Table A.40: F-T Loading data for 45%

45%					
Sample	**11**	**6**	**7**	**2**	**15**
Initial Mass(g)	3885	3887	3830	3866	3921
Final Mass (g)	3911	3900	3849	3886	3948
Mass loss (g)	-26	-13	-19	-20	-27
D1 (in)	4.003	3.975	4.018	4.013	4.004
D2 (in)	3.995	3.995	4.007	4.016	3.997
D3 (in)	4	4.012	4.015	4.009	4.007
Average (in)	3.999333	3.994	4.013333	4.012667	4.002667
Area (in^2)	12.56218	12.5287	12.65029	12.64608	12.58313
Load (lb)	48570	33880	48000	39740	52900
PSI	3866.366	2704.191	3794.381	3142.475	4204.041

Table A.41: F-T loading data for 50%

50%					
Sample	**3**	**15**	**2**	**12**	**4**
Initial Mass(g)	3845	3812	3846	3829	3827
Final Mass (g)	3867	3831	3863	3848	3846
Mass loss (g)	-22	-19	-17	-19	-19
D1 (in)	4.022	3.986	4.016	4.006	3.985
D2 (in)	4.022	3.997	4.016	4.009	3.996
D3 (in)	4.024	4.005	4.004	4.009	4.006
Average (in)	4.022667	3.996	4.012	4.008	3.995667
Area (in^2)	12.70919	12.54125	12.64188	12.61669	12.53916
Load (lb)	42490	36220	44690	43320	39630
PSI	3343.249	2888.069	3535.075	3433.548	3160.499

Table A.42: F-T loading data for Control

Control					
Sample	**15**	**8**	**3**	**7**	**14**
Initial Mass(g)	3890	3941	3946	3926	3910
Final Mass (g)	3911	3960	3969	3949	3942
Mass loss (g)	-21	-19	-23	-23	-32
D1 (in)	4	3.981	4.041	4.009	4.012
D2 (in)	4.001	3.992	4.014	4.006	4.007
D3 (in)	4.014	4.011	4.002	3.992	4.004
Average (in)	4.005	3.994667	4.019	4.002333	4.007667
Area (in^2)	12.59781	12.53288	12.68603	12.58104	12.61459
Load (lb)	87680	52340	69670	59850	82560
PSI	6959.942	4176.214	5491.866	4757.16	6544.804

Appendix B
Bond Strength Data

B.1 Bond Strength

Appendix B shows the data for the bond strength test.

Table B.1: Load for push through test 30% RAP

30%	Load (lb)
1	11410
2	12500
3	11760
Average	11890

Table B.2: Load for push through test 40% RAP

40%	Load (lb)
1	9680
2	11140
3	14110
Average	11643.33

Table B.3: Load for push through test 50% RAP

50%	Load (lb)
1	11750
2	11150
3	12360
Average	11753.33

Table B.4: Load for push through test control WYDOT mix

Control	Load (lb)
1	19700
2	19630
3	16610
4	22440
Average	19595

Appendix C
Coefficient of Thermal Expansion

C.1 Coefficient of Thermal Expansion

Appendix C shows the data for the coefficient of thermal expansion (CTE) test.

Table C.1: CTE data for 25%RAP

25% RAP				
	25-8	25-11	25-13	
deg C	displ.	displ.	displ.	
10	0	0	0	
20	0.001	0	0	
30	0.001	0	0.001	
40	0.001	0	0.001	
50	0.002	0.001	0.003	
40	0.002	0.002	0.003	
30	0.002	0.002	0.003	
20	0.001	0.001	0.001	
10	0.001	0	0.001	
L(cm)	19.82	19.98	19.93	
L(in)	7.80315	7.866142	7.846456693	
ΔL	0.002	0.001	0.003	
10-50	6.41E-06	3.18E-06	9.55845E-06	6.38E-06
ΔL	0.001	0.002	0.002	
50-10	3.2E-06	6.36E-06	6.3723E-06	5.31E-06

Table C.2: CTE data for 30% RAP

30% RAP				
	30-9	30-13	30-14	
deg C	displ.	Displ.	Displ.	
10	0	0	0	
20	0	0	0	
30	0	0	0	
40	0.001	0	0.001	
50	0.002	0.001	0.002	
40	0.002	0.002	0.002	
30	0.002	0.002	0.001	
20	0.001	0.001	0.001	
10	0	0.001	0	
L(cm)	19.95	20.03	20.05	
L(in)	7.854331	7.885826772	7.893701	
ΔL	0.002	0.001	0.002	
10-50	6.37E-06	3.17024E-06	6.33E-06	5.29E-06
ΔL	0.002	0.001	0.002	
50-10	6.37E-06	3.17024E-06	6.33E-06	5.29E-06

Table C.3: CTE data for 35% RAP

35% RAP				
	35-3	35-10	35-14	
deg C	displ.	displ.	displ.	
10	0	0	0	
20	0	0	0.001	
30	0.001	0.001	0.001	
40	0.001	0.002	0.001	
50	0.003	0.003	0.002	
40	0.003	0.003	0.002	
30	0.002	0.002	0.002	
20	0.001	0.002	0.001	
10	0.001	0	0	
L(cm)	20.15	20.06	20.07	
L(in)	7.933070866	7.897638	7.901575	
ΔL	0.003	0.003	0.002	
10-50	9.45409E-06	9.5E-06	6.33E-06	8.42615E-06
ΔL	0.002	0.003	0.002	
50-10	6.30273E-06	9.5E-06	6.33E-06	7.3757E-06

Table C.4: CTE data for 40% RAP

40% RAP				
	40-9	40-10	40-14	
deg C	displ.	displ.	displ.	
10	0	0	0	
20	0.001	0	0	
30	0.001	0	0.001	
40	0.001	0.001	0.001	
50	0.002	0.001	0.003	
40	0.002	0.002	0.003	
30	0.002	0.001	0.002	
20	0.001	0.001	0.001	
10	0	0	0	
L(cm)	19.86	20.05	19.83	
L(in)	7.818898	7.893701	7.807087	
ΔL	0.002	0.001	0.003	
10-50	6.39E-06	3.17E-06	9.61E-06	6.39E-06
ΔL	0.002	0.002	0.003	

Table C.5: CTE data for 45% RAP

45% RAP				
	45-2	45-7	45-15	
deg C	displ.	displ.	displ.	
10	0	0	0	
20	0.001	0	0	
30	0.001	0	0.001	
40	0.001	0	0.001	
50	0.002	0.001	0.002	
40	0.002	0.001	0.002	
30	0.002	0.001	0.002	
20	0.002	0	0.001	
10	0	0	0	
L(cm)	20.03	19.9	20.1	
L(in)	7.885827	7.834646	7.913386	
ΔL	0.002	0.001	0.002	
10-50	6.34E-06	3.19E-06	6.32E-06	5.28328E-06
ΔL	0.002	0.001	0.002	
50-10	6.34E-06	3.19E-06	6.32E-06	5.28328E-06

Table C.6: CTE data for 50% RAP

50% RAP				
	50-2	50-12	50-15	
deg C	displ.	displ.	displ.	
10	0	0	0	
20	0	0	0	
30	0	0	0.001	
40	0	0.001	0.001	
50	0.001	0.003	0.002	
40	0.001	0.002	0.002	
30	0.001	0.002	0.002	
20	0	0.001	0.001	
10	0	0	0	
L(cm)	19.9	19.61	19.84	
L(in)	7.834646	7.720472	7.811023622	
ΔL	0.001	0.003	0.002	
10-50	3.19E-06	9.71E-06	6.40121E-06	6.44E-06
ΔL	0.001	0.003	0.002	
50-10	3.19E-06	9.71E-06	6.40121E-06	6.44E-06

Table C.7: CTE data for control mix

Control				
	C-3	C-8	C-15	
deg C	displ.	displ.	displ.	
10	0	0	0	
20	0	0	0	
30	0.001	0	0	
40	0.001	0.001	0.001	
50	0.001	0.002	0.002	
40	0.001	0.002	0.002	
30	0.001	0.002	0.001	
20	0	0.002	0.001	
10	0	0.001	0	
L(cm)	19.95	19.79	19.81	
L(in)	7.854331	7.791339	7.799213	
ΔL	0.001	0.002	0.002	
10-50	3.18E-06	6.42E-06	6.41E-06	5.34E-06
ΔL	0.001	0.001	0.002	
50-10	3.18E-06	3.21E-06	6.41E-06	4.27E-06

Appendix D
Modulus of Elasticity

D.1 Modulus of Elasticity

Appendix D show the data for modulus of elasticity MOE

Table D.1 MOE data for 25% RAP

25%							57000*f'^.5	33*w^1.5*f'^.5
	Sample	mass (g)	Length (m)	Diameter (m)	wc	E (E-meter) Gpa	E (eq)	E (eq)
7	2	3891	0.2001	0.0968	165	39.8	4781092.796	5085143
Load	80090	lb	0.1996	0.0967		5772501.962		
f'c	7035.657	psi	0.2004	0.0966	3.807079			
			0.200	0.097	11.38344357			
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
5	5	3889	0.1989	0.0967		39.9	5417054.071	5761547
Load	102601	lb	0.1989	0.0966		5787005.735		
f'c	9031.848	psi	0.1988	0.0965	3.803142			
			0.199	0.097	11.35991191			
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
6	1	3874	0.2004	0.0966		37.7	4808300.998	5114081
Load	81060	lb	0.1999	0.0969		5467922.712		
f'c	7115.961	psi	0.2002	0.0967	3.808391333			
			0.200	0.097	11.39129287			

Table D.2 MOE data for 30% RAP

30%							57000*f'^.5	33*w^1.5*f'^.5
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
10	1	3959	0.2031	0.0969		37.6	4041003	4297988
Load	57530	lb	0.2044	0.097		5453418.939		
f'c	5026.072	psi	0.2047	0.097	3.817577667			
			0.204	0.097	11.4463137			
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
9	3	3917	0.2022	0.0968		38.5	4803211	5108667
Load	81000	lb	0.203	0.0969		5583952.903		
f'c	7100.903	psi	0.202	0.0967	3.811016			
			0.202	0.097	11.40699958			
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
8	2	3928	0.2021	0.0969		35.8	4555180	4844863
Load	72650	lb	0.2033	0.0967		5192351.011		
f'c	6386.478	psi	0.2029	0.0964	3.805766667			
			0.203	0.097	11.37559698			

Table D.3 MOE data for 35% RAP

35%							57000*f'^.5	33*w^1.5*f'^.5
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
11	4	3894	0.1986	0.0967		37.8	4547336	4836520
Load	72400	lb	0.1996	0.0967		5482426.486		
f'c	6364.501	psi	0.1995	0.0966	3.805766667			
			0.199	0.097	11.37559698			
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
12	3	3883	0.1996	0.0969		36.8	4543270	4832195
Load	72520	lb	0.1994	0.0969		5337388.748		
f'c	6353.124	psi	0.199	0.0967	3.812328333			
			0.199	0.097	11.41485699			
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
13	1	3898	0.21	0.0966		40	4535446	4823874
Load	72370	lb	0.21	0.0971		5801509.509		
f'c	6331.262	psi	0.2102	0.097	3.814953			
			0.210	0.097	11.43057993			

Table D.4 MOE data for 40% RAP

40%							57000*f'^.5	33*w^1.5*f'^.5
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
15	1	3844	0.1982	0.0969		34.1	4282275	4554603
Load	64250	lb	0.1976	0.0967		4945786.857		
f'c	5644.162	psi	0.1974	0.0967	3.807079			
			0.198	0.097	11.38344357			
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
14	3	3895	0.2006	0.00968		36.2	4440545	4722938
Load	69230	lb	0.2	0.0968		5250366.106		
f'c	6069.081	psi	0.2003	0.0969	3.811016			
			0.200	0.097	11.40699958			
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
16	2	3879	0.2204	0.0967		39.5	3967155	4219443
Load	55180	lb	0.21	0.0967		5728990.64		
f'c	4844.051	psi	0.2006	0.0968	3.808391333			
			0.210	0.097	11.39129287			

Table D.5 MOE data for 45% RAP

45%							57000*f'^.5	33*w^1.5*f'^.5
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter)	E (eq)	E (eq)
17	4	3834	0.1995	0.0967		34.5	4257081	4527807
Load	63540	lb	0.1967	0.0968		5003801.95		
f'c	5577.945	psi	0.1974	0.0967	3.808391			
			0.198	0.097	11.39129			
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter)	E (eq)	E (eq)
	1	3802	0.196	0.0967		34.8	4134903	4397859
Load	59780	lb	0.1974	0.0967		5047313.27		
f'c	5262.365	psi	0.1955	0.0964	3.803142			
			0.196	0.097	11.35991			
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter)	E (eq)	E (eq)
18	3	3887	0.1996	0.0966		38.5	4113718	4375326
Load	58680	lb	0.21	0.096		5583952.9		
f'c	5208.579	psi	0.2007	0.096	3.787394			
			0.203	0.096	11.26603			

Table D.6 MOE data for 50% RAP

50%							57000*f'^.5	33*w^1.5*f'^.5
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
20	4	3835	0.202	0.0971		33.3	3925270	4174895
Load	54170	lb	0.199	0.097		4829756.666		
f'c	4742.304	psi	0.2	0.0965	3.813641			
			0.200	0.097	11.42272			
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
21	5	3841	0.2	0.0968		34.5	4060615	4318846
Load	57970	lb	0.205	0.0969		5003801.952		
f'c	5074.975	psi	0.202	0.0969	3.813641			
			0.202	0.097	11.42272			
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
22	2	3799	0.1973	0.0968		34.5	3980913	4234076
Load	55640	lb	0.1982	0.0968		5003801.952		
f'c	4877.707	psi	0.1984	0.0968	3.811016			
			0.198	0.097	11.407			

Table D.7 MOE data for Control mix

Control							57000*f'^.5	33*w^1.5*f'^.5
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
	3	3955	0.1997	0.0967		46.6	6214362	6609560
Load	134840	lb	0.1981	0.0968		6758758.578		
f'c	11886.21	psi	0.1984	0.0961	3.800517333			
			0.199	0.097	11.34423767			
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
	1	3886	0.1958	0.0967		45.7	6236415	6633015
Load	136550	lb	0.1958	0.0969		6628224.614		
f'c	11970.72	psi	0.1956	0.0968	3.811016			
			0.196	0.097	11.40699958			
	Sample	mass (g)	Length (m)	Diameter (m)		E (E-meter) Gpa	E (eq)	E (eq)
	4	3970	0.2012	0.0967		46.1	6305718	6706725
Load	137590	lb	0.1999	0.096		6686239.709		
f'c	12238.25	psi	0.2006	0.0956	3.783457			
			0.201	0.096	11.24261882			