PERFORMANCE OF CEMENT TREATED RECYCLED AGGREGATES UNDER WETTING-DRYING CYCLES IN PAVEMENT BASE

By

SAIF BIN SALAH

Presented to the Faculty of the Graduate School of

The University of Texas at Arlington in Partial Fulfillment

of the Requirements

for the Degree of

MASTER OF SCIENCE IN CIVIL ENGINEERING

THE UNIVERSITY OF TEXAS AT ARLINGTON

APRIL 2017

Acknowledgements

First, I would like to express my deepest gratitude to my supervisor Dr. Sahadat Hossain, for his valuable time, guidance, encouragement, help and unconditional support throughout my graduate studies. Without his constant guidance and support, this dissertation would not have been completed.

I would like to give my special thanks to Dr. Laureano R. Hoyos and Dr. Xinbao Yu for their precious time, valuable suggestions and participation as my committee member.

Special thanks extended to Dr. Mohammad Faysal, Dr. Sonia Samir and Dr. Sadik Khan for their active cooperation and assistance in all stages of work. I would also like to thank my dearest friends Sangeeta Bhattacharjee, Nur Basit Zaman, MD Ashrafuzzaman Khan and Umme Zakira for their worthy friendship and the good times.

I wish to acknowledge the cooperation unconditional support from my sister Tania Salahuddin and brother Shateel Bin Salah throughout my graduate studies.

Finally, and most of all, I would like to dedicate this dissertation to my parents for all their love, encouragement, and great support. It is the best thing in my life to be a part of their family.

April 14, 2017

Abstract

PERFORMANCE OF CEMENT TREATED RECYCLED AGGREGATES UNDER WETTING-DRYING CYCLES IN PAVEMENT BASE

Saif Bin Salah

The University of Texas at Arlington, 2017

Supervising Professor: MD. Sahadat Hossain

Resilient Modulus (M_R) is one of the most important stiffness parameter to determine the thickness of a pavement layer (AASHTO 2003 pavement design guideline). Recent studies conducted by researchers on the variability of M_R with wetting-drying (WD) and freeze-thaw (FT) cycles show that long-term durability is an important criterion to be considered in designing pavement base with recycled materials. At present, the design procedure does not consider the effect of deterioration of pavement layers due to seasonal variations and limited studies have been conducted to evaluate the effect of environmental deterioration on the reclaimed asphalt pavement (RAP) and recycled crushed concrete aggregate (RCCA) mixtures stabilized with cement. The objective of this study is to evaluate the long-term durability of RCCA and RAP mix materials under repeated wetting-drying cycles. Three different combinations of RCCA and RAP materials containing 0%, 30% and 50% RAP content were used in this study. Each of these combinations were then stabilized with 4% and 6% cement content. After curing for 7 days the samples prepared from these material combinations were subjected to 4, 8, 16 and 30 wetting-drying (W-D) cycles. For the purpose of comparison, a different set of samples of the same combinations were prepared, cured for 7 days, and then further cured for 15, 25, 40 and 70 days. Resilient modulus tests were then conducted on all the samples at the end of these specified W-D cycles and curing periods. Environmental tests were also conducted to assess the effect of WD cycles on the washed-out water quality. These tests included total

suspended solids (TSS), total dissolved solids (TDS), turbidity, chemical oxygen demand (COD) and pH. M_R test results indicated that addition of 50% RAP into the mix reduced the resilient modulus (M_R) by about 39%. Increasing the cement content from 4% to 6% increased the M_R values by about 20-35% for all material combinations. Higher cement content also resulted in higher durability of the materials containing 30% and 50% of RAP. All the six material combinations used in this study showed adequate strength after 7-days of curing. But the mix containing 30% RAP + 70% RCCA 4% cement (30R_4C) and 50% RAP + 50% RCCA 4% cement (50R_4C) failed to meet the minimum layer coefficient value of 0.13 (AASHTO 2003) for pavement base layer after 8-16 wetting-drying cycles. Results obtained from the environmental tests after 30 WD cycles were found to be within the permissible values provided by EPA guidelines.

Table of Contents

List of Illustrations

List of Tables

Chapter 1

Introduction

1.1 Background and Problem Statement

Reclaimed asphalt pavement (RAP) and recycled crushed concrete aggregate (RCCA) are waste materials very abundantly produced owing to the demolition and rehabilitation projects carried out all over the United States. According to a report from the National Asphalt Pavement Association (Hansen and Copeland, 2015), the amount of RAP produced in 2014 was 71.9 million tons. The amount of RCCA generated from the construction and demolition is expected to be 123 million tons per year (USDOT, 2004). These non-biodegradable materials when discharged to landfills can pose significant threat to the environment.

Reusing RCCA as raw material for rip-rap, soil stabilization, pipe bedding and even for landscaping is now in practice in some of the states. The Texas Department of Transportation (TxDOT) has already started using RCCA as flex-base material for pavement base layer construction (Faysal et al. 2016*). The reclaimed asphalt pavement (RAP) materials on the other hand is being mainly reused in hot-mix and cold-mix processes (NAPA 2013). The average national usage rate of RAP in hot-mix asphalt (HMA) was only 12% on 2007 (Khosla and Visintine, 2011). There were only 10 state transportation departments which used 29% of RAP in intermediate layers. Still huge quantities of RAP material remains unused all over the United States, especially in Texas. Use of RAP as base course material would provide a viable cost effective alternative of utilizing this huge portion of unused RAP. This potential use of RAP was felt in early 90's and since then mechanical properties of RAP has been investigating extensively (Kolias 1996). Kolias (1996) investigated the compressive strength, tensile strength and modulus of elasticity of different RAP mixes with unbound granular materials and recommended further research on RAP mixes if it is stabilized with cement. Later on, a substantial amount of research on mechanical properties of different cement treated RAP mixes were reported in various studies (Taha et al. 2002, Guthrie et al. 2007*, and Grilli et al. 2013). Hoyos et al. (2011) investigated the influence of fiber inclusion to the mix and evaluated different engineering properties such as hydraulic conductivity, leachate and shear modulus. Research on fracture resistance and rutting potential (Research Report, FDOT, May 2007), resilient modulus response (Puppala et at. 2011), splitting tensile strength (Brand 2012), field evaluation (Nazarian et al. 1996), flexural strength and unrestrained shrinkage (Khay et al. 2014), dynamic modulus (Jones et. al. 2014) can be mentioned as the most recent works on RAP materials.

A combination of RAP and RCCA materials might be a viable option for use in pavement base construction. This issue was addressed in a study by Faysal et. al. (2016*) where different combinations of RAP and RCCA materials stabilized with cement were tested. It was concluded in this study that a maximum of 50% RAP if combined with RCCA and stabilized with 4-6% cement meets the AASHTO strength requirements for pavement base.

1.2 Problem Statement

All of the studies conducted on RAP and RCCA materials evaluated the short-term strength characteristics of these materials. But pavement engineers are nowadays more concerned about the long-term performance of pavement structures that is significantly affected by seasonal variations (Khoury and Zaman, 2007). Wetting and drying (WD) actions induced by these seasonal variations result in episodic moisture movement in various pavement layers. Moisture variation in pavements causes pavement distresses like edge cracking, edge drop and longitudinal cracking (Hedayati and Hossain, 2015* and Hossain et al. 2016*). This indicates possible changes in engineering properties that are associated with the pavement materials. Miller et al. (2003), Tao and Zhang (2006) and Puppala et. al. (2017) studied the durability of recycled aggregates under wetting-drying cycles. These researchers used unconfined compressive strength (UCS) as an indicator of durability which does not simulate actual field-loading conditions (Khoury and Zaman, 2007). Khoury and Zaman (2007) thus evaluated the effect of WD cycles on resilient modulus of virgin aggregates stabilized by lean stabilizers. Regression models have also been developed to predict resilient modulus values from aggregate properties and WD cycles (Maalouf et al. 2012). This research area is however not fully explored and additional studies are still needed (Little et al. 2005).

In this research work, RAP and RCCA materials were mixed at different proportions and stabilized with cement. These material combinations were then subjected to repeated wetting-drying (W-D) cycles. An experimental program was designed and executed to study the long-term performance of these materials under wetting-drying conditions.

1.3 Objective and Scope

The objective of this study is to evaluate the long-term durability of RCCA and RAP mix materials in terms of structural competency and environmental soundness when subjected to repeated wetting and drying cycles in flexible pavement base constructions. The following tasks have been performed to carry out the present research:

1. To collect available literature on strength and durability of RAP and RCCA materials.
2. To conduct basic engineering tests such as particle-size distribution and optimum moisture content (OMC) tests for material characterization.
3. To perform durability tests on the RAP-RCCA materials combinations which includes wetting-drying studies for 30 cycles.

4. To conduct resilient modulus (M_R) tests at various stages of the durability studies to observe the change in strength due to wetting-drying cycles.
5. To compare results from different material combinations and study the effect of RAP and cement content on the long-term durability of the materials.
6. To evaluate the environmental effects of using recycled materials as pavement base layer.

1.4 Thesis Organization

The thesis manuscript has been divided into six chapters:

Chapter 1 provides a background, problem statement, and objective and scope of this study.

Chapter 2 presents the literature review on previous studies conducted on recycled materials, available design guidelines, and conducted environmental tests. It also provides an insight on durability studies conducted recycles materials and their limitations.

Chapter 3 describes the experimental program, several test procedures such as optimum moisture content (OMC), maximum dry density (MDD), wetting-drying (W-D) processes, resilient modulus (M_R) and different types of environmental tests.

Chapter 4 presents test results, analysis and discussions on results.

Chapter 5 provides the summary and conclusion of current study and also includes future recommendation.

Chapter 2

Literature Review

2.1 Introduction

Now-a-days the use of recycled materials has become very popular in pavement construction. These materials are treated with cement and fibers to improve their performance, longevity, engineering properties, and cost effective at the same time. This chapter gives overview about the recycled base materials, pavement design criteria, different model recommended to be used to determine the strength parameters of pavement materials. The literature reviewed in this chapter was collected from different journals, design guidelines, and other research projects. At first a brief description about recycled pavement materials, and pavement structures will be depicted. Then pavement design methods will be explained briefly. After that cement treated base materials characteristics and properties will be reviewed. Next, different factors that affect the strength parameters of base materials will be described in brief. The recommended models suggested by different guidelines and research works to determine the value of strength parameters will be introduced which will be followed by different correlations between unconfined compressive strength and structural coefficient and also between unconfined compressive strength and resilient modulus of materials.

2.2 Recycled Apshalt Pavement (RAP)

The annual production of new asphalt pavement material in USA was approximately 500 million tons in 2007 which includes 40 million tons of recycled asphalt pavement material. The removed or reprocessed pavement material which contains asphalt and aggregate is called Recycled Asphalt Pavement (RAP) materials. According to Environmental Protection Agency (EPA), 80% of the total removed pavement materials are recycled each year. The recycling rate of reclaimed materials is higher than aluminum cans, plastic bottles and glass bottles. Approximately 100 million tons of asphalt pavement material was recycled (FHWA, 2011). The rate is even higher than the recycling rate of industrial waste products.

2.2.1 Use of RAP in USA

According to the survey conducted by North Carolina Department of Transportation (NCDOT) in 2007, majority of State transportation departments allow the use of RAP in HMA mixtures. The average national usage rate was 12% on 2007. There were only 10 state transportation departments which has used as high as 29% of RAP in intermediate layer. Although 35 state transportation departments could use 29% RAP in intermediate layer (Figure 2.1). Another survey conducted by Materials Engineering and Research Office of the Ministry of Transportation of Ontario, Canada, (MTO) on the US states found that for base and binder courses RAP percentage ranged from 20%- 50% for medium to low traffic roadways. Tests were conducted to evaluate the environmental soundness of these materials.

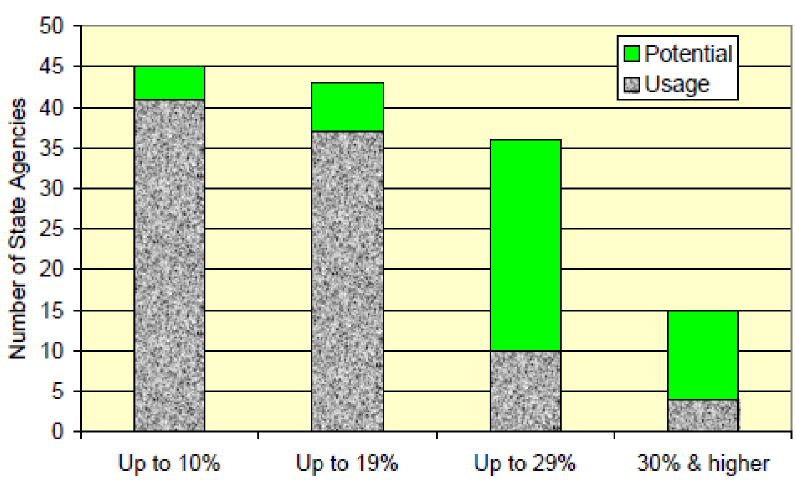


Figure 2.1 Usage and potential of various RAP percentages in the intermediate layer (NCDOT 2007)

2.2.2 Properties of RAP

In the following table, the physical and mechanical properties of the RAP are indicated. The typical unit weight of RAP ranges from 120 to 140 lb/ft^3 and the moisture content varies from 5 to 8%. Typically RAP material contains about 3 to 7% of hardened asphalt content. The ignition oven method specified in AASHTO T 308 was used to determine the asphalt content in 15 states department of transportation and solvent extraction method was used by the 9 states department of transportation. Hardening of asphalt content might have occurred because of oxidation, thixotropic effect etc. California Bearing Ratio (CBR) ranges from 20 to 25 (Table 2.1).

Table 2.1 Properties of RAP materials (Potturi, 2006)

Property	**Typical Range**
Unit Weight	120 to 140 pcf
Moisture Content	5 to 8%
Asphalt Content	3 to 7%
Asphalt Penetration	10 to 80 at 25^{o}C
Absolute Viscosity	4000 to 25000 poise at 60^{o}C
Compacted Unit Weight	100 to 125 pcf
California Bearing Ratio (CBR)	20 to 25% for 100% RAP

2.3 Recycled Crushed Concrete Aggregate (RCCA)

2.3.1 Use of RCCA in the USA.

The construction of buildings, bridges, and roadways continues to increase in the twenty-first century, especially in areas with ever-growing populations. Existing structures and highways require repair or replacement as they reach the end of their service life or simply no longer satisfy their intended purpose due to the growing population. As modern construction continues, two pressing issues will become more apparent to societies: an increasing demand for construction materials, especially concrete and asphalt aggregates, and an increasing production of construction and demolition waste. Already, the Federal Highway Administration (FHWA) estimates that two billion tons of new aggregate are produced each year in the United States. This demand is anticipated to increase to two and a half billion tons each year by 2020. With such a high demand for new aggregates, the concern arises of the depletion of the current sources of natural aggregates and the availability of new sources. Similarly, the construction waste produced in the United States is expected to increase. From building demolition alone, the annual production of construction waste is estimated to be 123 million tons (FHWA 2012). Currently, this waste is most commonly disposed of in landfills. To address both the concern of increasing demand for new aggregates and increasing production of waste, many states have begun to recognize that a more sustainable solution exists in recycling waste concrete for use as aggregate in new concrete, or recycled concrete aggregates (RCA). The solution helps address the question of how to sustain modern construction demands for aggregates as well as helps to reduce the amount of waste that enters already over-burdened landfills. Many states have begun to implement recycled concrete aggregates in some ways in new construction. As shown in Figure 1.1 from the FHWA, most states have recognized the many uses of RCA as a raw material, such as for rip-rap, soil stabilization, pipe bedding, and even landscape materials. As shown in Figure 1.2, many states have gone a step further in integrating RCA into roadway systems for use as aggregate course base material. However, as shown in Figure 1.3, only a small number of states have begun using RCA in Portland cement concrete for pavement construction. As shown in these figures, the state of Missouri does not currently integrate RCA in any function (FHWA). Currently, there are no accepted standards or guidelines in the United States for utilizing RCA in structural concrete.

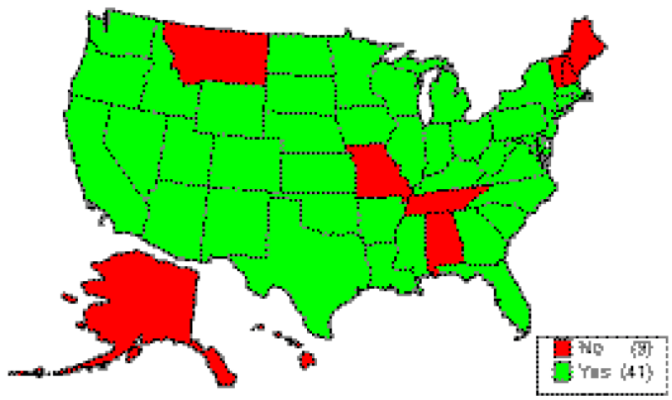


Figure 2.2 States using RCA as Aggregate (FHWA 2012)

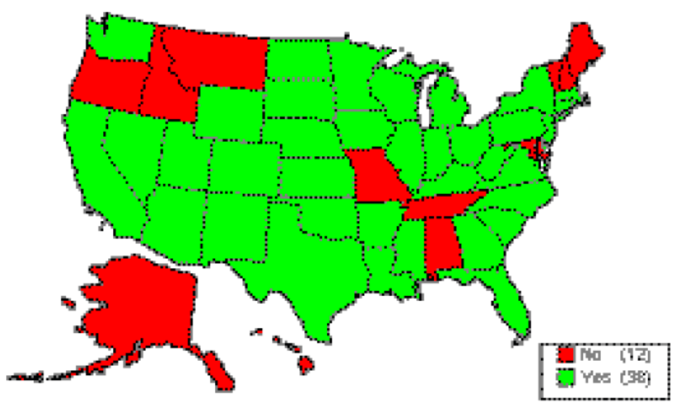


Figure 2.3 States using RCA as Base Aggregate (FHWA 2012)

2.3.2 Concerns regarding use of RCCA

RCAs are composed of both the original, or virgin, aggregate, as well as mortar which remains adhered to the surface of the aggregate. In the production of RCA, the removal of all this residual mortar would prove costly and detrimental to the integrity of the virgin aggregates within the concrete. Therefore, residual mortar is inevitable. Research has shown that this residual mortar causes high water absorption, low density, low specific gravity, and high porosity in RCAs compared to natural aggregates (Kou and Poon, 2012). These effects in the recycled aggregate can decrease hardened concrete properties of RAC. According to Fathifazl et al. (2008), the amount of residual mortar on the RCA can significantly affect the mechanical and durability properties of RAC. To reduce the negative impacts of this residual mortar, new mix design methods such as the equivalent mortar volume method can be used. Due to the variety of sources of RCA and the various functions, environment, and wear of the concrete structures and pavements from which the RCA can be obtained, characterizing this aggregate can be very difficult. Controlled studies must be performed to account for each of these variables on a regional basis, such as for each state's department of transportation, so that the aggregates within the area can be adequately characterized.

2.4 Pavement Structure

A typical pavement structure consists of several layers of different materials which receives load from the upper layer and distribute them to the lower layers. The purpose of upper layers is to reduce the stress level to the subgrade. Classification of pavement is done using its load distribution pattern. There are three types of pavements such as rigid pavement, flexible pavement and composite pavement. Flexible pavement generally consists of prepared or stabilized subgrade, base or sub-base course and surface course. Flexible pavement has higher deflection at the edges and lower deflection at center. On the other hand, rigid pavement consists of a prepared subgrade, base or sub-base course and a pavement slab. Pavement slab is usually a concrete slab which settles uniformly under loading. Composite pavement is a combination of both rigid pavement and flexible pavement. Rigid section is overlain by flexible pavement includes hot mix asphalt (HMA), open graded friction course or rubberized asphalt (Potturi, 2006). This flexible overlay works a thermal and moisture blanket and reduces the deflection and wearing of the rigid pavement layer.

2.4.1 Surface Course

It is the top layer of the pavement which is constructed on the base course and stays in contact with the traffic wheel load. For this reason, it has to resist the high traffic load, rutting, provide drainage control and also a smooth riding surface.

2.4.2 Base Course

This is the layer above the sub-base course if there is any, otherwise directly on the subgrade and immediate below the surface course to provide structural support. This layer consists of crushed virgin aggregate, crushed limestone, recycled crushed concrete aggregate and recycled asphalt pavement (RAP) treated with Portland cement, lime or other binder materials. Base material has to be selected in accordance with the specification. Using the recycle material, for base with treatment, will reduce the cost significantly by decreasing the thickness of the layer. It is necessary to study and find the optimum cement content to get the desired performance of the base layer.

2.4.3 Sub-Base Course

This layer is usually beneath the base layer to support the surface and base course. It consists of compacted layer of granular material with or without treatment of stabilizer. It prevents the fines from the subgrade to move into the base layer. The material quality of subgrade is usually lower than the base layer as it requires less strength. If the strength of the base layer is high enough to sustain under the wheel load then sub-base layer is neglected for economy.

As the stress, induced by the wheel load, reduces with depth especially in flexible pavement top layer is usually expensive and stronger layer than the materials of the bottom layers. While designing a pavement it is important to consider the load induced by the traffic and type of materials to be used to ensure the most economic and sustainable design. A typical cross section of pavement structure is shown in Figure 2.4.

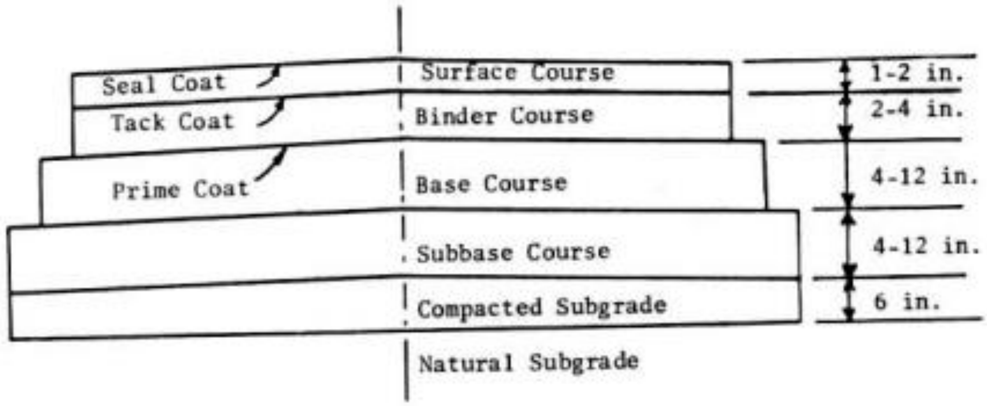


Figure 2.4 Typical pavement structure (Ordonez, 2006)

2.5 Pavement Design Criteria

Major component of pavement design is the thickness of the pavement layer. To determine the thickness of the pavement layer the criteria that involves are:

2.5.1 Imparted Load on Pavement

Equivalent single axle load (ESAL) is used to estimate the imposed load on the pavement using a fourth power formula. The concept of ESAL is developed by American Association of State Highway and Transportation Officials (AASHTO). The ESAL reference axle load is 18 kip single axle with two tires and is typically varies with the types of the trucks. The amount of traffic predicted over a design or analysis period and then converted into equivalent number of 18 kip single axle loads and totaled over the design period. Consider an 18-wheeler with tow tandem axles and one single axle exerts ESAL equivalent to 2.44. Different trucks have different wheel load condition which can be found in any pavement design guide book.

2.5.2 Strength and Stiffness of Subgrade

One of the most important parameters in pavement design is the strength and resilient values of the subgrade soil. In past, CBR, R-value, soil support value (SSV) and Triaxial strength parameter were used as pavement design parameter. These parameters mostly simulate the static load condition and the failure load does not represent the actual dynamic traffic load condition of the real life pavement. Soil failure does not occur in the field on a regular basis which is usually done in the laboratory test. Considering those factors, AASHTO 2003 recommended using resilient modulus (M_r) of soil or subgrade and base materials. The M_r value represents the dynamic modulus of soil and also considers the plastic deformation of the soil.

The parameters required for the design of a pavement structure are design variables, performance criteria, material properties, structural characteristics and reinforcement variables. Design variables are performance period, traffic, reliability and environmental effects. Performance criteria include serviceability criteria, allowable rutting, aggregate loss etc. Structural characteristics are known as drainage load transfer, and detachment between the pavement surface and subgrade. Material properties are resilient modulus, effective subgrade modulus, modulus of rupture of Portland Cement Composites (PCC). Reinforcement variables include different types of joints in concrete slab of rigid pavements.

2.6 Design Considerations for RAP and RCCA Materials

Current design guidelines are developed based only on the strength rather than the long time performance of the pavement. As a result, transportation department of different states using higher cement content to achieve high strength values. This high strength of relatively stiff cement treated aggregate base layers may guarantee the strength and resilient modulus but not necessarily the long term pavement performance (Guthrie, 2007*). Roadways which contain base layers treated with high cement content are subjected to rutting, shrinkage cracks, fatigue crack and transverse cracks which may not cause structural deficiency but allows water to penetrate inside the pavement layers and reduce the quality of the pavement. Tensile cracking occurs at the bottom of the pavement layers and rutting is the result of the accumulation of the pavement deformation. In recent studies, these problems such as rutting, fatigue crack etc. had been addressed by using fiber reinforcement with RAP material (Potturi, 2006). Fiber reinforced cement treated base material has improved tensile strength which reduce the propagation of cracks and reduce the associated cracking in the pavement surface layer.

2.6.1 Cement Treated RAP and RCCA

Recycled Asphalt pavement (RAP) consists of asphalt and aggregate which is generated by cold milling of the removed hot mix asphalt (HMA) pavement. Usually, it is used as a replacement of the aggregate base course and processed to meet the requirements of the specific gradation. Recycled Crushed Concrete Aggregates (RCCA) is produced by crushing of concrete to meet the specific particle size requirement. Its properties are different from the aggregate as cement is attached on the surface of the natural aggregate. Both RAP and RCCA caught the interest of the researchers as these could be a cost saving alternative to the virgin aggregate. RAP and RCCA materials must meet the minimum design criteria provided by the AASHTO guidelines and state transportation departments. Addition of cement to the base materials improves the strength and stiffness. But this higher value of stiffness i.e. resilient modulus does not ensure the proper performance and durability of the pavements against problems such as rutting and cracking.

2.6.2 Resilient Modulus and Permanent Deformation

The two important parameters to determine the pavement performance are resilient modulus and pavement deformation or rutting. The most common way to determine these properties are repeated load triaxial test according to the guideline of AASHTO T 307-99.

Resilient modulus defined as the ratio of the repeated deviator axial stress to the resilient or recoverable strain which can be expressed as:

$$Mr = \frac{\sigma_d}{\varepsilon_r}$$

Here, Mr = resilient modulus,

σd = repeated deviator stress ($\sigma 1$ –$\sigma 3$), and

εr = recoverable or resilient axial strain in the direction of principal stress

Permanent Deformation is usually characterized by assuming that the permanent strain is proportional to the resilient strain (Huang, 2007). It is expressed as:

εp (N) = μ εr N-α

Where, εp (N) = plastic or permanent strain due to single load application such as the Nth application,

εr resilient or recoverable strain at the 200th repetition,

N = Number of load applications,

μ and α = permanent deformation parameters.

According to AASHTO pavement design guidelines (1993), the value of resilient modulus Mr should be used for material characterization. It recommends the use of correlation between structural coefficients and resilient modulus. In few studies, it was found that the results obtained from different laboratory tests for modulus were different from the back calculated moduli. This might have occurred due to the cracks in the pavement structure (Lekarp et al., 2000).

The stress dependency of base materials is usually determined by using the K-θ model which is frequently used in pavement design. The nonlinear characteristics of pavement materials are described by K-θ model. The model is expressed as:

$Mr = K_1\theta^{k_2}$

Where,

K_1 and K_2 =material constants and

θ = bulk stress = $(\sigma_1 + \sigma_2 + \sigma_3) = (3\,\sigma_3 + \sigma_d)$

One of the limitations of this model is that it does not predict the volumetric strain. An improved Mr model was suggested by Uzan (1985) which is as follows:

$Mr = K_3\theta^{K_4}{\sigma_d}^{k_5}$

Where,

K_3, K_4, K_5 are material constants evaluated by a multiple regression analysis from set of repeated load Mr test and σ_d is the deviator stress.

This model is recommended by Mechanistic Empirical Pavement Design Guide (MEPDG), octahedral stress (τ_d) substitutes the deviator stress (σ_d) and the model is expressed as:

$Mr = K_6\ Pa \left[\frac{\theta}{P_a}\right]^{K_7} \left[\frac{\tau_{oct}}{P_a}\right]^{K_8}$

Where,

K_6, K_7, and K_8 are material constants.

P_a = atmospheric pressure e.g. 14.7 psi (normalizing stress)

This model is recommended by MEPDG to calculate k values which will be used as analysis input. The variable octahedral shear stress τ_{oct} is expressed as:

$\tau_{oct} = \frac{1}{9}\left[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2\right]$

If the tests are performed under isotropic confining pressure, the equation above can be simplified as:

$$\tau_{oct} = \frac{\sqrt{2}}{3}\sigma_d$$

When, $\sigma_2=\sigma_3$ and $\sigma_d = \sigma_1 - \sigma_3$

The typical stiffness values ranges from 100 to 300 MPa, based on the type of the granular materials, are used in some design methods (Lekarp et al., 2000). There are two other relationships between K_1 and K_2 suggested by (Chen et al., 1995) which are expressed as:

$\log K_1 = 4.7308 - 2.5179\ K_2$, (AASHTO T294-92I), and

$\log K_1 = 4.19 - 1.7304\ K_2$, (AASHTO T294-92I)

Rada and Witczak (1981) reported that the relationship between K_1-K_2 varies for different materials. They investigated the possibility of developing the equation of Mr from the physical properties of the material. They used six types of aggregates. Each of these was blended at different gradations. Each gradation was compacted at three different compaction methods to establish moisture-density relationship. It is found out that there is a possible relationship between physical properties and the Mr values of the materials. The largest variation was observed for crushed stone. But the mean values for all granular materials is K_1=9240 and K_2 = 0.52. There is a relationship same as Chen et al. (1995) which is semi-logarithmic and can be expressed as:

$\log K_1 = 4.66 - 1.82\ K_2$

Another findings of their study was that the effect of saturation on K_1 is significant than K_2. The value of K_1 and moduli reduces with increasing moisture content.

The value of bulk stress (θ), degree of saturation (S_r), and maximum dry density are major parameters to influence resilient modulus.

2.7 Resilient Modulus of Treated RAP and RCCA Materials

In another study by Taha et al., (2002), compaction level and unconfined compression strength of the mixtures of RAP and virgin aggregates were determined at different cement content. The laboratory test results of UCS were used to determine the value of resilient modulus. They used a correlation between UCS and Mr to get the value of resilient modulus. Based on the equation given in the AASHTO 1993, the values of the structural coefficient were determined using the obtained resilient modulus values.

Table 2.2 Suggested layer coefficients for existing flexible pavement layer materials (ASSHTO, 1993)

Material	Surface Condition	Coefficient
AC Surface	Little or no alligator cracking and/or only low-severity transverse cracking	0.35 - 0.40
	<10% low-severity alligator cracking and/or <5% medium- and high-severity transverse cracking	0.25 - 0.35
	>10% low-severity alligator cracking and/or <10 percent medium-severity alligator cracking and/or >5-10% medium- and high-severity transverse cracking	0.20 - 0.30
	>10% medium-severity alligator cracking and/or <10% high-severity alligator cracking and/or >10% medium- and high-severity transverse cracking	0.14 - 0.20
	>10% high-severity alligator cracking and/or >10% high-severity transverse cracking	0.08 - 0.15
Stabilized Base	Little or no alligator cracking and/or only low-severity transverse cracking	0.20 - 0.35
	<10% low-severity alligator cracking and/or <5% medium- and high-severity transverse cracking	0.15 - 0.25
	>10% low-severity alligator cracking and/or <10% medium-severity alligator cracking and/or >5-10% medium- and high-severity transverse cracking	0.15 - 0.20
	>10% medium-severity alligator cracking and/or <10% high-severity alligator cracking and/or >10% medium- and high-severity transverse cracking	0.10 - 0.20
	>10% high-severity alligator cracking and/or >10% high-severity transverse cracking	0.08 - 0.15
Granular Base/Subbase	No evidence of pumping, degradation, or contamination by fines	0.10 - 0.14
	Some evidence of pumping, degradation, or contamination by fines	0.00 - 0.10

In a separate study by Gnanendran and Woodburn (2003), resilient modulus, CBR and UCS tests were conducted on cement, lime and fly ash stabilized RAP materials. The resilient moduli, strength and CBR values increase with increasing amount of cement content or for each chemical treatment.

New Hampshire Department of Transportation (NHDOT), Janoo et. al. (1994) conducted experiment on RAP materials collected from the selected test section by NHDOT at Concord off Interstate 89. The resilient modulus of different layers were then determined from the Falling Weight Deflectometer (FWD) and other tests and subsequently used in the AASHTO 1993 design guideline for design of pavement. The results of the studies discussed above are summarized in the Table 2..

Table 2.3 Results summary of structural layer coefficients obtained from different studies

Reference	**Type of Recycled Material Tested**	**Tests Conducted**	**Stress levels**	**Resilient Modulus**	**a_2**
Lofti and Witczak	Cement-Treated Dense Graded Aggregate, which included Limestone	Resilient Modulus (M_r)	0.28 to 2.28 MPa of bulk stress	1260 MPa (4.5% cement)	0.27
Janoo (1994)	Reclaimed Stabilized Base	Back Calculation from Layer Modulus (FWD)	N/A	N/A	0.15-0.19
Janoo (1994)	Reclaimed Stabilized Base	CBR	N/A	N/A	0.13

Taha et al. (2002)	Cement Stabilized RAP Aggregates	Unconfined Compressive Strength Tests	N/A	3726 MPa (7% cement)	0.13
Gnanendran and Woodburn (2003)	Cement Stabilized RAP Aggregates	Resilient Modulus (M_r), CBR and UCS tests	0 to 140 kPa	310 to 590 MPa (0% to 3% cement)	N/A

Potturi (2006) investigated the effect of stabilization on the resilient modulus of RAP base materials and covered the designs of both rigid and flexible pavements. RAP materials used in this experiment met the requirement of TxDOT design guidelines. AASHTO T307-99 guideline was followed to perform the resilient modulus testing of RAP specimens. For the repeatability and reliability of the test each specimen was tested in identical condition and three identical specimens were tested to determine standard deviation and coefficients of variation. Tests were done on RAP materials with different cement content such as 0, 2, 4 & 6%. The standard deviation ranges from 1.8 to 5.2 MPa for untreated aggregate while 4.7 to 30 MPa for cement treated aggregate materials. The results are presented in the following figures:

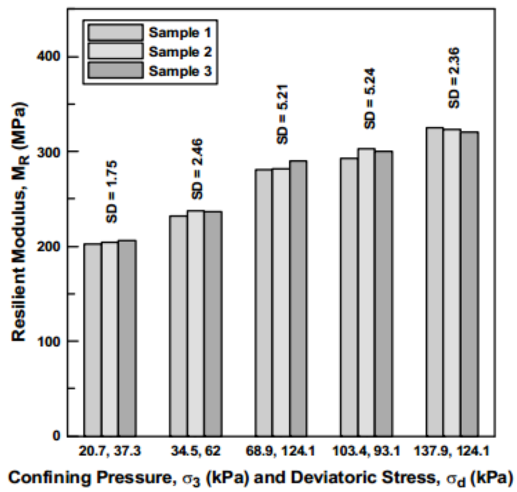


Figure 2.2 Repeatability of resilient modulus test results of untreated aggregates (Potturi, 2006)

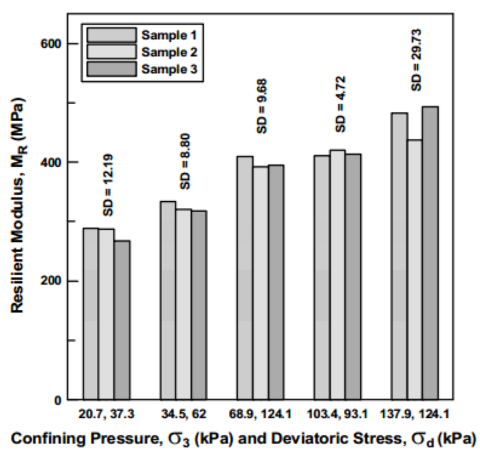


Figure 2.3 Repeatability of resilient modulus test results of cement treated aggregates (Potturi, 2006)

The increase in the value of M_r with increase in deviator stresses but the increment rate is moderate for higher confining stresses. This might have occurred because of the initial stiffening of the specimen under higher confinements and prevented additional stiffening of the specimen under higher deviator stresses. It might also be explained as in higher confinements the specimen is much stronger and it does not respond to the deviator stress. The resilient modulus increased with an increase in cement content such as for a confining pressure of 137.9 kPa, the cement content increased from 0 to 2% the value of Mr increased by 32%.

The value of resilient modulus (M_r) determined from the test were used to determine the value of structural coefficients a_2, from the following the AASHTO 2003 equation,

$a_2 = 0.249 \times \log M_r - 0.977$

Where, a_2 = Structural layer coefficient, and M_r = Resilient modulus (psi)

According to Janoo et. al. (1994), the value of structural coefficient a_2 ranged from 0.13 to 0.24 which showed an increasing rate with cement content and confining pressure. In the study of Potturi (2006), the structural coefficient ranged from 0.13 to 0.22.

Faysal et. al. (2016*) unconfined compressive strength tests on different combinations of RCCA (Grade 2) and RAP materials at cement contents varying from 0% to 6% at 2% intervals. Layer coefficients for the cement stabilized aggregates were determined to be used in the design of flexible pavements in accordance with the AASHTO design code.

Table 2.4 Resilient modulus of untreated and cement treated aggregates (Potturi, 2006)

Notation	Confining Pressure (kPa)	Average Resilient Moduli (MPa)	Structural Coefficient, a_2
Untreated	20.7	199	0.13
	34.7	235	0.15
	68.9	274	0.17
	103.4	300	0.18
	134.7	321	0.19
Treated	20.7	231	0.15
	34.7	265	0.16
	68.9	332	0.19
	103.4	360	0.20
	134.7	400	0.21
4% Cement Treated	20.7	247	0.16
	34.7	282	0.17
	68.9	360	0.20
	103.4	377	0.20
	134.7	430	0.22

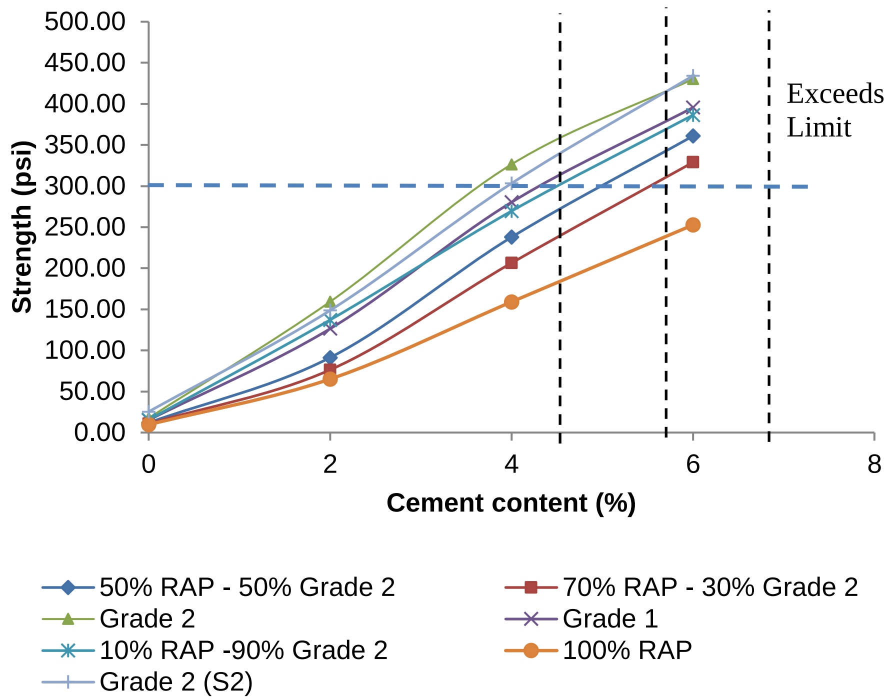


Figure 2.7 Unconfined compressive strength of different combination in untreated and treated condition. (Faysal et. al. 2016*)

Table 2.5 Determination of average structural coefficient (a_2) using the resilient modulus test results at 6% cement content.

Combination	Cement Content	Avg. RM (MPa)	a_2
100% Grade 2	4	196.07	0.13
	6	219.46	0.14
10% RAP+ 90% Gr 2	4	169.11	0.12
	6	237.32	0.15
50% RAP + 50% Gr 2	4	167.24	0.11
	6	218.67	0.15
70% RAP + 30% Gr 2	4	150.75	0.11
	6	220.40	0.14
100% RAP	4	144.3	0.10
	6	194.70	0.13

In this study it was concluded that with the inclusion of RAP content, strength decreases. The 50% RAP + 50% Gr 2 combination met the unconfined compressive strength requirement of 1947 kPa (300 psi) at a cement dosage of 5% and more. At 6% cement content, however, all of the combinations of Grade 2 and RAP materials except for the 100% RAP material met the minimum UCS requirements. In terms of structural layer coefficient (a_2), addition of 4% cement content does not meet the minimum AASHTO requirement of 0.13 for all combinations containing RAP materials. For the 50% RAP + 50% Gr 2 combination, this requirement is met at 6% cement content.

2.8 Durability Studies on RAP

The durability studies on reclaimed asphalt pavement include wetting and drying or freezing or thawing studies and leachate studies. The number of studies performed on the durability of recycled pavement is very rare and a few studies that are available in the literature will be discussed in this section. The most challenging issue for any stabilization technique is its durability or permanency of stabilization. If the leaching of the chemical stabilizer occurs through moisture movements in the base layer it will reflect in serious implications for durability and sustainability of the pavement. One form of moisture conditioning effects on chemically-treated soils is related to moisture fluctuations from seasonal changes and their impact on the performance of these soils. This aspect is often studied in soil stabilization projects as a part of the durability studies (Chittoori, 2008). The commonly used test for durability studies is ASTM D 559 or ASTM D 560 which measures the resistance to 12 cycles of wetting and drying or 12 cycles of freezing and thawing. In recent years many researchers have begun to use non- abrasion type (ASTM C 593-95) of durability studies which uses Vacuum Saturation Equipment to test the durability of stabilizer for strength (Imran et., al. 1999).

Puppala et. al. (2017) conducted durability studies on cement stabilized RAP materials. The specimens were subjected to durability testing as per the procedure outlined by ASTM D 559 method. According to the ASTM D 559 method, the specimens should be prepared and cured then submerged in water for 5 hours for wetting cycle and then oven dried at 160ºF for 48 hours for drying cycle. The test was continued until 14 wet-dry cycles were completed or until the sample failed. The samples are subjected to UCS tests after 0, 3, 7 and 14 cycles of wetting/drying studies and at the end of 14 leachate cycles. Leachate tests were conducted on several identically prepared and cured specimens. Leachate was collected after 3, 5, 7, 11, and 14 cycles of leaching.

Table 2.6 Summary of durability performance of different RAP mixtures (Puppala et. al. 2017)

Mix Type	# of cycles	Total Volumetric Change (%)	Retained UCS (psi)	# of cycles sample survived	Rank
100R_6C	3	0.64	200	14	IV
	7	0.30	185		
	14	0.49	145		
75R_2C	3	0.90	150	14	II
	7	0.89	135		
	14	0.48	120		
60R_0C	3	1.41	153	14	V
	7	0.81	145		
	14	0.79	123		
60R_2C	3	0.54	300	14	I
	7	0.56	285		
	14	0.09	272		
60R_7F	3	1.65	45	7	VI
	7	2.50	35		
	14	-	-		
50R_2C	3	0.36	260	14	III
	7	1.27	223		
	14	0.94	204		

Many state agencies like TXDOT have reported problems regarding disappearing of stabilizers from the base layers after certain years. Most of the research done in the past report that these durability studies are not due to abrasion of the pavement but rather because of chemical reversal of the stabilization process. In most of the cases the reversal of stabilizers is associated with moisture absorption into the stabilized materials. Capillary rise of water in stabilized surface is highly detrimental and can induce secondary reactions (McCallister and Petry, 1990). Due to the metastable nature of many of the mineral phases in chemical stabilization the water movement makes the alkali and alkali earth metals to reach out and there by decreases the strength of the stabilized layer.

Another important objective of the stabilization technique is to address the permanency of chemical stabilizer, i.e. the ability of the chemical additive to hold the recycled asphalt pavement for longer time period. Leaching of a chemical stabilizer through moisture movements will have serious implications on the durability and sustainability of the chemical treatment. One of the detrimental effects that a chemically treated soil may experience is the loss of the chemical stabilizer through leaching. Previous studies report that the leaching through moisture flows in subgrade soils result in variations of pH and Calcium and Magnesium ratios, which can influence the permanency of the chemical modifiers (McCallister, 1990). Studies addressing leaching of chemical stabilizer for recycled asphalt pavement (RAP) materials have not been researched till now.

Moisture absorption property of RAP material has significant impact on its strength and stiffness properties. It has been reported in literature that crushed or milled RAP can absorb a considerable amount of water if exposed to rain. Moisture contents up to 5 percent or higher have been observed for RAP stored in a stock pile (Smith, 1980). According to Decker (1999) during periods of extensive precipitation, the moisture content of some RAP stockpiles increase from 7 to 8 percent.

The most challenging issue for any stabilization technique is its durability or permanency of stabilization. If the leaching of the chemical stabilizer occurs through moisture movements in the base layer it will reflect in serious implications for durability and sustainability of the pavement. This aspect is often studied in soil stabilization projects as a part of the durability studies (Chittoori 2008; Pedarla et. al. 2011; Chittoori et. al. 2013). The commonly used test for durability studies is ASTM D 559 or ASTM D 560 which measures the resistance to 12 cycles of wetting and drying or 12 cycles of freezing and thawing. In most of the cases the reversal of stabilizers is associated with moisture absorption into the stabilized materials.

Studies conducted by Taha (2003), Potturi et. al. (2007) and Gutherie et. al. (2007) were only based on strength and stiffness properties of the stabilized RAP mixes. However, achievement of the specified strength and stiffness does not always ensure durability of these stabilized mixes. In order to accomplish this task, the long-term performance of the chemically treated RAP-base blended mixes was studied in this research. Treated RAP mixtures are subjected to several strength, stiffness and durability tests thereby conducting a comprehensive study on different mix designs.

2.9 Durability Studies on Other Recycled Aggregates

Khoury and Zaman (2002) investigated the effect of wet-dry cycles in low quality aggregates. Cylindrical specimens were cured for 3 and 28 days and subjected to different W-D cycles. Resilient modulus and unconfined compressive strength were evaluated. The resilient modulus values for 28 day cured specimens increased as W-D cycles increased up to 12, beyond which a reduction was observed. For 3 day cured specimens the resilient modulus increased with number of cycles. The resilient modulus values for 28 day cured specimens subjected to 30 cycles were approximately 5% lower than those not exposed to W-D conditions. On the other hand, for the 3 day cured specimens there was an increase of 55% in resilient modulus values compared to non-exposed specimens. Also, it was found that 12 to 30 W-D cycles could be considered adequate to have a noticeable negative effect on 28 day cured specimens; however, more than 30 cycles were needed for the 3 day cured specimens. Thus, the positive effect of curing time was more dominant on 3 day curing period and the detrimental effect of W-D cycles was more influential on the 28-day curing period.

(Hoyos, 2005) Here sulphate-rich expansive clay samples from southeast Arlington, Texas were4 stabilized with sulphate-resistant type V cement, low-calcium class F fly ash and quick lime were subjected to 0, 1, 2, 4, 8, 16 and 32 W-D cycles. UCS tests were conducted on the samples as shown in

The UCS of 5% treated soil shows an overall decreasing trend similar to those reported by Santoni et. al. (2002) on cement treated sand and Nunan and Humphrey (1990) on cement treated base aggregates.

Overall UCS of 10% stabilized soil shows an increasing trend with w-d cycles. This is indicative of the continuous bonding (pozzolanic reactions) taking place during w-d cycles. However the final moisture tends to increase with w-d cycles which indicates possible increased pore space within the treated soil after the cementation and agglomeration reactions. (Hoyos et al. 2005)

Santoni et. al. (2002) evaluated the effect of cement stabilization on silty-sand material under wet and dry conditions. The UCS of all the wet specimens were found to be about 67% lower than the dry specimens except for the specimens stabilized with 9% cement.

Table 2.7 Unconfined compressive strength (UCS) from UC tests (Hoyos et. al. 2011)

Table 33.3 Unconfined compressive strength (UCS) from UC tests

Number of w-d cycles	*UCS (psi)*	*UCS (kPa)*	*% Change*	*Initial w (%)*	*Final w (%)*	*Final pH (−)*	*Final pH (−)*
Natural soil							
0	26.5	182.9	0.0	21.0	21.0	7.4	7.4
1	16.9	116.6	−36.2	21.0	27.3	7.1	7.1
2	14.6	100.5	−45.1	21.0	28.2	7.6	7.0
4	12.1	83.5	−54.3	21.0	27.5	7.4	6.8
8	11.2	77.3	−57.7	21.0	26.9	7.1	6.6
16	10.7	73.6	−59.8	21.0	28.6	7.8	6.0
32	9.3	64.3	−64.8	21.0	26.2	7.5	5.6
5% Type V cement treated soil							
0	145.6	1004.6	0.0	35.0	34.3	9.9	9.6
1	82.4	568.5	−43.4	35.0	31.8	9.9	9.3
2	62.6	431.6	−57.0	35.0	34.9	10.1	9.6
4	59.3	409.4	−59.2	35.0	35.9	10.2	8.6
8	57.7	398.0	−60.4	35.0	31.8	9.9	8.4
16	48.8	336.8	−66.5	35.0	30.4	10.0	9.3
32	90.7	625.6	−37.7	35.0	36.8	10.3	8.3
10% Type V cement treated soil							
0	252.5	1742.3	0.0	29.0	29.5	10.1	10.0
1	174.2	1201.7	−31.0	29.0	30.8	10.1	9.7
2	198.3	1368.5	−21.5	29.0	30.2	10.1	9.5
4	210.7	1454.1	−16.5	29.0	28.8	10.0	9.4
8	277.7	1915.8	+10.0	29.0	32.3	10.1	9.2
16	279.3	1927.3	+10.6	29.0	33.2	10.0	8.7
32	310.6	2143.0	+23.0	29.0	36.1	10.0	9.4

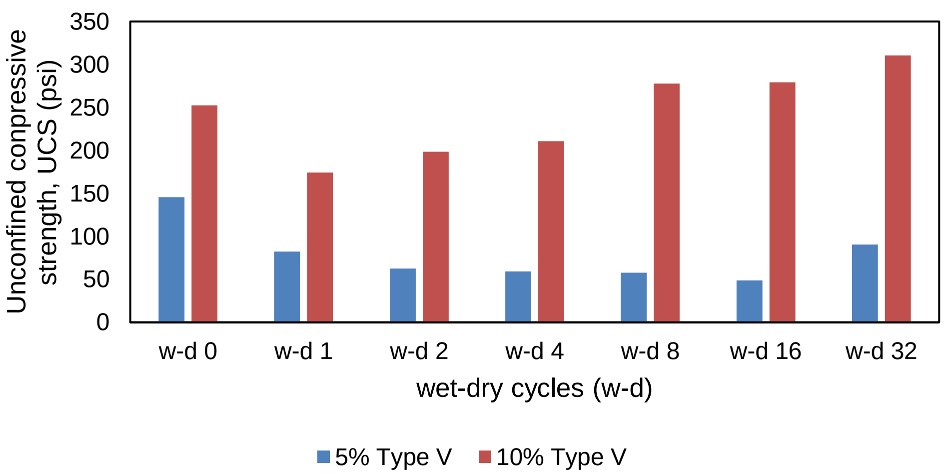


Figure 2.8 Variation of unconfined compressive strength (UCS) with wet-dry cycles (w-d)(Hoyos et. al. 2005).

Chapter 3

Methodology

3.1 Introduction

The experimental program was developed and conducted to determine the structural competency and environmental soundness of cement-treated recycled base materials under wetting-drying condition. Wetting-drying (W-D) for upto 30 cycles were applied. Resilient modulus (M_R) was determined for each of the material combinations after specific W-D cycles. pH, Total Suspended Solids (TSS), Total Dissolved Solids (TDS), Turbidity and Chemical Oxygen Demand (COD) were conducted to evaluate the environmental impacts of these materials. Different test methods, specifications, testing equipment are described in the following sections.

3.2 Sample Collection

Recycled crushed concrete aggregate (RCCA) was collected from stockpiles of Big City Crushed Concrete located in Goodnight Lane, Dallas, Texas which is one of the TxDOT approved recycled aggregate stockpile facilities.

RCCA materials were collected from stockpiles of Big City Crushed Concrete Company, Goodnight Lane, Dallas, Texas. Reclaimed asphalt pavement (RAP) was collected from the TxDOT specified stockpiles situated in Dallas County, Ellis County, and Rockwall County, Texas.

Table 3.2 Designation and location of collected recycled materials.

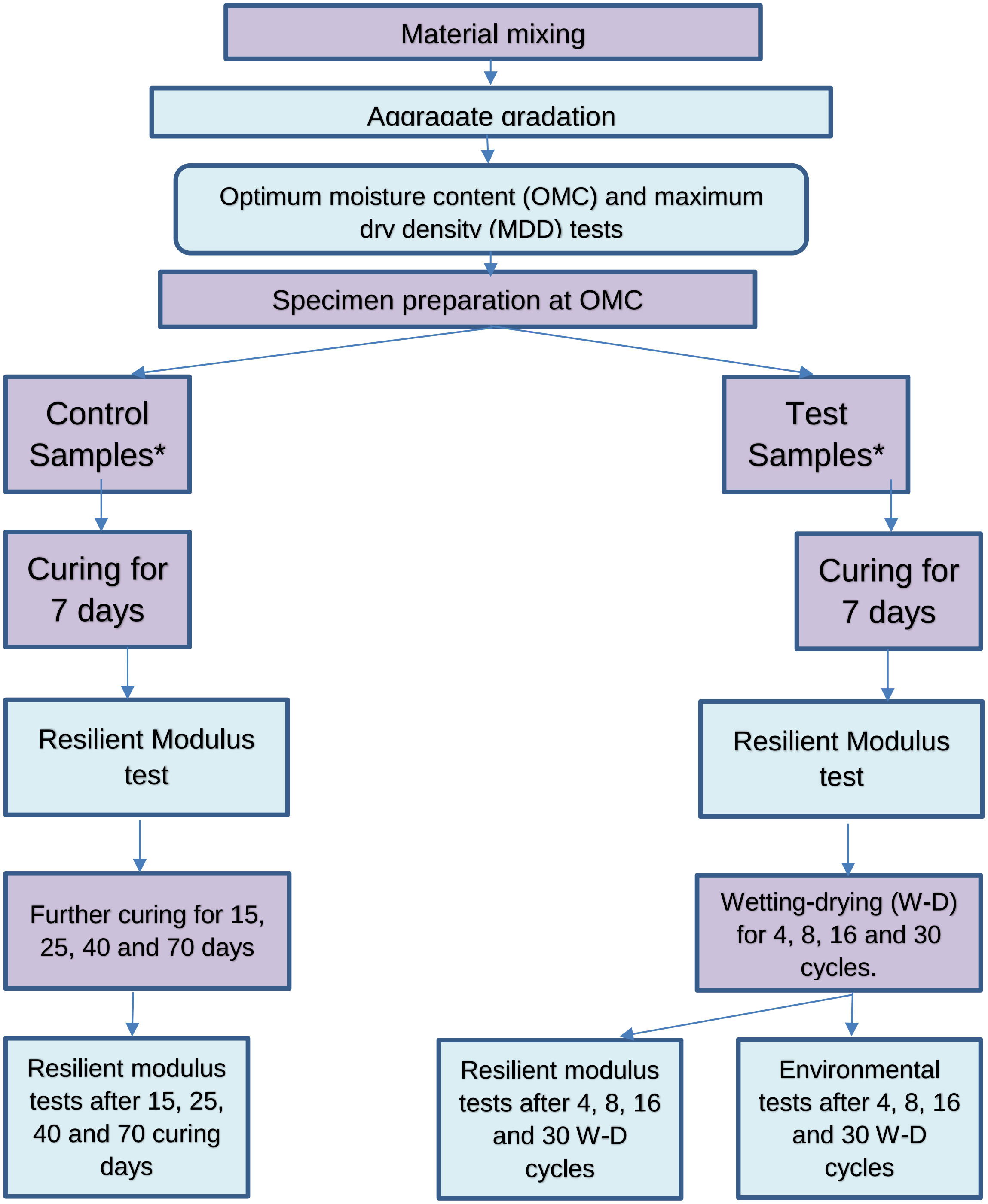
Stockpile Name	Material	Material ID
Big City Crushed Concrete	Recycled Crushed Concrete Aggregate	RCCA
Rockwell County	Reclaimed Asphalt Pavement	RAP

Figure 3.4 Sample collection (Big City Crushed Concrete, Dallas, Texas)

Figure 3.2 RAP sample collection from TxDOT Stockpile, Rockwall County

3.3 Experimental Program

The experimental program undertaken in this study aimed at evaluating the structural and environmental durability of RCCA and RAP materials in flexible pavement base construction in presence of repeated wetting-drying cycles. The RCCA and RAP materials used for this study contained particle size ranging from 1 inch (25 mm) to No. 200 (75 µm). For this research, three different combinations of RAP and RCCA materials were used – 100% RCCA, 70% RCCA + 30% RAP and 50% RCCA + 50% RAP. These material combinations were then stabilized at cement dosage of 4% and 6 %. Portland Type I/II cement was used to treat the base materials. For each of these material mixes, three “control samples” and three “test samples” were prepared. All the prepared samples were cured at 100% moisture controlled conditions for 7 days. After this curing period, the control samples were cured for upto 15, 25, 40 and 70 days. Simultaneously the test samples were subjected to 4, 8, 16 and 30 wetting-drying (W-D) cycles. At the end of these specified curing period and W-D cycles, the samples were tested for structural capacity and environmental soundness. A flow chart showing the experimental steps followed for a particular material combination is shown in Figure 3.3. Structural competency of the material was determined using the resilient modulus (M_R) test (Table 3.2). The environmental tests include total suspended solids (TSS), and total dissolved solids (TDS), chemical oxygen demand (COD), turbidity and pH which were conducted on the leachate samples. A tabular presentation of these environmental tests is given in Table 3.3. Repeatability of the tests were ensured by performing the same test on three identical specimens for each type of mixes.



*Three identical samples were prepared and tested for ensuring repeatability of tests.

Figure 3.3 Flow chart of the experimental steps followed for a typical material mix.

Table 3.2 Experimental program for resilient modulus (M_R) tests

Material	Cement Content (%)	Mix ID	Test Samples: Wetting-drying (WD) cycles					Control Samples: Curing (days)				
			0	4	8	16	30	7	15	25	40	70
100% RCCA	4%	0R_4C	√	√	√	√	√	√	√	√	√	√
	6%	0R_6C	√	√	√	√	√	√	√	√	√	√
70% RCCA + 30% RAP	4%	30R_4C	√	√	√	√	√	√	√	√	√	√
	6%	30R_6C	√	√	√	√	√	√	√	√	√	√
50% RCCA + 50% RAP	4%	50R_4C	√	√	√	√	√	√	√	√	√	√
	6%	50R_6C	√	√	√	√	√	√	√	√	√	√

Each of the material combinations were given a Mix ID in the form of XR_YC. Here the alphabets ‘R’ and ‘C’ represents Reclaimed Asphalt Pavement (RAP) and cement respectively. ‘X’ and ‘Y’ denotes the percentage of RAP and cement present in a particular mix. For example, the material combination containing 70% RCCA and 30% RAP with 4% cement content has a Mix ID of 30R_4C.

Table 3.3 Experimental program for environmental tests

Material	Cement content (%)	Mix ID	Environmental Tests	Test Samples Wetting-drying (WD) cycles				
				0	4	8	16	30
100% RCCA	4	0R_4C	COD	√	√	√	√	√
			TDS	√	√	√	√	√
			TSS	√	√	√	√	√
			Turbidity	√	√	√	√	√
			pH	√	√	√	√	√
	6	0R_6C	COD	√	√	√	√	√
			TDS	√	√	√	√	√
			TSS	√	√	√	√	√
			Turbidity	√	√	√	√	√
			pH	√	√	√	√	√
70% RCCA + 30% RAP	4	30R_4C	COD	√	√	√	√	√
			TDS	√	√	√	√	√
			TSS	√	√	√	√	√
			Turbidity	√	√	√	√	√
			pH	√	√	√	√	√
	6	30R_6C	COD	√	√	√	√	√
			TDS	√	√	√	√	√
			TSS	√	√	√	√	√
			Turbidity	√	√	√	√	√
			pH	√	√	√	√	√
50% RCCA + 50% RAP	4	50R_4C	COD	√	√	√	√	√
			TDS	√	√	√	√	√
			TSS	√	√	√	√	√
			Turbidity	√	√	√	√	√
			pH	√	√	√	√	√
	6	50R_6C	COD	√	√	√	√	√
			TDS	√	√	√	√	√
			TSS	√	√	√	√	√
			Turbidity	√	√	√	√	√
			pH	√	√	√	√	√

3.4 Aggregate Gradation

Particles size distribution of greater than No. 200 (0.075mm) sieve is determined by sieve analysis. The sieve analysis was conducted by following the guideline of Tex 110E Standard test method for particle size analysis of soil/particles. If less than 1% materials by weight passing through the No. 200 sieve, then hydrometer analysis is required. In this case, the amount of percent passing No. 200 sieve was less than 1% so hydrometer analysis was not necessary.

The amount of materials retained in each sieve was weighed and percent passing through the each sieve is calculated. The material retained on each sieve was divided by the weight of total sample and then subtracted by the total percentage of material. The percent of material passing through each sieve was plotted against the sieve size in a semi-log graph paper.

3.5 Laboratory Compaction and Moisture Density Relationships

According to TxDOT Tex0-113-E Laboratory Compaction Characteristics and Moisture-Density Relationship test procedure, the maximum dry density and optimum moisture content was determined. The compaction effort for TxDOT is more than standard proctor method but less than modified proctor compaction tests. The difference in the compaction energy is included in Table 3.4.

Table 3.4 Compaction energy on different laboratory compaction procedures

Method	Compaction Energy (ft-lb/in^3)	Reference
Standard Proctor	7.18	ASTM D-698 A
Modified Proctor	32.41	ASTM D-1557
TxDOT	13.25	TEX-113-E

The compaction test was performed using a mold of 6 inches diameter and 8 inches height and a hammer of 10lbs dropping from a height of 18 inches which applies 50 blows in each layer of four layers. The compaction was done on at least four samples at different moisture content. Moisture content was determined after the compaction of the samples and dry density was determined. After that moisture vs dry-density curve was plotted to determine the corresponding optimum moisture content and maximum dry density from the peak of the curve.

3.6 Wetting-drying (WD) Methodology

The effect of successive wetting-drying (W-D) cycles on strength properties of cement stabilized RCCA-RAP mix materials was investigated in this study. Due to unavailability of standard procedures for wetting-drying of stabilized base materials, experimental methods reported by researchers in recent times (Khoury and Zaman, 2002; Faysal et al. 2017b) has been adopted in this study. RCCA and RAP materials mixed in 50%-50% proportions and stabilized with 4% (MIX4) and 6% (MIX6) cement content have been used for this purpose. For each of these material combinations, three “control samples” and three “test samples” were prepared, cured for 7 days and then tested for resilient modulus as per AASHTO T 307-99 test procedures. The test samples were then subjected to wetting-drying (W-D) cycles. Each WD cycle consisted of drying the sample in the oven (71ºC/160ºF) for 24 hours followed by submerging it in potable water for 24 hours. For this study, the numbers of WD cycles considered were 0, 4, 8, 16 and 30 cycles. After completing a specified number of cycles, the samples were tests for resilient modulus (M_R). On the other hand, the control samples were cured following conventional process. After curing for upto for upto 15, 25, 40 and 70 days, the samples were tested again for M_R. These curing periods represent the time corresponding to 4, 8, 16 and 30 wetting-drying (WD) cycles. In this study, each sample was subjected to multiple resilient modulus tests after specific times. This approach was considered reasonable since resilient modulus tests involve very low levels of strain (Khoury and Zaman, 2007).

3.7 Resilient Modulus Test

Resilient Modulus (M_r) is a key parameter for pavement layer thickness design. This test was conducted using the AASHTO T 307-99 guidelines (AASHTO 2003).

3.7.1 Specimen Preparation for Testing

According to the Item 247 of Texas pavement Design Guideline, the type D Recycled Crushed Concrete Aggregate (RCCA) and RAP materials obtained from different sources and were used as individual or in mixes at different percentage to prepare the specimen. These materials were tested for resilient modulus with or without stabilization using cement. Repeatability of the tests were ensured by replicating three specimens for each RCCA, RAP and cement combination. The size of the specimens were 6 inch in diameter and 12 inch in height for all of the combinations of RCCA, RAP and cement content.

All of the specimens were subjected to compaction at optimum moisture content to achieve the maximum dry density from the moisture-density test results. Sample was compacted at 6 lifts with each lift having a height of 2 inch and subjected to 50 blows. The height of each lift is controlled by the automatic compactor itself. The maximum size of the particle has been kept limited to 1.2 inch which is one-fifth of the maximum diameter of the mold. Density of the compacted specimens were within +/-5% of the maximum dry density which signifies the attainment of satisfactory compaction.

The procedure stated above was used to prepare the specimen of RCCA, RAP, and different combination of these materials with or without stabilized. The test specimens were extracted from the mold using the extruder and then wrapped with plastic to avoid any disturbance and stored in the moist room for curing for seven days. The moist room has a controlled relative humidity of about 100% and a constant temperature of 70°F during the curing period of seven days. After seven days, the specimens were tested for their resilient modulus.

3.7.2 Resilient Modulus Testing Equipment

The resilient modulus of compacted specimen were determined using the automated system which meets the AASHTO T307-99 requirements. The whole system consists of two major components a fully automated unit and a computerized data acquisition system. The automatic unit consists of two LOADTRAC units, one Cyclic-RM unit, a load frame, actuator, a triaxial cell, two Linear Variable Displacement Transducer (LVDT) and electro-pneumatic air pressure controlling unit.

The cyclic load is applied using cyclic-RM unit with haversine pulse. The load pulse applied for 0.1s and the rest period is 0.9s. The actuator consists of load cell, the capacity of which is 1000 lbf to apply up to 40 psi stress on 6 inch dia and 12 inch height cylindrical specimen. Confining pressure is applied by electro-pneumatic air pressure regulator. This regulator can increase air pressure automatically in triaxial chamber. The axial deformation of the sample is measured from outside using two LVDTs attached on the piston rod at equal distance and opposite to each other.

RM6 software was installed to give initial inputs and data acquisition during the test. As the sample gets stiffer with time during the test, the system has controller to maintain the load and get it corrected to meet corrected values. The whole equipment setup which was used to do Resilient Modulus tests is shown in Figure . Confining pressure is applied by controlled air pressure. There are two different types of loading sequences are specified in AASHTO T307-99 such as subgrade soil and base materials to simulate traffic condition in pavement foundation. The amount of applied stress is higher for granular base or sub-base material than subgrade soil. The first loading sequence is preconditioning which has 500 to 1000 cycles. In this study, 500 cycle was selected for preconditioning. After the preconditioning the total load will be applied in 15 load sequences while each load sequences contains 100 cycles in accordance with AASHTO T307-99 code. This test was conducted on all of treated or untreated combinations of RCCA and RAP materials.

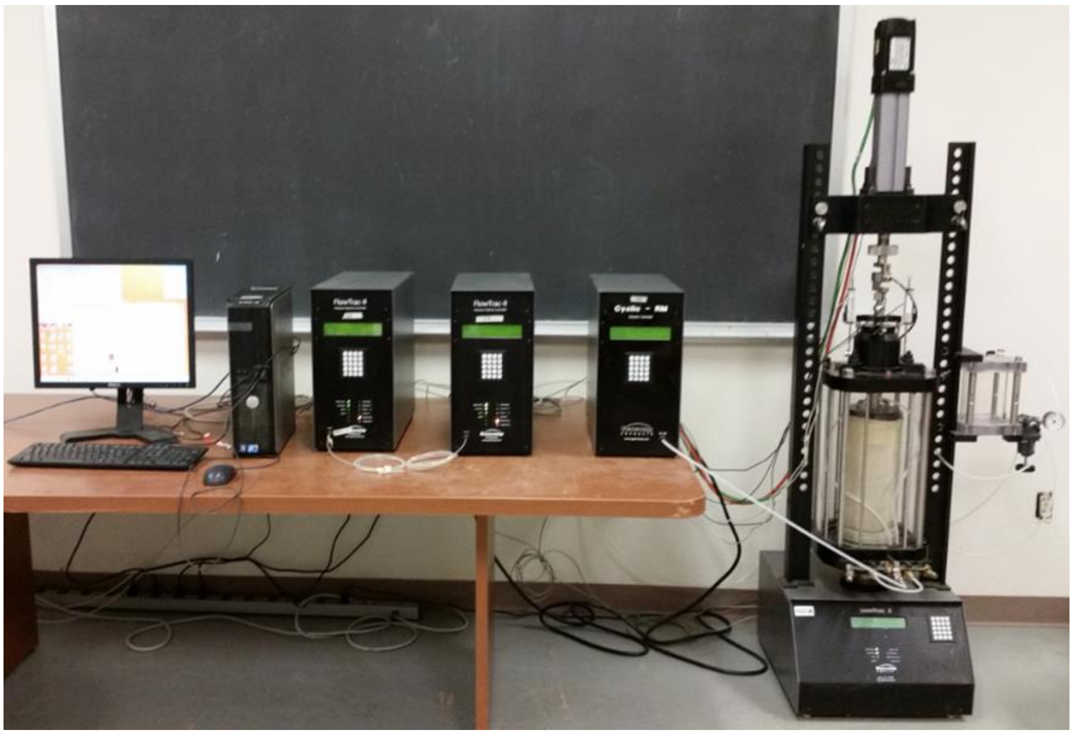


Figure 3.4 Resilient Modulus testing machine

The test sequences for resilient modulus test according to AASHTO T307-99 is mentioned in Table 3.5.

Table 3.5 Resilient modulus test sequences and stress values for base and subbase materials (AASHTO T307-99)

Sequence No.	**Confining Pressure (psi)**	**Max. Axial Stress (psi)**	**No. of Cycles**
Pre-conditioning	15	15	500-1000
1	3	3	100
2	3	6	100
3	3	9	100
4	5	5	100
5	5	10	100
6	5	15	100
7	10	10	100
8	10	20	100
9	10	30	100
10	15	10	100
11	15	15	100
12	15	30	100
13	20	15	100
14	20	20	100
15	20	40	100

3.7.3 Data Analysis of Resilient Modulus Tests

Determination of resilient moduli in each load sequence under different confining and deviator stresses were calculated in accordance with AASHTO T307-99 code. The RM6 software can automatically generate the resilient modulus vs bulk stress graph and the test result chart while each value of M_r is the average of the last five cycles. The displacement results obtained from two LVDTs were averaged and divided by the specimen height to determine the accumulated strain of each reading. The difference between the maximum axial strain and the last axial strain in the 200 axial strains of each load cycle is the resilient strain of this load cycle (Li 2011). Axial stress is determined by dividing the load with the area of the specimen. Cyclic stress is the difference between the maximum axial stress and minimum axial stress in 200 readings for each load applications. Resilient modulus of each load cycle was calculated through dividing the cyclic stress by the resilient strain. According to AASHTO test procedure requirements, the obtained resilient modulus data were used to develop prediction models. One of them is the "k-θ model" proposed by Moosazedh and Witczak (1981).

$$M_r = k_1\theta^{k_2}$$

Where k_1 and k_2 are model parameters and θ is the bulk stress expressed as a combination of confining (σ_c) and deviator stresses (σ_d) in the form $3\sigma_c + \sigma_d$. The other model used in this study is the improved three-parameter model.

$$M_r = k_3\sigma_c^{k_4}\sigma_d^{k_5}$$

Where k_3, k_4 and k_5 are model parameters. Statistical analysis was conducted to examine the accuracy of these models.

3.8 Environmental Tests

Leachate tests were conducted on the selected combinations of RCCA and RAP materials untreated or treated with cement. Leachate tests included pH, total dissolved and suspended solids, turbidity, and chemical oxygen demand.

3.8.1 pH Tests

pH is the measure of the acidity in the subsequent materials. pH test will be conducted in accordance with ASTM D1287. The value of pH ranges between 0 to 14 and 7 is considered as the neutral value. If the value is less than 7 is considered acidic and more than 7 is considered as alkaline. The value of pH was measured using a dual channel pH conductivity meter device. The following Figure 3.5 includes the pH measurement device. pH test was conducted by inserting the probe into the leachate sample collected after soaking the specimen in water for 24 hours.

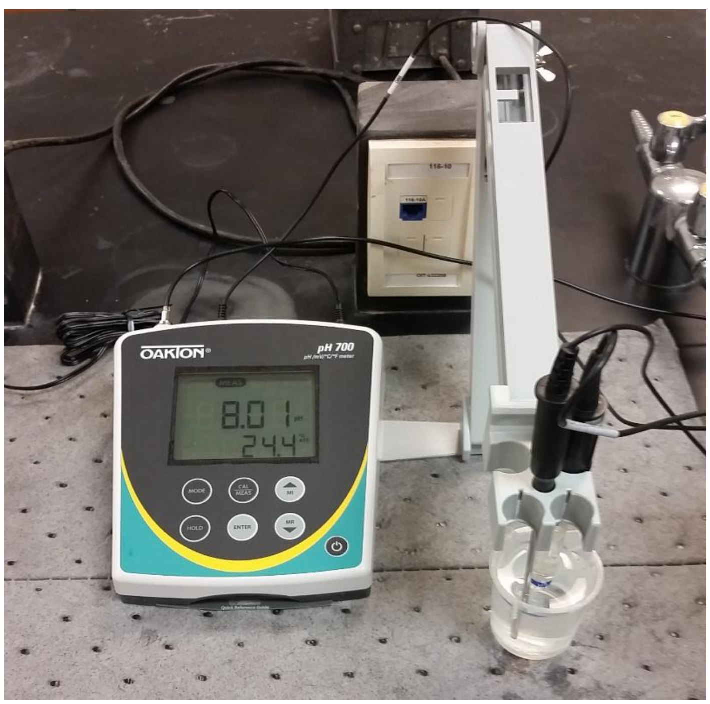


Figure 3.5 Dual channel pH/ion/conductivity meter

3.8.2 Total Suspended and Dissolved Solids (TSS & TDS)

According to the ASTM D 5907-03 specification of Standard Test Method for Filterable and non-filterable materials total dissolved and suspended solids tests were conducted. Glass fiber filter paper is used to remove the suspended solids by passing the water sample through the filter. Suspended solids were retained on the filter paper whereas filtrates were passed through the filter paper.

3.8.3 turbidity

Turbidity was measured using 2100P Turbidimeter (Figure 3.6). The amount of particles present in water represents turbidity. It is measured by using the shine of a light passing through the sample.

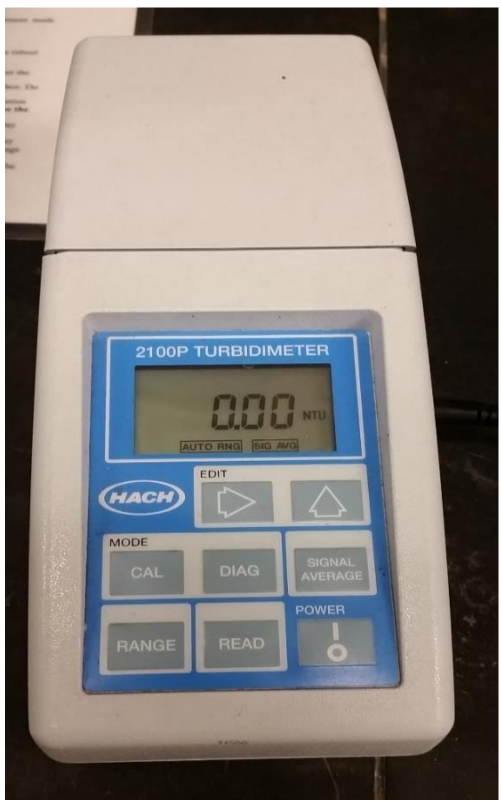


Figure 3.6 2100P Turbidimeter

3.8.4 Chemical Oxygen Demand (COD)

According to the specification of the ASTM D 1252, test method was conducted to determine oxygen that is consumed by the impurities in the water. First a Transmittance vs concentration of COD was calibrated and calibration curve was produced. Then the samples were poured into COD vials and heated for two hours as digester period in the COD reactor (Ordonez 2006) (Figure 3.7).

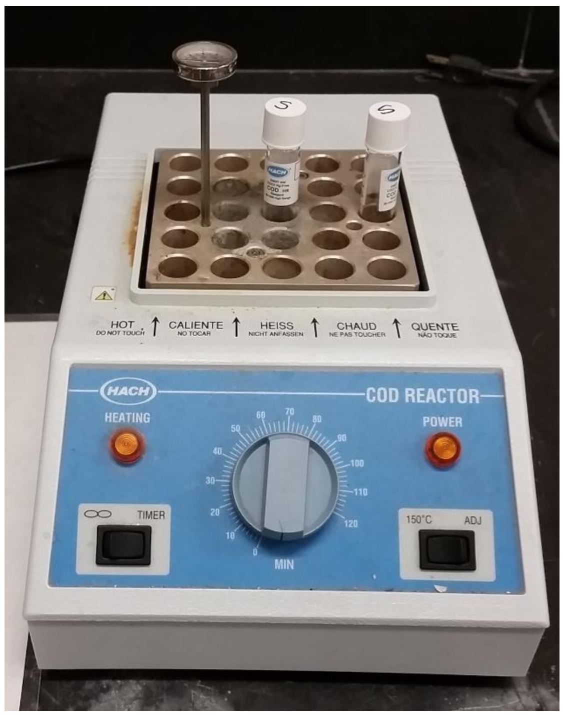


Figure 3.7 COD Reactor

Then the vials were removed from the digester and waited for 20 minutes to cool it down to room temperature. After that the vials were inserted into the digital reactor and the value of transmittance and absorbance readings were taken from the device.

Chapter 4

Results and Analysis

4.1 Introduction

The results obtained from the particle size distribution, optimum moisture content, resilient modulus and environmental tests have been presented in this section. The resilient modulus data has been plotted in regression models and their variation with RAP content, cement content, curing period, wetting-drying cycles etc. has also been discussed here.

4.2 Particle Size Distribution

Varying particle size has a significant effect on the strength characteristics of granular materials (UFGS, 2010). Sieve analysis tests were conducted on the collected RCCA and RAP materials following Standard TxDOT Specifications (Tex-110E). Figure 4.1 shows the particle-size distribution curves for both the materials. It is evident that for both RCCA and RAP, less than 1% of the material passes through the No. 200 sieve. Therefore, as per TxDOT specification (Item 276), hydrometer analysis was not deemed necessary for this study.

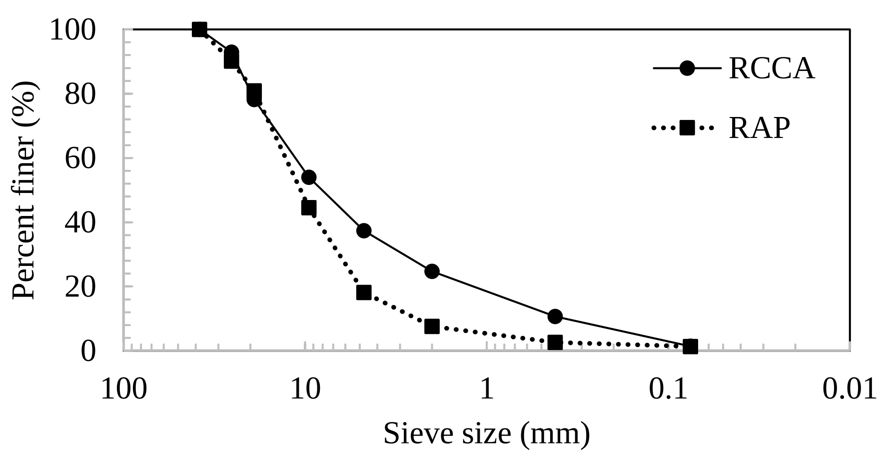


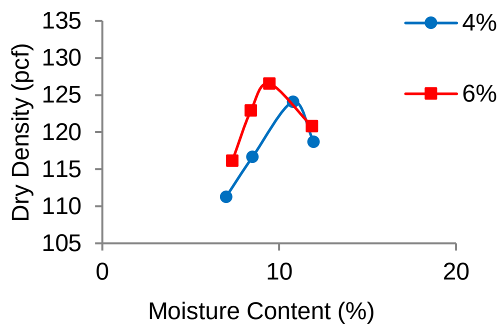
Figure 4.1 Particle size distribution for RAP and RCCA material

4.3 Moisture Density Tests

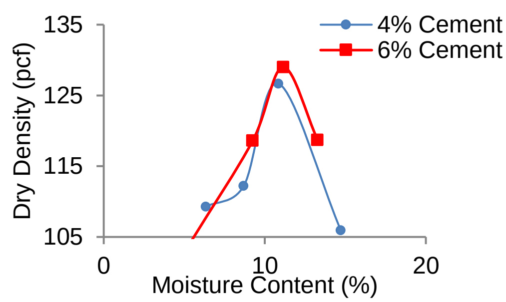
The moisture content at which compaction of a certain material yields its maximum dry density (MDD) is the Optimum Moisture Content (OMC). In this study, OMC and MDD tests were conducted following the Tex-113 E guidelines on each of the material combinations at different cement contents as shown in Table 3.2. Compaction energy required for compaction is 13.25 ft-lb/in3. The dimensions of the mold were 6 inch diameter and 8 inch height. The compaction tests were done at least on 4 different moisture contents, and the dry density was determined for different moisture content. The obtained dry densities were plotted against the moisture contents, and the optimum moisture contents were determined from the peak of the trend curve. The value obtained for different combination of base materials are shown in Figure 4.2, Figure 4.3. The value of the optimum moisture content was found to be 8.5% with a maximum dry density value of 1950 kg/m^3 (122 pcf).

4.4 Specimen Preparation

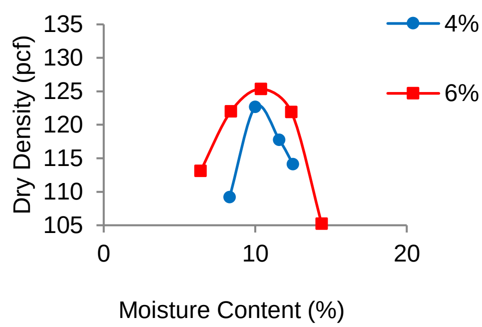
TxDOT guideline (Tex- 113 E) was followed for specimen preparation. The mold used to prepare UCS samples was 6 in. (152.4 mm) in diameter and 8 in. (203.2 mm) in height, but for the resilient modulus test the mold height was 12 in. (254 mm). An automated mechanical compactor which meets the TxDOT specifications was used for compacting. Prepared specimens were kept in moist room for 7 days in accordance with Soil-Cement Testing Procedure (Tex- 120 E) before testing



(a)



(b)



(c)

Figure 4.2 Optimum moisture content (OMC) and maximum dry density (MDD) plots for (a) 100% RCCA, (b) 30% RAP + 70% RCCA and (c) 50% RAP + 50% RCCA combinations.

4.5 Resilient Modulus Test Results

The resilient modulus response of all the material combinations were obtained at confinements of 3. 5, 10, 15 and 20 psi. For each of these confining pressures, three deviator stresses were applied. The results obtained from the resilient modulus tests are presented in APPENDIX A Figure A1 to Figure A12. It was found that both the confining and deviator stresses have noteworthy effects on resilient modulus response. At higher confinements, samples become denser and hence stronger which attributed to the increase of resilient modulus. Also at a constant confining pressure, resilient modulus increased with the increase of deviator stress as the samples yield lower axial strain due to strain hardening, though the influence of deviator stress was less pronounced at higher confinements.

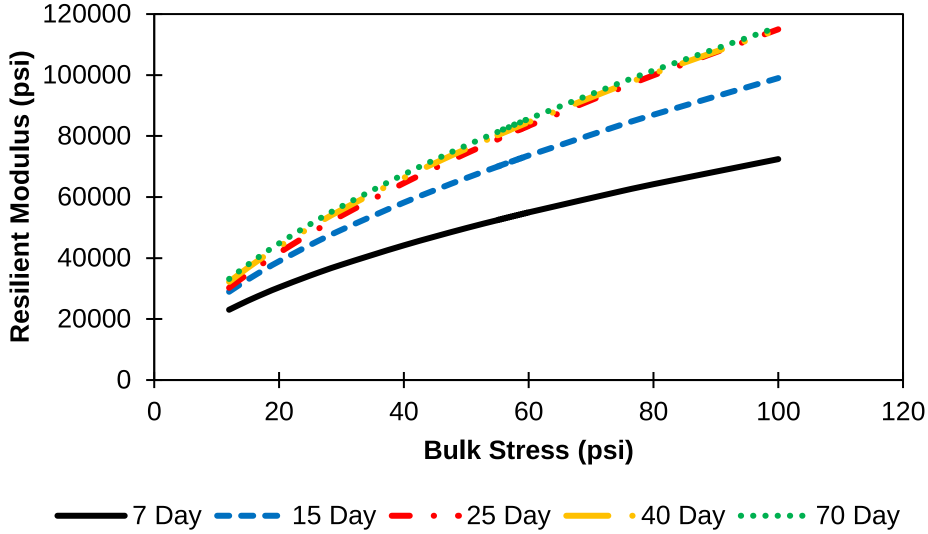
4.5.1 Prediction Models

The resilient modulus values for all the samples were obtained at 15 sets of confining and deviator stresses. For each material combination and testing conditions, three identical samples were tested. For all the cases, the coefficient of variation (COV) of the M_R values was found to be within 0.15% - 8.63% which shows good repeatability of the performed tests. As per the AASHTO test procedure requirements, the obtained resilient modulus data were used to develop prediction models. One of them is the "k-θ model" proposed by Moosazedh and Witczak (1981): $M_R = k_1\theta^{k2}$, where k_1 and k_2 are model parameters and θ is the bulk stress expressed as a combination of confining (σ_c) and deviator stresses (σ_d) in the form $3\sigma_c + \sigma_d$. This bulk stress most practically represents the stress conditions in flexible pavement base layers (Cetin 2010). Figure 4.2 – 4.7 shows the k-θ models developed for Control and Test samples of MIX4 and MIX6. Another model used in this study is the improved three-parameter model (Puppala et. al., 1996): $M_R = k_3\sigma_c^{k4}\sigma_d^{k5}$, where k_3, k_4 and k_5 are model parameters. Statistical analysis was conducted to examine the accuracy of these models. The model parameters along with the calculated statistical parameters are presented in APPENDIX A Table A1 to Table A2.

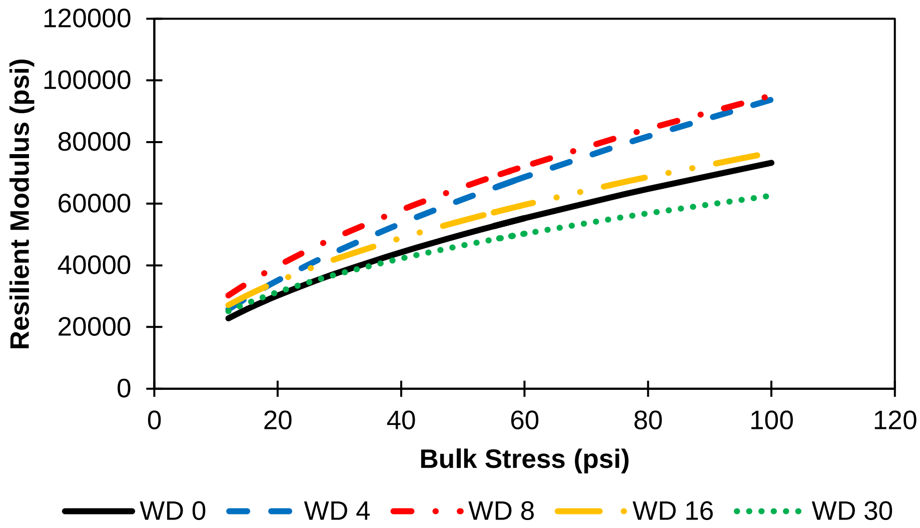
k-θ model parameter $logk_1$ indicates magnitudes while k_2 indicated the non-linear nature of the stress dependency (Potturi 2006). Three parameter model parameter $logk_3$ indicates the magnitude of the resilient moduli while k_4, and k_5 represents the non-linear nature of the stress dependency.

The value of R^2 was found to be greater than 0.8 for all cases. The three-parameter model had higher values of R^2 than the two-parameter model for all cases. This is because the three-parameter model considers the individual effects of confining and deviator stresses on resilient modulus in contrast to the k-θ model that considers only a combined bulk stress.

The two-parameter models obtained for all the material combinations have been plotted in Figure 4.3 – 4.9. Increasing curing period and wetting-drying cycles causes the model curves to shift to the left or right. This indicates changes in resilient modulus values of these material. For all the material mix, the model curved shifted upwards with increasing curing period. Longer curing time improves stiffness properties of the materials and thus increases the resilient modulus. But the change in resilient modulus values due to wetting-drying (W-D) cycles did not follow a specific trend for all the combinations. A more detailed analysis on this trend is presented in the following sections. However, the resilient modulus (M_R) values obtained at a specific bulk stress of 30 psi (0.207 MPa) was used for this purpose. This stress level is considered ideal for pavement base layers in accordance with NCHRP 1-28A guidelines.

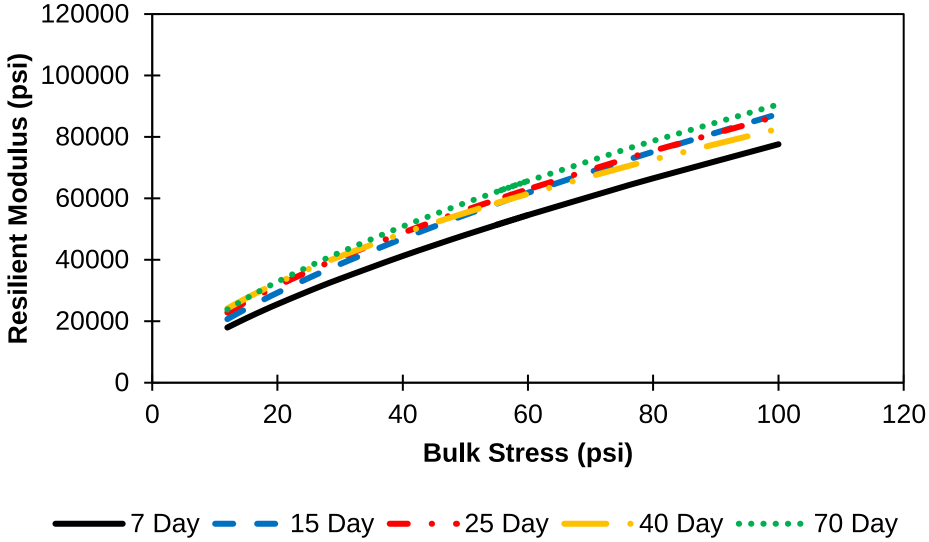


(a)

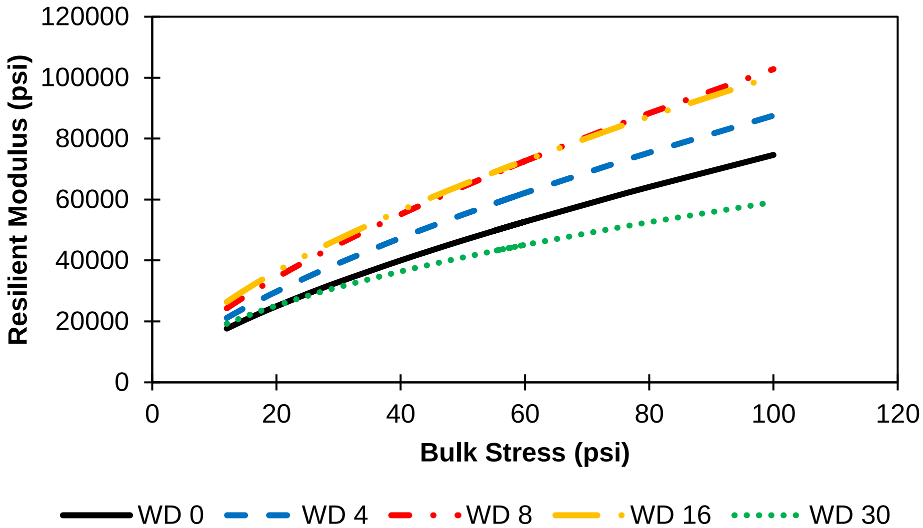


(b)

Figure 4.3 Graphical plot of the k-$\boldsymbol{\theta}$ model for 0R_6C combinations subjected to (a) curing and (b) wetting-drying (W-D) cycles.

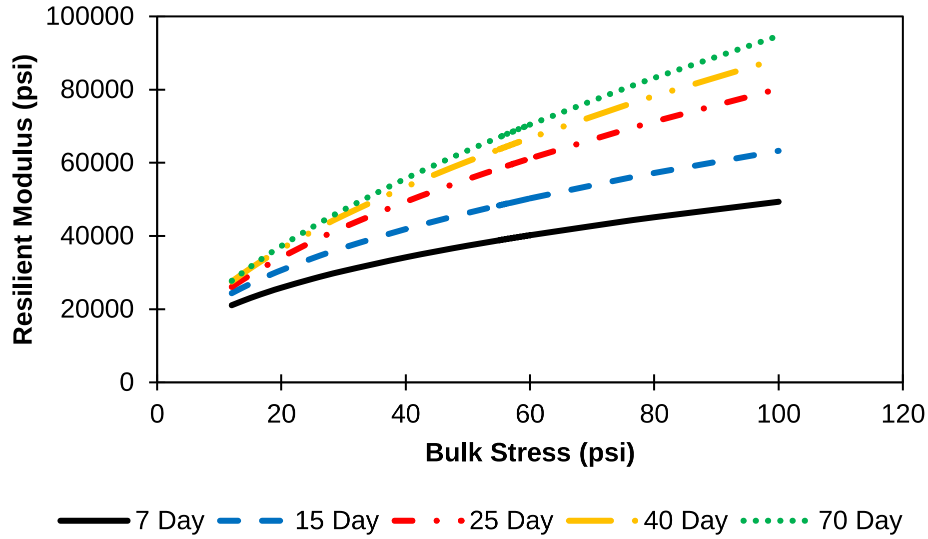


(a)

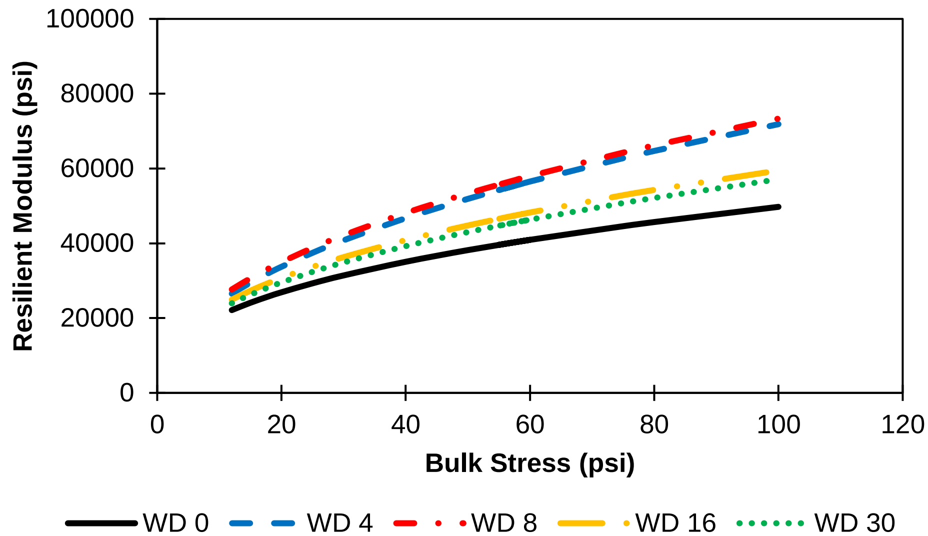


(b)

Figure 4.4 Graphical plot of the k-$\boldsymbol{\theta}$ model for 0R_4C combinations subjected to (a) curing and (b) wetting-drying (W-D) cycles.

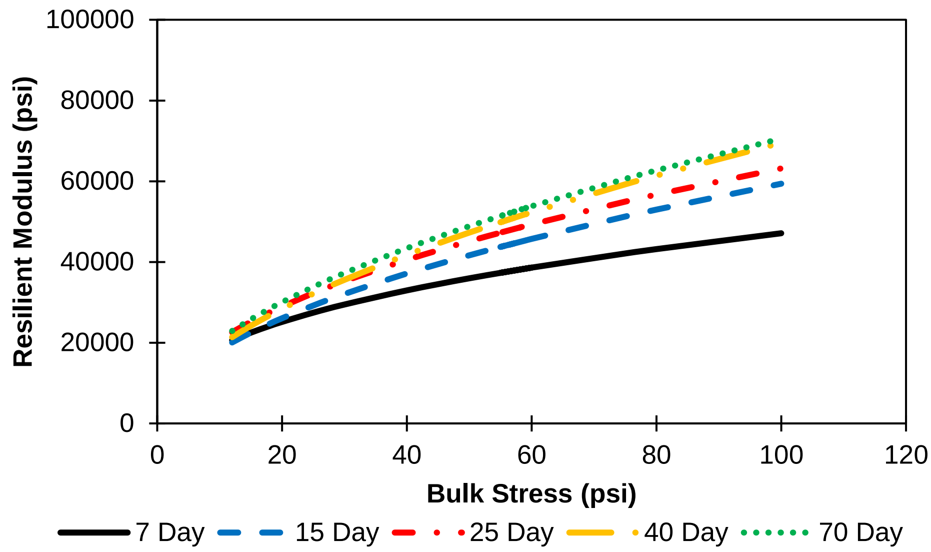


(a)

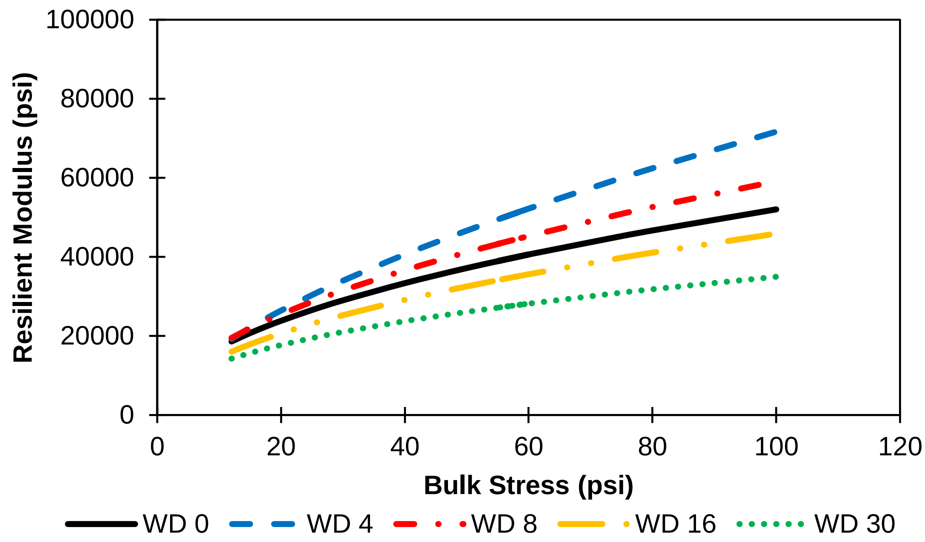


(b)

Figure 4.5 Graphical plot of the k-$\boldsymbol{\theta}$ model for 30R_6C combinations subjected to (a) curing and (b) wetting-drying (W-D) cycles.

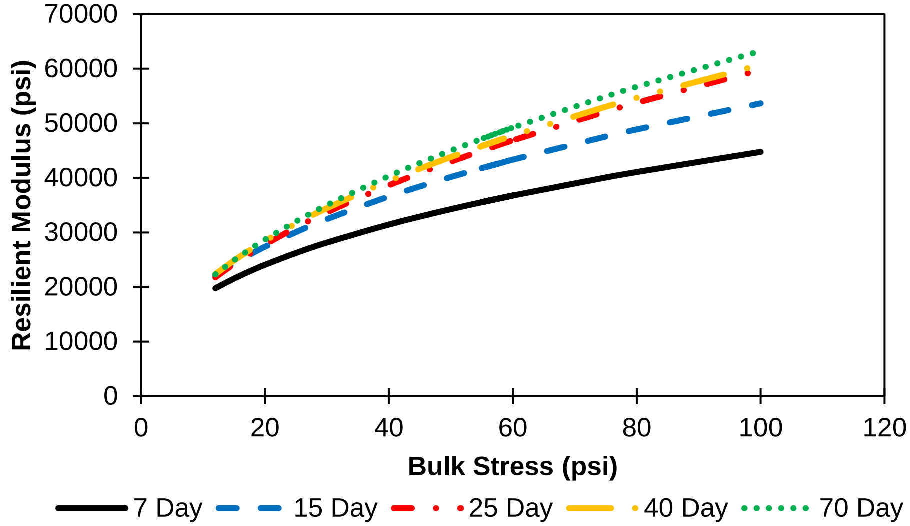


(a)

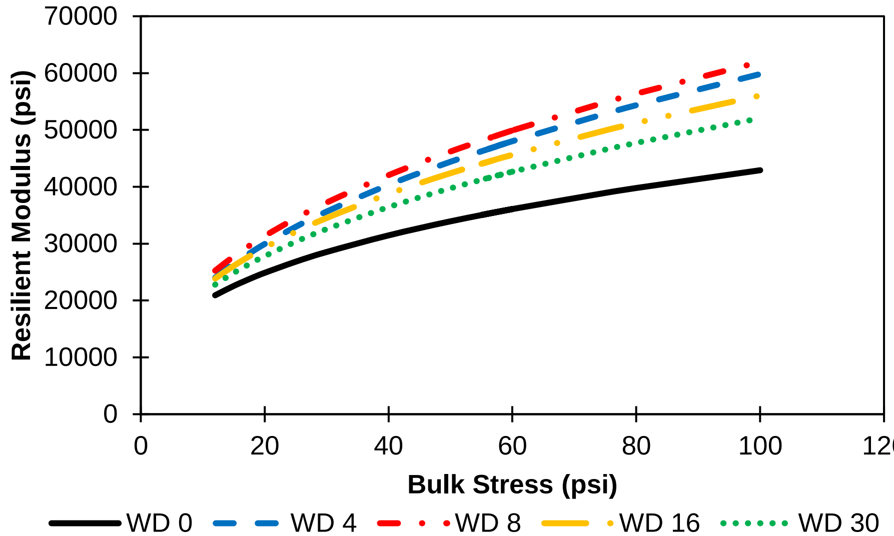


(b)

Figure 4.6 Graphical plot of the k-$\boldsymbol{\theta}$ model for 30R_4C combinations subjected to (a) curing and (b) wetting-drying (W-D) cycles.

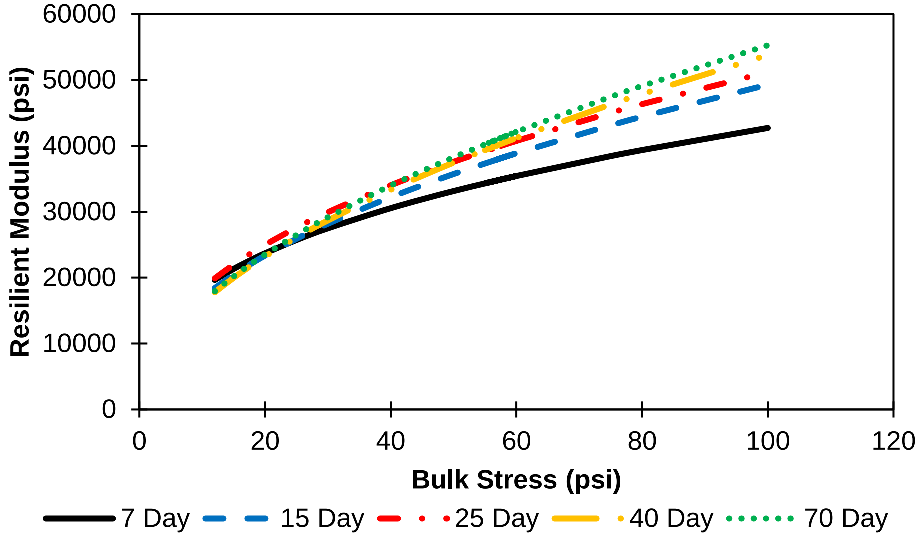


(a)

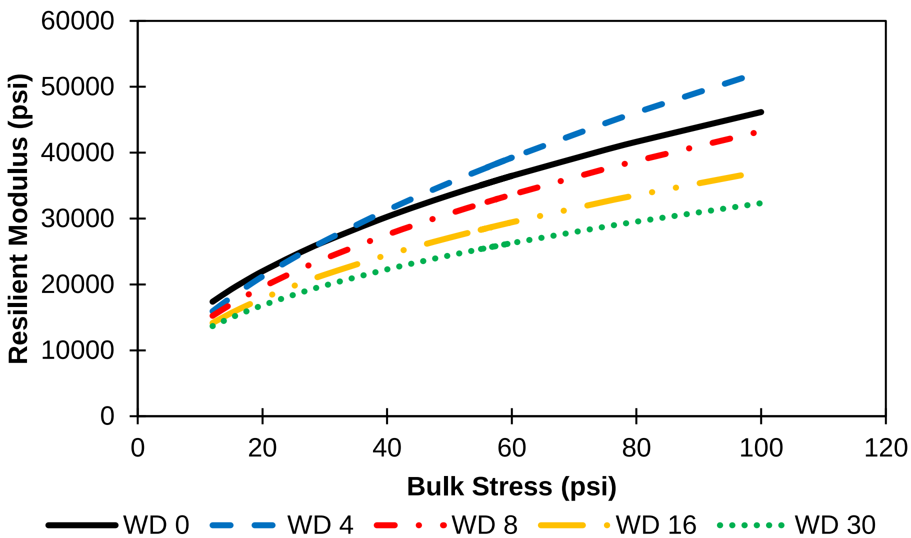


(b)

Figure 4.7 Graphical plot of the k-$\boldsymbol{\theta}$ model for 50R_6C combinations subjected to (a) curing and (b) wetting-drying (W-D) cycles.



(a)



(b)

Figure 4.8 Graphical plot of the k-θ model for 50R_4C combinations subjected to (a) curing and (b) wetting-drying (W-D) cycles.

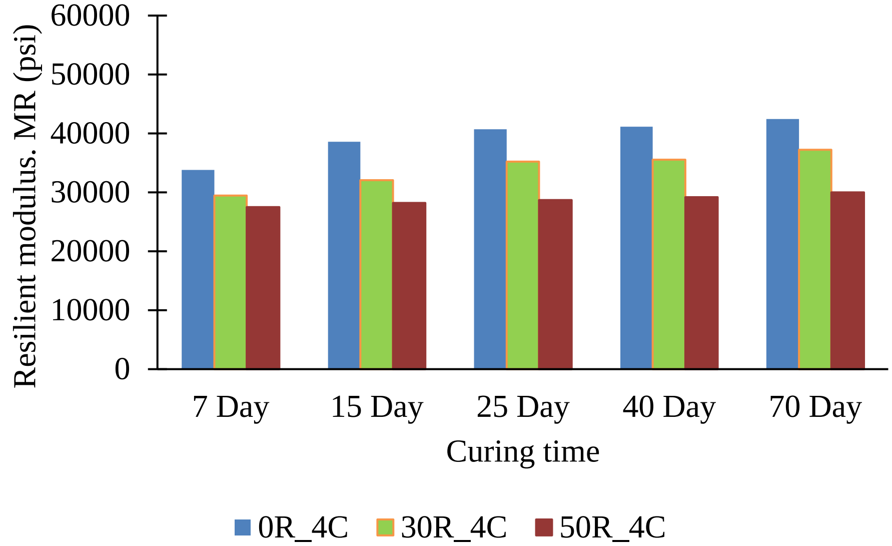
4.5.2 Effect of curing on Resilient Modulus

Three identical control samples were prepared for each of the material combinations and cured for 7, 14, 25, 40 and 70 days. At the end of these curing periods, these control samples were tested for resilient modulus. The resilient modulus values obtained at a bulk stress of 30 psi are plotted in Figure 4.9. It was observed that conventional curing process had a positive effect on the resilient modulus of all the combination mixes.

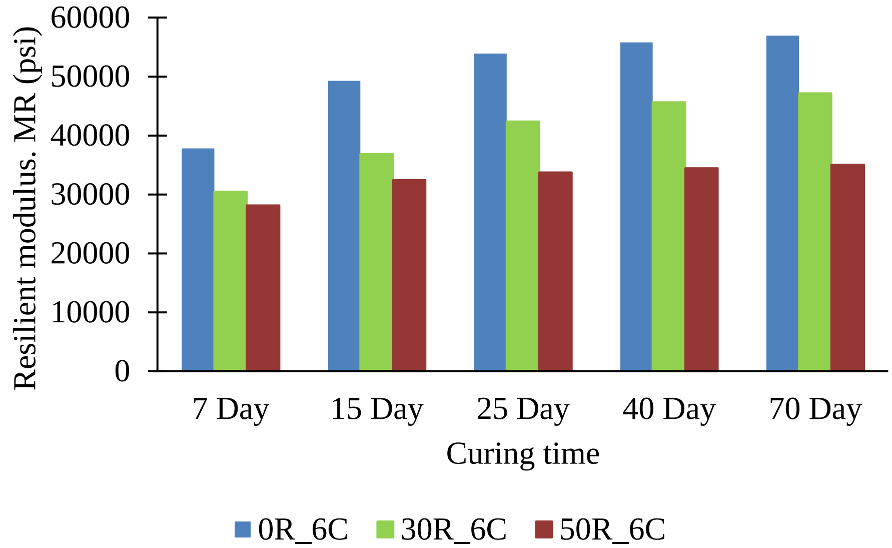
For the 100% RCCA 4% cement mix (0R_4C), the 7-day resilient modulus value increased by about 14%, 20%, 22% and 25% after 15, 25, 40 and 70 days respectively. However, for the 100% RCCA 6% cement mix (0R_ 6C), the 7-day resilient modulus value increased by about 30%, 42%, 48%% and 50% after 15, 25, 40 and 70 days respectively.

For the 30% RAP - 70% RCCA 4% cement mix (30R_4C), the 7-day resilient modulus value increased by about 9%, 19%, 21% and 26% after 15, 25, 40 and 70 days respectively. However, for the 30% RAP - 70% RCCA 6% cement mix (30R_6C), the 7-day resilient modulus value increased by about 21%, 39%, 49%% and 54% after 15, 25, 40 and 70 days respectively.

For the 50% RAP - 50% RCCA 4% cement mix (50R_4C), the 7-day resilient modulus value increased by about 2.5%, 5.4%, 6.1% and 9% after 15, 25, 40 and 70 days respectively. However, for the 50% RAP - 50% RCCA 6% cement mix (50R_6C), the 7-day resilient modulus value increased by about 15%, 19%, 22%% and 24% after 15, 25, 40 and 70 days respectively.



(a)



(b)

Figure 4.9 Effect of curing on resilient modulus of material combinations stabilized with (a) 4% and (b) 6% cement content.

The resilient modulus of all the combinations increased with curing time. The strength gain was prominent upto 25 days for most of the combinations. But this increasing trend of resilient modulus continued upto 70 days of curing for all material combinations. The primary reason behind this is the curing action of cement. Similar results were also obtained by Kaniraj and Havanagi (2001) who concluded that cement stabilized aggregates if properly cured can gain strength for upto 90 days.

Each of these cured samples were tested for resilient modulus after 7, 14, 25, 40 and 70 days of curing. Any detrimental effect of such repeated testing procedure would have reduced the stiffness properties of the samples, and hence the resilient modulus. Since no such reduction in resilient modulus was observed even after 70 days, it can be concluded that this repeated testing procedure is a non-destructive method of determining the durability of stabilized aggregates. The resilient modulus test induces very low levels of strain in the samples which does not deteriorate the samples significantly. (Khoury and Zaman, 2005). Camargo et. al. (2013) studied the effect of cement kiln dust (CKD) on Recycled Pavement Material (RPM) consisting of milled asphalt and limestone base course. The blends were cured for 7, 28 and 56 days. Resilient modulus was found to increase with increasing curing periods for upto 56 days.

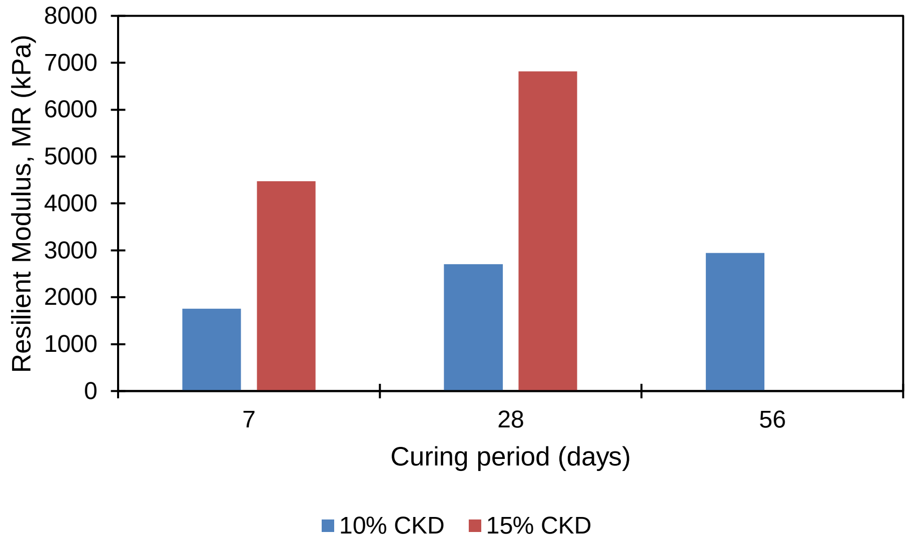


Figure**: Variation of resilient modulus of CKD stabilized RPM with curing period. (Camargo et. al. 2013).

4.5.3 Effect of Wetting-Drying (W-D) cycles on Resilient Modulus

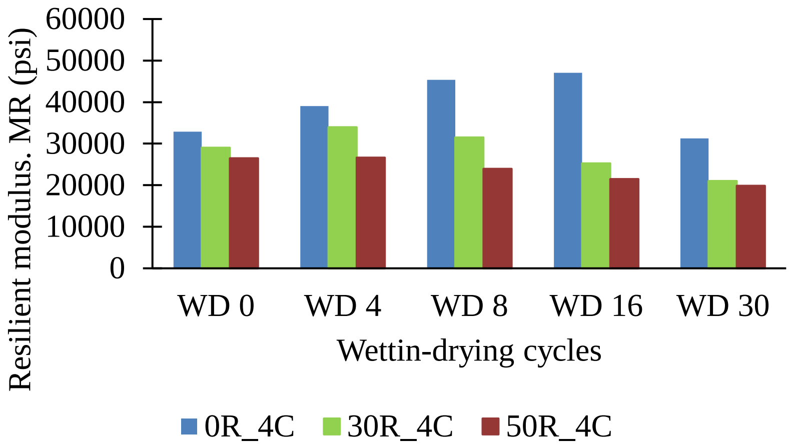
Three identical test samples were prepared for each of the material combinations and cured for 7 days. At the end of the curing period, these test samples were then subjected to 4, 8, 16 and 30 wetting-drying (W-D) cycles. At the end of these specified W-D cycles, the samples were tested for resilient modulus. The k-θ models developed for these test samples are plotted in Figure 4.3-4.8. Also the resilient modulus values obtained at a bulk stress of 30 psi are plotted in Figure 4.10. It was observed that the wetting-drying process had both positive and negative effects on the resilient modulus of the combination mixes.

For the 100% RCCA 4% cement mix (0R_4C), the 7-day (0 W-D cycles) resilient modulus value increased by about 19%, 38% and 43% by the end of 4, 8 and 16 W-D cycles respectively. However, the resilient modulus value obtained after 30 W-D cycles was found to be lower by about 33% as compared to the resilient modulus after 16 W-D cycles. This value is even lower than the 7-day resilient modulus value by about 5%. Also for the 100% RCCA 6% cement mix (0R_6C), the 7-day (0 W-D cycles) resilient modulus value increased by about 19% and 31% after 4 and 8 W-D cycles respectively. However, this M_R value obtained after 8 W-D cycles dropped by about 14% and 12% by the end of 16 and 30 W-D cycles respectively.

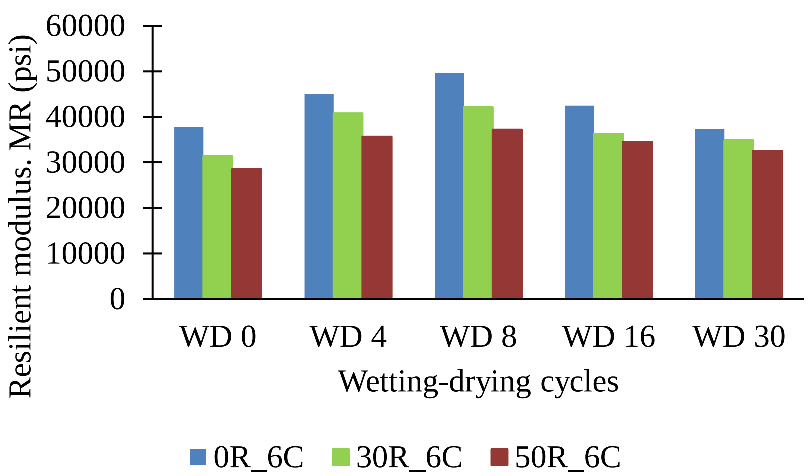
For the 30% RAP - 70% RCCA 4% cement mix (30R_4C), the 7-day (0 W-D cycles) resilient modulus value increased by about 17% after 4 W-D cycles. However, this M_R value at 4 W-D cycles dropped by about 7%, 25% and 38% by the end of 8, 16 and 30 W-D cycles respectively. But for the 30% RAP - 70% RCCA 6% cement mix (30R_6C), the 7-day (0 W-D cycles) resilient modulus value increased by about 30% and 34% after 4 and 8 W-D cycles respectively. However, the M_R value at 8 W-D cycles dropped by about 14% and 17% by the end of 16 and 30 W-D cycles respectively.

For the 50% RAP - 50% RCCA 4% cement mix (50R_4C), the 7-day (0 W-D cycles) resilient modulus value increased only by about 1% after 4 W-D cycles. However, this M_R value at 4 W-D cycles dropped by about 10%, 19% and 26% by the end of 8, 16 and 30 W-D cycles respectively. But for the 50% RAP - 50% RCCA 6% cement mix (50R_6C), the 7-day (0 W-D cycles) resilient modulus value increased by about 25% and 30% after 4 and 8 W-D cycles respectively. However, this M_R value at 8 W-D cycles dropped by about 14% and 17% by the end of 16 and 30 W-D cycles respectively.

For all the combination materials, the resilient modulus values initially increased with increasing wetting-drying cycles. This was most prominent in the 0R_6C mix where the resilient modulus increased upto 16 W-D cycles. In case of 50R_4C mix, this increase continued upto 4 W-D cycles. The reason behind this phenomenon is moisture intrusion resulting from the initial W-D cycles contributes towards cement hydration rather than weakening the materials. This induces higher stiffness properties into the samples which increases resilient modulus. Additional wet-dry cycles caused a reduction in M_R values. This indicates that the wetting-drying process is having adverse effects on the binding properties of the mix.



(a)



(b)

Figure 4.10 Effect of wetting-drying cycles on resilient modulus of material combinations stabilized with (a) 4% and (b) 6% cement content.

Khoury and Zaman (2007) conducted resilient modulus tests on Sawyers specimens stabilized with 15% cement kiln dust (CKD) after subjecting the samples to wetting-drying cycles. They observed that the initial resilient modulus value increased by about 25% after 8 W-D cycles. A further increase in W-D cycles decreased the resilient modulus values upto 30 W-D cycles. They concluded that the initial wetting-drying phase induced enhanced pozzolanic reactions within the material mix which increased the stiffness. Further wetting-drying processes had adverse effects on binding properties of cement and thus caused in a reduction in resilient modulus values. The results are presented in Figure 4.11.

Table 9. A summary of the statistical analysis of CKD-stabilized Sawyer specimens subjected to W-D cycles.

	W-D cycles	$[M_r = k_1 \times (k_2)_3^{\sigma} \times k_3^{\theta})]$		R^2	Adjusted R^2	F value	Pr	Significant	$M_r^{\dagger}$
Sawyer with 15% CKD	0	k_1	2537	0.95	0.95	168.35	<0.0001	Yes	4027
		k_2	0.99563						
		k_3	1.00168						
	8	k_1	3586	0.88	0.86	60.43	<0.0001	Yes	5037
		k_2	0.99653						
		k_3	1.00128						
	16	k_1	2947	0.84	0.83	45.82	0.0715	Yes	4253
		k_2	0.99389						
		k_3	1.00184						
	30	k_1	2528	0.27	0.19	3.2	0.066	Yes	3165
		k_2	1.00027						
		k_3	1.00036						

† M_r values calculated at $\sigma_3 = 104$ kPa and $\theta = 547.5$ kPa.

Figure 4.11 Variation of resilient modulus with wetting-drying cycles for Sawyer specimens stabilized with CKD. (Khoury and Zaman, 2007)

4.5.4 Effect of RAP content

For the current study, RCCA and RAP were mixed in three different combinations – 100%, 30% RAP + 70% RCCA and 50% RAP+ 50% RCCA. Each of these combinations was there stabilized with 4% and 6% cement contents. Figure 4.10 shows the resilient modulus results of the control and test samples of these material mixes respectively. It was observed that at 4% cement content, increasing the RAP content from 0% (0R_4C) to 30%

(30R_4C) decreased the resilient modulus values by about 12.5%. At 6% cement content, this decrease in resilient modulus values was about 18%. However, increasing the RAP content to 50% further decreased the resilient modulus values by about 6-9 % at all cement contents. RAP materials are relatively lighter than RCCA and has a lower specific gravity (Faysal 2017). RAP aggregates are coated with asphalt which accounts for a slippery surface. Addition of RAP thus reduces the stiffness of the mix and hence yield lower values of resilient modulus. These results are in accordance with those obtained by Faysal (2017). He conducted resilient modulus tests on RAP-RCCA materials collected from different sources. He concluded that addition of more than 10% RAP content significantly reduces the resilient modulus values of materials stabilized at all cement contents.

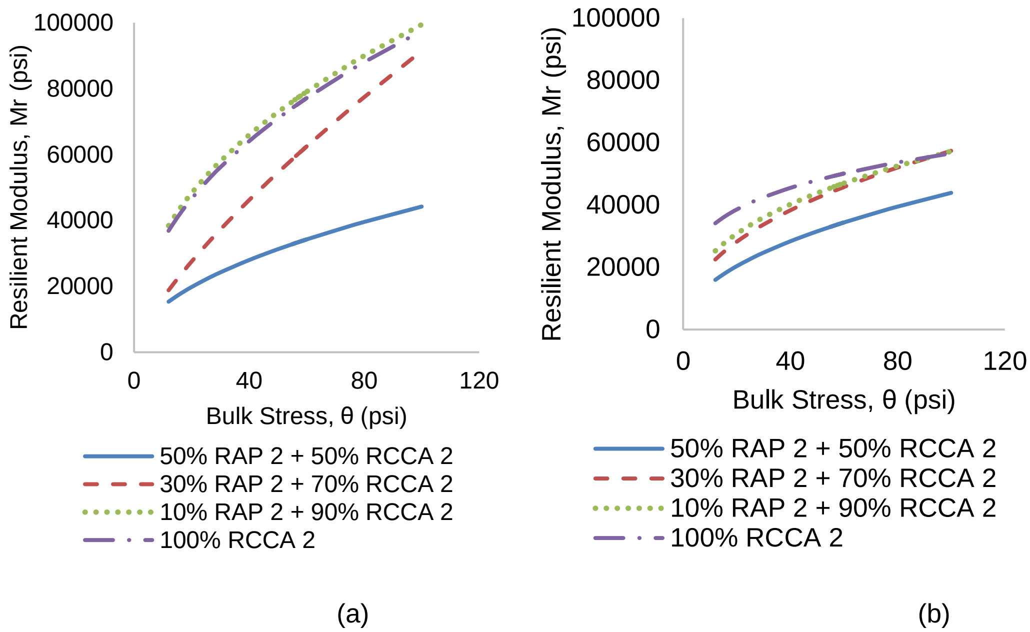


Figure 4.12 Two parameter model (k-θ) for different RAP 2-RCCA 2 (Source 2) combinations stabilized at (a) 6 % and (b) 4% cement content (Faysal, 2017).

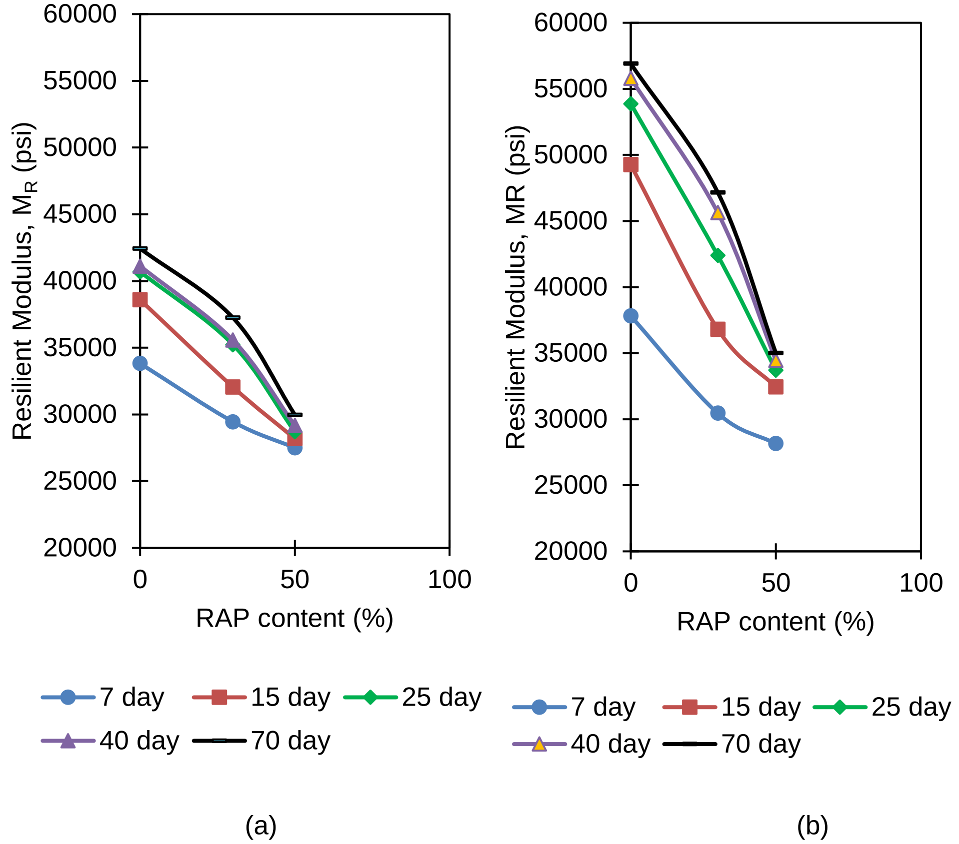


Figure 4.13 Variation of resilient modulus with RAP content at different curing periods for materials stabilized with (a) 4% and (b) 6% cement content.

The resilient modulus results of the control samples are shown in Figure 4.9. With increase in curing time, the resilient modulus values of all the combinations also increases. This increase is higher in material combinations containing lower RAP content. After 15, 25, 40 and 70 days of curing, the 7-day resilient modulus values of 50R_4C mix was found to be lower than 30R_4C mix by about 14%, 23%, 22% and 24% respectively. During the same periods, the resilient modulus values of 30R_4C mix was found to be lower than 0R_4C mix by about 20%, 16%, 16% and 14% respectively.

After 15, 25, 40 and 70 days of curing, the 7-day resilient modulus values of 50R_6C mix was found to be lower than 30R_6C mix by about 14%, 26%, 33% and 35% respectively. During the same periods, the resilient modulus values of 30R_6C mix was found to be lower than 0R_6C mix by about 33%, 27%, 22% and 20% respectively.

4.5.5 Effect of cement content

The resilient modulus data obtained for the control samples have been shown in Figure 4.9. It is observed that for all material combinations, an increase in cement content from 4% to 6% increased the resilient modulus values. For 100% RCCA, the 7-day resilient modulus of the 6% cement mix (0R_6C) is found to be about 17% higher than that of the 4% cement mix (0R_4C). Also for 30% RAP + 70% RCCA, the resilient modulus of the 30R_6C mix is found to be about 12% higher than that of the 30R_4C mix. Similar results were also obtained for the 50% RAP + 50% RCCA combination. Faysal (2017) conducted resilient modulus tests on RAP-RCCA materials mixes stabilized with 0, 2, 4 and 6% cement content. He concluded in this study that increasing the cement content from 4% to 6% does not have any significant effect on the resilient modulus values. This is more prominent at higher RAP content (Faysal, 2017).

The present study is focused on assessing the durability of these aggregate mixes. Control samples were thus prepared and tested after curing for specified periods. It is observed that after 15, 25, 40 and 70 days of curing, the resilient modulus values of 0R_6C is higher by about 28%, 32%, 36% and 34% than that of 0R_4C mix respectively. For 30% RAP + 70% RCCA, the resilient modulus values of 30R_6C after 15, 25, 40 and 70 days of curing were found to be higher than 30R_4C by about 15%, 20%, 28% and 25% respectively. During the same periods, the resilient modulus values of 50R_6C mix was found to be higher than 50R_4C mix by about 15%, 17%, 18% and 14% respectively.

In all cases, higher cement content results in an increase in resilient modulus values. However, this increase is higher at longer curing periods. Curing contributes towards further hydration of cement which results in higher stiffness of the material mix. The mix containing 6% cement content hydrates and gains strength at a higher rate than the mix containing 4% cement content. So longer the curing period, higher is the percentage increase in stiffness. However, this curing effect is diminished by the presence of RAP in the combination mix. This can be understood from the results of 30% RAP and 50% RAP mix. The difference in resilient modulus values between 50R_6C and 50R_4C remains almost constant even after 70 days of curing. This can be attributed to the fact that asphalt coatings of RAP materials do not bond as well with cement as does the RCCA materials.

Wetting-drying cycles were also conducted on the same material combinations and then tested for resilient modulus. For 0R_4C mix, the 7-day (0 W-D cycles) resilient modulus value increased by about 43% by the end of 16 W-D cycles. However, the resilient modulus value dropped by about 33% from 16 to 30 W-D cycles. This value is even lower than the 7-day resilient modulus value by about 5%. Also for the 0R_6C, the 7-day (0 W-D cycles) resilient modulus value increased upto 8 W-D cycles and then started to decrease. At the end of 30 W-D cycle, the M_R value was about 12% lower than the value at 8 W-D cycles.

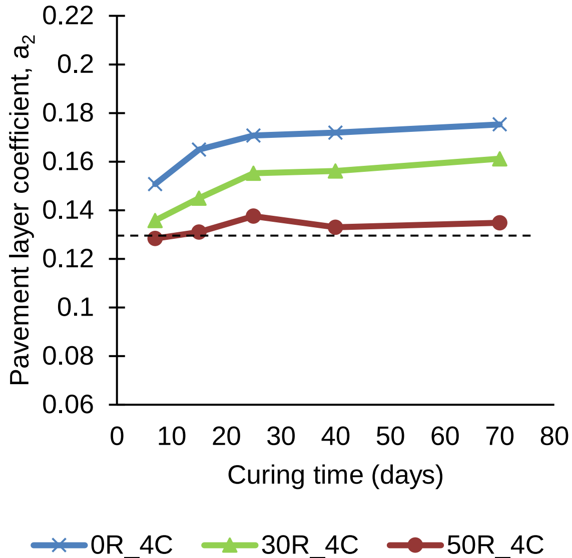
For the combinations containing 30% RAP content, the resilient modulus of 30R_6C mix increased upto 8 W-D cycles beyond which a drop was noticed upto 30 W-D cycles. The value obtained at the end of 30 W-D cycles was only about 2% lower than the initial 7-day value. However the resilient modulus of 30R_4C mix increased upto 4 W-D cycles. Futher wetting-drying process reduced the M_R values. The minimum value found after 30 W-D cycles was about 27% lower than the 7-day value. Similar trends were also observed for 50% RAP content. The lowest value of M_R for the 50R_4C mix was found after 30 W-D cycles which was about 25% lower than the initial 7-day value. This indicates that in the presence of wetting-drying cycles, increasing the cement content from 4% to 6% improves the durability properties of materials containing higher percentages of RAP content.

4.5.6 Pavement Layer Coefficient

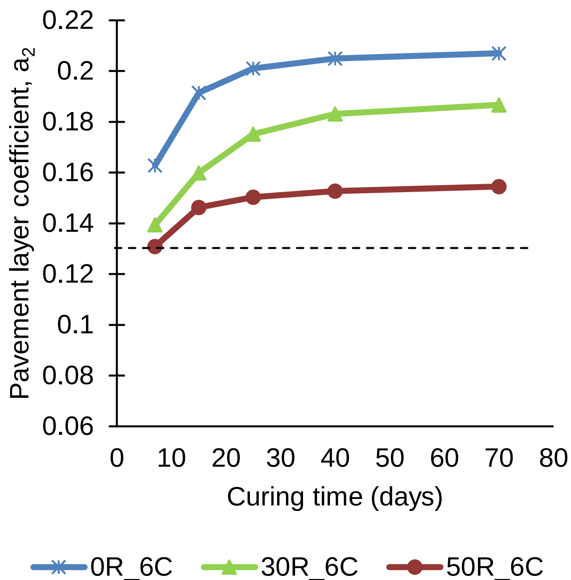
Resilient modulus (M_R) is a key parameter to determine the structural layer coefficient 'a_2' that is used for designing pavement base layer thickness. The layer coefficient a_2 was calculated in this study from M_R values using the correlation (AASHTO, 2003):

$$a_2 = 0.249*\log [M_R \text{ (psi)}]-0.977 \quad (1)$$

The resilient modulus (M_R) at a specific bulk stress of 30 psi (0.207 MPa) was used for this purpose. This stress level is considered ideal for pavement base layers in accordance with NCHRP 1-28A guidelines. Figure 4.14 shows a plot of the structural layer coefficient a_2 for the control samples of all material combinations. It is observed that higher values of a_2 is obtained for combinations having lower contents of RAP. Conventional curing has positive effects on the layer coefficient value for all cases. At the end of 70 days of curing, the final values of a_2 for 0R_4C, 30R_4C and 50R_4C were found to be 0.18, 0.16 and 0.13 respectively. For the mixes 0R_6C, 30R_6C and 50R_6C, these values were found to be 0.21, 0.19 and 0.15 respectively. It is also observed that increasing the cement content from 4% to 6% also increased the values of a_2 for all material combinations.

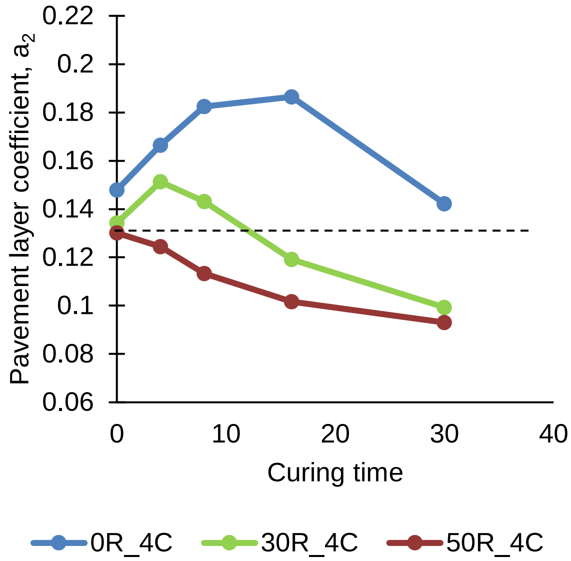


(a)

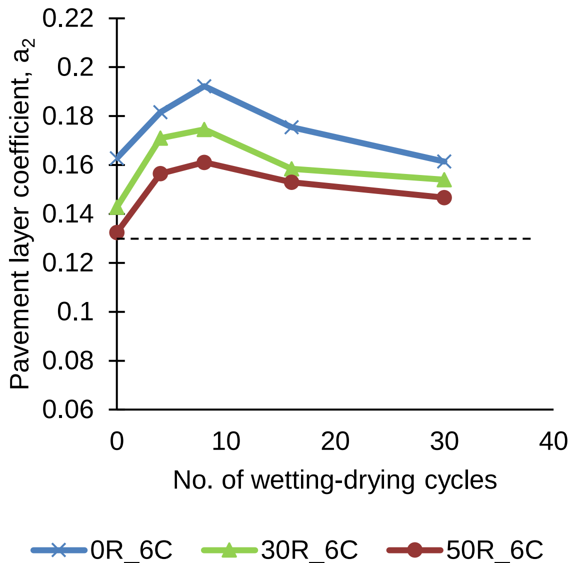


(b)

Figure 4.14: Effect of curing on pavement layer coefficient (a_2) of material combinations stabilized with (a) 4% and (b) 6% cement content.



(a)



(b)

Figure 4.15 Effect of wetting-drying cycles on pavement layer coefficient (a_2) of material combinations stabilized with (a) 4% and (b) 6% cement content.

The layer coefficient (a_2) values for the test samples showed a different trend. The variation of layer coefficient a_2 with wetting-drying (W-D) cycles for all combinations have been plotted in Figure 4.15.

But for the combinations 0R_4C, 30R_4C and 50R_4C final values of a_2 obtained after 30 W-D cycles is lower than the initial values obtained at 0 W-D cycles. In case of 50R_4C, the value of a_2 reduced to about 0.09. For flexible pavement base, a minimum value of 0.13 for layer coefficient a_2 is considered acceptable (Faysal et al. 2016*). It is thus evident that the 50R_4C becomes structurally incompetent for pavement base construction after 30 W-D cycles.

4.5.7 Effect of Moisture content

After each wetting process, the samples were rested to drain out the free-flowing water until a constant weight is reached. This constant weight was then used to determine the percentage of absorbed moisture in the samples. The percentages were calculated in terms of the original 7-day cured weight of the samples. For all material combinations, the moisture absorption rate is higher upto the first 6-8 wetting-drying cycles. This occurred maybe due to the hydration effect of the cement that tends to absorb more water. It is also evident from Figure 4.16 that the stiffness of the samples increased during the first 4-16 wetting-drying cycles for all the combination materials.

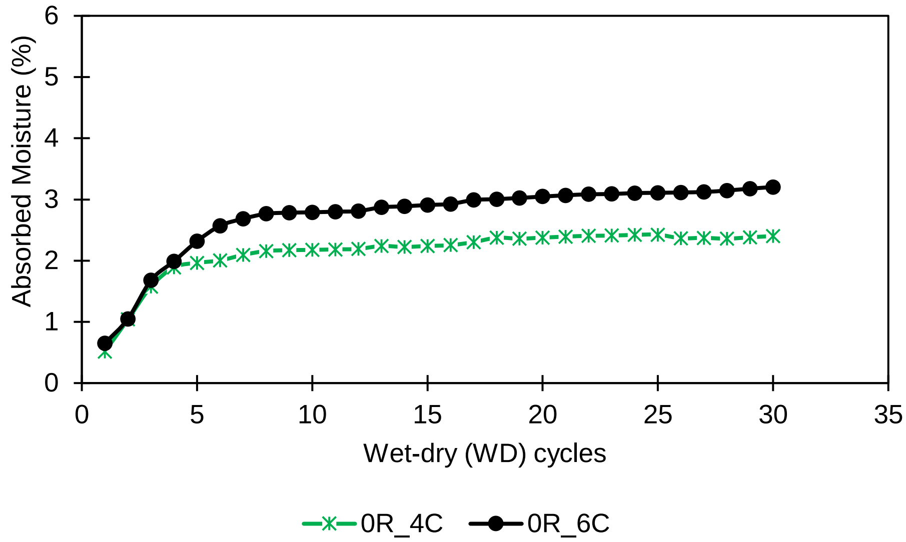


Figure 4.16 Absorbed moisture (%) with wetting-drying (W-D) cycles for 100% RCCA combinations.

Figure 4.17 shows the variation in absorbed moisture for the 100% RCCA combinations. Upto the first 7 W-D cycles, the added moisture content raised to about 2-2.8%. At the end of 30 wetting-drying cycles, the added moisture content was about 2.2-3%.

Addition of 30% RAP to the combinations increased the amount of moisture absorption. In case of the 30R_4C and 30R_6C samples, the moisture absorbed at the end of 30 W-D cycles was about 3% and 4% respectively.

Increasing the RAP content to 50% increased the moisture absorption by much. For the 50R_4C and 50R_6C combinations, the rate of moisture variation was high upto 8 W-D cycles. This is the same period during which resilient modulus values of these samples also increased due to the wetting-drying process (Figure 4.10). From 8 to 30 wetting-drying cycles, the absorbed moisture content varied from 4.7-5%. This value is higher than those obtained for the 100% RCCA and 30% RAP + 70% RCCA combinations.

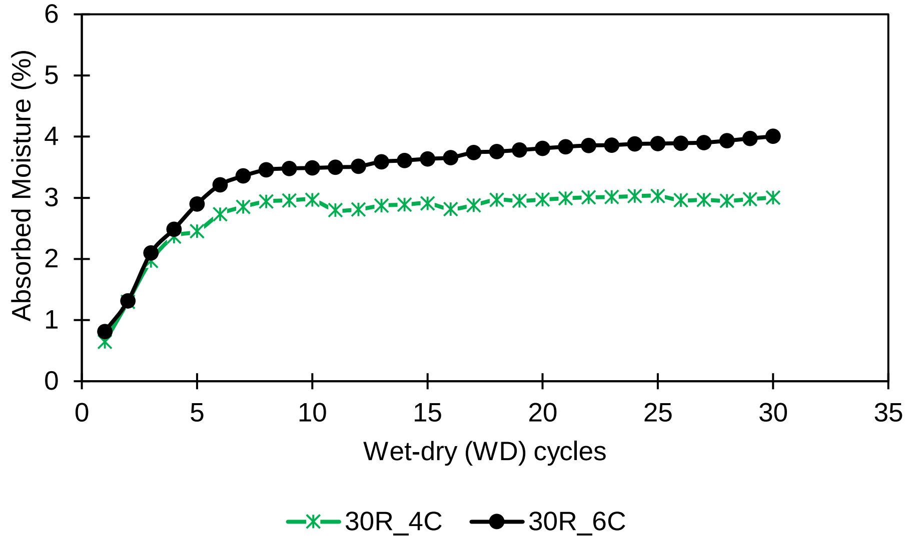


Figure 4.17 Absorbed moisture (%) with wetting-drying (W-D) cycles for 30% RAP + 70% RCCA combinations.

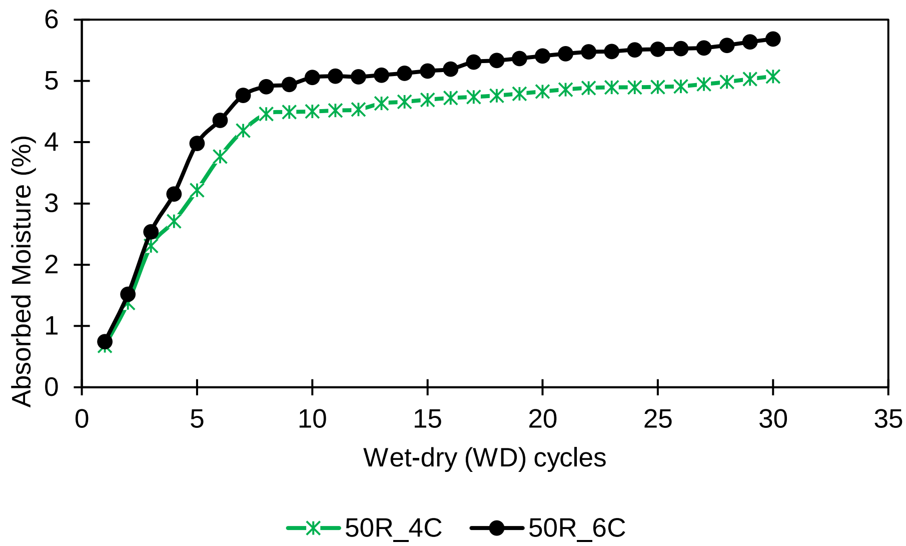


Figure 4.18 Absorbed moisture (%) with wetting-drying (W-D) cycles for 50% RAP + 50% RCCA combinations.

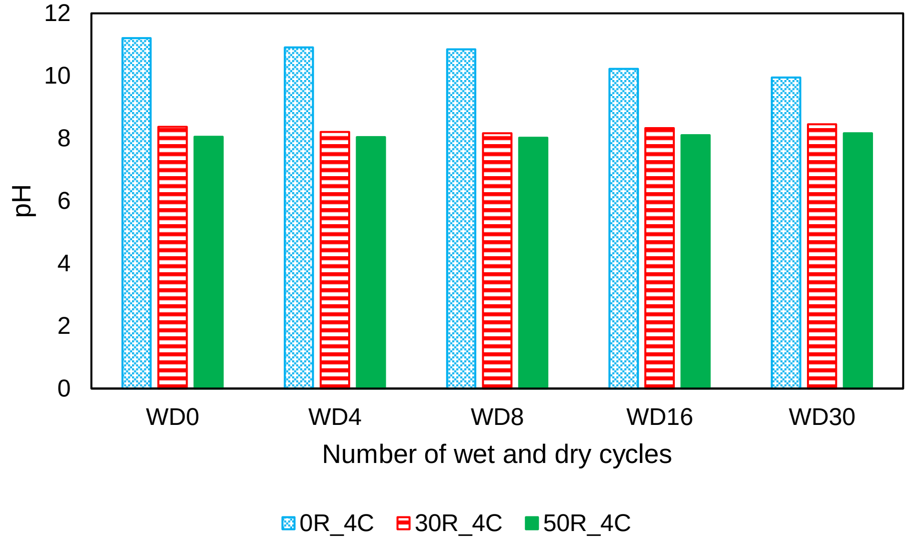
4.6 Environmental Test Results

Environmental tests were conducted on the test samples prepared from the RCCA-RAP material mix as shown in Table 2. These samples were subjected to 0, 4, 8, 16 and 30 WD cycles. At the end of these specified cycles, the samples were soaked in deionized water for 24 hours and leachate samples were collected. Environmental tests such as total suspended solids (TSS), and total dissolved solids (TDS), chemical oxygen demand (COD), turbidity and pH tests were conducted on the collected leachate samples as per the ASTM standard test methods.

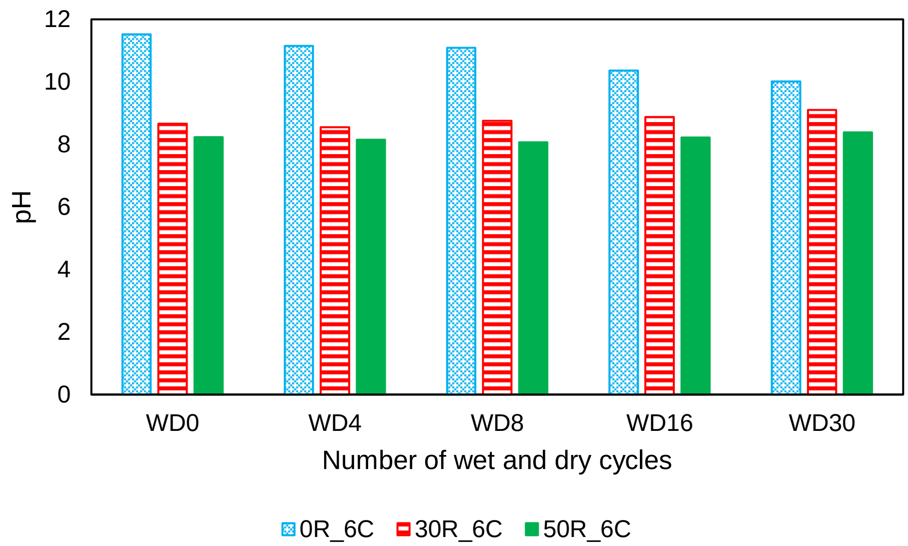
4.6.1 pH

pH is the measure of the acidity or alkalinity of water or leachate samples. pH test was performed in on the obtained leachate samples in accordance with ASTM D1287. The results obtained for the pH tests have been presented in Figure 4.19.

It can be observed that for all combinations, the value of pH shows a decreasing trend with increasing wetting-drying (W-D) cycles. Hydration reaction taking place between water and calcium carbonate forms soluble calcium hydroxide which might have caused the decrease in pH values (Faysal et al. 2017a). This hydration process improves bonding between aggregates which in return prevents flowing out of calcium ions and thus reduces pH is the water (Puppala, 2017).



(a)



(b)

Figure 4.19 Change in pH with wetting-drying (WD) cycles for materials stabilized at (a) 4% and (b) 6% cement content.

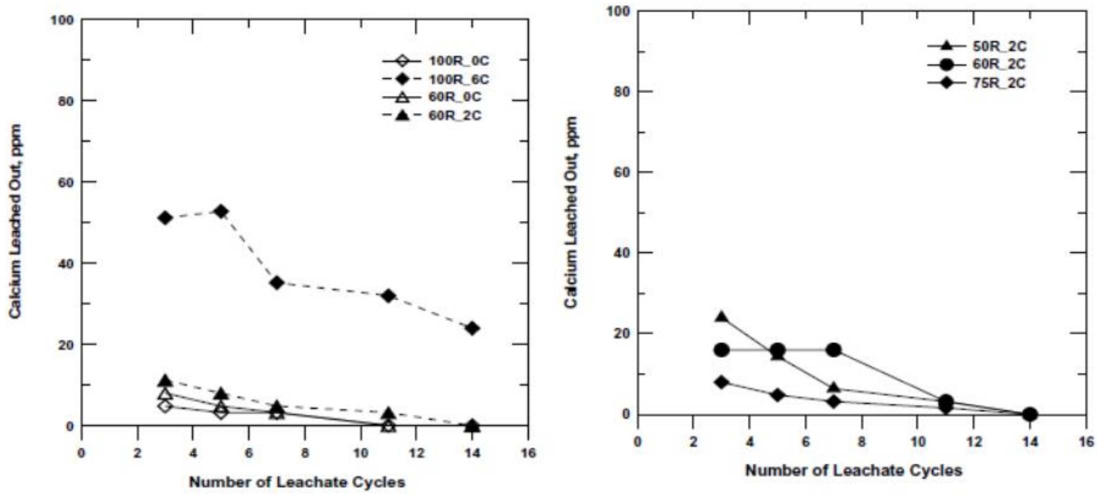


Figure 4.20 Calcium leached out at different leachate cycles (Puppala 2017)

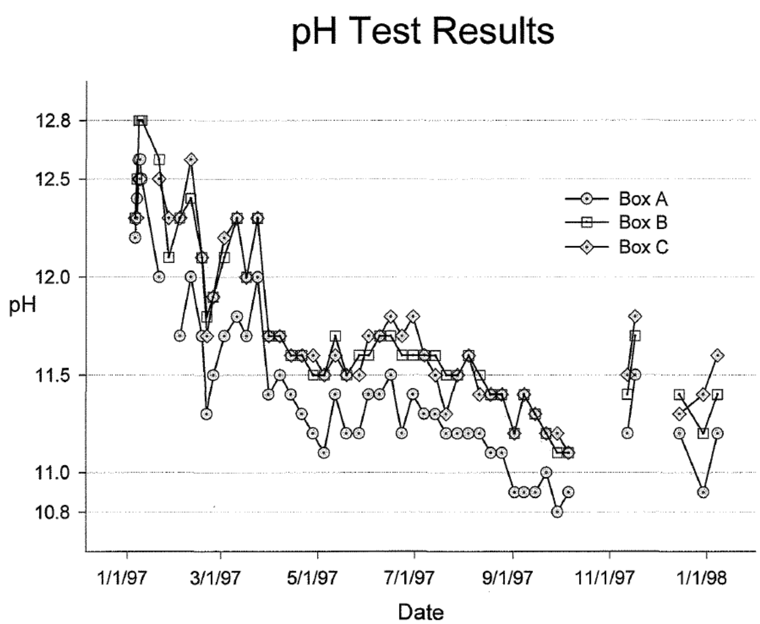


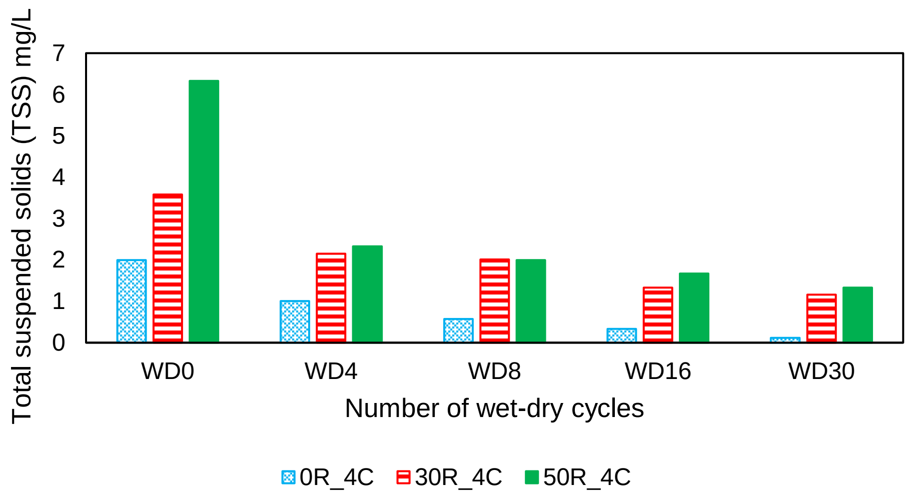
Figure 4.21 Variation of pH with time for recycled PCC (Steffes, 1999).

Test results also indicate that pH value obtained for the 30% RAP + 70% RCCA and 50% RAP + 50% RCCA combinations varies from 7.6 to 8.2. This is within range of 6 to 9 as per EPA guidelines for storm water sampling (EPA 2005). However, for the 100% RCCA material, the pH value remains above 10 even after 30 wetting-drying (W-D) cycles. The highest value of 11.8 is obtained for 0R_6C mix at 0 W-D cycles. Steffes (1999) studied the pH characteristics of recycled crushed PCC as drainable base material. He observed pH values as high as 12.5 which gradually dropped to about 11.5 after 10 weeks (70 days) of testing.

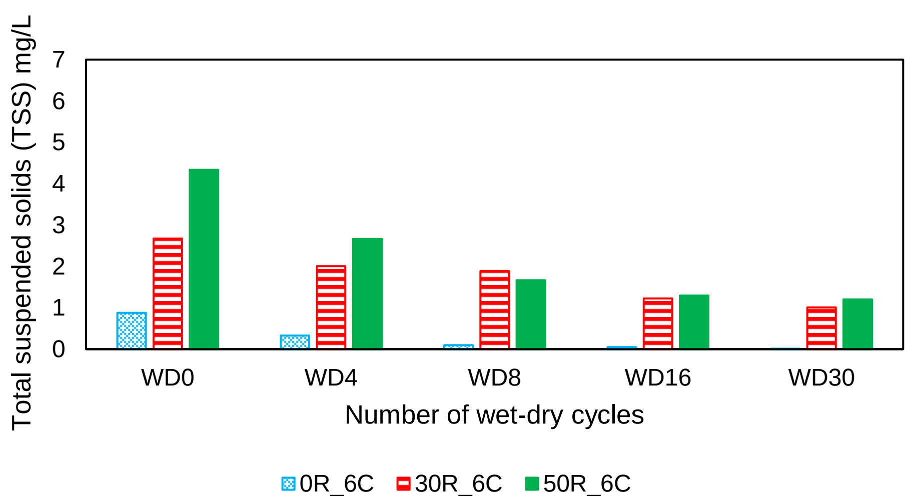
The 100% RCCA material does not meet the EPA (2005) guideline for storm water. However, this test procedure was conducted on rested water in contrast to the actual field conditions where storm water will be on the flow. These lab results are thus very much conservative. (Hoyos et. al. 2011)

4.6.2 Total Suspended (TSS) and Dissolved (TDS) Solids

Total suspended solids (TSS) test was conducted on the leachate samples in accordance with ASTM D5907-13 standard method for non-filterable matter. TSS test results show that the value of TSS in leachate samples decreases by 80% and 83% for MIX4 and MIX6 respectively with the increase in the number of wet and dry cycles from 0 to 30. This might have occurred because of the improved inter particle bond and well developed matrix due to hydration of cementitious materials within the specimens. According to the test results, the value of TSS is higher for the leachate samples obtained for MIX4 than that for MIX6 which might have attributed to the fact that the specimens prepared using 4% cement content is weaker than the prepared specimens at 6% cement content as shown in FIG. 6.

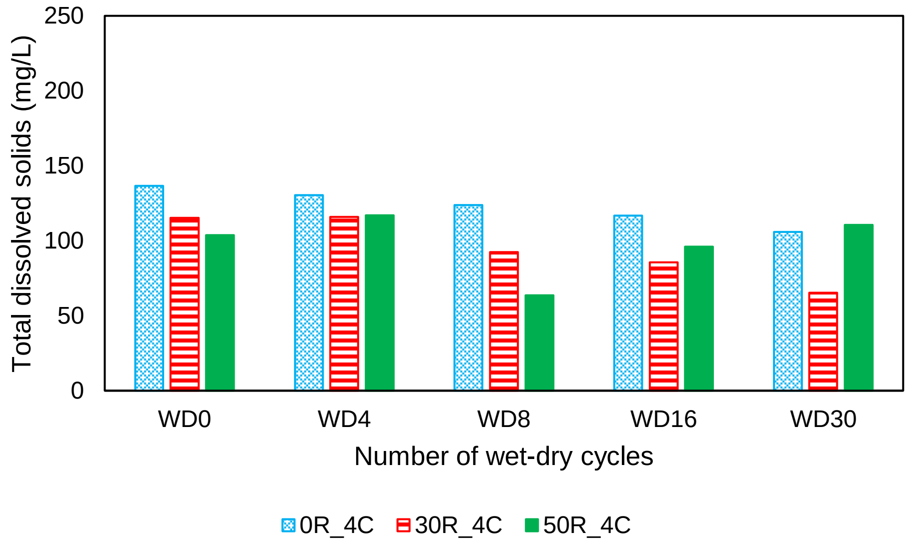


(a)

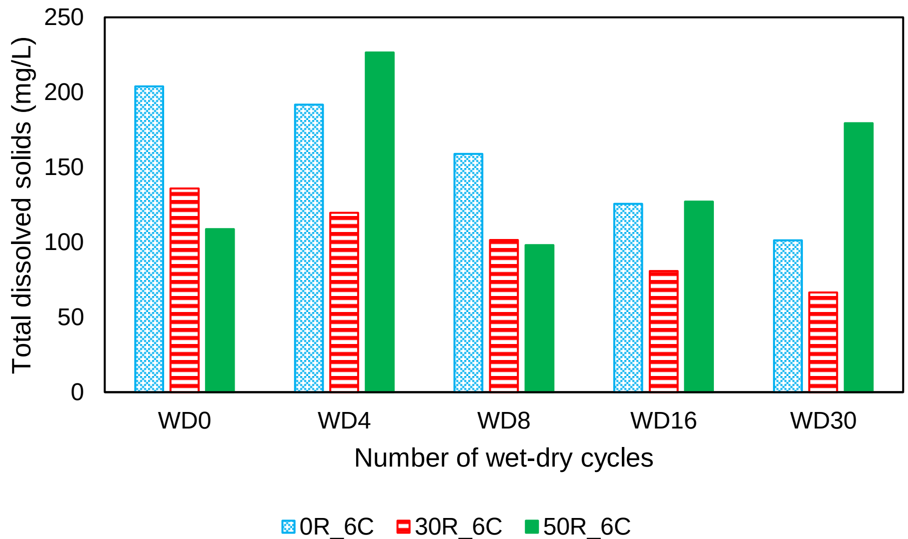


(b)

Figure 4.22 Change in total suspended solids (TSS) with wetting-drying (WD) cycle.



(a)



(b)

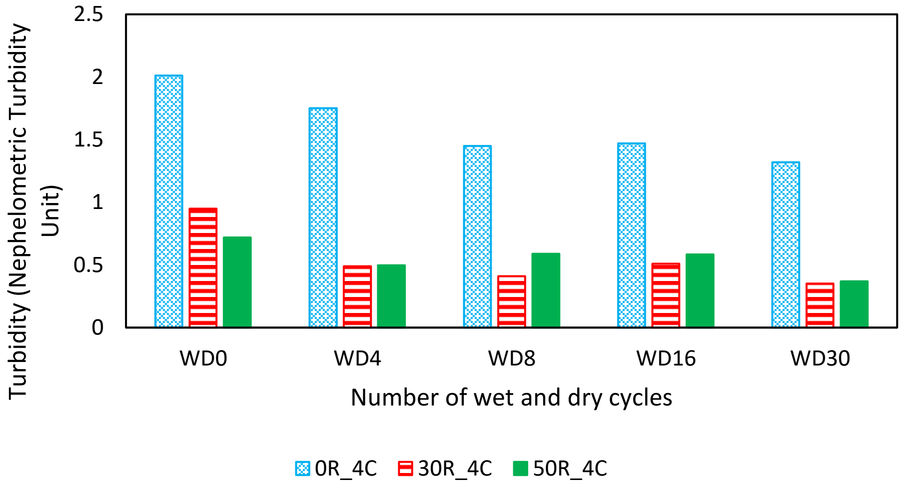
Figure 4.23 Change in total dissolved solids with wetting-drying (WD) cycle

Total dissolved solids (TDS) or filterable parameter is one of the important parameters for the treatment of raw water, wastewater and in monitoring of streams. TDS tests were performed in accordance with the ASTM D5907-13 standard method for filterable matter. Filtrate obtained after passing the leachate through the glass fiber filter paper of 1.5 µm nominal pore size. Dissolved solids represent the amount of cementitious materials washed out from the specimen due to its reaction with water (Faysal et al. 2017a). TDS tests were conducted on the collected leachate samples after selected number of wet and dry cycles such as 0, 4, 8, 16, and 30.

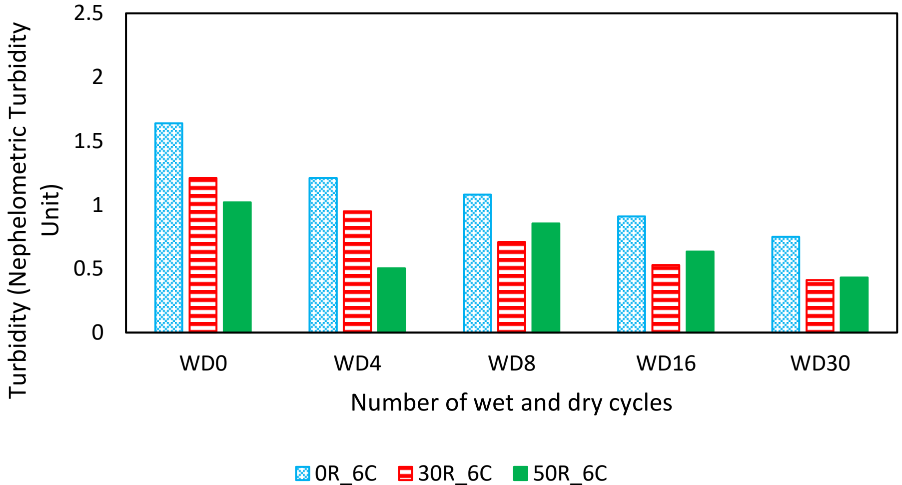
The value of TDS changes with the cement content and number of wet and dry cycles (FIG. 7). The lower cement content results in lower value of TDS than the specimens contain higher cement content. TDS value obtained at different cycles follows similar trend as COD. TDS values increase with increase in wet-dry cycles until 4 after that it reduces after completion of 8 wet-dry cycles. During the first 4 wet-dry cycles the value of TDS increase due to rapid hydration process. The value of TDS increases by 75% and 83% for MIX4 and MIX6 samples respectively with the increase in WD cycles from 8 to 30 cycles. The increased value of TDS denotes the degradation of strength of specimens, which complies with trend obtained from resilient modulus test results at different number of wet and dry cycles. TDS test results are well within the limit of 500 $x10^{-3}$ kg/m^3 as per EPA guidelines (EPA 2005).

4.6.3 Turbidity

Turbidity is a parameter to determine the amount of suspended matter such as soil particles, different types of organic and inorganic matter and microorganisms present in water. Turbidity was measured using a HACH 2100P Model portable turbidimeter that operates on the nephelometric principle of turbidity measurement in Nephelometric Turbidity Unit (NTU). This equipment measures the optical property of water such as the amount of light scattered and absorbed while passing through the water sample. The variations in turbidity test results due to different combinations of cement-stabilized materials are included in FIG. 8



(a)



(b)

Figure 4.24 Change in turbidity with wetting-drying (WD) cycle

. It can be inferred that the value of turbidity decreases with increase in cement content. The maximum value of turbidity is 1 NTU that is less than 5 NTU. This satisfies the EPA guidelines.

4.6.4 Chemical Oxygen Demand (COD)

The COD concentration in the leachate samples was determined using Spectronic 20D+ model Spectrometer. Changes in COD with different number of wet and dry cycles is shown in FIG. 5. Test results indicate that the value of COD increases until 4 wet-dry cycles and COD decreases from 4 to 8 wet–dry cycles. MIX4 samples release higher concentration of chemical when subjected to 4 wet-dry cycles after that there is a reduction in the released chemical compound. The amount of chemical compound releases into the leachate for MIX6 samples up to 4 wet-dry cycles is higher than the cycles from 4 to 8 wet-dry cycles which might have occurred because of the reduction in the rate of hydration. The value of COD obtained for MIX4 samples is less than the MIX6 samples which indicates that specimens stabilized with higher cement content releases more chemical compound in the leachate.

The value of resilient modulus decreases with increase in the number of WD cycles. Khoury and Zaman (2007) reported that this reduction in strength is due to the adverse effects of repeated wetting-drying processes on binding properties of cement. The value of resilient modulus decreases after the completion of 8 WD cycles for the specimens stabilized using 4% and 6% cement. The decrease in the resilient modulus shows the degradation of the specimens which led to the increased value of COD. From 8 to 30 wet-dry cycles the value of COD increased by 1.5 and 2.2 times for MIX4 and MIX6 samples respectively. The value of COD obtained is less than 120 mg/L which is within the EPA guidelines.

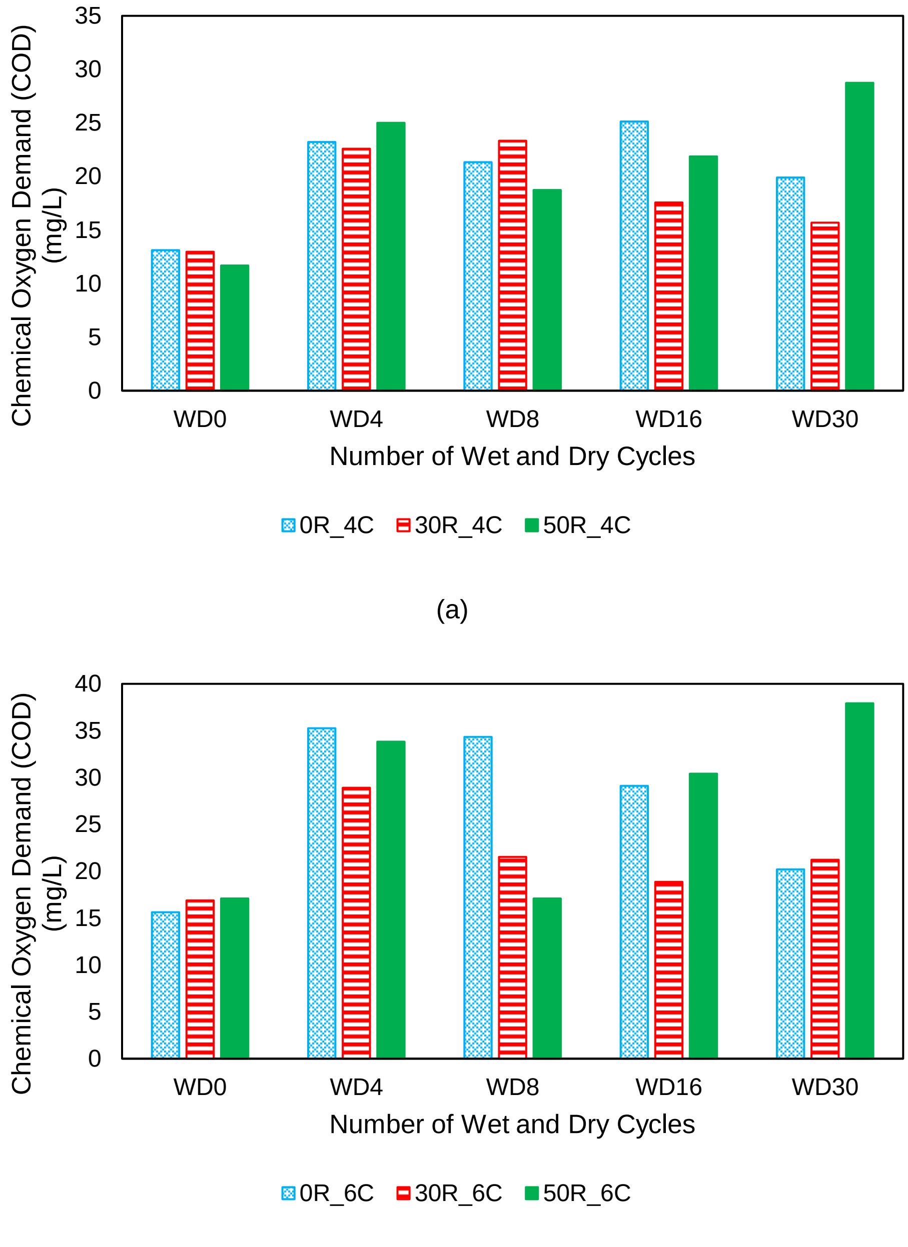


Figure 4.25 Change in chemical oxygen demand (COD) with wetting-drying (WD) cycles.

Chapter 5

Conclusion and Future Recommendations

Use of recycled crushed concrete aggregate (RCCA) and reclaimed asphalt pavement (RAP) as an alternative to virgin aggregates in pavement constructions is not yet a common practice in the US. The reason behind this is the lack of proper design guidelines. The most accepted design guideline was developed by American Association for State Highway and Transportation Officials (AASHTO 2003) for virgin aggregates. Considering the highly variable nature of recycled materials, Faysal (2017) developed a design chart for using RAP and RCCA materials in pavement base constructions. Faysal (2017) also added that long-term performance of these materials under various field conditions might affect the design considerations. The overall objective of the present study was to evaluate the long-term performance of cement treated RAP-RCCA material combinations when subjected to repeated wetting-drying (W-D) cycles. Resilient modulus (M_R) tests were conducted to monitor the variation of stiffness properties of these materials. For the purpose of comparison, similar tests were conducted on a different set of samples of the same material combinations subjected to conventional curing for the same period of time. Environmental tests were also conducted to check the possible deterioration of storm water quality. According to the test results, RCCA is a more durable materials than RAP when subjected to wetting-drying processes. All the six material combinations used in this study showed adequate strength after 7-days of curing. But two out of these six combinations failed to meet the AASHTO specifications for pavement base layer after 8-16 wetting-drying cycles.

5.1 Summary and Conclusion

A summary of conducted research test results are as follows:

1. The RCCA and RAP materials used for this study were collected from Big City Crushed Concrete and Rockwell County stockpiles respectively. These are specific stockpile sites from where recycled materials are collected for different projects conducted by TxDOT.
2. Basic engineering tests were conducted on the collected RAP and RCCA materials for characterization. These tests include grain-size distribution, specific gravity and maximum dry density tests. Effects of these properties on the long-term and short-term strength of the materials combinations has also been studies.
3. Three different combinations of RAP and RCCA materials containing 0%, 30% and 50% RAP content were selected for this study. These materials were then stabilized with 4% and 6% cement content. Selection of these percentage composition was done based on previous studies and to ensure maximized use of RAP and RCCA materials.
4. Samples prepared from these material combinations were subjected to two types of long-term effects – conventional curing and wetting-drying (W-D) cycles. At the end of specified curing and W-D cycles, the samples were tested for resilient modulus (M_R). All the M_R data were fitted in prediction models as per AASHTO requirements. According to previous studies, the maximum bulk stress that can be achieved in the field was 30 psi. For this reason, the value used to compare the resilient modulus was kept limited to bulk stress of 30 psi. Environmental tests were also conducted to monitor the quality of the washed-out water.
5. Resilient modulus of all the RCCA-RAP materials combinations increased with increasing curing periods. The 7-day resilient modulus increased by about 50% for the

100% RCCA combination after 70 days of curing. However, this increase was as low as 9% when 50% RAP content was added to the mix.

6. The resilient modulus values of all the material combinations initially increased upto a certain number of wetting-drying (W-D) cycles. This is due to the prolonged cement hydration from moisture intrusion. For 100% RCCA combination, the M_R values increased by about 31-43% after 8-16 W-D cycles. However, in case of mix with 50% RAP content, M_R values increased by 25-30% during the initial 4-8 W-D cycles.
7. At all cases, resilient modulus (M_R) reduces with inclusion of RAP into the mix with RCCA materials. The value of 7-day M_R decreases by 23% with inclusion of 50% RAP at any cement content. However, this reduction in M_R was about 39% by the end of 70 days of curing.
8. Higher cement content resulted in higher stiffness in the combination mix. Increasing the cement dosage from 4% to 6% increased the 7-day M_R and 70-day M_R values by about 15% and 20-35% for all material mix respectively. This indicates effect of cement content is more pronounced after longer curing periods. When subjected to wetting-drying (W-D) cycles, increasing cement content enhanced the durability of mix containing 30% and 50% RAP content.
9. Pavement layer coefficient (a_2) values were calculated using the AASHTO (2003) equation. AASHTO specifications require a minimum a_2 value of 0.13 for pavement base materials. For all combination mix, the a_2 values obtained after 7 days of curing was more than 0.13. However, for the 30% RAP mix stabilized at 4% cement content (30R-4C), the a_2 value dropped below 0.13 from 8-16 W-D cycles. Also for the 50% RAP mix at 4% cement content (50R_4C), the a_2 value was lower than 0.13 only after 4-8 W-D cycles. This indicates that the 30R_4C and 50R_4C material combinations are not structurally durable when subjected to long-term wetting-drying processes.

10. The environmental effects of using these recycled materials have also been evaluated. Environmental tests were conducted on the water collected after submerging the samples for 24 hours. Environmental tests include chemical oxygen demand (COD), total suspended solids (TSS), total dissolved solids (TDS), turbidity and pH. Even after 30 wetting-drying (W-D) cycles, the test results were well within the range of EPA guidelines for storm water sampling.

5.2 Recommendation for Future Study

1. The current study was conducted using cement as stabilizer agent. However, foam asphalt can be used for future study as an alternative stabilizer.
2. Effect of asphalt might be neutralized using sand. A future study can incorporate sand to neutralize effect of asphalt.
3. The MLR model was developed and verified using laboratory testing results. A future study might incorporate field test section where resilient modulus can be obtained using falling weight deflectometer (FWD). The field test data might be used to evaluate the model and add additional factors as required.
4. The current study was conducted only for the recycled materials available in north Texas region. The similar study can be conducted on the recycled materials available throughout the region to make a comprehensive design chart for entire Texas.

APPENDIX A

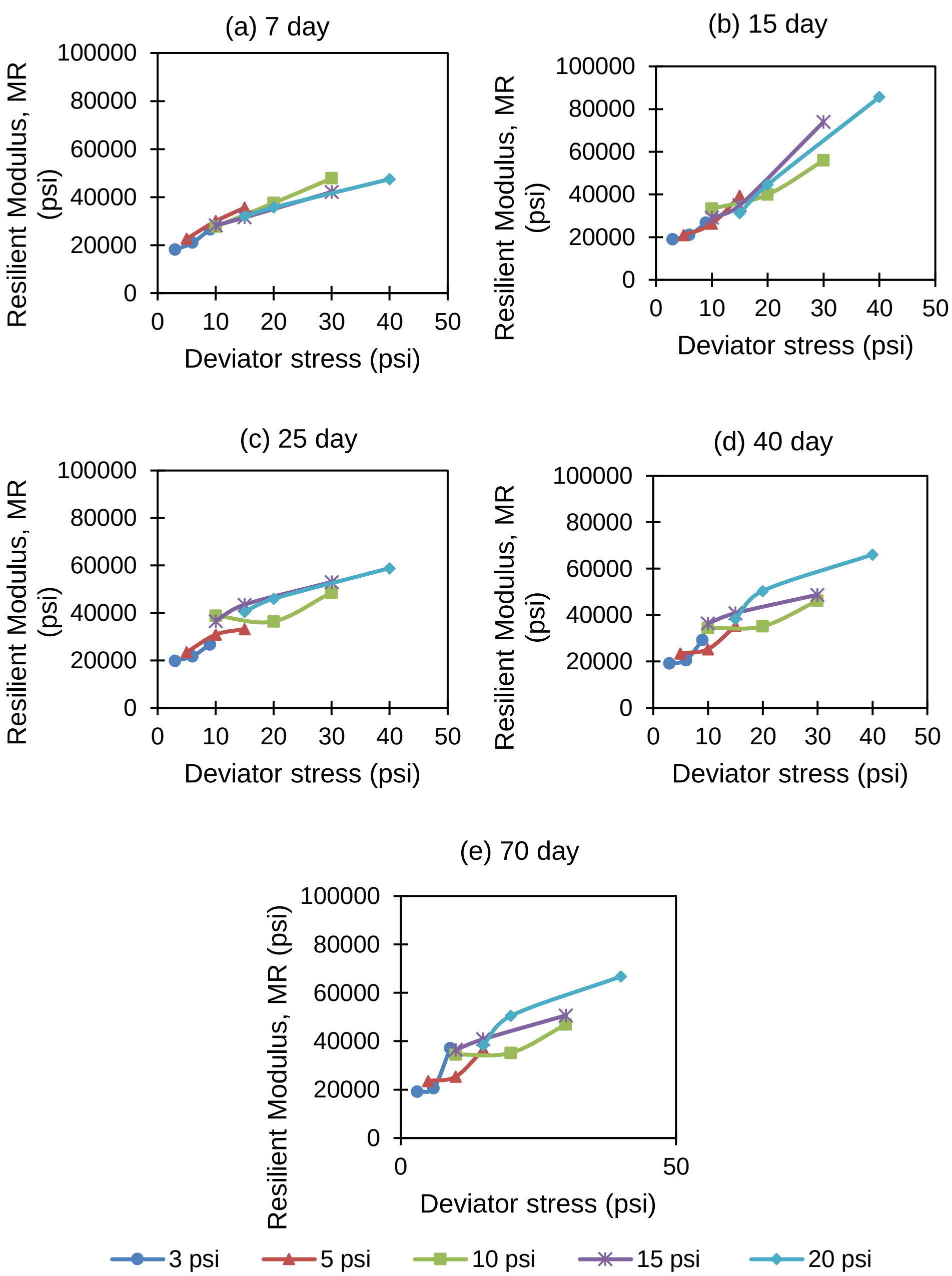

(a) 7 day
(b) 15 day
(c) 25 day
(d) 40 day
(e) 70 day
Resilient Modulus, MR (psi)
Deviator stress (psi)
100000
80000
60000
40000
20000
0
0
10
20
30
40
50
3 psi
5 psi
10 psi
15 psi
20 psi

Figure A-1 Resilient modulus test results at confining stresses of 3, 5, 10, 15 and 20 psi for the control samples of mix 0R_4C after (a) 7, (b) 15, (c) 25, (d) 40 and (e) 70 days.

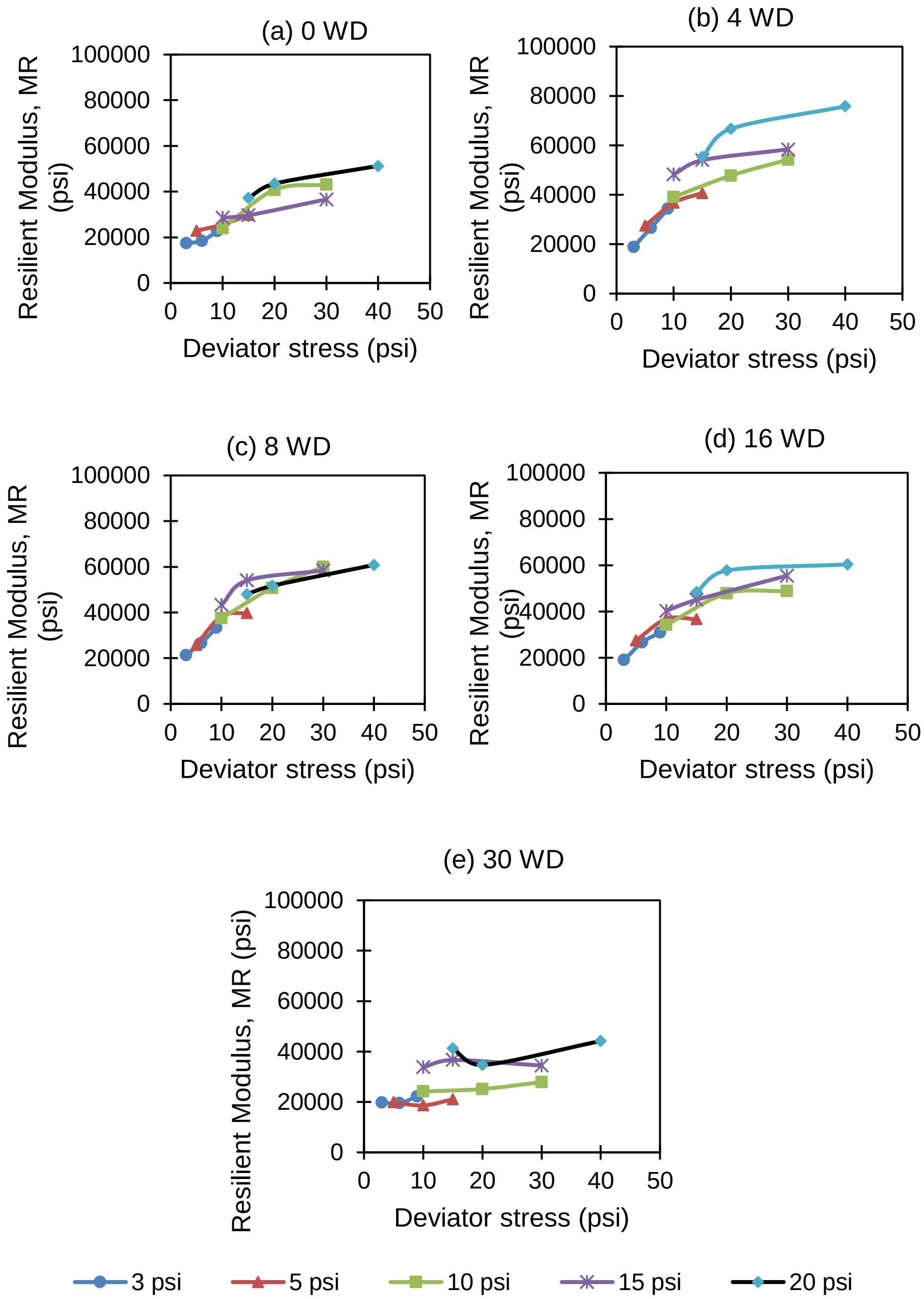


Figure A-2 Resilient modulus test results at confining stresses of 3, 5, 10, 15 and 20 psi for the test samples of mix 0R_4C after (a) 0, (b) 4, (c) 8, (d) 16 and (e) 30 W-D cycles.

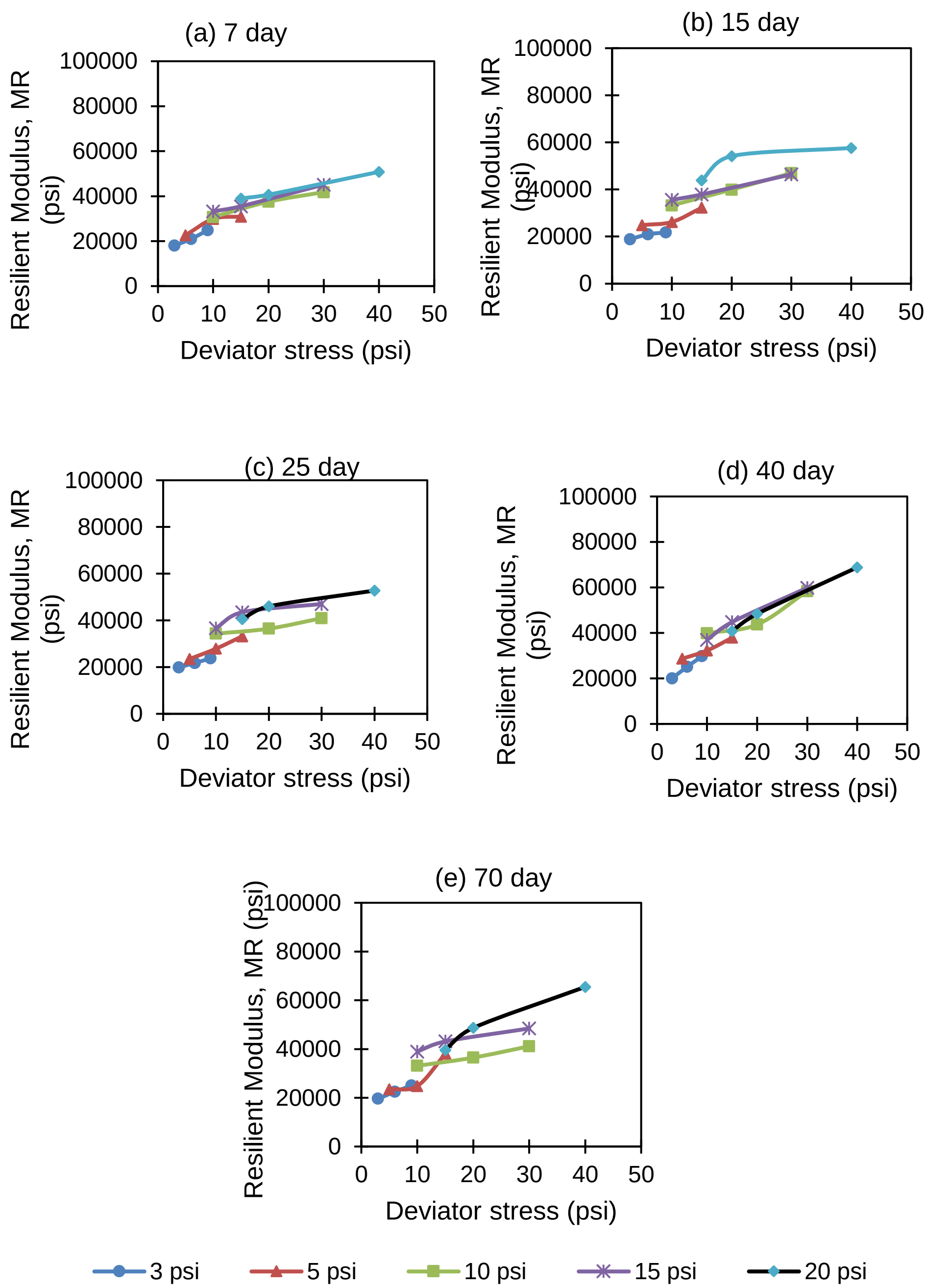


Figure A-3 Resilient modulus test results at confining stresses of 3, 5, 10, 15 and 20 psi for the control samples of mix 0R_6C after (a) 7, (b) 15, (c) 25, (d) 40 and (e) 70 days.

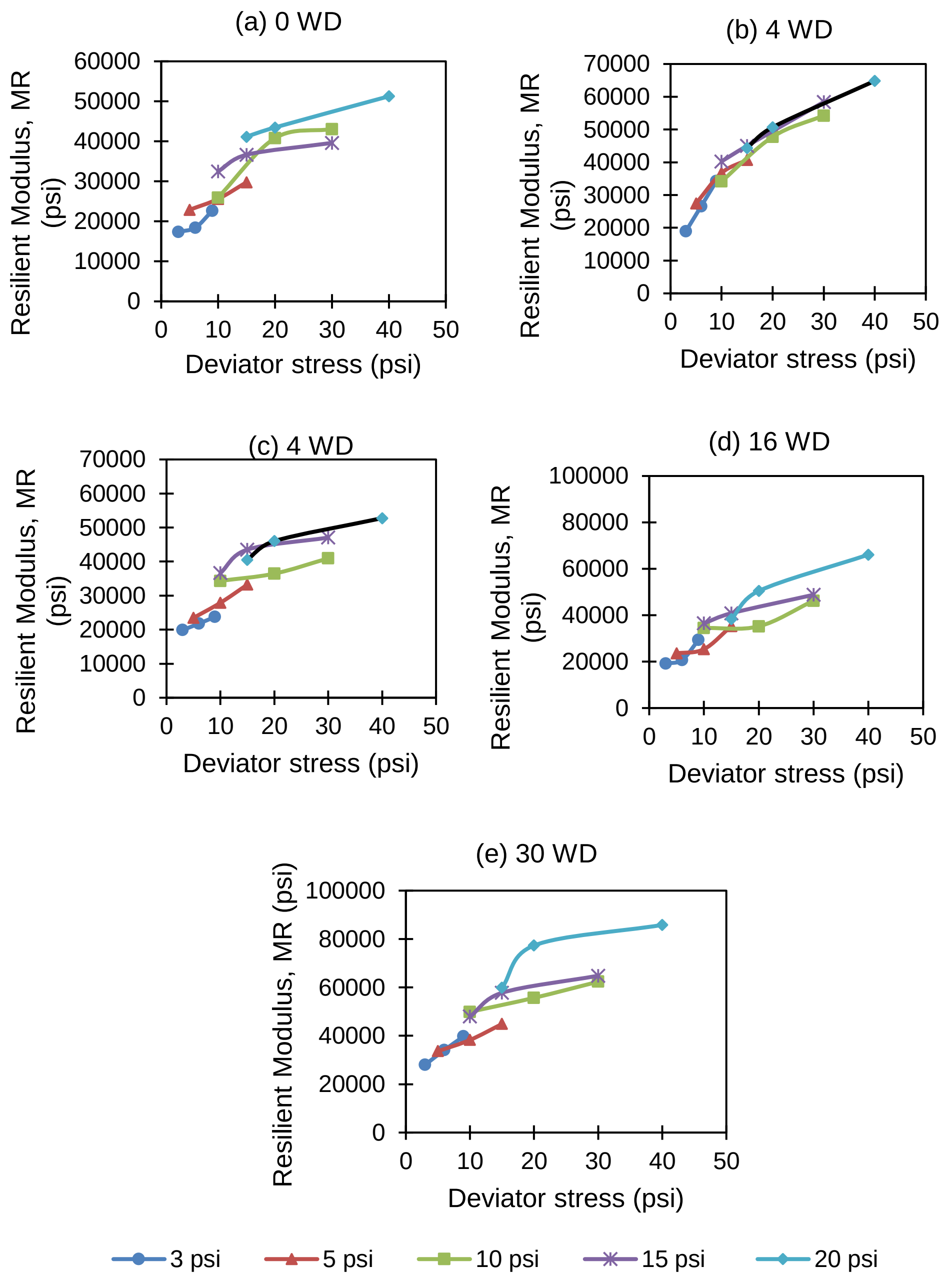


Figure A-4 Resilient modulus test results at confining stresses of 3, 5, 10, 15 and 20 psi for the test samples of mix 0R_6C after (a) 0, (b) 4, (c) 8, (d) 16 and (e) 30 W-D cycles.

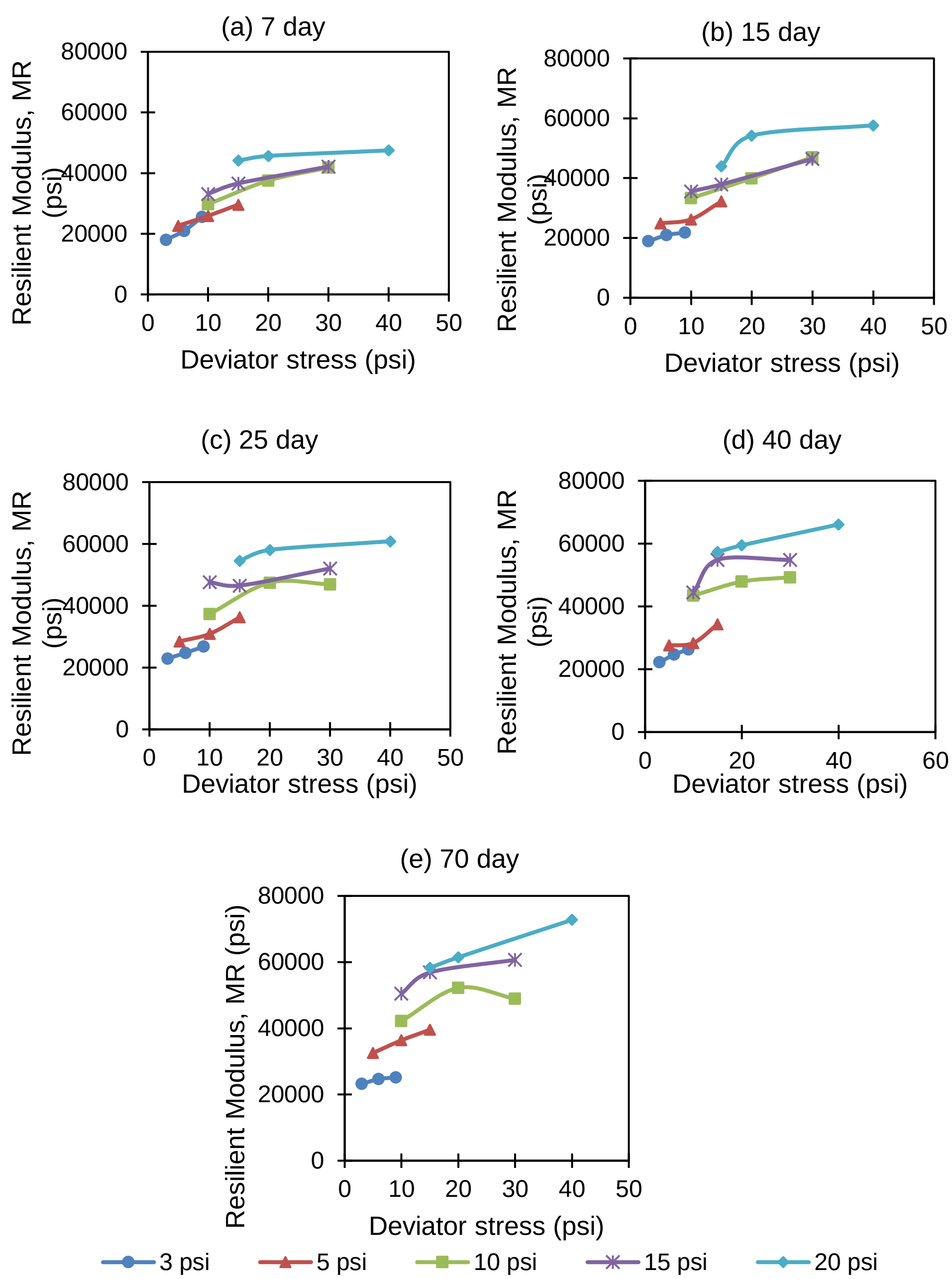


Figure A-5 Resilient modulus test results at confining stresses of 3, 5, 10, 15 and 20 psi for the control samples of mix 30R_4C after (a) 7, (b) 15, (c) 25, (d) 40 and (e) 70 days.

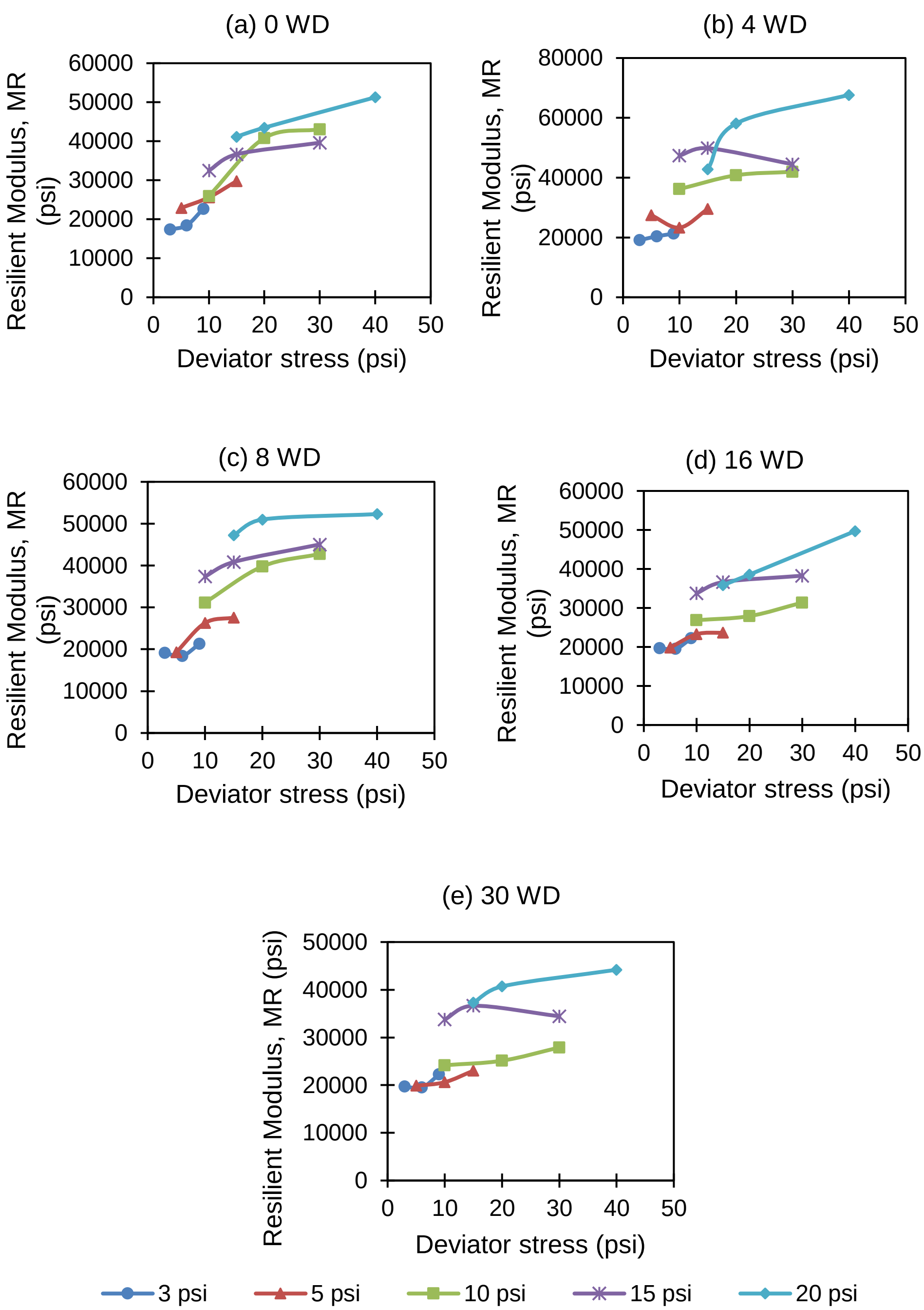


Figure A-6 Resilient modulus test results at confining stresses of 3, 5, 10, 15 and 20 psi for the test samples of mix 30R_4C after (a) 0, (b) 4, (c) 8, (d) 16 and (e) 30 W-D cycles.

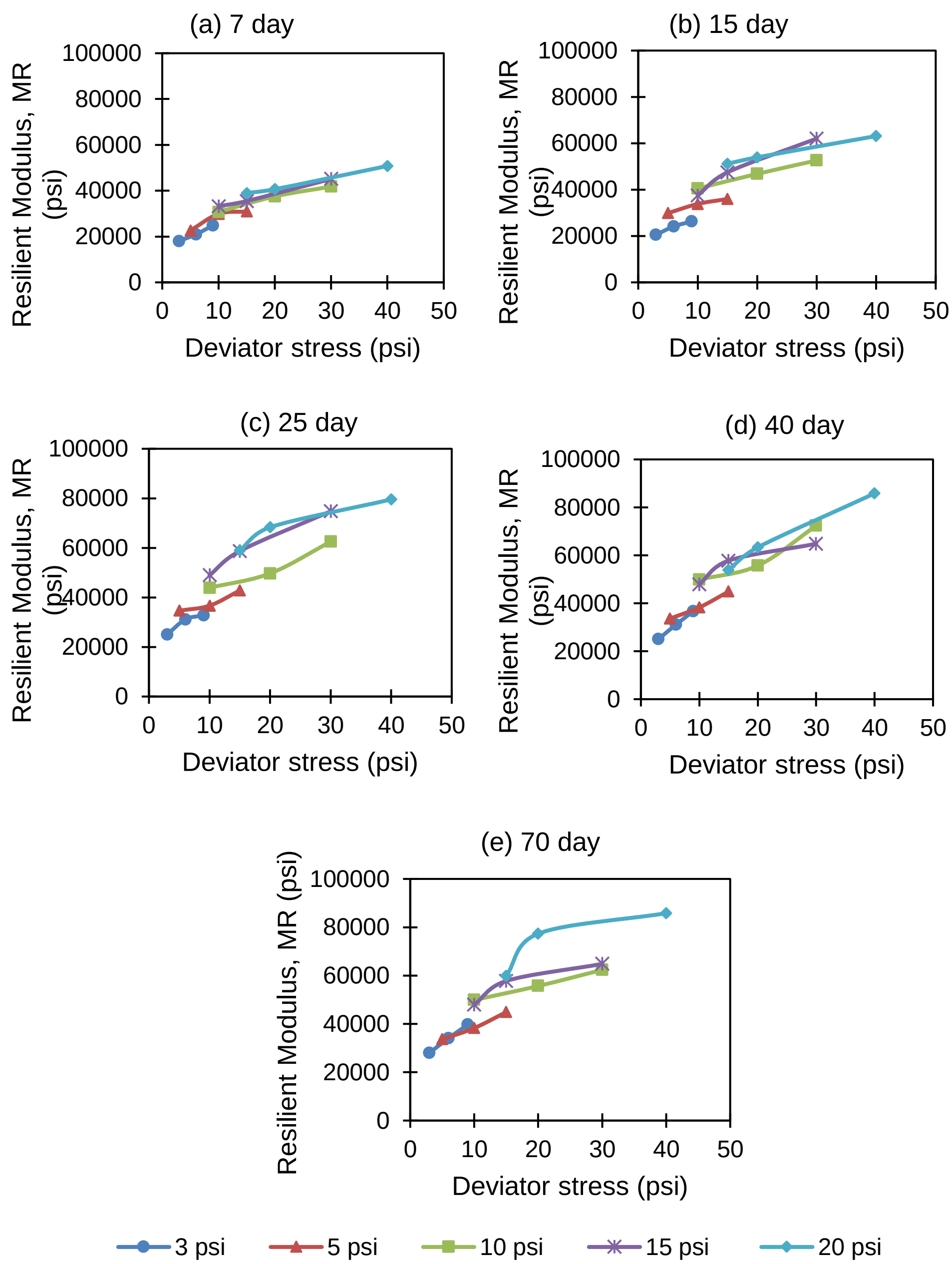


Figure A-7 Resilient modulus test results at confining stresses of 3, 5, 10, 15 and 20 psi for the control samples of mix 30R_6C after (a) 7, (b) 15, (c) 25, (d) 40 and (e) 70 days.

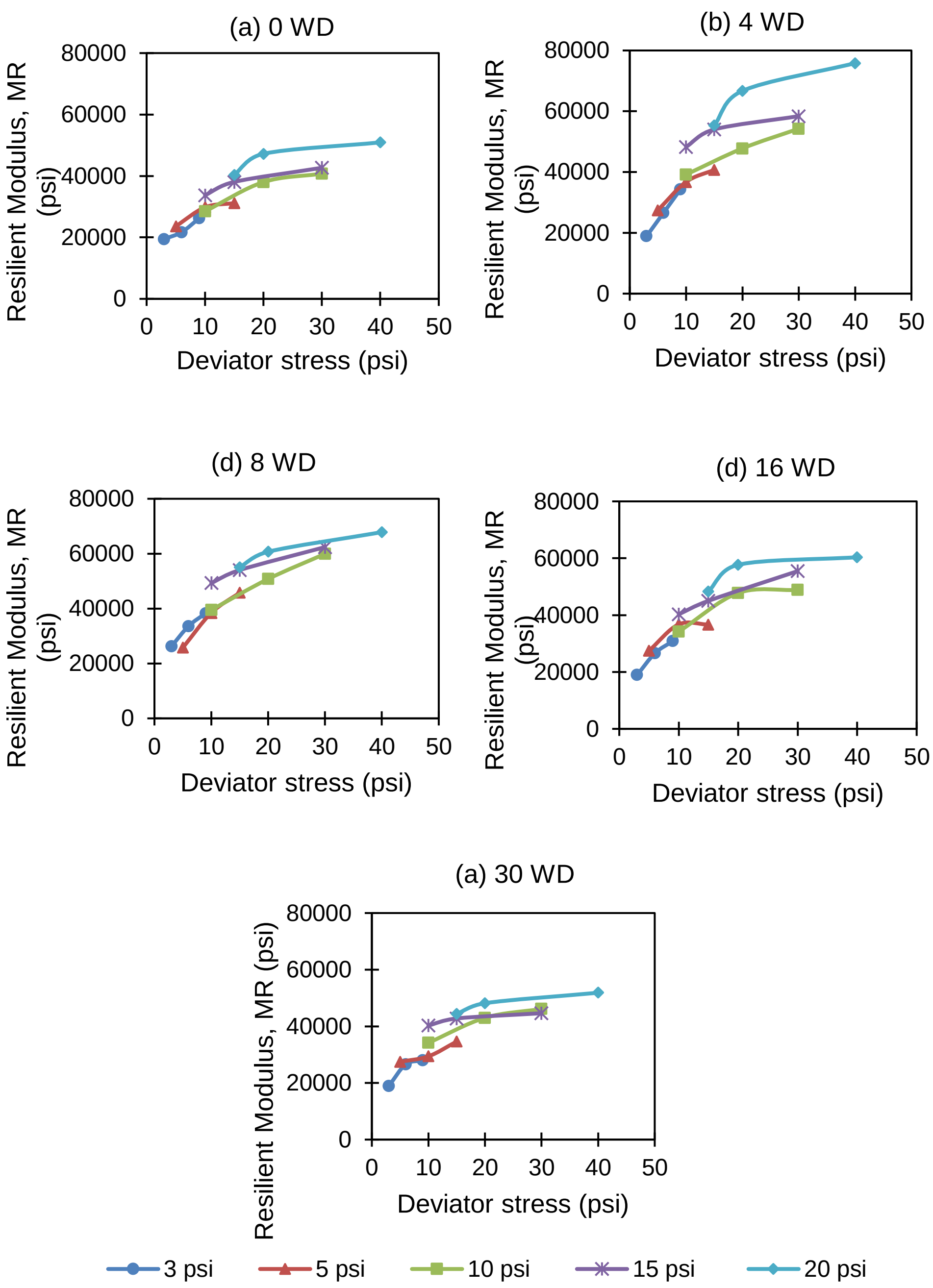


Figure A-8 Resilient modulus test results at confining stresses of 3, 5, 10, 15 and 20 psi for the test samples of mix 30R_6C after (a) 0, (b) 4, (c) 8, (d) 16 and (e) 30 W-D cycles.

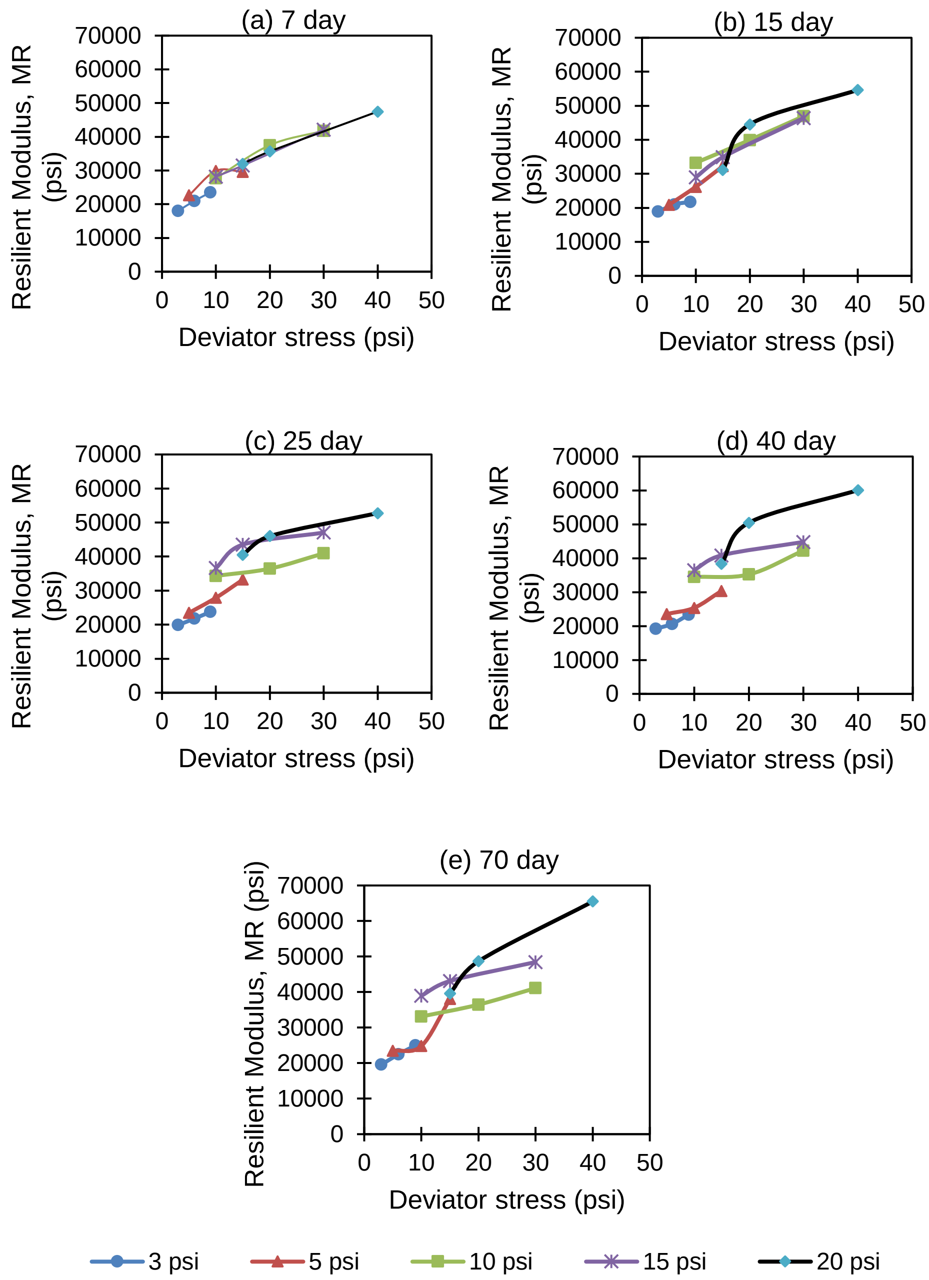


Figure A-9 Resilient modulus test results at confining stresses of 3, 5, 10, 15 and 20 psi for control samples of mix 50R_4C after (a) 7, (b) 15, (c) 25, (d) 40 and (e) 70 days.

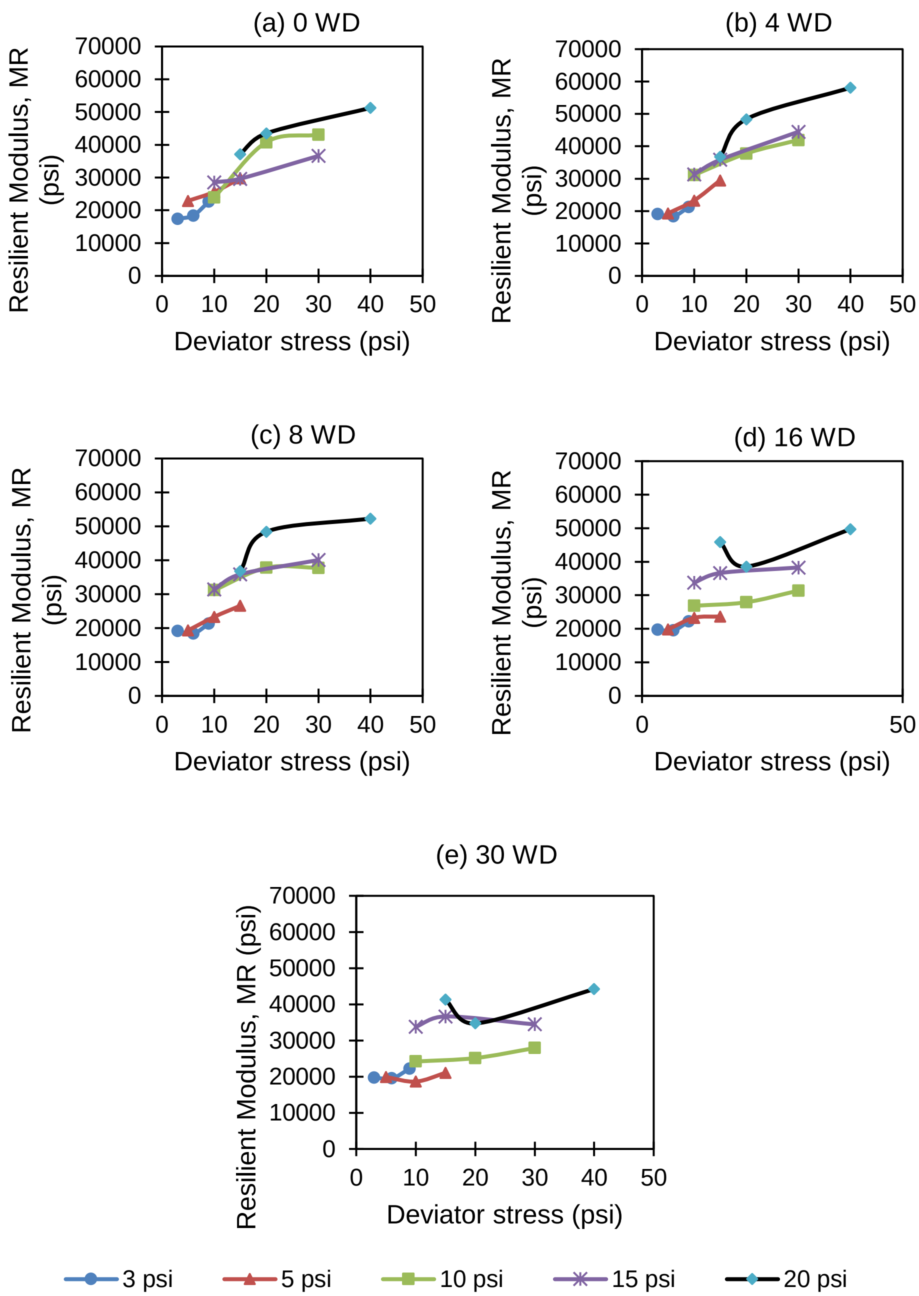


Figure A-10 Resilient modulus test results at confining stresses of 3, 5, 10, 15 and 20 psi for the test samples of mix 50R_4C after (a) 0, (b) 4, (c) 8, (d) 16 and (e) 30 W-D cycles.

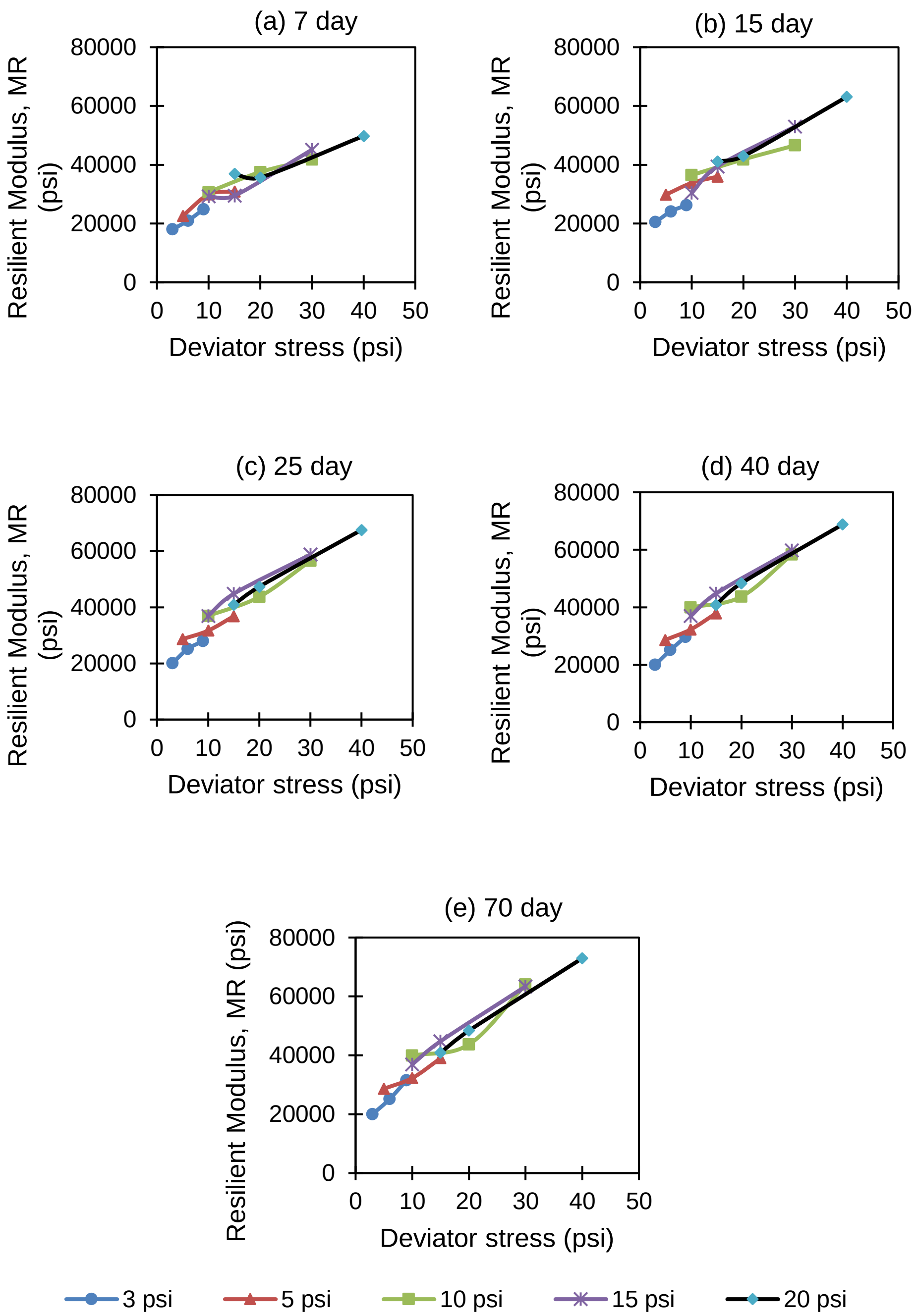


Figure A-11 Resilient modulus test results at confining stresses of 3, 5, 10, 15 and 20 psi for the control samples of mix 50R_6C after (a) 7, (b) 15, (c) 25, (d) 40 and (e) 70 days.

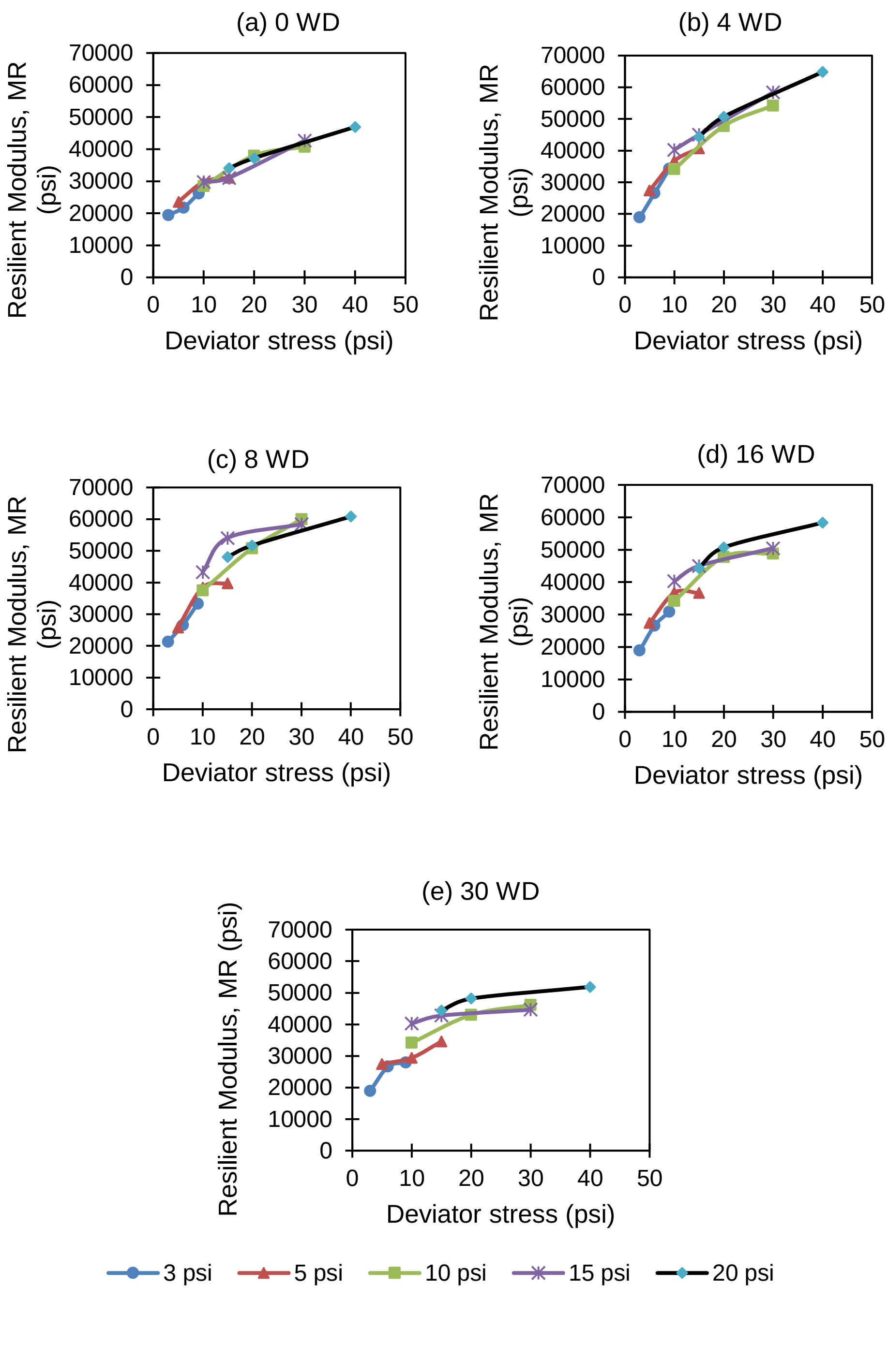


Figure A-12 Resilient modulus test results at confining stresses of 3, 5, 10, 15 and 20 psi for the test samples of mix 50R_6C after (a) 0, (b) 4, (c) 8, (d) 16 and (e) 30 W-D cycles.

Table A-1 Model parameters for 0R_4C

		Two Parameter Model			Three Parameter Model			
		k_1	k_2	R^2	k_3	k_4	k_5	R^2
Number of W-D Cycles	0	5528.7	0.46	0.83	11074	0.04	0.41	0.89
	4	3956.8	0.56	0.92	7866.7	0.24	0.34	0.95
	8	4487	0.52	0.94	8874.5	0.28	0.24	0.95
	16	4669.4	0.49	0.91	9788.2	0.39	0.11	0.91
	30	5088	0.45	0.91	10384	0.40	0.03	0.91
Curing	7 day	7918.2	0.37	0.82	11074	0.03	0.37	0.97
	15 day	5814.8	0.46	0.83	9218.9	0.08	0.41	0.94
	25 day	6575.6	0.44	0.97	11825	0.24	0.20	0.98
	40 day	4861.7	0.52	0.95	9581.8	0.28	0.25	0.96
	70 day	4797.4	0.53	0.94	9294.5	0.24	0.30	0.96

Table A-2 Model parameters for 0R_6C

		Two Parameter Model			Three Parameter Model			
		k_1	k_2	R^2	k_3	k_4	k_5	R^2
Number of W-D Cycles	0	5528.7	0.46	0.83	11074	0.04	0.41	0.89
	4	3956.8	0.56	0.92	7866.7	0.24	0.34	0.95
	8	4487	0.52	0.94	8874.5	0.28	0.24	0.95
	16	4669.4	0.49	0.91	9788.2	0.39	0.11	0.91
	30	5088	0.45	0.91	10384	0.40	0.03	0.91
Curing	7 day	7918.2	0.37	0.82	11074	0.03	0.37	0.97
	15 day	5814.8	0.46	0.83	9218.9	0.08	0.41	0.94
	25 day	6575.6	0.44	0.97	11825	0.24	0.20	0.98
	40 day	4861.7	0.52	0.95	9581.8	0.28	0.25	0.96
	70 day	4797.4	0.53	0.94	9294.5	0.24	0.30	0.96

Table A-3 Model parameters for 30R_4C

		Two Parameter Model			Three Parameter Model			
		k_1	k_2	R^2	k_3	k_4	k_5	R^2
Number of W-D Cycles	0	5528.7	0.46	0.83	11074	0.04	0.41	0.89
	4	3956.8	0.56	0.92	7866.7	0.24	0.34	0.95
	8	4487	0.52	0.94	8874.5	0.28	0.24	0.95
	16	4669.4	0.49	0.91	9788.2	0.39	0.11	0.91
	30	5088	0.45	0.91	10384	0.40	0.03	0.91
Curing	7 day	7918.2	0.37	0.82	11074	0.03	0.37	0.97
	15 day	5814.8	0.46	0.83	9218.9	0.08	0.41	0.94
	25 day	6575.6	0.44	0.97	11825	0.24	0.20	0.98
	40 day	4861.7	0.52	0.95	9581.8	0.28	0.25	0.96
	70 day	4797.4	0.53	0.94	9294.5	0.24	0.30	0.96

Table A-4 Model parameters for 30R_6C

		Two Parameter Model			Three Parameter Model			
		k_1	k_2	R^2	k_3	k_4	k_5	R^2
Number of W-D Cycles	0	5528.7	0.46	0.83	11074	0.04	0.41	0.89
	4	3956.8	0.56	0.92	7866.7	0.24	0.34	0.95
	8	4487	0.52	0.94	8874.5	0.28	0.24	0.95
	16	4669.4	0.49	0.91	9788.2	0.39	0.11	0.91
	30	5088	0.45	0.91	10384	0.40	0.03	0.91
Curing	7 day	7918.2	0.37	0.82	11074	0.03	0.37	0.97
	15 day	5814.8	0.46	0.83	9218.9	0.08	0.41	0.94
	25 day	6575.6	0.44	0.97	11825	0.24	0.20	0.98
	40 day	4861.7	0.52	0.95	9581.8	0.28	0.25	0.96
	70 day	4797.4	0.53	0.94	9294.5	0.24	0.30	0.96

Table A-5 Model parameters for 50R_4C

		Two Parameter Model			Three Parameter Model			
		k_1	k_2	R^2	k_3	k_4	k_5	R^2
Number of W-D Cycles	0	5528.7	0.46	0.83	11074	0.04	0.41	0.89
	4	3956.8	0.56	0.92	7866.7	0.24	0.34	0.95
	8	4487	0.52	0.94	8874.5	0.28	0.24	0.95
	16	4669.4	0.49	0.91	9788.2	0.39	0.11	0.91
	30	5088	0.45	0.91	10384	0.40	0.03	0.91
Curing	7 day	7918.2	0.37	0.82	11074	0.03	0.37	0.97
	15 day	5814.8	0.46	0.83	9218.9	0.08	0.41	0.94
	25 day	6575.6	0.44	0.97	11825	0.24	0.20	0.98
	40 day	4861.7	0.52	0.95	9581.8	0.28	0.25	0.96
	70 day	4797.4	0.53	0.94	9294.5	0.24	0.30	0.96

Table A-6 Model parameters for 50R_6C

		Two Parameter Model			Three Parameter Model			
		k_1	k_2	R^2	k_3	k_4	k_5	R^2
W-D Cycles	0	9006	0.34	0.82	12482.5	0.041	0.32	0.97
	4	8249.41	0.43	0.84	13021	0.08	0.38	0.97
	8	8816.9	0.42	0.90	14379.7	0.12	0.32	0.96
	16	8764.35	0.40	0.89	14162.8	0.14	0.27	0.96
	30	8646	0.39	0.94	14427.4	0.20	0.19	0.96
Curing	7 day	7587.86	0.39	0.82	11096	0.06	0.36	0.95
	15 day	7828	0.42	0.82	11863	0.10	0.35	0.93
	25 day	6664.81	0.48	0.87	10810.1	0.10	0.41	0.97
	40 day	6939.94	0.47	0.85	11072.5	0.08	0.42	0.96
	70 day	6581	0.49	0.81	10302	0.09	0.46	0.96

REFERENCE

1. AASHTO T307-99 (2003),"Standard Method of Test for Determining the Resilient Modulus of Soils and Aggregate Materials." American Association of State Highway and Transportation Officials, Washington, D.C.
2. AASHTO, T308-10 (2010). Standard method of test for determining the asphalt binder content of hot mix asphalt (HMA) by the ignition method. Washington DC, USA: American Association of State Highway and Transportation Officials.
3. American Association of State Highway, & Transportation Officials. (1993). *AASHTO Guide for Design of Pavement Structures, 1993* (Vol. 1). AASHTO.
4. ASTM. (2011). *Standard test method for pH of engine coolants and anti-rusts,* ASTM D1287-11. West Conshohocken, PA.
5. ASTM. (2012). *Standard test methods for chemical oxygen demand (dichromate oxygen demand) of water,* ASTM D1252. West Conshohocken, PA.
6. ASTM. (2013). *Standard test methods for filterable matter (total dissolved solids) and non-filterable matter (Total suspended solids) in water,* ASTM D5907-13. West Conshohocken, PA.
7. Camargo, F. F., Edil, T. B., & Benson, C. H. (2013). Strength and stiffness of recycled materials stabilised with fly ash: A laboratory study. *Road Materials and Pavement Design*, *14*(3), 504-517.
8. Cetin, B., Aydilek, A. H., & Guney, Y. (2010). Stabilization of recycled base materials with high carbon fly ash. *Resources, Conservation and Recycling*, *54*(11), 878-892.

9. Chen, D., Chen, D. H., Laguros, J. G., & Zaman, M. M. (1995). Characterization of base/subbase materials under repetitive loading. *Journal of testing and evaluation*, *23*(3), 180-188.
10. Chittoori, B.C.S (2008). “Clay mineralogy effects on long-term performance of chemically treated expansive clays.” Dissertation, University of Texas at Arlington, Arlington, Texas
11. Chittoori, B.S., Puppala, A.J., Wejrengsikul, T., Hoyos, L.R. (2013) “Experimental Studies on Stabilized Clays at Various Leaching Cycles.” *ASCE, Journal of Geotechnical and Geoenvironmental Engineering*, October, Vol 139, No. 10, pp. 1665-1675.
12. Copeland, A. (2011). *Reclaimed asphalt pavement in asphalt mixtures: state of the practice* (No. FHWA-HRT-11-021).
13. Decker, D. (1999). Handling RAP in an HMA Facility. *Roads & Bridges*, *37*(3).
14. Decker, D. S., and Young, T. J. (1996). “Handling RAP in an HMA facility.” Proceedings of the Canadian Technical Asphalt Association, Edmonton, Alberta.
15. EPA. (2005). Benchmarks for storm-water sampling, Business Environmental Resource Center, City of Sacramento, California.
16. Euch Khay, S. E., Euch Ben Said, S. E., Loulizi, A., & Neji, J. (2014). Laboratory Investigation of Cement-Treated Reclaimed Asphalt Pavement Material. *Journal of Materials in Civil Engineering*, *27*(6), 04014192.
17. Fathifazl, G., Abbas, A., Razaqpur, A. G., Isgor, O. B., Fournier, B., & Foo, S. (2009). New mixture proportioning method for concrete made with coarse recycled concrete aggrega

18. Faysal, M. (2017). "Structural competency and environmental soundness of recycled base materials in North Texas." Dissertation, University of Texas at Arlington, Arlington, Texas.

19. Faysal, M., Hossain, S., Salah, S. B., Khan, M. S., & Flores, C. M. (2017). Characterization of Strength and Long-Term Durability of Recycled Flex Base Materials. In *Transportation Research Board 96th Annual Meeting* (No. 17-04301).

20. Faysal, M., Mahedi, M., Aramoon, A., Thian, B., Hossain, M. S., & Khan, M. S. (2016a). "Strength Characterization of Untreated and Cement-Treated Recycled Flex-Base Materials." Geotechnical and Structural Engineering Congress 2016. 1233-1244.

21. Faysal, M., Mahedi, M., Aramoon, Al., Thian, B., Hossain, M.S., and Khan, M.S. (2016b). "Strength Characterization of Untreated and Cement Treated Recycled Flex-Base Materials." Geotechnical and Structural Engineering Congress 2016.

22. Gnanendran, C. T., & Woodburn, L. J. (2003). Recycled Aggregate for Pavement Construction and the Influence of Stabilization. *Publication of: ARRB Transport Research, Limited*.

23. Grilli, A., Bocci, E., & Graziani, A. (2013). Influence of reclaimed asphalt content on the mechanical behaviour of cement-treated mixtures. Road Materials and Pavement Design, 14(3), 666-678.

24. Guthrie, S.W., Cooley, D., and Eggett, D.L., 2007, "Effects of Reclaimed Asphalt Pavement on Mechanical Properties of Base Materials," J. Transp. Res. Board, No. 2005, pp. 44-52.

25. Guthrie, W.S., Brown, A.V., and Eggett, D.L. (2007). "Cement stabilization of aggregate base material blended with reclaimed asphalt pavement." Transportation Research Record No. 2026, Transportation Research Board, Washington, DC, 47-53.

26. Hedayati, M., & Hossain, S. (2015). Data based model to estimate subgrade moisture variation case study: Low volume pavement in North Texas. *Transportation Geotechnics*, *3*, 48-57.

27. Hossain, S., Ahmed, A., Khan, M. S., Aramoon, A., & Thian, B. (2016). Expansive Subgrade Behavior on a State Highway in North Texas. In *Geotechnical and Structural Engineering Congress 2016*.

28. Hoyos, L. R., Puppala, A. J., and Ordonez, C. O. (2011). "Characterization of cement-fiber-treated reclaimed asphalt pavement aggregates: Preliminary investigation." Journal of Materials in Civil Engineering 23.7 (2011): 977-989.

29. Hoyos, L.R., Laikram, A., and Puppala, A.J (2005). "Assessment of Seasonal Effects on Engineering Behavior of Chemically Treated Sulfate Rich Expansive Clay." *Proc., GEOPROB2005: International Conference on Problematic Soils*, 25-27.

30. Huang, Y., Bird, R.N., Heidrich, O. (2007). A review of the use of recycled solid waste materials in asphalt pavements. Resources, Conservation and Recycling, Vol. 52, No. 1, pp 58-73.

31. Janoo, V., Berg, R. L., Simonsen, E., & Harrison, A. (1994). Seasonal changes in moisture content in airport and highway pavements. In *Symposium and Workshop on Time Domain Reflectometry in Environmental, Infrastructure, and Mining Applications, Evanston, IL* (pp. 357-362).

32. Jones, D., Wu, R., & Louw, S. (2014). Full-Depth Recycling Study: Test Track Construction and First Level Analysis of Phase 1 HVS and Laboratory Testing.

33. Kaniraj, S. R., & Havanagi, V. G. July 2001. Behavior of cement-stabilized fiber-reinforced fly ash–soil mixtures. *Journal of Geotechnical and Geoenvironmental En*

34. Khosla, N. P., & Visintine, B. (2011). Effect pf the Use of Higher Percentages of RAP in NCDOT Hot Mix Asphalt.

35. Khoury, N., & Zaman, M. (2002). Effect of wet-dry cycles on resilient modulus of class C coal fly ash-stabilized aggregate base. Transportation Research Record: Journal of the Transportation Research Board, (1787), 13-21.

36. Khoury, N., & Zaman, M. (2005). Durability effects on flexural behavior of fly ash stabilized limestone aggregate. Journal of Testing and Evaluation, 34(3), 1-9.

37. Khoury, N., & Zaman, M. M. (2007). Durability of stabilized base courses subjected to wet–dry cycles. International Journal of Pavement Engineering, 8(4), 265-276.

38. Kolias, S. (1996). Mechanical properties of cement-treated mixtures of milled bituminous concrete and crushed aggregates. Materials and Structures, 29(7), 411-417.

39. Kou, S. C., & Poon, C. S. (2012). Enhancing the durability properties of concrete prepared with coarse recycled aggregate. *Construction and Building Materials*, *35*, 69-76.

40. Lad, P. P. (2012). Development of a new device to evaluate stabilization durability of expansive soils by addressing wetting/drying and leachate issues. The University of Texas at Arlington.

41. Lekarp, F., Isacsson, U., & Dawson, A. (2000). State of the art. I: Resilient response of unbound aggregates. *Journal of transportation engineering*,*126*(1), 66-75

42. Little, D.N., Males, E.H., Pursinski, J.R. and Stewart, B., Cementitious stabilization, http://www.nationalacademies.org/trb/publications/millennium/00016.pdf. Accessed January 2005.

43. Maalouf, M., Khoury, N., Laguros, J. G., & Kumin, H. (2012). Support vector regression to predict the performance of stabilized aggregate bases subject to wet–dry cycles. International journal for numerical and analytical methods in geomechanics, 36(6), 675-696.

44. McCallister, L. D. (1990). *The effects of leaching on lime treated expansive clays* (Doctoral dissertation, University of Texas at Arlington).

45. McCallister, L. D., & Petry, T. M. (1990). *Property changes in lime treated expansive clays under continuous leaching*. TEXAS UNIV AT ARLINGTON.

46. Miller, G. A., Zaman, M. M., Rahman, J., & Tan, N. K. (2003). Laboratory and field evaluation of soil stabilization using cement kiln dust.

47. Nazarian, S., Pezo, R., Melarkode, S., & Picornell, M. (1996). Testing methodology for resilient modulus of base materials. *Transportation Research Record: Journal of the Transportation Research Board*, (1547), 46-52

48. Nunan, T. A., & Humphrey, D. H. (1990). A REVIEW AND EXPERIMENTATION OF GRAVEL STABILIZATION METHODS. EXECUTIVE SUMMARY (No. FHWA ME-90-2)

49. Ordonez, C. A. (2006). *Characterization of cemented and fiber-reinforced RAP aggregate materials for base/sub-base applications*. The University of Texas at Arlington.

50. Pedarla, A., Chittoori, B., and Puppala, A.J. (2011) “Influence of Mineralogy and Plasticity Index on the Stabilization Effectiveness of Expansive Clays”, Journal of the Transportation Research Board, Vol. 2212, pp. 91-99

51. Potturi, A. K. (2006). *Evaluation of resilient modulus of cement and cement-fiber treated reclaimed asphalt pavement (RAP) aggregates using repeated load triaxial test*. MS Thesis, University of Texas at Arlington.

52. Potturi, A., Puppala, A.J., Hoyos, L.R. (2007). “Resilient characteristics of cement treated reclaimed asphalt pavement aggregates.” Transportation Research Board Annual meeting, National Research Council, National Academy of Science, Washington, D.C.

53. Puppala, A. J., Hoyos, L. R., & Potturi, A. K. (2011). Resilient moduli response of moderately cement-treated reclaimed asphalt pavement aggregates. *Journal of Materials in Civil Engineering*, *23*(7), 990-998.

54. Puppala, A. J., Pedarla, A., Chittoori, B., Ganne, V. K., & Nazarian, S. (2017). Long-Term Durability Studies on Chemically Treated Reclaimed Asphalt (RAP) Material as a Base Layer for Pavements. In *Transportation Research Board 96th Annual Meeting* (No. 17-05105).

55. Rada, G., & Witczak, M. W. (1981). *Comprehensive evaluation of laboratory resilient moduli results for granular material* (No. 810).

56. Santoni, R., Tingle, J., & Webster, S. (2002). Stabilization of silty sand with nontraditional additives. *Transportation Research Record: Journal of the Transportation Research Board*, (1787), 61-70.

57. Smith, R.W. (1980). “State of the art hot recycling.” Proceedings of the National Seminar on Asphalt Pavement Recycling, Transportation research board, Record No. 780, Washington, D.C.

58. Specifications, U. F. G. (2010). Concrete Pavement for Airfields and Other Heavy-Duty Pavements. UFGS 32-13-11. Construction Criteria Base, National Institute of Building Sciences, Washington, DC.

59. Taha, R, Al-Harthy, A., Al-Shamsi, K., and Al-Zubeidi, M. (2002). “Cement stabilization of reclaimed asphalt pavement aggregate for road bases and subbases.” J. Materials in Civil Eng., 14(3), 239-245.

60. Taha, R. (2003). “Evaluation of cement kiln dust-stabilized reclaimed asphalt pavement aggregate systems in road bases.” Proceedings Eighth International Conference on LowVolume Roads, Reno, Nevada..

61. Tao, M., & Zhang, Z. (2006). Durability of Cement-Stabilized Low-Plastic Soils. In Transportation Research Board 85th Annual Meeting (No. 06-1255).

62. Texas Department of Transportation (1999). *Laboratory compaction characteristics and moisture- density relationship of base materials*. Tex-113-E, Austin, TX.

63. Texas Department of Transportation (1999). *Soil-cement testing procedure.* Tex-120-E, Austin, TX.

64. Texas Department of Transportation. (1999). Determining the Specific Gravity of soils. Tex-108-E, Austin, TX.

65. Texas Department of Transportation. (1999). *Particle size analysis of soils.* Tex-110-E, Austin, TX.

66. Texas Department of Transportation. (2016). Tex-236-F: Test Procedure for determining asphalt paving mixtures by the ignition method. Austin, Texas: Texas Department of Transportation.

67. Transportation Applications of Recycled Concrete Aggregate—FHWA. (2004). “State of the Practice National Review.” U.S. Department of Transportation Federal Highway Administration, Washington, DC, USA, 2004; pp. 1-47.

68. Uzan, J. (1985). Characterization of granular material. *Transportation Research Record*, (1022).