RECLAIMED STABILIZED BASE:

STABILIZING AGENT SELECTION AND DESIGN

Ali Khorasani[1], Ehsan Ghazanfari[2] and John E. Lens[3]

[1]Graduate Research Assistant, Department of Civil and Environmental Engineering, The University of Vermont, Burlington, Vermont, 05405 (E-mail: Ali.khorasani@uvm.edu)

[2]Associate Professor, Department of Civil and Environmental Engineering, The University of Vermont, Burlington, Vermont, 05405 (E-mail: Ehsan.ghazanfari@uvm.edu)

[3]Professor of Practice, Department of Civil and Environmental Engineering, The University of Vermont, Burlington, Vermont, 05405 (E-mail: john.lens@uvm.edu)

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16. Abstract

Reclaimed stabilized base (RSB) is a common technique utilized to rehabilitate roadways. RSB involves reclaiming the base material and adding a stabilizing agent to increase the strength and durability of the base/subbase structure. This technique involves pulverizing some of the existing pavement along with a portion of the underlying base material, adding aggregate material as necessary to accommodate grade changes, adding a stabilizing agent, and thoroughly mixing the components in place to create a homogeneous mixture. Utilizing RSB can lead to increased service life of the pavement and consequently significant maintenance cost savings compared to pavement overlay without improving the base conditions. However, determining the appropriate stabilizing agent and design for various subbase materials remains a challenge, particularly for cold winter climate locations.

RSB is shown to reduce and relocate the tensile strains at the bottom of pavement layer, resulting in a pavement structure with reduced distress potential. Traditional chemical stabilizers create a cementing compound by reacting with the base material or on their own. The addition of the right amount of chemical stabilizer is key to successful RSB implementation, as inadequate amount leads to insufficient binding and therefore insufficient strength and durability, and more than adequate amount leads to increased cracking potential due to increased rigidity of the product. In contrast to the chemical stabilizers (i.e., calcium-based stabilizers), bituminous stabilizers (e.g., asphalt emulsion) do not react chemically with the base material, rather coat the aggregates and provide adhesive bonding.

In this project, the suitability of a few stabilizing agents (i.e., cement, Liquid Calcium Chloride (LCC), Asphalt Emulsion (AE)) for common subbase materials encountered in Vermont roadways with three different gradations were investigated by performing laboratory experiments and the appropriate type and percentage of stabilizing agent were determined. In both cement and LCC stabilization, reduction of the gravel content below 45% led to reduction in Unconfined Compressive Strength (UCS) values of molded specimens. Moreover, 2-3% cement and 4% LCC content were determined to be the optimum additive content for cement and LCC stabilization respectively.

Adding RAP up to 30% to the soils with gradations within the typical soils used for FDR (about 45% gravel content) did not substantially reduce the UCS of cement-stabilized mixtures while the LCC-stabilized mixtures showed higher range of strength reduction. The Marshall Stability tests indicated that the 4% AE content to be the optimum additive content to use for base stabilization.

They also showed that the Marshall values of the specimens prepared using pure subbase were higher than specimens incorporating 30 % RAP for all the percentages of AE contents beyond 3%.

In addition to the laboratory work, to further maximize the benefits of the RSB, a Finite Element Analysis model of a typical 3-layer pavement structure loaded with circular static loads were developed in ABAQUS and the performance of the pavement for pre-and post-stabilization cases were analyzed. The model input included the pavement properties i.e., layer thickness, elastic modulus, Poisson's ratio and the load magnitude and model predictions included the range of deformation and strains along the depth of the pavement.

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ORGANIZATION OF THE REPORT

This report is structured as follows: Chapter 1 gives an introduction about road rehabilitation and benefits of RSB. Chapter 2 includes the material collection and research methodology for all types of soil and stabilizing agents used at different phases of the project. The results of all the tests on aggregate, gradation and stabilizing agent combinations are presented and discussed in Chapter 3. Chapter 4, starting with a brief literature review, presents the process and methodology of developing a Finite Element Analysis (FEA) model of a 3-layer pavement structure in ABAQUS and its implications. The conclusions of this research project are provided in Chapter 5. Finally, Chapter 6 includes recommendations on RSB implementation in the State of Vermont.

Executive Summary

Pavement “tenting” refers to raised surfaces on both sides of a transverse crack. There have been examples of low performance of RSB projects in the state of Vermont and costumer complaints relating to significant pavement “tenting”, or heaving, which significantly lower the ride quality. Previous forensic investigation by VTrans in 2014 indicated that “Pavement tenting” is related to shrinkage of the cement treated base (Vtrans report, 2014). Shrinkage cracks that form in the cement treated base protrude upward through the pavement layers. Factors which increase the probability of shrinkage include compacting material at high moisture levels, using a soil with a high percentage of clay, rapid moisture loss (improper curing procedures), inadequate compaction levels and using excessive amounts of cement.

The objectives of this project were to (i) evaluate base treatment efficacy using cement along with two other prevalently used additives i.e., asphalt emulsion and liquid calcium chloride and investigate the proper mix design (ii) determine optimum stabilizing agent for gaining certain strengths associated with the type or gradation of the base/subbase material, and (iii) establish the effect of RAP on the strength properties of the pavement layer and the range of strength reduction with the increments of the RAP content.

In this study the range of improvements in strength properties (i.e., Unconfined Compressive Strength (UCS)) of the base/subbase materials with three different gradations, stabilized with various RAP and stabilization additives contents were evaluated. For specimens stabilized with cement and calcium chloride, the UCS of the molded specimens compacted inside the standard 4” mold in compliance with ASTM D1633 – 17 were selected as the standard testing criteria. For specimens stabilized with cement and asphalt emulsion the available standards and procedures in the literature for sample preparation and curing were followed. As for the liquid Calcium Chloride (LCC) stabilized mixtures, due to lack of sufficient information on the sample preparation and curing of the specimens, different curing procedures were explored, and two alternative curing procedures were established.

The results of the tests on both chemical stabilizers (i.e., cement, LLC) highlighted the role of the subbase/base materials gradation, provided the optimum moisture content of the mixture, and determined the contribution of stabilizing agent content to the strength of the stabilized base/subbase layer. The optimum LCC and cement content to be utilized for subbase/base material in a typical RSB project were reported.

For cement-stabilized specimens, fine graded subbase showed the lowest range of Unconfined Compressive Strength (UCS). The subbase material containing 5% clay exhibited significantly higher UCS compared to the coarse and fine aggregate subbase specimens. In general, to attain the UCS of around 300 psi, the required cement percentages for addition were found to be between 2% to 4% for different tested subbase specimens. For

liquid calcium chloride (LLC)-stabilized specimens, the LLC content of 4% was found to provide the highest UCS. Addition of RAP significantly decreased the UCS of LCC stabilized specimens.

The Marshall stability values of samples prepared with and without RAP content and compacted inside the Marshall molds were opted as the testing criteria for bituminous stabilized (Asphalt Emulsion (AE)) specimens. The AE treated specimens were cured upon compaction for 4 days at 40 ^{o}C and then were tested under a constant load at a rate of 50.8 mm/minute in compliance with AASHTO Designation: R 68-15 (2019) and ASTM D6927-15. Marshall Stability values and Marshall indices were reported from the load-deformation data of the tested specimens. It was found that incorporating the AE contents beyond 4% on both pure subbase specimens and specimens containing 30% RAP adversely affects the strength of the subbase material. The 4 % AE content was found to be the optimum AE content for the soils with gradations that is typical in RSB projects.

Finite Element Analysis (FEM) of a typical 3-layer pavement structure loaded with circular static loads were developed in ABAQUS and the performance of the pavement for pre-and post-stabilization cases were analyzed. The model input included the pavement properties i.e., layer thickness, elastic modulus, Poisson's ratio and the load magnitude and model predictions included the range of deformation and strains along the depth of the pavement. The comparative analysis of the pavement for pre- and post-stabilization cases using 2% cement indicated that with improved modulus of elasticity, the stress resulting from the static load shifts to a lower depth, implying that the HMA will be subjected to lower distress. Moreover, in a stabilized base layer compared to none-stabilized (unbound) layer, a reduction in the values of horizontal stress along the HMA can be expected. The reduced tensile stress will result in service life improvement of the pavement. Finally, in the stabilized base layer, the vertical deformation along the depth was found to be reduced by approximately 30 %. This will result in less rutting of the pavement. Overall, the FEM analysis highlighted the benefits of stabilization, but additional model improvement and inclusion of other additives would further extend the benefits of the developed model.

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CHAPTER 1. INTRODUCTION

Stabilized Full Depth Reclamation (SFDR) also known as Reclaimed Stabilized Base RSB) in the state of Vermont is an ever increasingly used technique that amends base aggregate material, through the incorporation of a stabilizing additive, to produce a homogeneous base layer with enhanced characteristics for strength, stability, and durability. Improvements to strength and stability translate into improved performance of the supported pavement structures. It is essential that the stabilizing additive provide permanent and durable improvement in elastic and strength characteristics. Increased structural capacity derived from stabilizing a base layer can result in an overall more economical equivalent pavement structure with a surfacing layer that is thinner but not excessively thin (Wegman, 2017).

Traditional Stabilization additives are generally classified into two broad categories (i) chemical additives (e.g., lime, cement, fly-ash, Liquid Calcium Chloride (LCC)) and (ii) bituminous additives (e.g., Asphalt Emulsion (AE), foamed asphalt). The non-traditional stabilizers (e.g., sulfonated oils, enzymes, polymers, and potassium compounds) have also been proposed and used in several projects with mixed level of success (Little & Nair, 2009).

The addition of the right amount of stabilizer corresponding the soil type and gradation, determination of the optimum moisture content of the mix and proper curing of the mixture after compaction is key to successful RSB implementation. Failure to control mentioned factors, for instance, low quantity of the stabilizing agent leads to insufficient binding and therefore insufficient strength and durability. On the other hand, more than adequate amount leads to increased cracking potential due to increased rigidity of the product (Little & Nair, 2009; Wegman et al., 2017). To come up with the optimum percentage of stabilizing agents suitable for the soils encountered in the state of Vermont (low plasticity index) as well as the standard test methodologies for conducting laboratory tests (i) literature, (ii) reports of the past research project of VTrans and (ii) the reports of the surveys conducted in this research project which are presented in this chapter were reviewed and the scope of the research project were outlined.

While there were adequate information and standard procedures for sample preparation, curing and testing of the base treatment using cement, lime and asphalt emulsion in the literature, however, there is a gap with such standards and procedures for LCC. Chapters 2 and 3 describe the materials being used for this research, stabilizing agents, research methodologies as well as the process of establishing the laboratory test procedure for LCC.

1.1. ROAD REHABILITATION

1.1.1. SUSTAINABLE APPROACH TO ROAD REHABILITATION

The finite nature and depletion of natural aggregate resources calls for sustainable approaches to road construction and rehabilitation. The annual consumption of aggregate materials is estimated about 1.5 billion tons for the construction of new infrastructures and pavements in the United States (USGS 2005). According to the report of USDOT, due to the rapid increase in construction of different types of infrastructures, it is estimated that more than 2.5 billion tons of aggregates will be consumed by 2020. Moreover, declining landfill spaces is another issue resulting from the tremendous amount of waste generated from the pavement rehabilitation and structural demolition, which emphasize reusing these materials as an alternative to natural aggregates. Among varied materials, Reclaimed Asphalt Pavement (RAP) and Recycled Crushed Concrete Aggregate (RCCA) are the most commonly recycled materials in the United States (Cetin et al., 2010; Faysal, 2017; Hanks & Magni, 1989; Hoppe et al., 2015; Hoyos et al., 2011; Vuong & Brimble, 2000).

1.1.2. RECLAIMED ASPHALT PAVEMENT (RAP) APPLICATIONS

Existing asphalt pavement materials are commonly removed during resurfacing, rehabilitation, or reconstruction operations. Once removed and processed, the pavement material becomes RAP, which contains valuable asphalt binder and aggregate (see figure 1.1). In the early 1990s, FHWA and the U.S. Environmental Protection Agency estimated that more than 90 million tons of asphalt pavement were reclaimed (i.e., converted into material suited for use) every year, and over 80 percent of RAP was recycled, making asphalt the most frequently recycled material. RAP is most commonly used as an aggregate and virgin asphalt binder substitute in recycled asphalt paving, but it is also used as a granular base or subbase, stabilized base aggregate, and embankment or fill material. It can also be used in other construction applications (Copeland, 2011).

a b

Figure 1.1. (a) RAP and (b) RAP production (Source: [Http://all-county-paving.com)](Http://all-county-paving.com)

There are many applications for RAP. The three major areas where RAP is used is (i) as an aggregate and asphalt binder substitute in Hot Mix Asphalt (HMA) production, (ii) a granular base or subbase aggregate in road construction/rehabilitation, and (iii) as embankment or fill material. In these applications, reclaimed material is either directly used after milling or it is required to undergo some screening and processes depending on the purpose it serves. These processes are essential to achieve the properties of virgin materials out of recycled ones which is one of the approaches to sustainable exploitation of the finite resources. The focus of this research project is utilization of RAP through in-place modification or stabilization of the reclaimed materials resulting in improved elastic and strength characteristics of the pavement in road rehabilitation projects. The techniques are commonly utilized for in-place recycling of the deteriorated pavements are defined in the following sections and the "Stabilized Full Depth Reclamation (SFDR)" as the primary method of base stabilization will be discussed in depth (Arulrajah et al., 2013; Faysal, 2017; Hoppe et al., 2015; Mohammadinia et al., 2015).

1.1.3. FULL DEPTH RECLAMATION (FDR)

Full Depth Reclamation (FDR) is defined by Asphalt Recycling & Reclaiming Association (ARRA) as:
"A pavement rehabilitation technique in which the full flexible pavement section and a predetermined portion of the underlying materials are uniformly crushed, pulverized or blended, resulting in a stabilized base course."

The roads are desirably maintained and preserved at reasonable intervals before they are distressed to the levels that causes significant ride quality reduction; accelerated degradation and resulting in the need for major recycling works. If preservation maintenance is not applied early enough or if there are base/subbase/subgrade problems, the road will inevitably be deteriorated to a level that a major rehabilitation work is necessary. FDR can be categorized as one of the major rehabilitation techniques (Morian et al., 2012)

Generally, the type of FDR construction is dependent on the existing pavement condition, availability of materials (i.e., aggregate, reclaimed material), traffic demand, and cost. The basic form of FDR consists of in-situ pulverization of existing pavement and underlying layers, uniform blending of pulverized material, grading, and compaction with water being the only additive added to the pulverized and blended material (Morian et al., 2012)

1.1.4. STABILIZED FULL DEPTH RECLAMATION (SFDR)

When full depth reclamation involves adding a stabilizing agent (e.g., cement, lime, calcium chloride, emulsion, foamed asphalt) it is called Stabilized Full Depth Reclamation (SFDR). The term SFDR will be substituted for

“Reclaimed stabilized base (RSB)” as it is the conventional terminology in the state of Vermont. RSB refers to a treatment process that involves the removal of a portion of the upper pavement layer(s) via a milling process. The remaining pavement is then “reclaimed” using a mixing action similar to a conventional rototiller. A stabilizing agent is then added to the reclaimed material and mixed in with the in-place subgrade material. The cost associated with this treatment is approximately $850k per mile versus a full depth reconstruction valued at between $4M and $5M per mile (Ismail et al., 2014; Little et al., 1995; Wegman et al., 2017).

RSB process is shown in Figure 1.2 and construction machineries in one of the Vermont Agency of Transportation (VTrans) RSB projects in Figure 1.3.

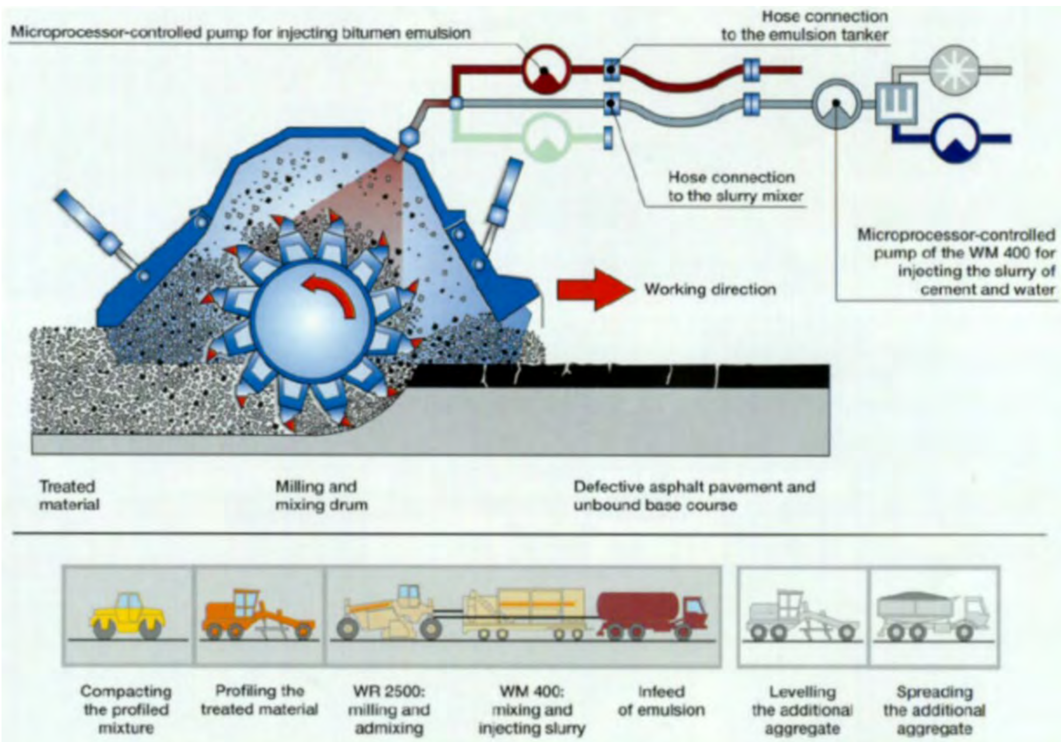


Figure 1.2. Schematic of the SFDR process for a single machine (Source: Wegman et al., 2017)

Equipment train with reclaim grinder/mixer supplied by water tanker truck with sheepsfoot compactor behind

Wirtgen WR250 reclaimer used on the project.

Figure 1.3. Stowe-Morristown RSB Project

As an example of calcium chloride stabilized base work. The Brandon-Goshen project, a 7.6-mile roadway reconstruction of Route 73 from Brandon to Goshen, Vermont consisted of the following steps:

- Remove about 2 inches of existing asphalt pavement as recycled asphalt pavement (RAP) mix.
- Grind/mix the remaining asphalt and underlying materials to about 10 inches deep and compact.
- Shim the roadway with added gravel material to design width and profile.
- Grind/mix this combination of materials about 8 inches deep, with the calcium chloride added at a rate of 0.75 to 0.9 gallons per square yard of roadway surface.
- Finish grade and compact.
- Spray a cap of calcium chloride at a rate of 0.1 to 0.25 gallons/square yard to achieve a final calcium chloride application rate of 1 gallon per square yard.

The pavement surface above this base consists of:

- Cold mix (RAP, cement, emulsion) at 3 inches thick.
- 2-3/4 inches Type II asphalt (mountain portion of road) o r2-1/2 inches Type II asphalt (village portion) followed by 1-1/2 inch of Type IV asphalt (village portion)
- Finish with a paver placed three-quarter inch thick surface treatment layer.

Figure 1.4 shows the work process and observations in this project.

(a) Equipment train with grinder/mixer supplied by tanker truck and followed by sheepsfoot compactor

(b) Teeth on grinding/mixing drum of Wirth 2500S reclaimer used on the project.

(c) Tanker with calcium chloride and hose to reclaimer

(d) Control panel inside Wirth 2500 S cab showing fluid injection and grinding controls

(e) Typical base composition behind grinding/mixing drum

(f) Road grader and smooth drum compactor following sheepsfoot compactor

(g) Road grader fine grading base following grinding/mixing train

(h) Smooth drum compactor following road grader

Figure 1.4. Photos of construction process in Brandon-Goshen RSB project

Another example of cement and asphalt emulsion stabilized base work, the Stowe-Morristown Roadway Project, a 7.5-mile roadway reconstruction of Route 100 from Stowe to Morristown, consists of the following:

1. Removing about 4 inches of existing asphalt pavement as recycled asphalt pavement (RAP) mix.
2. Grinding/mix the remaining asphalt (8 to 9 inches) and underlying materials to about 10 inches deep and then compact that mixture. The target moisture content of the mixture is approximately 6%, aiming toward an optimum moisture content per the corresponding maximum proctor density.
3. Shim the roadway with added gravel material to design width and profile, followed by grinding/mixing this combination of materials in another pass to about 8 inches deep, then compaction, followed by placing asphalt pavement.
4. The cement is to be placed via a mechanical spreader trailing behind a truck.

Figure 1.5 shows examples of various construction stages in Stowe-Morristown RSB Project.

(a) Equipment train with reclaim grinder/mixer supplied by water tanker truck with sheepsfoot compactor behind

(b) Wirtgen WR250 reclaimer used on the project

(c) Remaining asphalt beside reclaim pass is about 9 inches thick in this section

(d) Sheepsfoot compactor following reclaimer, with smooth drum roller behind

Figure 1.5 Survey/visit of Stowe-Morristown RSB Project

1.1.5. COLD IN-PLACE RECYCLING (CIR)

When no hot mix asphalt is involved and only the depth of pulverization is limited to HMA layer by removing and reusing a portion of an in-place Hot Mix Asphalt (HMA) layer to produce a restored pavement layer, the recycling method is called Cold In-place Recycling (CIR). In This method the top few inches (about 3 to 5 inches) of the existing HMA surface is pulverized, mixed with a bituminous stabilization additive such as

Foamed Asphalt (FA) or Asphalt Emulsion (AE) and finally graded and compacted. The CIR utilizes the 100 % of the generated RAP. With CIR the underlying layers of the pavement, i.e., subbase and subgrade are not modified, hence, the range of improvement is not as much as FDR methods.(Wegman et al., 2017).

1.2. BENEFITS OF RSB

The benefits of RSB are generally categorized as (i) economical, (ii) technological, and (iii) environmental. The RSB process lends itself well to sustainable road concepts. First, in RSB all the existing road materials are recycled. Second, several potential stabilization materials, such as fly ash and lime kiln dust, are also recycled products. Hence, the is the potential of recycling multiple materials in the RSB process. (Morian et al., 2012).

Several factors contribute to the renewed interest in RSB including improved equipment, stabilization technology, sustainability, and costs relative to more conventional rehabilitation strategies. FDR also presents highway agencies with an effective tool for achieving sustainability of their road system. Figure 1.6 provides an indication of benefits from FDR as an effective tool for achieving sustainability of the road systems. These benefits can be realized in the form of both preservation of resources and reduction in roadway rehabilitation.

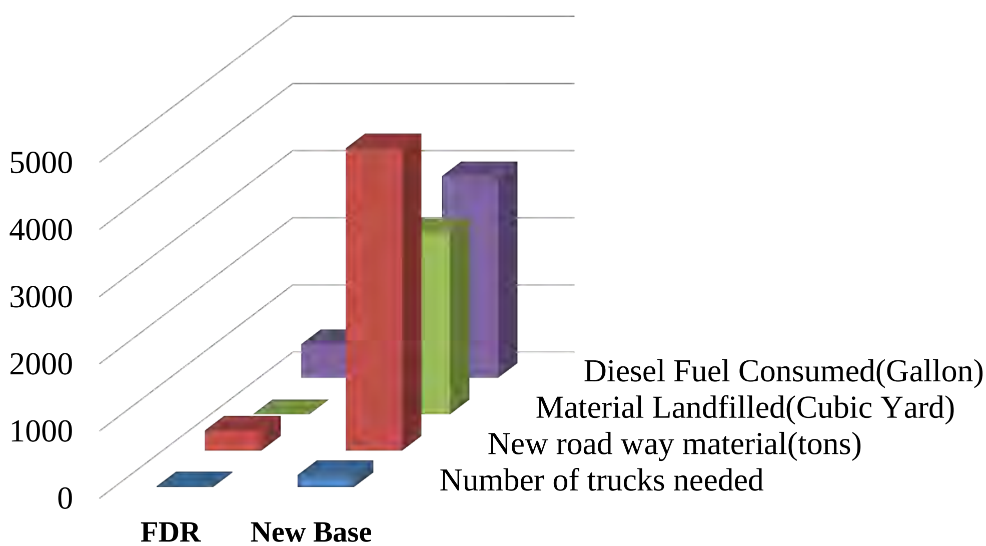


Figure 1.6. FDR vs new base construction (adopted and modified from (Morian et al., 2012)

According to Morian et al. 2012, the following objectives can be addressed by FDR:

- Increase Capacity
- Increase Structural Strength and Stability

- Improve Pavement Condition
- Improve Serviceability
- Extend Service Life

Wegman et al. 2017, expanded the "Improve Pavement Condition" when utilizing RSB as follows:

- All forms of cracking; fatigue, edge, slippage, block, longitudinal, and reflective
- Reduced ride quality due to swells, bumps, sags, patches, and depressions.
- Permanent deformations including, rutting, corrugations, and shoving.
- Loss of bond between pavement layers
- Moisture damage (stripping)
- Loss of surface integrity due to raveling, potholes, and bleeding
- Inadequate structural capacity
- Addressing subgrade instability by increasing structural capacity of the base and surfacing layers

As for "Increase capacity" and "Increase Structural Strength and Stability" little et al. 1995, reported that RAB improves the shear strength, modulus (i.e., stiffness), and fatigue resistance of the pavement (Little et al., 1995). One of the contributing mechanisms to the improved strength of the pavement is that RSB reduces and relocates the tensile strains at the bottom of the pavement layer, resulting in a pavement structure with reduced distress potential as is shown in Figure1.7 (Wegman et al., 2017) (Jones et al., 2015).

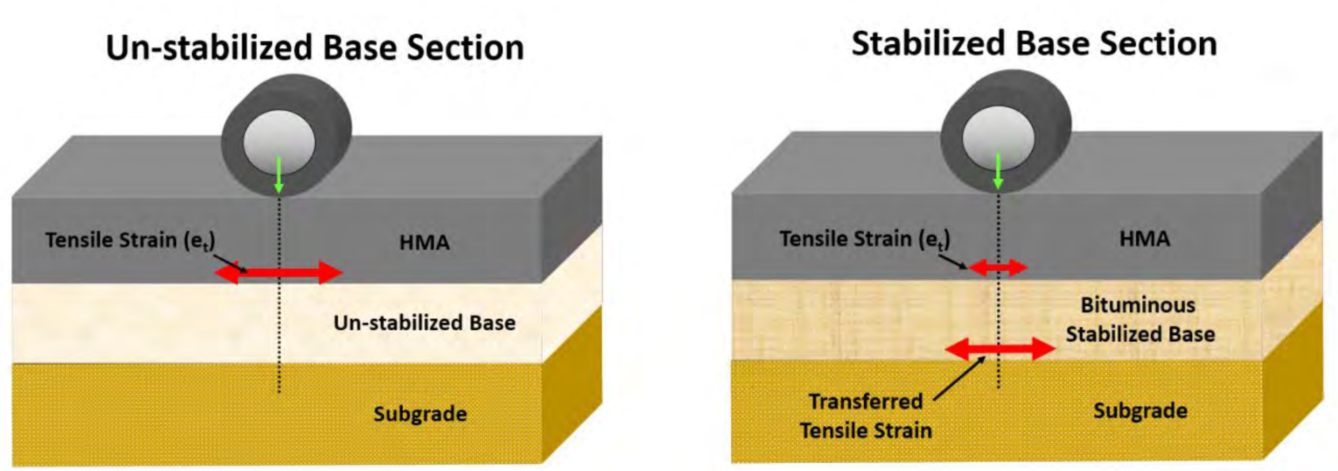


Figure 1.7. Horizontal tensile stresses in stabilized base vs un-stabilized pavement. Source: Wegman et al, 2017

1.3. STABILIZATION ADDITIVES

Stabilization additives are generally classified into two broad categories (i) chemical additives and (ii) bituminous additives. Traditional chemical stabilizers (e.g., lime, cement, fly-ash) create a cementing compound by reacting with the base material or on their own. Considering the reaction between calcium oxide (CaO) component of chemical stabilizers ((e.g. lime, cement, fly-ash) and clay particles to reduce plasticity, cement and fly-ash are usually appropriate for aggregate materials with plasticity index of less than 20, while lime is appropriate for materials with plasticity index of greater than 20. (Ismail et al., 2014; Wegman et al., 2017). The non-traditional stabilizers (e.g., sulfonated oils, enzymes, polymers, and potassium compounds) have also been proposed and used in several projects with mixed level of success (Little & Nair, 2009).

The addition of the right amount of chemical stabilizer is key to successful RSB implementation, as inadequate amount leads to insufficient binding and therefore insufficient strength and durability, and more than adequate amount leads to increased cracking potential due to increased rigidity of the product (Little & Nair, 2009; Wegman et al., 2017). In contrast to the chemical stabilizers (i.e., calcium-based stabilizers), bituminous stabilizers (e.g., asphalt emulsion, foamed asphalt) do not react chemically with the base material, rather coat the aggregates and provide adhesive bonding.

Utilizing RSB can lead to increased service life of the pavement and consequently significant maintenance cost savings compared to pavement overlay without improving the base conditions (Ismail et al., 2014; Little & Nair, 2009; Stroup-Gardiner, 2011; Wegman et al., 2017). However, determining the appropriate stabilizing agent and design for various subbase materials remains a challenge, particularly for cold winter climate locations. In addition, seasonal temperature range and rate of loading are important factors affecting strength and stiffness improvement in bituminous based RSB (Stroup-Gardiner, 2011)

Figure 1.8 shows a successful performance of stabilized base using emulsion in a section of I-94 (Wegman et al., 2017)

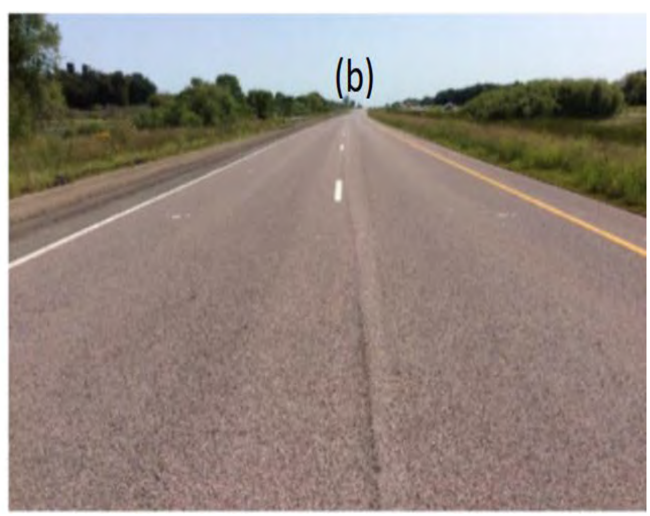


Figure 1.8. Example of a successful RSB application: (a) I-94 before rehabilitation (14-year-old pavement with cracked HMA), and (b) I-94 after rehabilitation (~10 years later), emulsion stabilized and a constructed 3" HMA surface (Wegman et al., 2017).

Several studies have investigated the use of cement as stabilizing agent for base material (Jones et al., 2015; Miller et al., 2006; Stroup-Gardiner, 2011) Limited research on using emulsion (Bleakley et al., 2016; Wegman et al., 2017). calcium chloride (Kipp & Fitch, 2008), lime (Puppala et al., 2017), and other stabilizing agents such as fly ash (Saha & Mandal, 2017; Saride et al., 2015) are available in the literature.

Malick et al, 2002 evaluated the performance of a laboratory mix design of reclaimed materials (from the western part of Maine) stabilized with lime and cement with asphalt emulsion and reported that mixes with additives developed strength faster and exhibited higher shear and stripping resistance than mixes with water only (Mallick et al., 2002)The improved strength and durability of the reclaimed stabilized base significantly varies by the type of base material, the type of stabilizing agent, and the operation conditions such as weather and projected traffic levels. In addition, the type of aggregate and its uniformity significantly influences the choice of stabilizer. In general, for fine aggregates, chemical stabilization is a better choice. In contrast, bituminous stabilization is more appropriate for cleaner aggregates (e.g., sand, gravel) with no silt or clay. Often times performing a laboratory mix design is necessary to investigate the appropriate stabilizing agent, its percentage and application rate for a specific subbase. Adding insufficient amount of stabilizing agent will not provide targeted improvement, while adding too much stabilizer can be a catalyst for cracking in chemical stabilizers, and lead to an unstable matrix in bituminous stabilizers.

1.4. SURVEY OF PREVIOUS RSB PROJECTS

In addition to the survey/visit of Brandon-Goshen project, several Mapillary imageries from previously reclaimed projects in Vermont (provided by VTrans) were reviewed to evaluate the extent/severity of transverse cracking (See Figure 1.9 for examples).

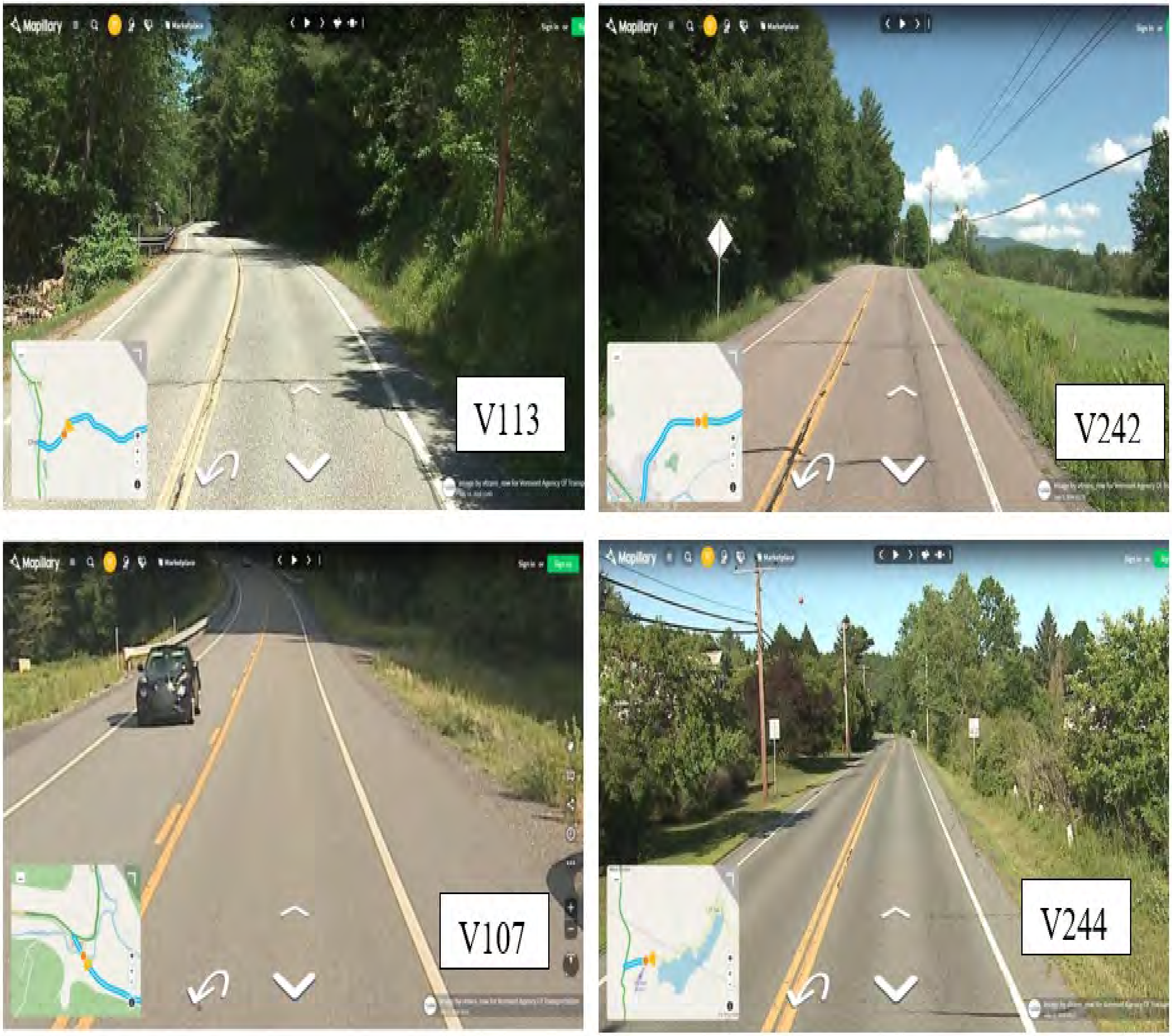


Figure 1.9. Examples of reviewed Mapillary images

Few projects were identified (See Table 1.1) for field survey in March/April 2020; the most critical time for surveying because of frost heave effects expected during mud season/after winter extremes).

Table 1.1. List of RSB survey sites

Road	From	To	Project Name	Project No.	Project Year
V113	0	7	Chelsea-Vershire	STP 2331	2010
V114	21.4	29.2	Brighton-Warren Gore	STP 2724	2010
V244	0	5.6	Thetford-Fairlee	STP 2710	2011
V017	8.4	15.7	Addison-New Haven	STP 9632	2013
V108	17.6	25.9	Cambridge-Bakersfield	STP 2926	2013
V242	0	6.4	Montgomery-Westfield	STP 2906	2013
V107	0	10.1	Stockbridge-Bethel	STP 2910	2015

The selected sites were intended to be surveyed using Pavement Condition Index that include:

- assessing surface conditions and noting distresses that provide insight into the condition of underlying materials and performance of the rehabilitated pavement.
- looking for possible signs of shrinkage cracking, bonding failure/slippage, surface roughness, surface raveling, early load associated distresses
- general observations on the pavement performance.

Reconnaissance site visits were performed on March 7, 8, 12, and 19, and April 4, 2020, covering 14 roadway segments which had been reconstructed using RSB techniques between 2009 and 2017. The main points are summarized below:

- The most distressed section observed was the 7-mile section of Vermont Route 113 between Chelsea and Vershire exhibited the heaving during the first visit on March7.
- Heaving was noticeably reduced on the April 4 visit.
- Section conditions ranged from frequent heaving/tenting on the Route 113 stretch and considerable pavement cracking evident throughout that project area, corresponding to a rate of poor, per the Vermont Performance Measures to minimal distress, corresponding fair or better per the Vermont Performance Measures, observed in most other sections, aside from localized distressed areas over short distances.

Figure 1.10 shows examples of the roadway conditions observed during these visits:

(a) V113 Chelsea-Vershire at about mile 2.60 (centerline tenting on March 7, 2020)

(b) V12A Roxbury-Northfield at Roxbury village (edge heaving on March 8, 2020)

(c) Vermont 12 Worcester-Elmore at southern portion on March 12, 2020

(d) Vermont 12 south of Worcester-Elmore reclaimed stabilized base project March 12, 2020

Figure 1.10. Examples of surveyed roadways

Significantly low performance of RSB projects i.e., transverse cracking, heaving and tenting observed at the 7-mile section of Vermont Route 113 between Chelsea and Vershire, was aligned with the previously reported forensic investigation by VTrans (Vtrans report, 2014), where “Pavement tenting” was found to be related to shrinkage of the cement treated base. Shrinkage cracks that form in the cement treated base protrude upward through the pavement layers. Factors which increase the probability of shrinkage include compacting material at high moisture levels, using a soil with a high percentage of clay, rapid moisture loss (improper curing procedures), inadequate compaction levels and using excessive amounts of cement.

1.1. SUMMARY

In general, per the Vermont Performance Measures, the performance of the most pavements reconstructed using RSB has been rated as satisfactory. However, there are sections rated as poor and fair corresponding to the pavement distresses i.e., transverse cracking, heaving and tenting which significantly lower the ride quality. The main culprit in the poor performance of the RSB is improper mix design. The objectives of this project were to (i) evaluate base treatment efficacy using cement along with two other prevalently used additives i.e., asphalt emulsion and liquid calcium chloride and investigate the proper mix design (ii) determine optimum stabilizing agent for gaining certain strengths associated with the type or gradation of the base/subbase material, and (iii) establish the effect of RAP on the strength properties of the pavement layer and the range of strength reduction with the increments of the RAP content.

CHAPTER 2. RESEARCH METHODOLOGY

The objectives of this project were to (i) evaluate base treatment efficacy using cement along with two other prevalently used additives i.e., asphalt emulsion and liquid calcium chloride and investigate the proper mix design (ii) determine optimum stabilizing agent for gaining certain strengths associated with the type or gradation of the base/subbase material, and (iii) establish the effect of RAP on the strength properties of the pavement layer and the range of strength reduction with the increments of the RAP content.

In this chapter, all the components of RSB, i.e., aggregate, RAP and stabilizing agents are categorized and presented. The first section of the Chapter 2 describes the types of aggregate and RAP materials being used for laboratory testing along with the performed test to classify and determine the materials properties. In part 2, the test methodologies used throughout this project are outlined based on the stabilizing agent types.

2.1. MATERIALS

In order to explore the effect of particle size and gradation on the load bearing capacity of the stabilized pavement, three different gradations were selected for the mix designs with specimens of chemical and bituminous stabilizing agents. These materials include the field materials which is the subbase materials provided from different regions of the Vermont where RSB projects have been in progress or completed as well as three types of the manufactured subbase materials.

2.1.1. FIELD SUBBASE MATERIAL

Samples from Groton-Newbury and Winhall projects were provided by VTrans for performing the laboratory tests. Considering the quantity of these materials, they were used only for making cement-stabilized specimens.

Table 2.1 summarizes the samples' information and Figure 2.1 shows the GN2, GN3, and W3 subbase samples that were used for the laboratory testing of the finest gradation being used in this research project.

Table 2.1. Summary of samples' information

Sample ID	Project location	Sample Description	Notes
GN1	Groton- Newbury	TS-4 S2 (Subbase)	Gravely Sand
GN2	Groton- Newbury	TP-101 S2	Gravel, Sand w/cobble (2" to 9" diameter); Lt/brn M; Test pit depth represented: 2.05 to 2.9 feet
GN3	Groton- Newbury	TP-102 S2 (Subbase)	Sand w/cobble (up to 3" diameter); Lt/brn M; Test pit depth represented: 1.9 to 2.9 feet
W3	Winhall	Subbase	Gravely Sand

GN2 Subbase ample

GN3 Subbase sample

W3 subbase sample

Figure 2.1. Photos of the received GN2, GN3, and W3 Subbase samples.

2.1.2. MATERIAL TESTS AND CLASSIFICATION

Mechanical sieve analysis was run on all soil samples in accordance with ASTM C136: *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates* and ASHTO T27_T11 of the soil samples followed by Standard Proctor Compaction test in compliance with ASTM D698 *"Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort"*. Considering the course proportion of the subbase materials, ASTM D698, method A or B was selected which is compaction of aggregate passing sieve number 4 or 3/8" in 4" mold respectively for compaction and optimum moisture determination. Figure 2.2 shows the gradation curves of the field material and Figure 2.2 shows compaction curves.

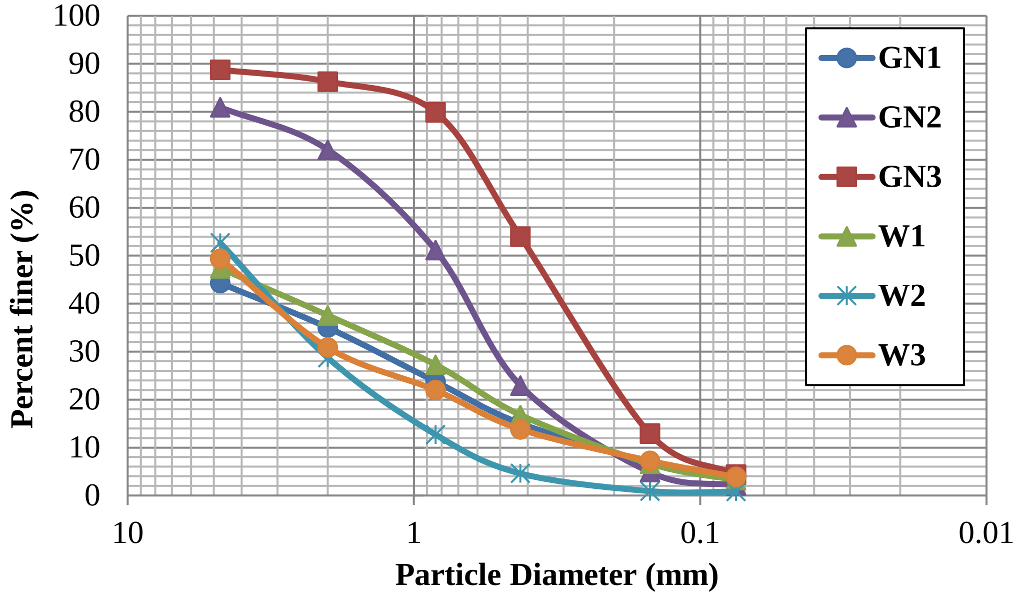


Figure 2.2. Gradation curves for soil samples

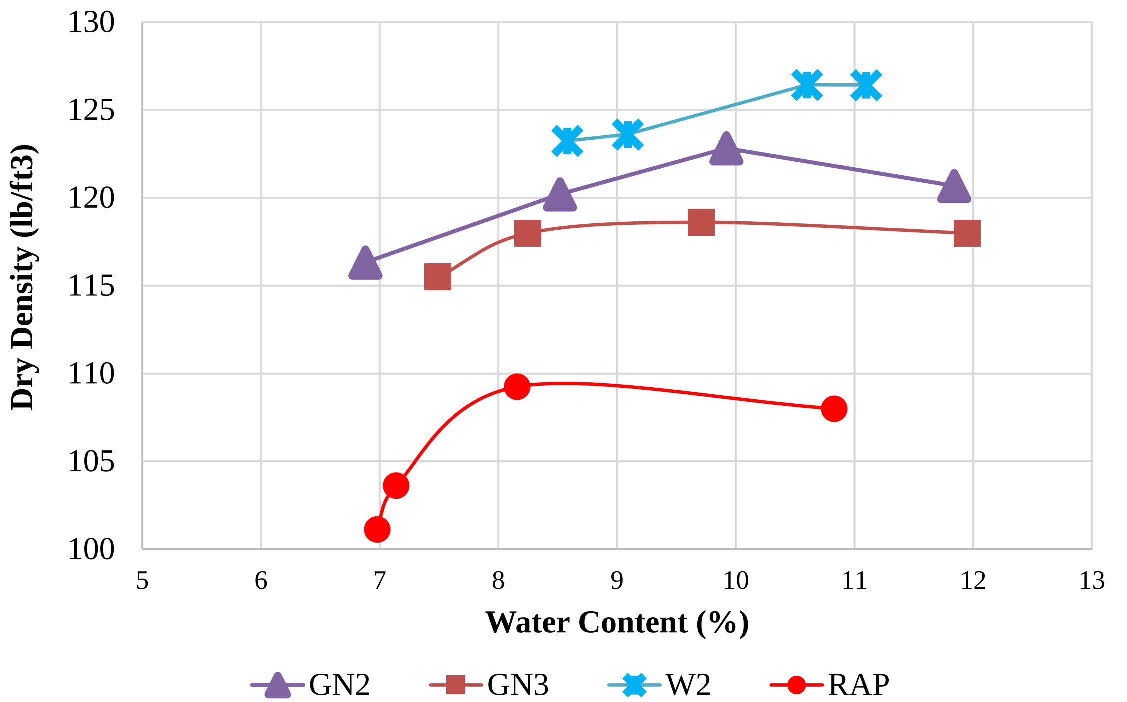


Figure 2.3. Compaction curves for soil samples

GN1, GN2 and W3 soils were used only for unconfined compressive strength on cement stabilized specimens. These subbase materials were passed from 3/4" and sieve number 4. The material to be used for making samples from these three soils is the material passing the sieve number 4, however, a stone correction of adding 20 % of the material retained on sieve no 4 and passing ¾ inches was added to adjust for filtered coarse portion

(VTrans report 2015). This is the finest grain to be used for preparing specimens. The results of the tests on these soils are shown in the Chapter 3.

2.1.3. MANUFACTURED SUBBASE MATERIALS

To factor in all the design parameters and exploring the effect of aggregate type and gradation, stabilizing agent, curing procedure, as well as establishing the potential correlation of these variables, large quantities of aggregate (subbase material); way more than the available field subbase samples i.e., GN1, GN2, GN3, W2, W3 were required. Hence, the required quantity of subbase material to the end of the project was supplied by producing “manufactured subbases” in the UVM’s Geomaterial Laboratory by mixing three types of soils provided by the aggregate supply vendor; “Livingston Farm” in Bristol VT. Figure 2.4 shows the acquired aggregate material.

(a) (b) (c)

Figure 2.4: (a) Livingston Farm aggregates, (b) bucketed river stone gravel, crushed gravel, and sand and (c) sieve analysis.

Laboratory tests to evaluate the RSB mixtures were performed on five soil type categories and gradations as is shown in Table 2.2. GN2 and GN3 were the field subbase materials which were used for laboratory tests on cement stabilizations, additionally, along with some other field subbase materials (i.e., GN1, W1, W2 and W3) they served as the determinants of the targeted gradations to be used for preparing the laboratory test specimens. The first specimens of cement stabilized soils were prepared with the finest gradation range i.e., the subbase materials GN2 and GN3 passing sieve number 4. In other words, a gradation with zero gravel content according to ASCS classification criterion.

Table 2.2. Proportions of the acquired aggregates in manufactured subbases.

Soil Name	Proportions				Gradation / Classification (USCS)
	Crushed Gravel (%)	Gravel (%)	Sand (%)	Kaolinite Clay (%)	
GN2	-	-	-	-	Passing sieve number 4
GN3	-	-	-	-	
SUBBASE I	60	20	20	0	Well graded sand with gravel
SUBBASE II	60	20	15	5	Well graded gravel with clay and sand
SUBBASE III	55	10	35	0	Well graded sand with silt and gravel

To obtain a subbase with a gradation close to the aforementioned subbase materials supplied by VTrans, multiple combinations of the three aggregate sizes under the names, “Crushed gravel”, “River Stone Gravel” and “Sand” were tried, and gradation curves were developed. Figure 2.5 shows the gradation curves of the three components of the manufactured subbases and Table 2.3 summarizes their soil classification information. RAP-S1 is the RAP used for making the specimens of soil-RAP stabilized with different additives.

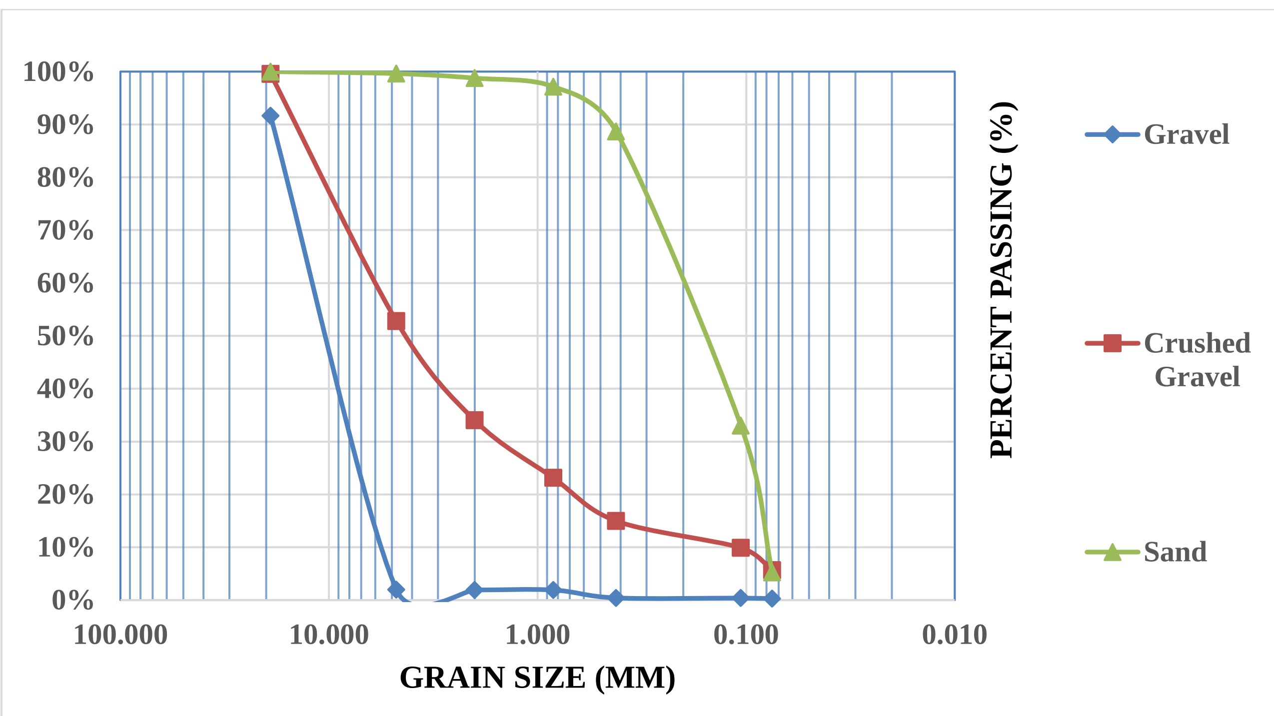


Figure 2.5. Gradation of the three components of the manufactured subbases.

Classification of the three components of the manufactured subbase and RAP.

Sample	Classification	USCS
River Stone Gravel	A-1a	GP
Crushed Gravel	A-1a	GW
Sand	A-3	SP
RAP_S1	A-1a	GW

Attempts to assess the liquid limit and plastic limit of select subbase samples were made in accordance with ASTM 4318 standards. However, due to negligible fine content, neither the liquid limit nor plastic limit was possible to be determined.

2.1.3.1. SUBBASE I

The first designed manufactured subbase material was created by mixing 20 % of Sand, 20% of Gravel and 60% of Crushed Gravel under the name "Subbase I." Figure 2.6 shows how Subbase I, II and III fall withing the gradation of the previously used/supplied subbase materials by VTrans for this research project from the undergoing s (e.g., Groton-Newbury (G.N) and Winhall sites (W), Brandon Goshen (B.G)) or the materials provided by VTrans' contractors (e.g., Kubricky (Kub.). As it can be seen from Figure 2.6, the gradation of the "Subbase I" falls within those of previously used/supplied subbase material by VTrans. With **45 % gravel** and 52 % sand is classified as ***"well graded sand with gravel"*** according to USCS.

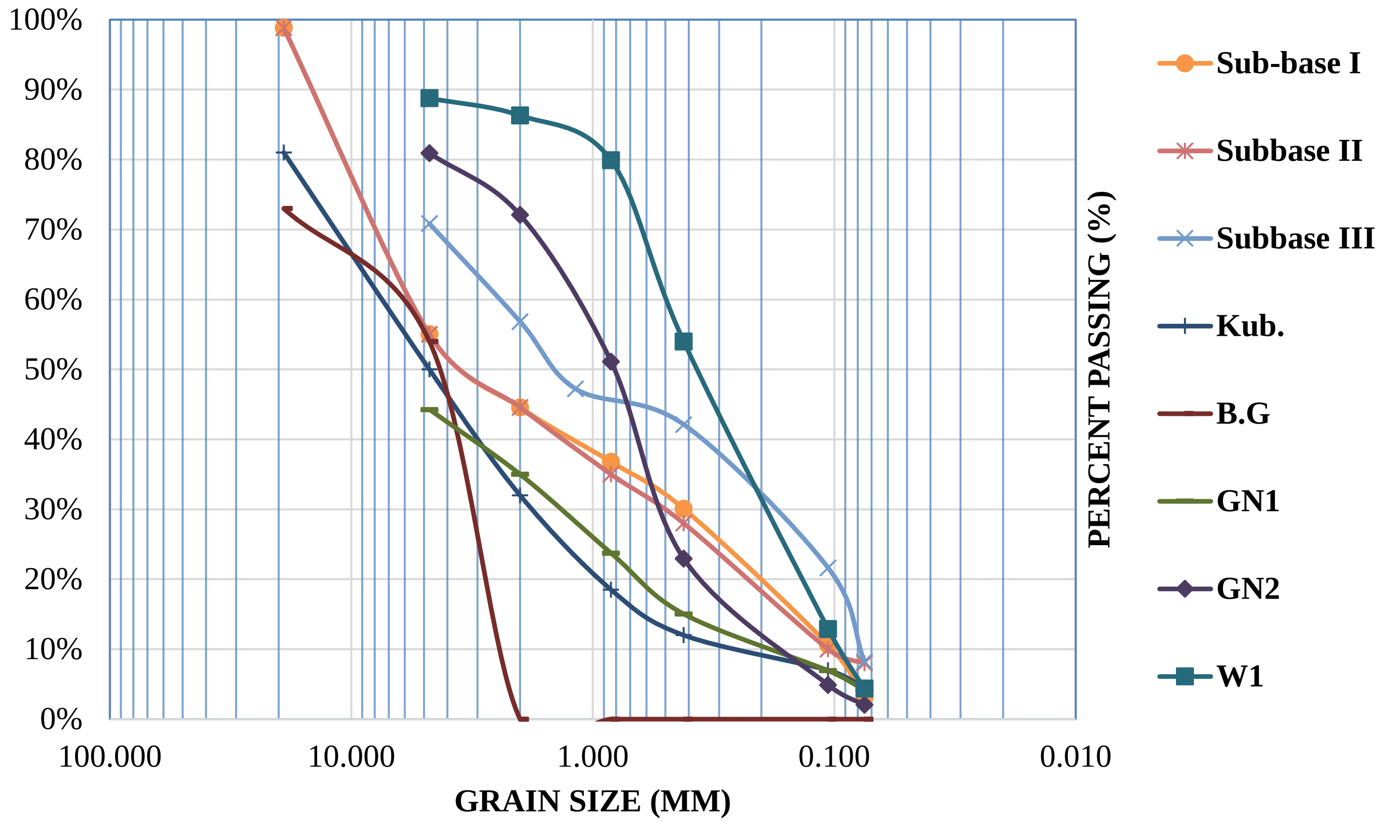


Figure 2.6. Gradation curves of Subbase I, II and III compared with other subbase materials.

Standard proctor compaction test was performed in compliance with ASTM D698 to determine the maximum dry density and optimum moisture content of the designed manufactured subbase materials. The compaction curve of the first manufactured subbase; "Subbase I" is shown in Figure 2.7.

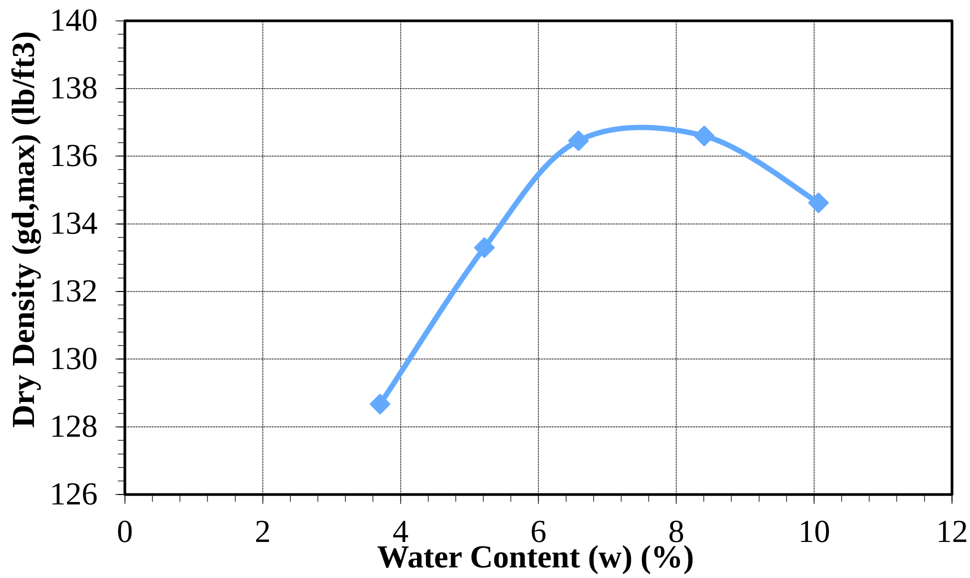


Figure 2.7: Compaction curve for the Subbase I

2.1.3.2. SUBBASE II

Subbase II was created by replacing 5% of the Sand portion in Subbase I with Kaolinite clay. The objective of creating this soil was to explore the effect of cementation of clay at low percentages on the bonding of the stabilized base and its contribution to dry density and unconfined compressive strength of the mixtures stabilized with different additives. The gradation of this soil is very close to Subbase I as it can be seen on Figure 2.6, however, the added clay changes the classification name to "***well graded gravel with clay and sand***" with **45% gravel** and 53% sand and 8 percent fine 5% out of which is Kaolinite clay. This added clay also decreases the OMC by 1%. The dry density vs water content (DD-WC) is shown in Figure 2.8.

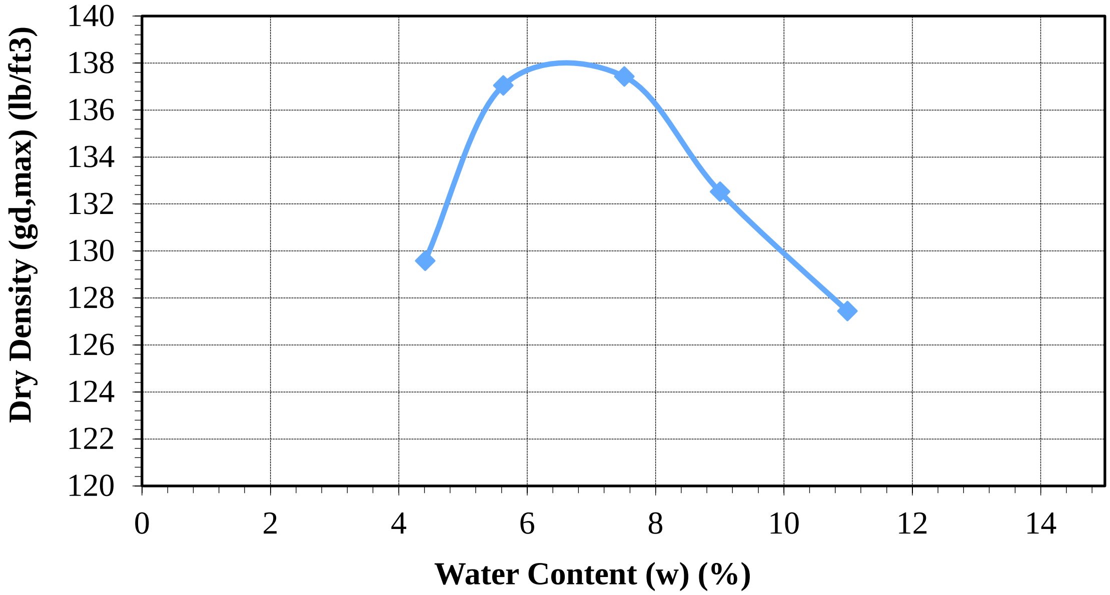


Figure 2.8: Compaction curve for the II

2.1.3.3. SUBBASE III

To cover a wide range of gradation, Subbase III was created by mixing the three soil samples in proportions that produce a gradation with **29% gravel** and 63 % sand and is classified as ***"well graded sand with silt and gravel."***

Particle size distribution of Subbase III is shown on Figure 2.9 and 2.6 and DD-MC plot on Figure 2.10.

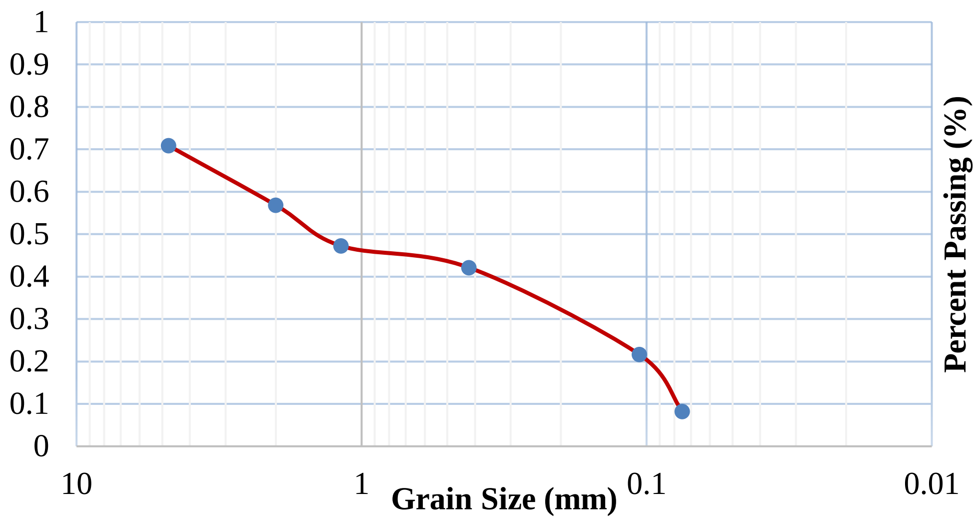


Figure 2.9. Gradation curves for SUBBASE II (mid graine aggregate)

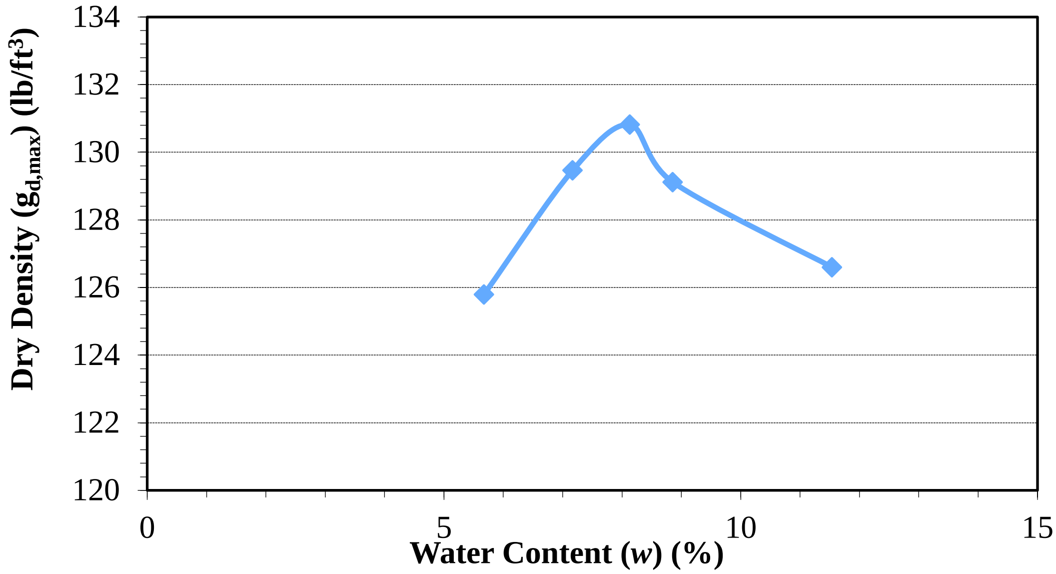


Figure 2.10: Compaction curve for the SUBBASE II

2.1.4. RAP

The research group received two series of RAP samples from station 194+00 of Stowe-Morristown RSB project. The buckets were labeled as sample 1 and sample 2. To use the sample S1 for preparing specimens and making sure that the gradation of the RAP and consequently the gradation of the subbase-RAP mixture falls within the gradation of the typical subbase materials; sieve analysis was performed on it. The RAP was classified as A-1a (AASTHO), ***well graded gravel*** (USCS). The gradation curve of the RAP sample is shown in Figure 2.11.

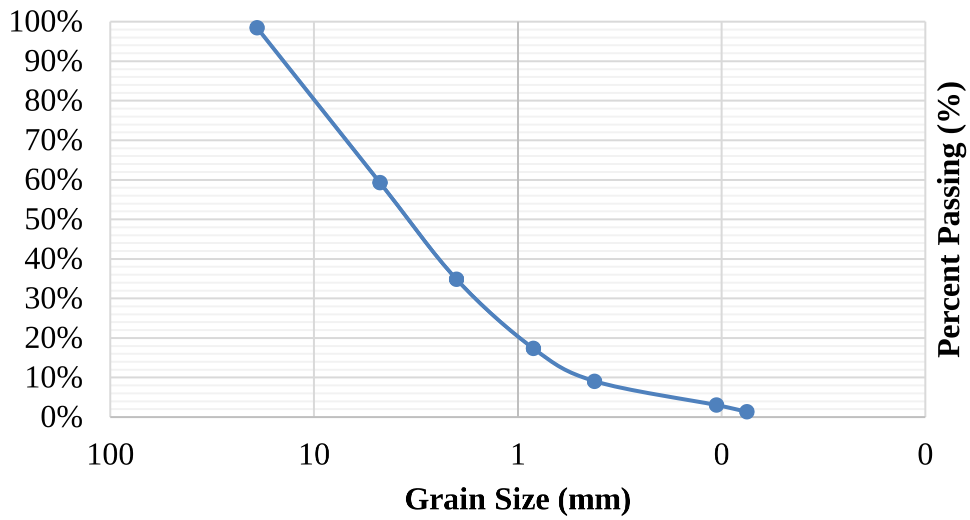


Figure 2.11: Gradation curve of RAP S1

This RAP was used was mixed at various proportions with the subbase materials to prepare the RAP-Subbase specimens.

2.2. TESTING METHODOLOGY

In this project two chemical additive i.e., cement and Liquid Calcium Chloride (LCC) and one bituminous additive i.e., Asphalt Emulsion (AE) were explored. Since sample preparation curing and testing standards and procedures are different for each additive, this section is structured based on the stabilizing agent type being used.

2.2.1. CEMENT STABILIZATION

2.2.1.1. SAMPLE PREPARATION

All types of soils and gradations presented in Table 2.1 were used for laboratory experiments on soil-cement mixtures. The summary of these materials is shown in table 2.4.

Table 2.4. Soil types being used for soil-cement specimens.

Soil No.	Soil Name	Gradation
1	GN2	Passing sieve number 4
2	GN3	
3	SUBBASE I	Well graded sand with gravel
4	SUBBASE II	Well graded gravel with clay and sand
5	SUBBASES III	Well graded sand with silt and gravel

The soil-cement specimens were prepared according to Designation: D1633 – 17 "*Standard Test Methods for Compressive Strength of Molded Soil-Cement Cylinders*" as well as a document provided by the VTrans from a 2015 report summary on reclaimed stabilized base laboratory testing. 2300-g batches of soil-cement mixtures were prepared at the optimum moisture content of the soil and were applied to the mold in three distinct layers. A total of 25 blows were delivered to each layer by a standard proctor hammer using the compaction machine.

2.2.1.2. CURING

Identical curing procedures were applied to all soil-cement and soil-cement-RAP specimens of all different soils and gradations. Upon compaction, the specimens were cured for 24 hours inside the mold in the fog room, and then extracted out of the mold and stored for 6 more days inside the fog room in compliance with ASTM

D1633. This condition provides sufficient moisture necessary to complete the chemical reactions of the cement in the soil-cement specimens. However, for the control specimens with 0% cement content, the excessive moisture available in the fog room prevents the specimens from drying out and gaining strength. Thus, these specimens were too brittle and testing them was not possible. This issue regarding the curing procedure of the control specimens according to ASTM D1633 has been reported by other researchers in the literature as well.

Another issue reported by researchers is the adverse effect of soaking of the specimens before testing. According to ASTM D1633, cement-stabilized specimens should be soaked in water at the end of curing time and before testing for four hours. Considering the low cement content of the specimens (4 % by the weight of the aggregate being the highest cement percentage), this step was skipped for all cement stabilized specimens. Figure 2.12 shows sample extraction and curing of the specimens.

(a) (b)

Figure 2.12: Extraction (a) and curing of cement and calcium chloride specimens(b)

2.2.1.3. TESTING

The testing criterion to evaluate the RSB mixtures stabilized with cement were selected to be the Unconfined Compressive Strength (UCS). After the end of the curing period, the specimens were tested for their UCS using the triaxial machine by applying the load to produce an axial strain at a rate of 0.5 percent per minute which is the lower limit specified in Standard Method of Test for Unconfined Compressive Strength of Cohesive Soil—AASHTO Designation: T 208-15 (2019). See the Figure 2.13.

Figure 2.13: Unconfined compressive strength testing of 4% cement stabilized specimen

2.2.2. LIQUID CALCIUM CHLORIDE STABILIZATION

2.2.2.1. MATERIAL

Three types of soils were used at different phases of the project for sample preparation and testing of LCC mixtures. The list of the soils being used for soil-LLC tests are shown in table 2.5.

Table 2.5. Summary of the soil samples being used for soil-LLC specimens.

Soil No.	Soil Name	Gradation
1	SUBBASE I	Well graded sand with gravel
2	SUBBASE II	Well graded gravel with clay and sand
3	SUBBASES II	Well graded sand with silt and gravel

2.2.2.2. ESTABLISHMENT OF THE CURING PROCEDURE

Although ASTM/AASHTO standards and several procedures are available for preparation and testing of samples stabilized with cement, lime, quick lime, or hydrated lime, such standards/procedures are not available for Liquid Calcium Chloride (LCC)-stabilized specimens. Therefore, the existing standards/procedures for lime, quick lime, or hydrated lime have usually been adopted and modified for preparation and testing of LCC by a few researchers who worked on this additive. The curing temperature and duration significantly contributes to the strength gain of lime-stabilized specimens. The first laboratory experiments on LCC attempted to establish this trend for LCC.

2.2.2.3. SAMPLE PREPARATION

A 35% liquid calcium chloride solution (typical in SFDR) was used to make LCC-stabilized specimens. The percentage of the LCC in each specimen was defined as the mass of 35%-LCC solution to the mass of the soil. Equivalent to the water portion in LCC (65% of the solution) was reduced from the mass of the additional water required to reach the optimum moisture content of the given soil. Based on the procedures of lime stabilization and findings of this research project, the curing temperature and time significantly contribute to the strength gain of lime/liquid calcium chloride specimens. (Shepard et al., 1991),(Choi, 2006). Therefore, usually, two sets of accelerated and normally cured specimens are made. These procedures were followed as the initial curing method with Subbase I to determine the proper curing method of LCC-stabilized mixtures.

2300-g batches of soil-LCC mixtures were prepared at the optimum moisture content of the soil and were applied to the 4” Standard Proctor molds in three distinct layers. A total of 25 blows were delivered to each layer by a standard proctor hammer using the compaction machine. Compacted specimens were extracted after 24 hours- when the specimens have gained enough strength and are not damaged during extraction.

2.2.2.4. CURING

Different curing procedures were explored to finally come up with a standard curing procedure applicable to all types of soil and the correlation of curing methods were finally established. The summary of the curing methods to be used within the subbase samples are shown in the Table 2.6 and in the following sections the process of curing procedure development is described with the order of the subbase materials to be used for the laboratory experiments.

Table 2.6. Summary of the curing methods.

Curing methods	Curing period (days)	Subbase I			Subbase II			SUBBASE II		
		2%	4%	6%	2%	4%	6%	2%	4%	6%
Curing in the Fog room	28	✓	✓	✓						
Curing in plastic wrap at room temperature	28	✓	✓	✓						
Curing at room temperature	28				✓	✓	✓	✓	✓	✓
Curing in the oven at 40 C (Accelerated curing)	7				✓	✓	✓	✓	✓	✓
Curing in oven at 40 C	28						✓			

The first two curing methods were adopted from the lime curing methods in the literature and were tested on LCC specimens of Subbase I. According to these methods, two replicate specimens at 2%, 4%, and 6% LCC were mixed and compacted and then cured at different conditions. The first curing method was curing the specimens at the temperature of 20.8℃ and humidity of 97.5 % in the curing chamber for 28 days. The second subset were cured in an air-tight plastic bags in the room temperature (22 ℃) for the same length of time (28 days).

The 28-days cured specimens were tested for unconfined compressive strength using the triaxial machine. As can be seen in Figure 2.14 the specimens that were cured in both conditions were not dried out and consequently did not gain much strength. This can be explained by the high deliquescent nature of calcium chloride (affinity for water) which tends to retain the moisture of the specimens and this is the very reason for which the LCC is used for dust control applications.

Figure 2.14. The high moisture retained inside specimens cured inside plastic wrap and fog room after 28 days.

These observations led to trying the next curing methods which were adopted as the standard curing method for LLC specimens throughout this research project. The curing methods were (i) curing the specimens in the ambient temperature of the lab (22 ^oC) for 28 days and (ii) curing the specimens inside the oven at 40 ^oC for seven days.

2.2.3. ASPHALT EMULSION

2.2.3.1. MATERIAL

The laboratory experiments on the Asphalt Emulsion (AE) treated soils were performed only on the Subbase I. The gradation and OMC of this soil was presented in Chapter 2.

The objective of the laboratory experiments on asphalt emulsion-stabilized tests were to determine the optimum asphalt emulsion content to be used for stabilization of base/subbase materials. To determine the trial emulsion content, the following formula were used (Barbod, 2014)

% Emulsion = $\frac{[(0.6*B)+(.01*C)]*100}{A}$

Where:

% Emulsion = Estimate initial percent asphalt emulsion by dry weight of the aggregate

A = Percent residue of emulsion by distillation

B = Percent of dry aggregate passing 4.75 mm (No 4 sieve).

C = 100 – B = Percent of dry aggregate retained on 4.75 mm (No 4 sieve).

Source (asphalt product exhibit 1009)

With the 57 % percent asphalt residue of the RS-1 anionic asphalt emulsion type were used,

A= 57%, B=55% and C=45% the trial emulsion content would be:

% Emulsion = $\frac{[(\mathbf{0.6*0.55})+(\mathbf{.01*0.45})]*\mathbf{100}}{\mathbf{0.57}}$ **= 5.5 %**

2.2.3.2. SAMPLE PREPARATION/MIX DESIGN

Specimens with two percentages above and below the initial emulsion content were prepared. The soil samples were oven-dried to prepare 1200 g batches of soil-asphalt emulsion mixtures sufficient for 1 Marshall mold for each AE content percentage. The oven dried soil sieved in compliance with AASHTO Designation: R 68-15 (2019) and ASTM D6927-15. The amount of added water (to prepare at optimum moisture content) was reduced to account for the 50% of water contribution by emulsion.

To prepare the mixture, required water added to the soil and mixed. After about one minute the AE was added to the moist soil. Batches of 1200 g of the subbase material with 3, 4, 5, 6 and 7% AE were prepared. (See Figure 5.1). In addition, replicate specimens were prepared with 70 % subbase material and 30 % RAP. All the specimens were mixed and compacted at room temperature. For all the specimens, using the manual Marshall assembly, 75 blows of the compaction hammer with a free fall of 457.2 mm (18 in) were applied and then the base plate and collar were removed, the mold was reversed, and another 75 blows were applied.

Paper filters were placed at the bottom of the Marshall molds and the mixture was transferred to the molds placing another filter on top of the mixture. The mixture was compacted immediately using Marshall compaction mold (75 blows, flip the mold, another 75 blows). See Figure 2.15.

a) (b) (c) (d)

Figure 2.15: (a) soil-emulsion mixture (3 %), (b) Marshall compaction mold and rammer (c) specimen extraction, and (d) curing of the specimen.

2.2.3.3. CURING

The compacted specimens were cured inside the mold at room temperature for 24 hours. After 24 hours, they were extracted out of the mold and cured inside the oven at 104 °F (40 °C) for 96 hours. See Figures 2.15 and 2.16.

Figure 2.16. Photo of the prepared asphalt emulsion specimens

2.2.3.4. TESTING

The cured specimens were tested for resistance to plastic flow at room temperature using triaxial machine and Marshall breaking head at constant rate of 50.8 mm/min in accordance with AASHTO T245-15a. The output of the tests was used for creating the Load-Deformation plots. The cumulative results of these tests are presented in Chapter 3. Figure 2.17 shows the Marshall Breaking head mounted on the triaxial machine.

Figure 2.17. Testing of the prepared asphalt emulsion specimens

CHAPTER 3. RESULTS AND DISCUSSION

In Chapter 2 the materials used for this research i.e., (i) aggregate type and range of gradation and (ii) stabilizing agents were introduced and the results of tests on the material (Proctor test, sieve and analysis and index tests) were presented. The research methodology (i.e., sample preparation, curing and testing method) pertinent to each stabilizing agent were also described in section 2 of chapter 2. The results and discussion related to those tests are presented in this chapter for each stabilizing agent type.

3.1. MATERIAL PROPERTIES

The information regarding the soils being used for all the tests in this research is summarized in Table 3.1 based on their course content order. Throughout this chapter the effect of gradation and stabilizing agent type and percentage will be presented by the categories of the soil types/gradations.

Table 3.1. Summary of the soil and their properties

Control Specimens (0% Additive)	Gravel content (%)	Classification	Added Clay (%)	Optimum Moisture Content (%)	Maximum Dry Density (lb/ft^3)
GN2	0	Sand	0	8	NA
GN3	0	Sand	0	9	NA
Subbase III	29	Well graded sand with silt and gravel	0	8	131.6
Subbase I	45	Well graded sand with gravel	0	7.5	137.6
Subbase II	45	Well graded gravel with clay and sand	5	6.55	140.8

As it can be seen in the above Table, increasing the portion of the gravel (course portion of the soil) corresponds to increased dry density and decreased OMC. The observations on the test results indicates that the type and percentage of the stabilizing agents does not dramatically contribute to the change in dry density compared to the dry density of the control specimens. However, adding clay to the Subbase I showed three units increase in the maximum dry density.

Note 1.

In reporting the data in the tables and plots presented in this chapter, for select percentages, repeat specimens were prepared and tested. In case of repeat specimens, in the tables that summarizes the values of UCS, the averages of the repeat specimens were used and the data that were the most consistent with the trend and repeated data were used for creating the plots that compare the values of UCS with the increments of the additive content or RAP.

Note 2.

Although the results of the mixes with different OMCs were not reported in this report, the observations of the researchers indicates that the precise determination of the OMC of the mixture crucially contribute to strength gain.

3.2. CEMENT STABILIZATION

The laboratory tests were started with evaluation of the improvement in the strength of the soil-cement mixtures where the course portion of the soil was removed. For this phase, existing subbase samples from the RSB project sites were used. In order to expand the scale of laboratory testing and determining the effect of gradation and course content with cement treatment, the manufactured soils were utilized as well. The test results of the soil-cement mixtures categorized based on the gradation of the soils are presented in what follows.

3.2.1. GN2 AND GN3 SOILS

Table 3.2 Summarizes the testing matrix of the specimens of GN2 and GN3. Specimens were prepared cured and tested following the standard procedures described in Chapter 2. The test results on these two soils are summarized in Tables 3.3 and 3.4 and are also illustrated in Figures 3.1 and 3.2.

Tables 3.2 The testing matrix of GN2 and GN3.

Test#	Mixture	Cement%	RAP%
1	Pure Soil	0%	0
2	Soil-Cement	2%	0
3	Soil-Cement	3%	0
4	Soil-Cement	3%	0

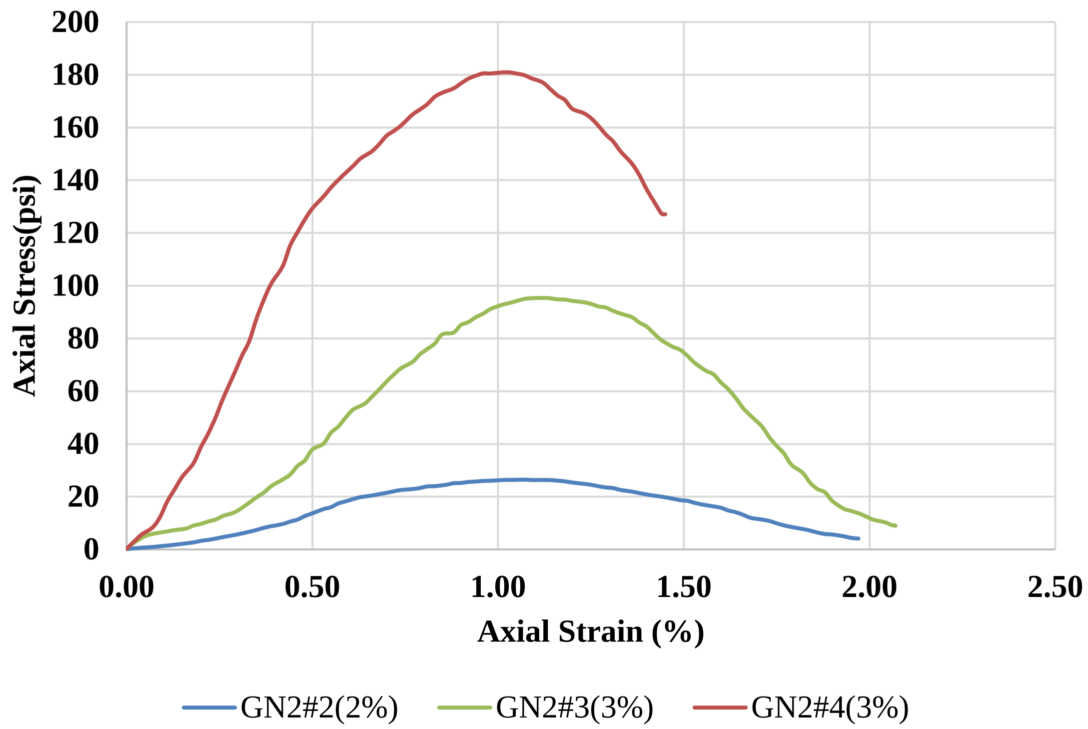


Figure 3.1: Variation of axial stress vs axial strain for GN2 Subbase-cement samples.

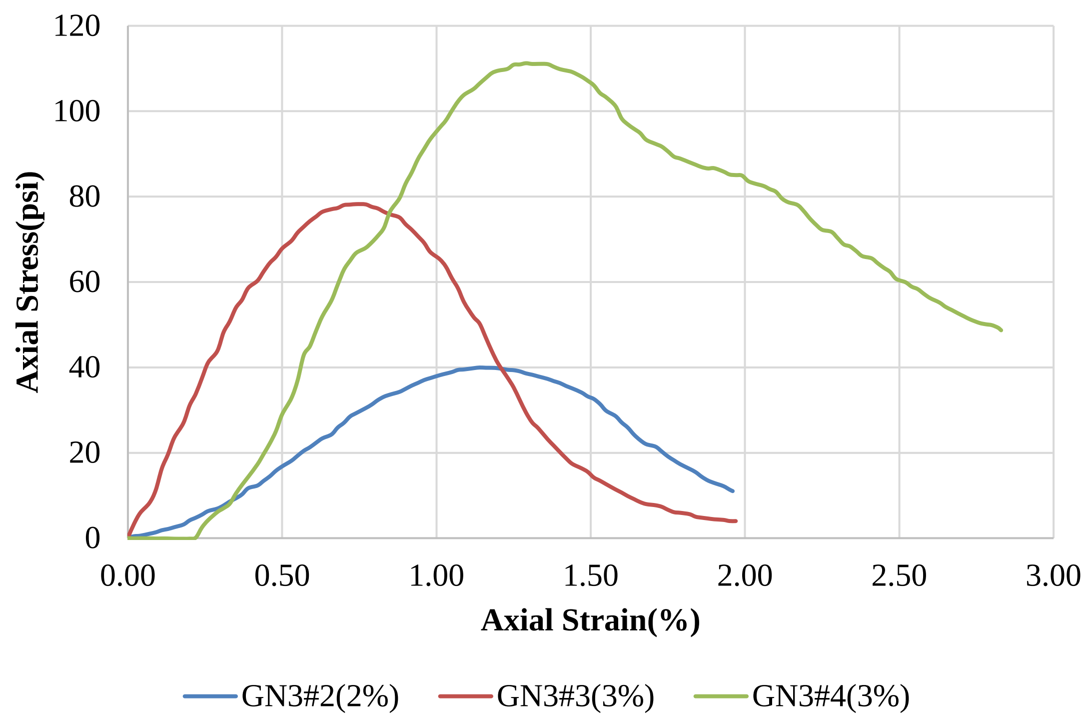


Figure 3.2: Axial stress-strain curves for GN3 Subbase-cement samples.

Table 3.3. Values of UCS for GN2

Sample	Cement Content (%)	OMC (%)	RAP (%)	Subbase (%)	UCS (psi)
1	0	8	0	100	NA
2	2	8	0	100	26
3	3	8	0	100	95
4	3	8	0	100	180

Table 3.4. Values of UCS for GN3.

Sample	Cement Content (%)	OMC (%)	RAP (%)	Subbase (%)	UCS (psi)
1	0	9	0	100	NA
2	2	9	0	100	39
3	3	9	0	100	75
4	3	9	0	100	111

As it was described in Chapter 2 GN2 and GN3 are the soils with 0% gravel since the molded specimens were prepared using the soil passing sieve number 4. These specimens make up the lowest UCS among all the soil-cement specimens. The range of UCS values for the 2% cement specimens is between 26 to 39 psi and for the 3% cement specimens is between 111 to 180 psi. These values are way below the 300-psi threshold that is used in many pavement designs. The low UCS values can be explained by the missing course portion. This was also confirmed by the test results of Subbases I, II and III which will cover a relatively wide range of gradations in terms of aggregate size (course proportion) in the following sections.

3.2.2. SUBBASE I

3.2.2.1. PURE SUBBASE

Subbase I and Subbase II are the soil samples with higher portions of the course material among the soils being used in this project (45% gravel). Gradation and OMC of these soils were explained in Chapter 2. Subbase I was classified as ***well graded sand with gravel***. Table 3.5 shows the test matrix of cement-stabilized specimens prepared using this soil.

Table 3.5. Cement stabilized specimens' matrix using Subbase I.

Test#	Mixture	Cement%	RAP%
1	Pure Soil	0	0
2	Soil-Cement	1%	0
3	Soil-Cement	2%	0
4	Soil-Cement	3%	0
5	Soil-Cement	4%	0
6	Soil-RAP-Cement	2%	15%
7	Soil-RAP-Cement	2%	30%

Specimens were tested following the same procedure explained in 3.1 and the plots of the axial stress vs axial strain are shown in Table 3.6 and Figure 3.3.

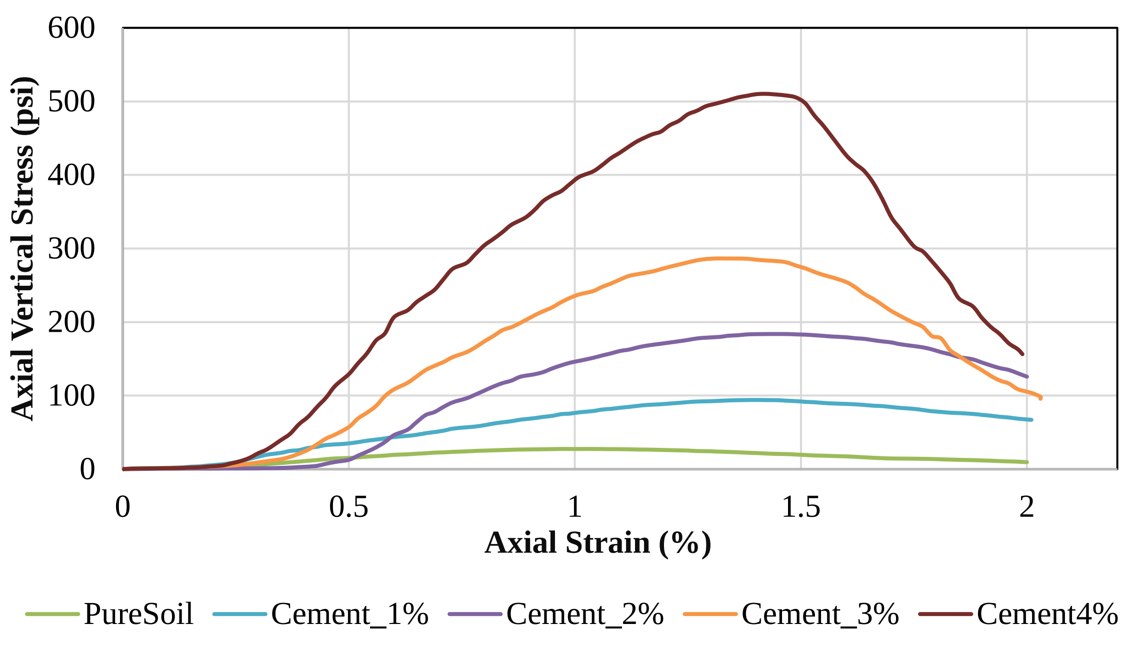


Figure 3.3: Axial stress-strain curves for cement stabilized specimens of Subbase I.

The test results show a significant increase in UCS of soil-cement specimens of subbase I compared to GN1 and GN2. With the same cement percentages, since the sample preparation and curing have been the same, this jump can be explained just by the change in aggregate size. Subbase I contained 45 % gravel compared to GN2 and GN3 with 0% gravel. A significant jump in the UCS values is evident with adding this portion of the course material. The range of UCS values for the 2% cement specimens jumped from 63 psi (average value for GN2 and Gn3) to 184 psi. This increase for the 3% cement specimens is from 145 psi to 286 psi.

The Linear relation of the UCS corresponding with cement content for the Subbase I is shown in Figure 3.4.

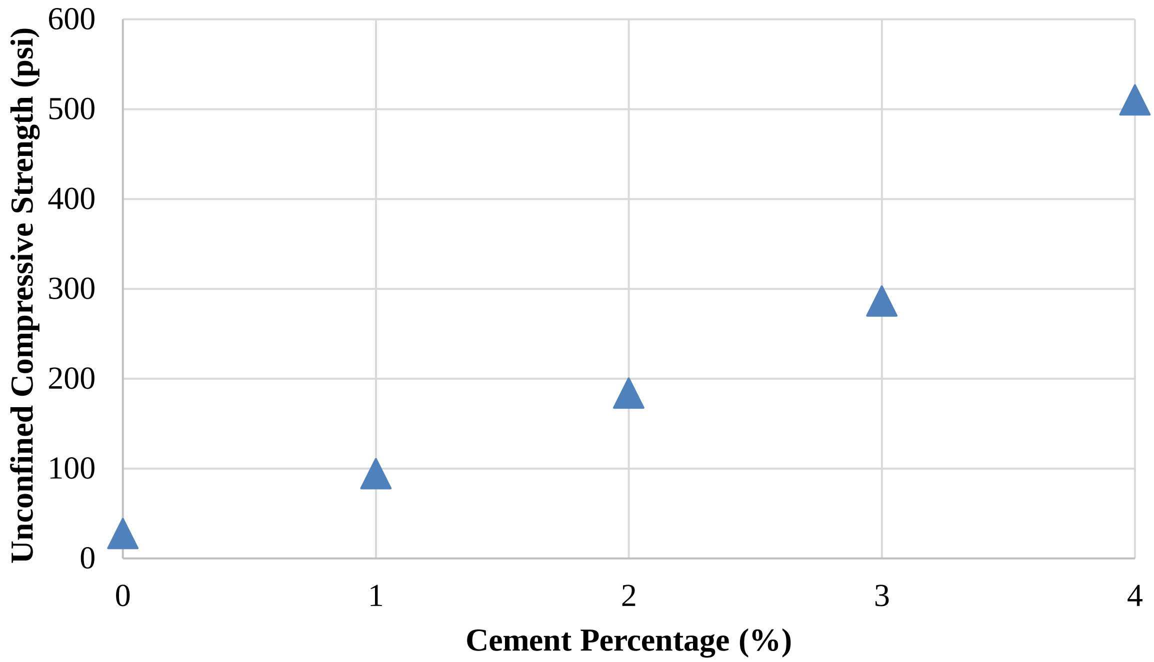


Figure 3.4: Linear relation of cement content and UCS.

The plot in Figure 3.4 can serve as a good tool for determination of the optimum cement content for the targeted strength when a field soil is within a gradation close to Subbase I as this gradation is the common gradation to be used in RSB projects according to the literature in this field. For instance, given the UCS target of 300 psi, 3% would be the optimum percentage of the stabilizing agent (cement).

3.2.2.2. SUBBASE + RAP

As explained in Chapter 2, a RAP sample classified as A-1a (AASTHO), ***well graded grave*l** (USCS) was used for preparing Subbase-RAP specimens in order to evaluate the effect of RAP content on the strength properties of the pavement layer stabilized with different type and percentages of the stabilizing agents. Samples containing 15 and 30% RAP were prepared using Subbase I at the OMC and tested in accordance with the same standards procedures employed for making soil-cement specimens. The same curing procedure was applied for these specimens as well. Figure 3.5 illustrates the variations of the stress with incorporation of 15 and 20% RAP to the Subbase material.

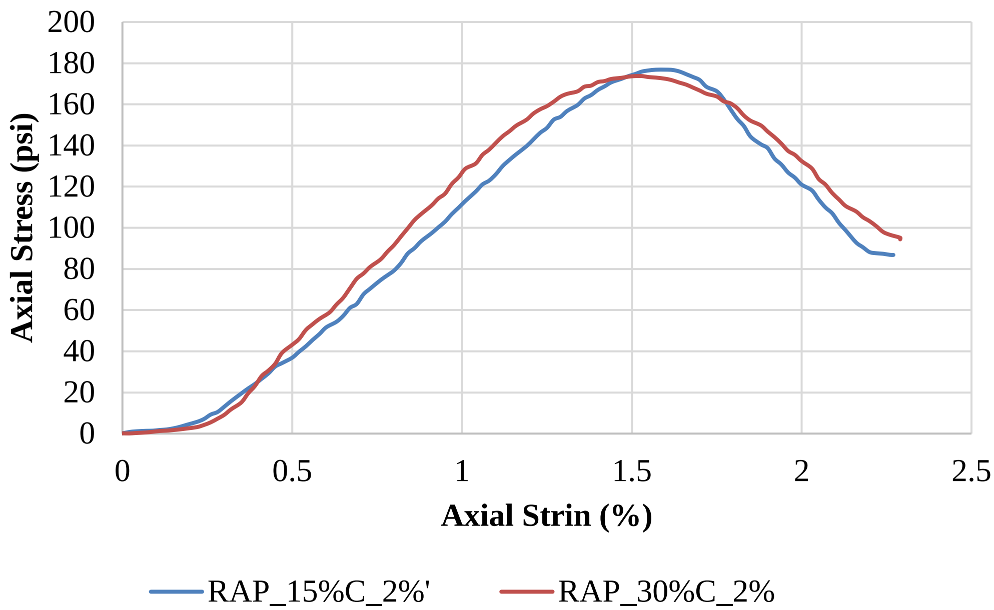


Figure 3.5: Axial stress-strain curves for cement stabilized specimens of Subbase I, containing 15 and 30 % RAP.

Based on the obtained laboratory results in this study, adding up to 30 % RAP to a soil with gradation similar to Subbase I does not compromise the UCS significantly.

Finally, All the UCS values of tested specimens with and without RAP content were summarized in Table 3.6 to be used for comparing the results of this gradation with the data on the test results of soils with different gradations stabilized with cement in the following sections.

Table 3.6 UCS values for Subbase I.

Sample	Cement Content (%)	OMC (%)	RAP (%)	Subbase (%)	UCS (psi)
1	0	7.5	0	100	28
2	1	7.5	0	100	94
3	2	7.5	0	100	184
4	3	7.5	0	100	286
5	4	7.5	0	100	510
6	2	7.5	15	85	177
7	2	7.5	30	70	173

3.2.3. SUBBASE II

3.2.3.1. PURE SUBBASE II

The sample preparation and test methods for Subbase II were identical to what described in sections in Chapter 2 and sections 3.2.1 and 3.2.2. The information for the specimens of this soil is summarized in Table 3.7.

Table 3.7. Cement stabilized specimens' matrix of Subbase II.

Test#	Mixture	Cement%	RAP%
1	Pure Soil	0	0
2	Soil-Cement	1%	0
3	Soil-Cement	2%	0
4	Soil-Cement	3%	0
5	Soil-Cement	4%	0
6	Soil-Cement + RAP	2%	20%
7	Soil-Cement + RAP	2%	30%
8	Soil-Cement + RAP	2%	40%

The gradation of this soil was almost the same as Subbase I except for the 5% clay added to this soil compared to Subbase I. Surprisingly, this 5% clay content substantially increased the UCS. This additional strength gain with the same course proportion may be justified by: (i) role of the clay particle as a filler and improving the particle size distribution, and (ii) contribution of Kaolinite clay in chemical reaction and acting as additional cement content. Figure 3.6 illustrates the variation in UCS with increments of the cement content. The UCS values of different soils with the same contents is compared in the summary of this Chapter.

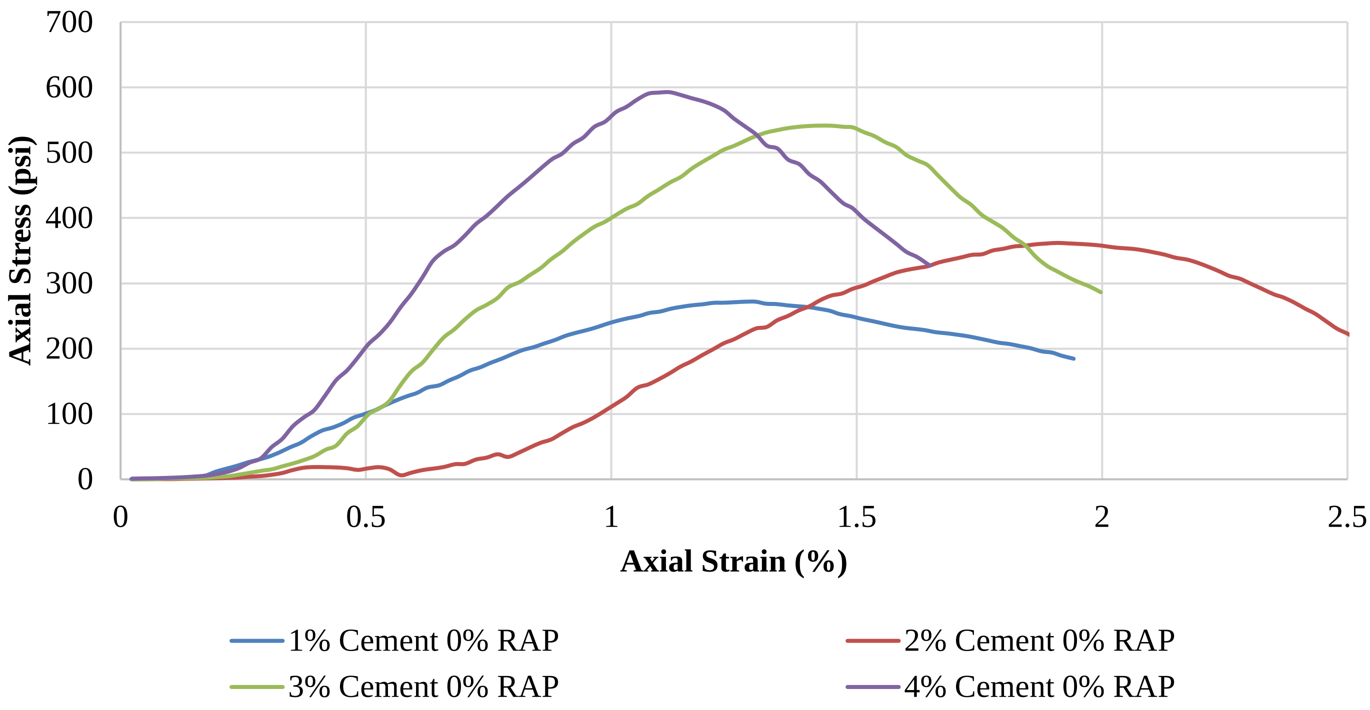


Figure 3.6. Unconfined compressive strength of soil-cement specimens of Subbase II.

Using the available USC, the correlation of the UCS and cement content for Subbase II established as well. See Figure 3.7. The remarkable point here is that the 300-psi targeted UCS which obtained with 2% cement content with Subbase I can be obtained with approximately 1 percent cement using Subbase II.

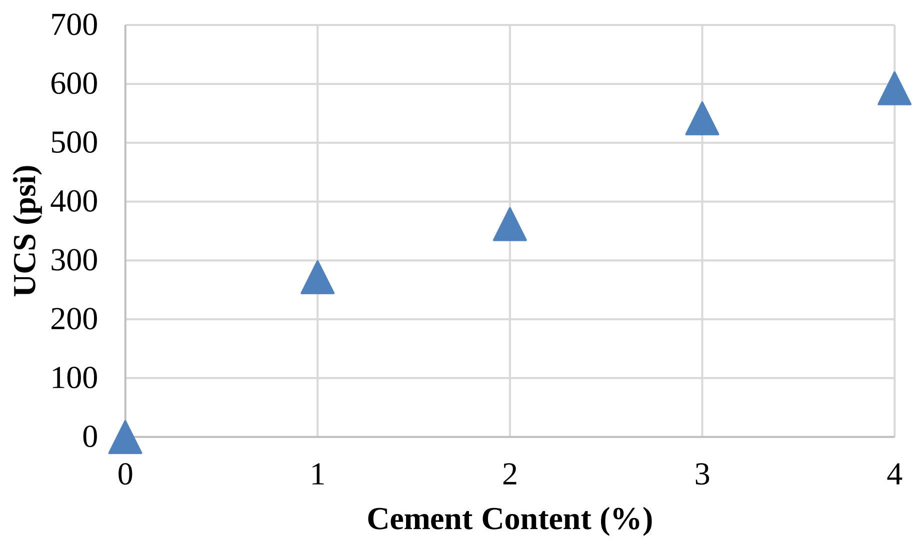


Figure 3.7. UCS VS cement content for Subbase II

3.2.3.2. SUBBASE II + RAP

For the sake of comparison and observing the trends of increase/reduction of UCS, all the results of cement specimens with and without RAP are plotted together in Figure 3.8 and Table 3.8. The UCS reduction trends with the increments of RAP content in specimens with 2% cement are shown in the cumulative plot. Through comparing the UCSs, one can read from this plot that incorporating up to 30% RAP to subbase II stabilized with 2% cement will still keep the UCS above300 psi.

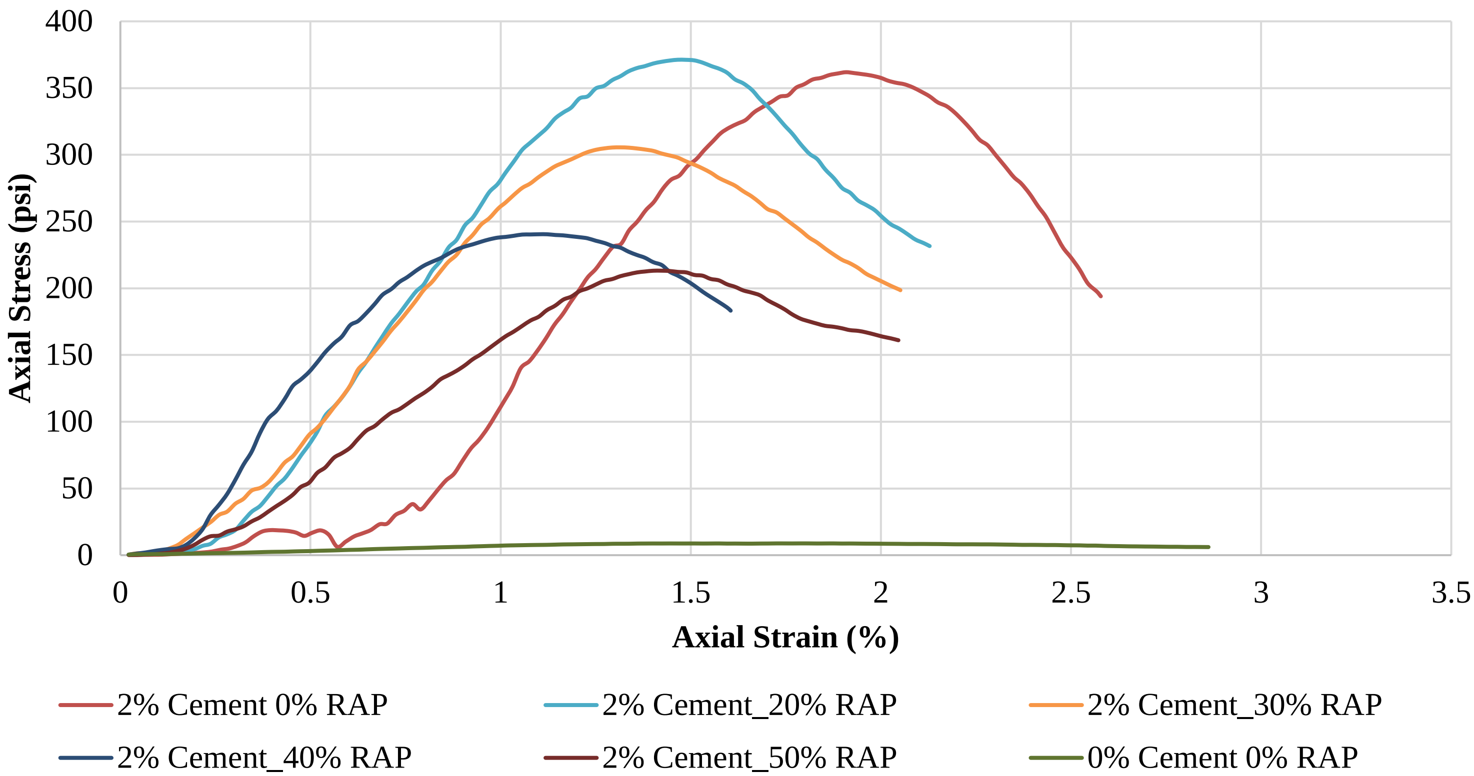


Figure 3.8. Unconfined compressive strength of soil-cement specimens with increments of RAP content.

Table 3.8. The peak UCS values of cement stabilized specimens with and without RAP content for Subbase II.

Sample	Cement Content (%)	OWC (%)	RAP (%)	Subbase (%)	UCS (psi)
1	0	6.5	0	100	9
2	1	6.5	0	100	272
3	2	6.5	0	100	362
4	3	6.5	0	100	541

5	4	6.5	0	100	593
6	2	6.5	20	80	313
7	2	6.5	30	70	305
8	2	6.5	40	60	240
9	2	6.5	50	50	213

3.2.4. SUBBASE III

3.2.4.1. PURE SUBBASE III

The sample preparation, additive percentage, curing and test method for Subbase III were also identical to the Subbase I and II and the only difference was the gradation of the Subbase material. The test results on this soil showed a new range of UCS values which is presented in the following sections. Information pertinent to the samples are shown in table 3.9.

Table 3.9. Cement stabilized specimens' matrix for Subbase III.

Test#	Mixture	Cement%	RAP%
1	Pure Soil	0	0
2	Soil-Cement	2%	0
3	Soil-Cement	3%	0
4	Soil-Cement	4%	0
5	Soil-Cement + RAP	2%	20%
6	Soil-Cement + RAP	2%	30%
7	Soil-Cement + RAP	2%	40%

The results of the tests on specimens prepared with Subbase III and zero RAP content are shown in Figure 3.9.

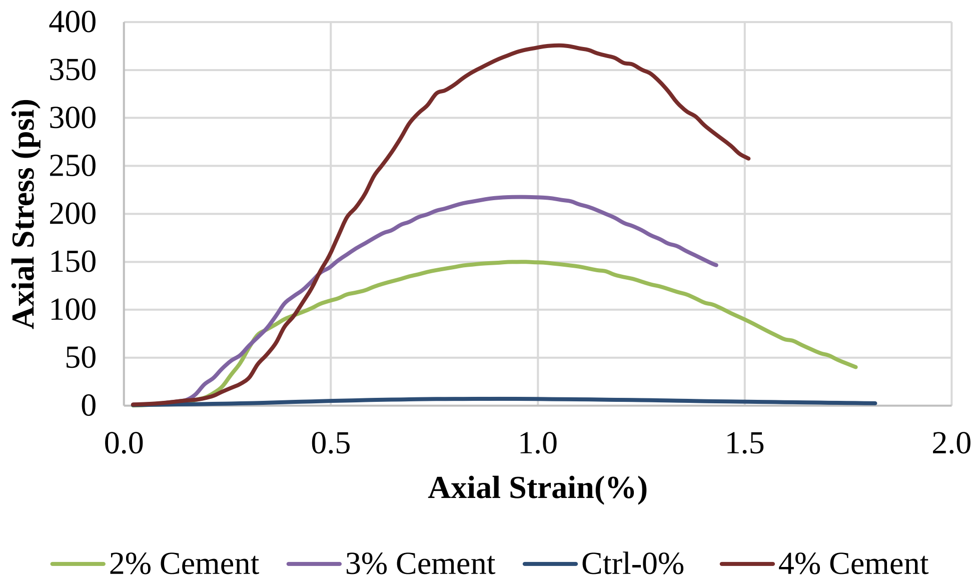


Figure 3.9. UCS values of Subbase III specimens

Subbase III with **29% gravel** and 63% compared to Subbase I and II contains 16% lower gravel content and remarkably lower UCS. The range of UCS for this soil as is shown on the above plot for 2%-cement specimen is 149 psi and for the 3%-cement specimen is 217 psi. To reach the targeted UCS cement contents beyond 3% is required while this limit is gained with 2 and 3% additive with subbase II and Subbase I respectively.

Using the available USC, the correlation of the UCS and cement content for Subbase III established as well. See Figure 3.10. The 300-psi targeted UCS which obtained with 2% cement content with Subbase and 1% cement using Subbase II is gained with 4% cement which implies the risk of shrinkage. Thus, this gradation is recommended to be avoid in stabilization with cement.

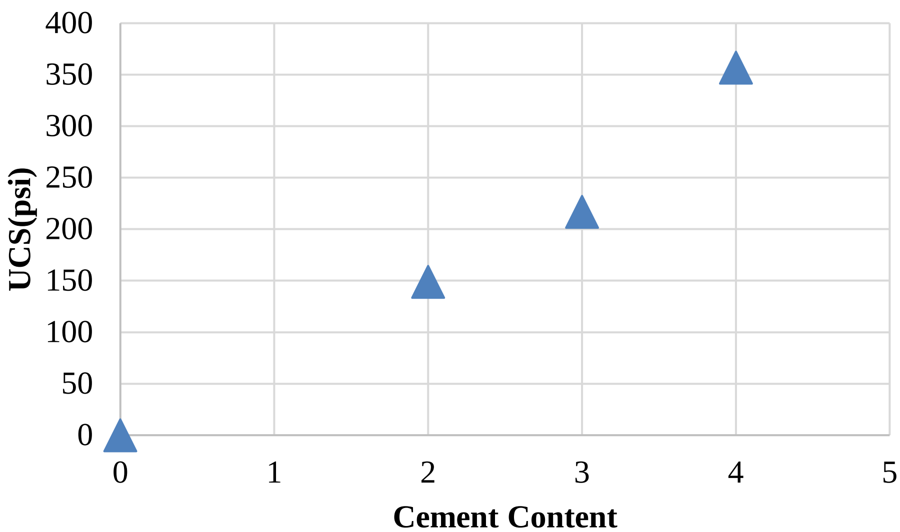


Figure 3.10. UCS VS cement content for Subbase III

Additionally, incorporating RAP to this soil with cement additive of 2% showed higher compromise of strength as is presented in the following section.

3.2.4.2. SUBBASE III +RAP

The cumulative plots of UCS of specimens with and without RAP is shown in Figure 3.11

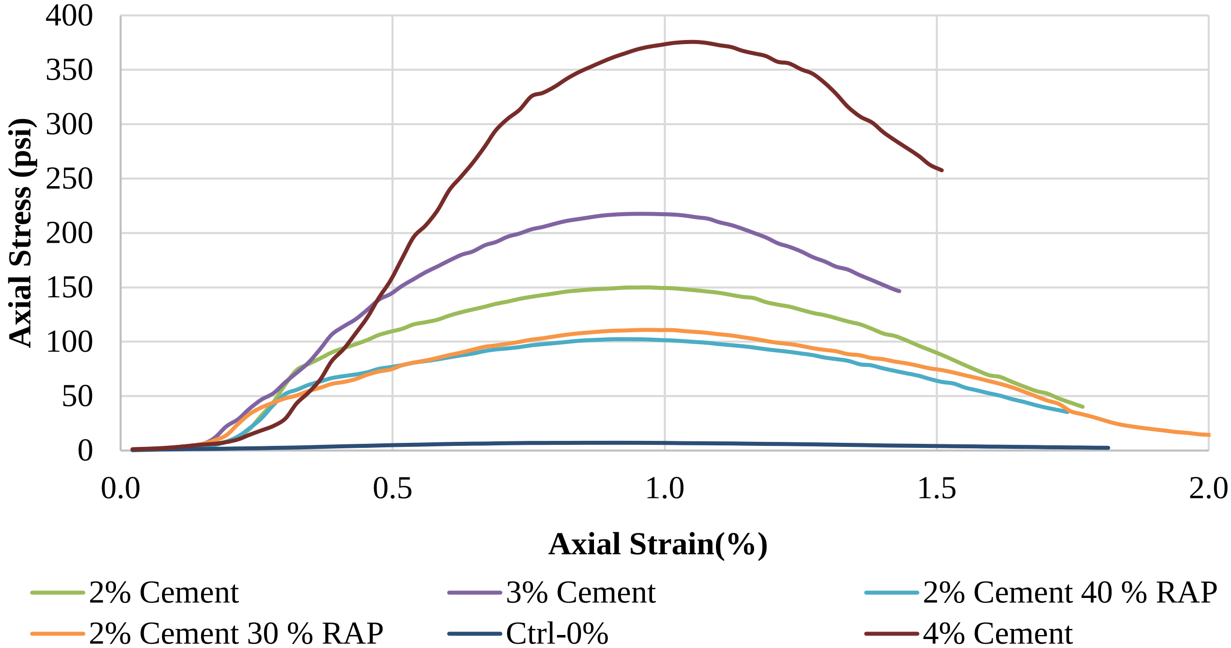


Figure 3.11. Reduction of UCS values with increments of RAP content for Subbase III

Table 3.10. The peak UCS values of cement stabilized specimens for Subbase III.

Sample	Cement Content (%)	OMC (%)	RAP (%)	Subbase (%)	UCS (psi)
1	0	8	0	100	7.15
2	2	8	0	100	149.0
3	3	8	0	100	217.0
4	4	8	0	100	357
5	2	8	20	80	NA*
6	2	8	30	70	110
7	2	8	40	60	102

*: Data is not available due to testing machine failure.

As it is shown in Figure 3.10 and Table 3.10 the trend of reduction in UCS of 2%-cement specimens with the increments of RAP is not very dramatic with up to 40% RAP content.

3.2.5. SUMMARY

Identical procedure and standards were applied for sample preparation, curing and testing of the specimens (ASTM D1633). The variable parameters were additive percentage and the soil gradation i.e., the course proportion of the soil. In addition to GN2 and GN3 subbase samples, three other subbase samples i.e., Subbase I, Subbase II and Subbase III were created. The objective of conducting the tests on different gradations was to evaluate the effect of gradation on UCS of cement-stabilized specimens and potentially establish the optimum cement content for a specific range of gradation.

All the data on cement stabilization tests presented in sections 3.1 through 3.4 are summarized in Figure 3.12 and Table 3.11 to make the analysis and drawing conclusions easier.

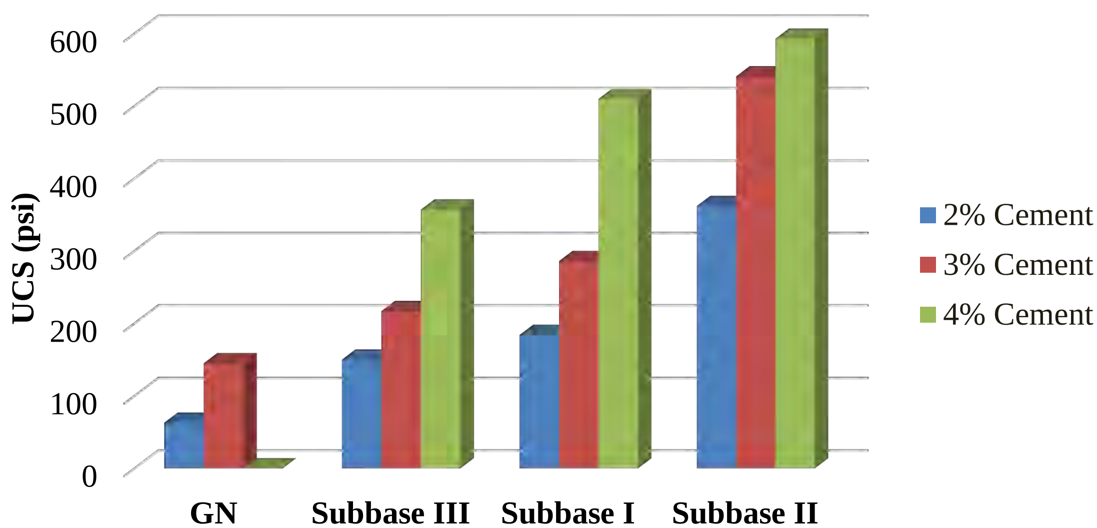


Figure 3.12. Variation of UCS with gradation and percent cement

Table 3.11. UCS of the cement-stabilized specimens

UCS (psi)	2% Cement	3 % Cement	4% Cement
GN[1]	63	145	NA
Subbase III	150	217	357
Subbase I	184	286	510
Subbase II	362	541	593
[1]: The average values for GN2 and GN3 as the soils are very similar in terms of their properties and gradation.			

The highlights of the laboratory experiments on cement-stabilized specimens are summarized below:

Note:

GN2 and GN3 refer to the samples of project site soils passing sieve no. 4 (0% gravel)

Subbase I refers to the manufactured Subbase with 45% gravel content.

Subbase II refers to the manufactured Subbase with 45% gravel content and 5% added Kaolinite clay.

Subbase III refers to the manufactured Subbase with 29% gravel content.

1. Specimens prepared, cured, and tested following the same procedures, but with different aggregate gradation exhibited substantially different UCS.
2. Specimens were prepared considering VTrans' past experiences in both avoiding the excessive cement content (maximum of 4%) and gaining the target UCS value of around 300 psi.
3. Fine graded soil with 0% gravel (GN2 and GN3 passing sieve number 4) showed the lowest ranges of UCS. The average UCS of these soils for 2% and 3% cement content were 63 and 145 psi, respectively.
4. A significant increase in the UCS values is evident with increasing the course portion of the soil from 0% (GN) to 45% (Subbase I). The range of UCS values for the 2% cement specimens increased from 63 psi (average value for GN2 and Gn3) to 184 psi (almost three folds). This increase for the 3% cement specimens was from 145 psi to 286 psi (almost two folds).
5. Adding 5% clay to the Subbase I reduced the OMC by 1% and increased the UCS significantly.
6. The Subbase II showed the highest UCS values, compared to other subbase specimens.
7. To obtain a UCS of around 300 psi, with Subbase I, II, and III, required cement percentages are around 3%, 2%, and 4% respectively.
8. Kaolinite clay at low percentages (5% in the case of this study) can serve as additional cement content, but more economic substitute. This can be considered for further investigation for its economic benefits.
9. Incorporating up to 30% RAP into subbase II stabilized with 2% cement, the UCS will likely stay above 300 psi.
10. The range of UCS for Subbase III is 149 psi for 2% cement and for the 3%-cement specimen is 217 psi. To reach the targeted UCS of around 300 psi, cement contents beyond 3% is required while this target strength is achieved with about 2% and 3% cement additive with Subbase II and Subbase I, respectively.
11. Gaining 300 psi or higher UCS with Subbase III requires cement contents beyond 3% which is associated with increased rigidity, cracking potential and heaving. Thus, RSB in pavements where the

subbase layer is poor in course contents are not proper for RSB and grade improvement should be considered by adding sufficient quantity of gravel.

12. For the 2% cement-additive specimens, Subbase III, with higher portions of sand and lower gravel portion compared to Subbases I and II, showed a higher reduction in UCS when RAP was incorporated into this soil.

3.3. LIQUID CALCIUM CHLORIDE

3.3.1. SUBBASE I

As stated in Chapter 2, due to lack of sufficient information on sample preparation, curing and testing of LLC-stabilized mixtures, the first experiments aimed at establishing a standard laboratory procedure, in particular, a proper curing method. The first samples were prepared using Subbase I and cured under conditions summarized in Table 3.12. Also, Table 3.12 outlines the test matrix for the Subbase I.

Table 3.12. The summary of prepared specimens using Subbase I.

Test#	Mixture	OMC (%)	Calcium Chloride %	Curing Conditions
1	Soil-Calcium Chloride	7.5	2%	28 days at room temperature inside plastic wrap
2	Soil-Calcium Chloride	7.5	4%	28 days at room temperature inside plastic wrap
3	Soil-Calcium Chloride	7.5	6%	28 days at room temperature inside plastic wrap
4	Soil-Calcium Chloride	7.5	2%	28 days inside the fog room at 22°C and humidity of 97%
5	Soil-Calcium Chloride	7.5	4%	28 days inside the fog room at 22°C and humidity of 97%
6	Soil-Calcium Chloride	7.5	6%	28 days inside the fog room at 22°C and humidity of 97%

The stress-strain plots from unconfined compressive strength of the specimens stabilized using LCC (cured inside curing chamber and plastic wraps) are shown in Figure 3.13. As it was evident from the retained moisture inside the specimens even after testing (at end of the curing time), the specimens cured inside plastic wrap had less chance of drying out, and consequently gained lower strength compared to the specimens cured in the fog room.

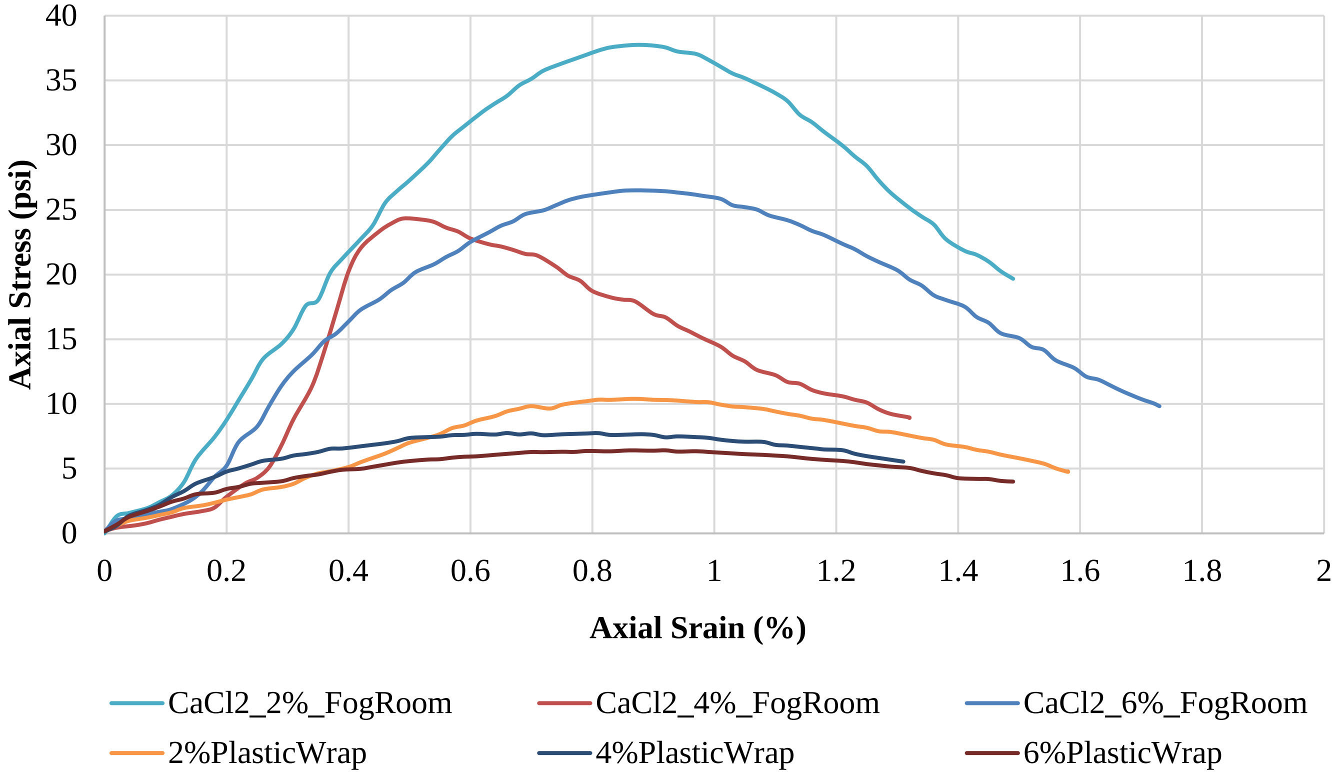


Figure 3.13. Unconfined compressive testing results for the 28-days cured calcium chloride specimens.

These observations of the examined curing method proved that unlike cement and lime stabilization, preserving the moisture of the specimens during the curing of the LCC specimens is not relevant, and curing is more about drying the specimens out. These preliminary results led the research team to cure the specimens under conditions in which they have a chance to lose moisture during the curing period and gain strength. The next curing procedure was tried on Subbase II and the rest of the specimens and turned out to be the appropriate curing method for LCC stabilized laboratory test specimens.

3.3.2. SUBBASE II

After determining the curing procedure, the first tests on LCC stabilized mixtures were preformed using the Subbase II. The information about the prepared samples and curing conditions are summarized in Table 3.13. Figure 3.13 shows the curing and testing of the specimens.

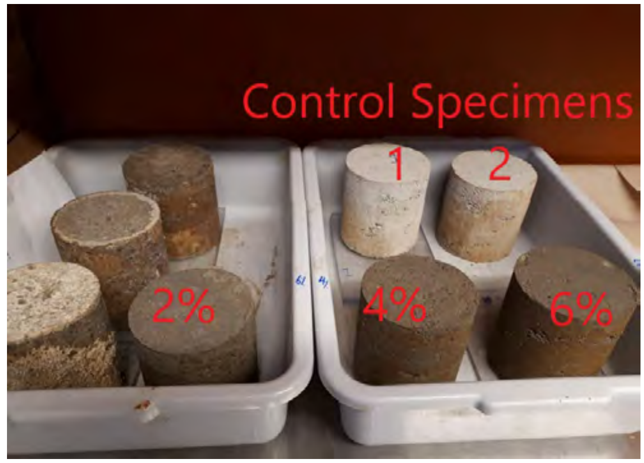
3.3.2.1. PURE SUBBASE II

Table 3.13. Testing matrix of the prepared/tested calcium chloride-stabilized specimens of Subbase II.

Test#	Mixture	OMC (%)	Calcium Chloride %	Curing Conditions
1	Control specimens	6.5	0	7 days at room temp.
2	Control specimens	6.5	0	28 days at room temp.
3	Soil-Calcium Chloride	6.5	2%	28 days at room temp.
4	Soil-Calcium Chloride	6.5	4%	28 days at room temp.
5	Soil-Calcium Chloride	6.5	6%	28 days at room temp.
6	Soil-Calcium Chloride	6.5	2%	7 days in oven and 104°F (40C)
7	Soil-Calcium Chloride	6.5	4%	7 days in oven and 104°F (40C)
8	Soil-Calcium Chloride	6.5	6%	7 days in oven and 104°F (40C)

The curing process and testing of the specimens cured at room temperature and oven are shown in Figure 3.14. As it is shown in Figures 3.14(a) the tardiness in drying of LCC specimens is clearly evident compared to control specimens. Even though curing the specimens for 28 days at room temperature is relatively an effective way, however, compared to control specimen, after 28 days, LCC specimens seemed to have retained a little bit more moisture. In terms of UCS, however, LCC specimens of 2 and 4% LCC showed higher LCC compared to control specimens thanks to the properties of LCC.

Day 1



Day 7

Day 14

Day 21

(a)

(b)

(c)

Figure 3.14. (a) curing for 28-days at room temperature, (b) curing for 7-days in oven at 104°F, and (c) UCS testing of prepared specimens

Figure 3.15 shows the cumulative plots of axial stress vs axial strain of UCS tests performed on two sets of liquid calcium chloride stabilized specimens cured under two different curing conditions-28 days at room and 7 days at oven.

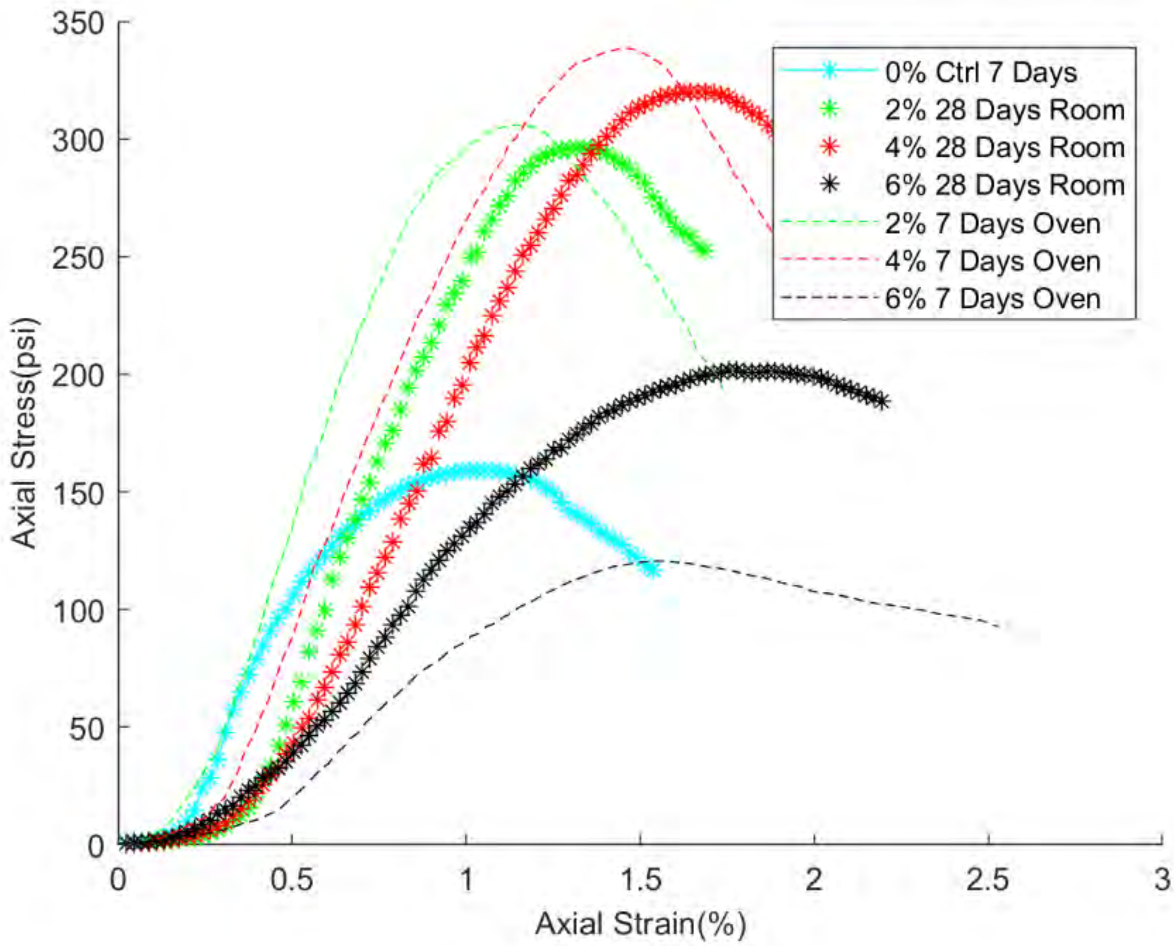


Figure 3.15. Variation of axial stress versus axial strain for calcium chloride-stabilized specimens cured for 7 and 28 days.

Comparing the curves of axial stress associated with the UCS values of 2, 4, and 6% LCC content, there is a relatively good consistency between the UCS test results of both curing methods. (See star and dash curves). The second remarkable fact is that 4% LCC is associated with the highest UCS for both curing methods. Comparing UCS gained by 2%, 4%, and 6% calcium chloride stabilized specimens, it is evident that the 6% $CaCl_2$ had the lowest strength gain after curing in both conditions. In addition to lower strength, performing the moisture content tests on the tested specimens (after 28 days curing at room or 7 days curing in oven at 104°F) indicated that the 6% $CaCl_2$ had preserved the highest moisture content. (See Figure 3.16)

Figure 3.16. The moisture content of 2%, 4%, and 6 % specimens after breaking the specimens.

In the next step, curing the 6% LCC content specimens at oven for a longer period (28 days at curing in oven at 104^{o}F) were attempted to investigate whether the low strength of the 6% specimens underlies lower drying out or not. Additional 6% LCC specimen were prepared and cured inside the oven for 28 days at 104^{o}F and tested. This specimen as is shown in Figure 3.17 had the chance to cure perfectly and reached the UCS of 392 psi which is higher than all the tested specimens (all percentages). This high value of UCS indicates a direct correlation of the UCS and curing/drying out of the soil-LCC specimens and necessity of a proper assisted drying out for LLC stabilized soil in order to reach the desired strength.

Figure 3.17. The dried, tested 6% LLC 28 days oven cured specimen.

The correlation of the LCC content vs UCS is shown in Figure 3.18. This plot can serve as a tool for determination of the optimum LCC content for a soil within the range of the gradation of the Subbase II. The target UCS of 300 psi for this soil is obtained with the LLC content of 3 to 4% of 35% solution LLC by the weight of the aggregate.

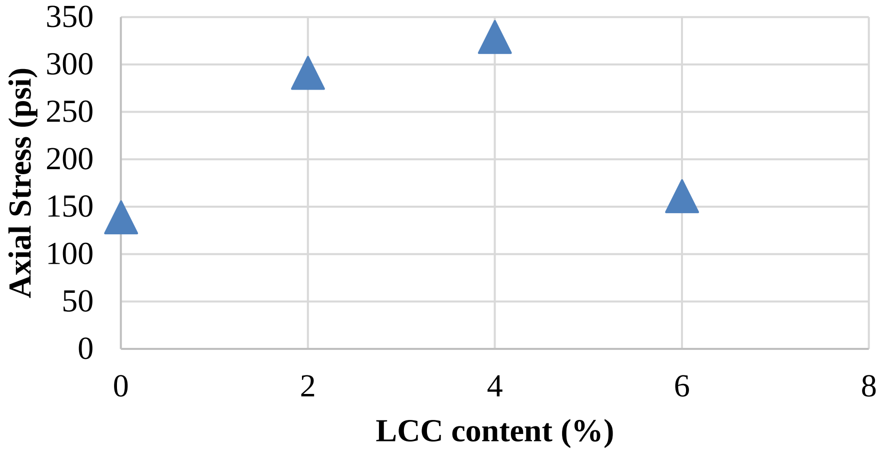


Figure 3.18. The correlation of LCC content and UCS.

3.3.2.2. SUBBASE II +RAP

Specimens containing 20, 30, and 40% RAP were prepared at the LCC content of 4% and the test results are shown in Figure 3.19 and 3.20 as well as in the Table 3.14.

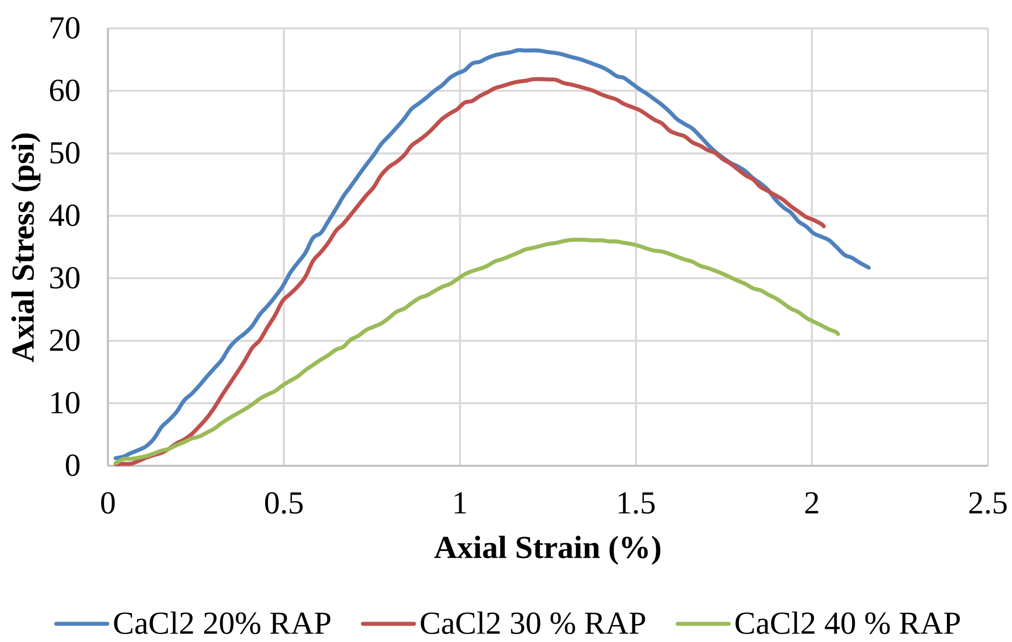


Figure 3.19. Axial stress vs axial strain curves of LCC-RAP specimens of Subbase III

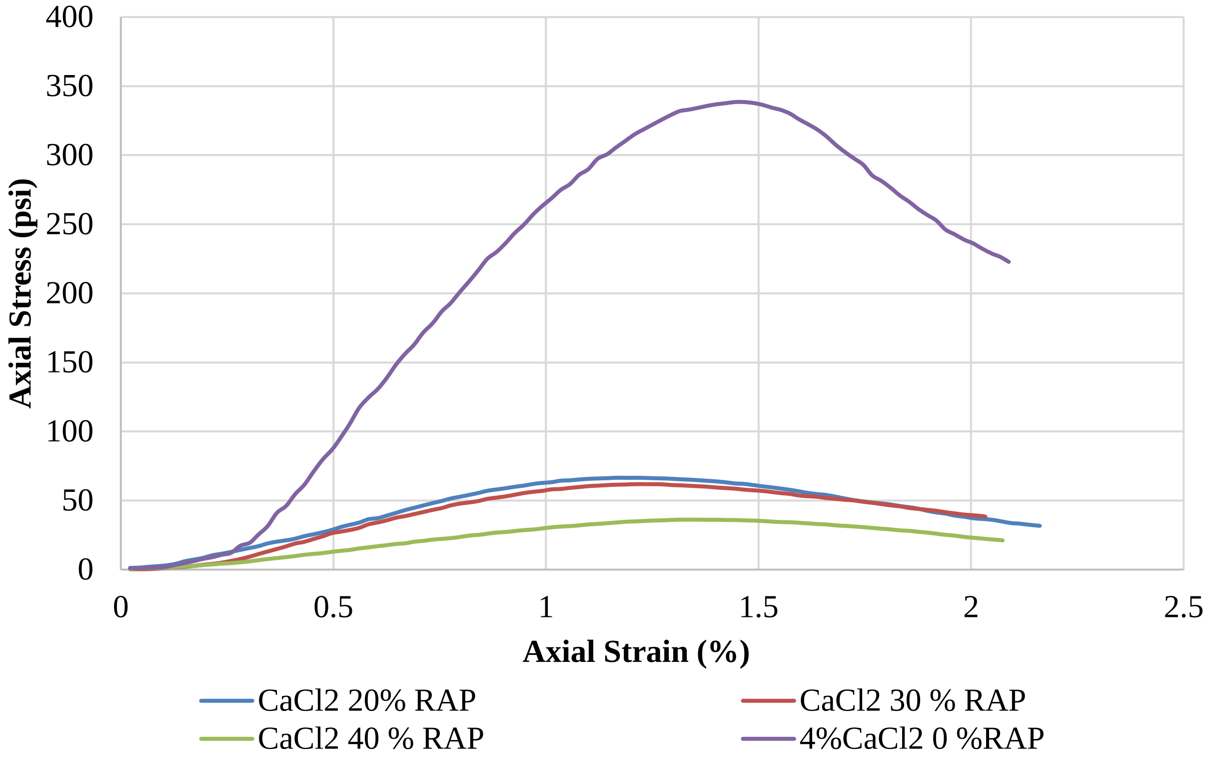


Figure 3.20. Axial stress vs axial strain curves of LCC-RAP specimens of Subbase II with 4 % LCC content compared to specimens containing increments of RAP content.

Table 3.14 summarizes the UCS values of LCC specimens with and without RAP cured at both curing conditions. According to this table, with 4% LCC the specimens containing RAP showed values way below the 4% specimens of pure subbase. This can be justified with contribution of RAP in trapping the moisture and interfering with the drying out.

Table 3.14. UCS test results of LCC specimens.

Test #	Mixture	Calcium Chloride %	RAP content (%)	Curing Conditions	UCS (psi)	Avg UCS (psi)
1	Control specimens	0	0	7 days at room temp.	118	139
2	Control specimens	0	0	28 days at room temp.	159	
3	Soil-Calcium Chloride	2%	0	28 days at room temp.	276	291
4	Soil-Calcium Chloride	2%	0	7 days in oven and 104°F (40C)	306	
5	Soil-Calcium Chloride	4%	0	28 days at room temp.	319	329
6	Soil-Calcium Chloride	4%	0	7 days in oven and 104°F (40C)	338	
7	Soil-Calcium Chloride	6%	0	28 days at room temp.	201	161
8	Soil-Calcium Chloride	6%	0	7 days in oven and 104°F (40C)	120	
9	Soil-Calcium Chloride	6%	0	28 days in oven and 104°F (40C)	392	392
10	Soil-Calcium Chloride	4%	20	28 days at room temp.	66	66
11	Soil-Calcium Chloride	4%	30	28 days at room temp.	61	61
12	Soil-Calcium Chloride	4%	40	28 days at room temp.	36	36

3.3.3. SUBBASE III

Table 3.15. Testing matrix of prepared/tested calcium chloride-stabilized specimens of Subbase III.

Test#	Mixture	Calcium Chloride %	Curing Conditions
1	Control specimens	0	7 days at room temp.
2	Control specimens	0	28 days at room temp.
3	Soil-Calcium Chloride	2%	28 days at room temp.
4	Soil-Calcium Chloride	4%	28 days at room temp.
5	Soil-Calcium Chloride	6%	28 days at room temp.
6	Soil-Calcium Chloride	2%	7 days in oven and 104°F (40C)
7	Soil-Calcium Chloride	4%	7 days in oven and 104°F (40C)
8	Soil-Calcium Chloride	6%	7 days in oven and 104°F (40C)

3.3.3.1. PURE SUBBASE III

Subbase III with 29% gravel and 63% Sand tends to lose moisture harder that a more gravely soil. Hence, in case of LCC stabilization, drying out the specimens requires more time/ higher temperature. As it is shown in Figure 3.21, the specimens seem to be moist until the end of the testing period. Obviously, this soil showed lower values of UCS compared to Subbase II.

Figure 3.21. Moist specimens of Subbase III after the end of the curing period

The UCS test results for specimens of LCC with two curing procedures are shown in Figures 3.22 to 3.24.

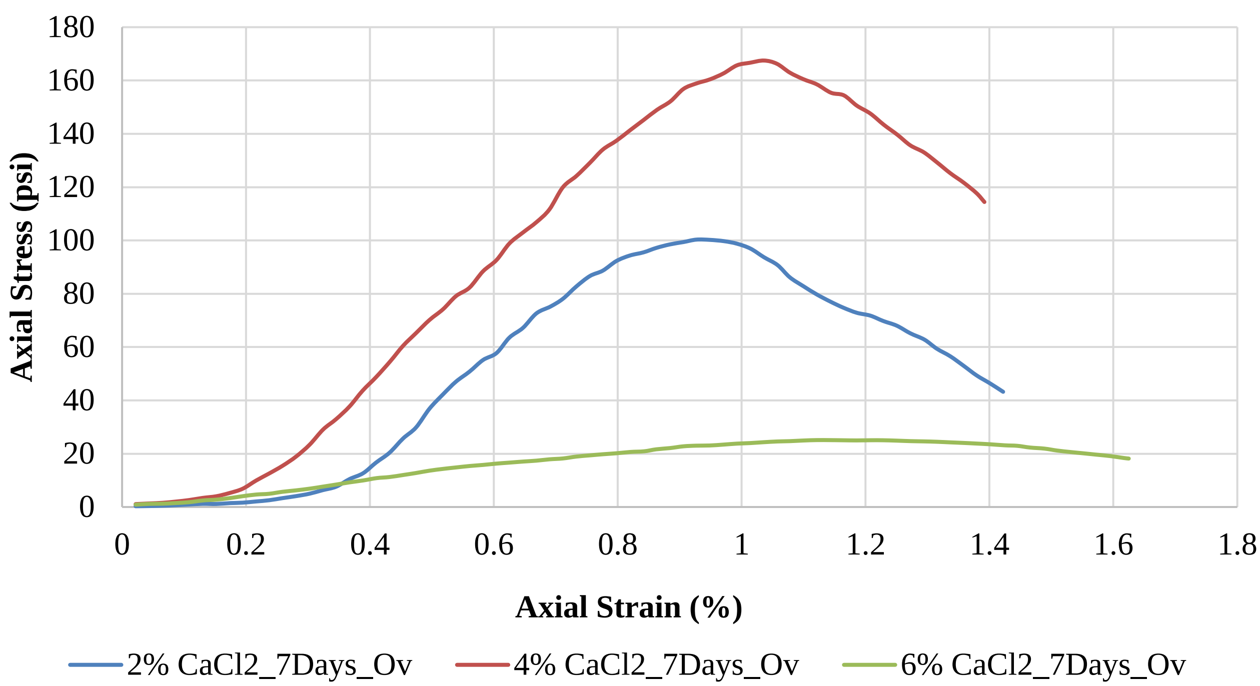


Figure 3.22. Axial stress vs axial strain curves for oven cured specimens

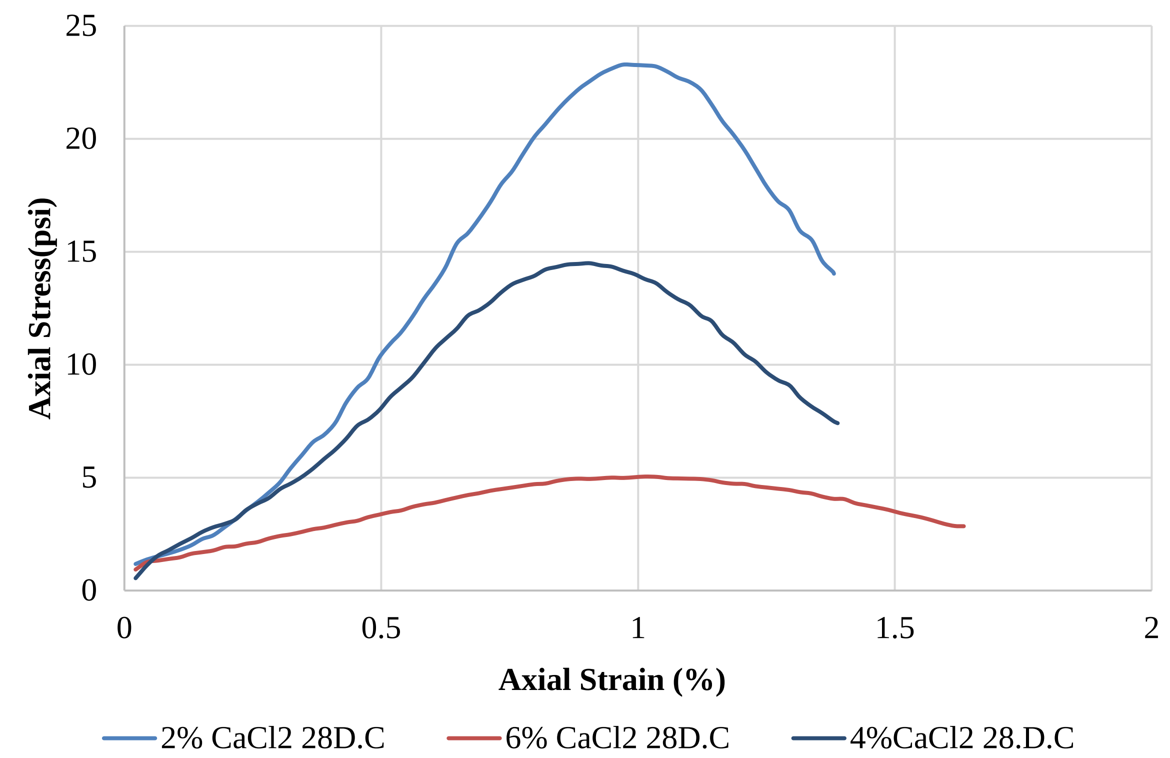


Figure 3.23. Axial stress vs axial strain curves

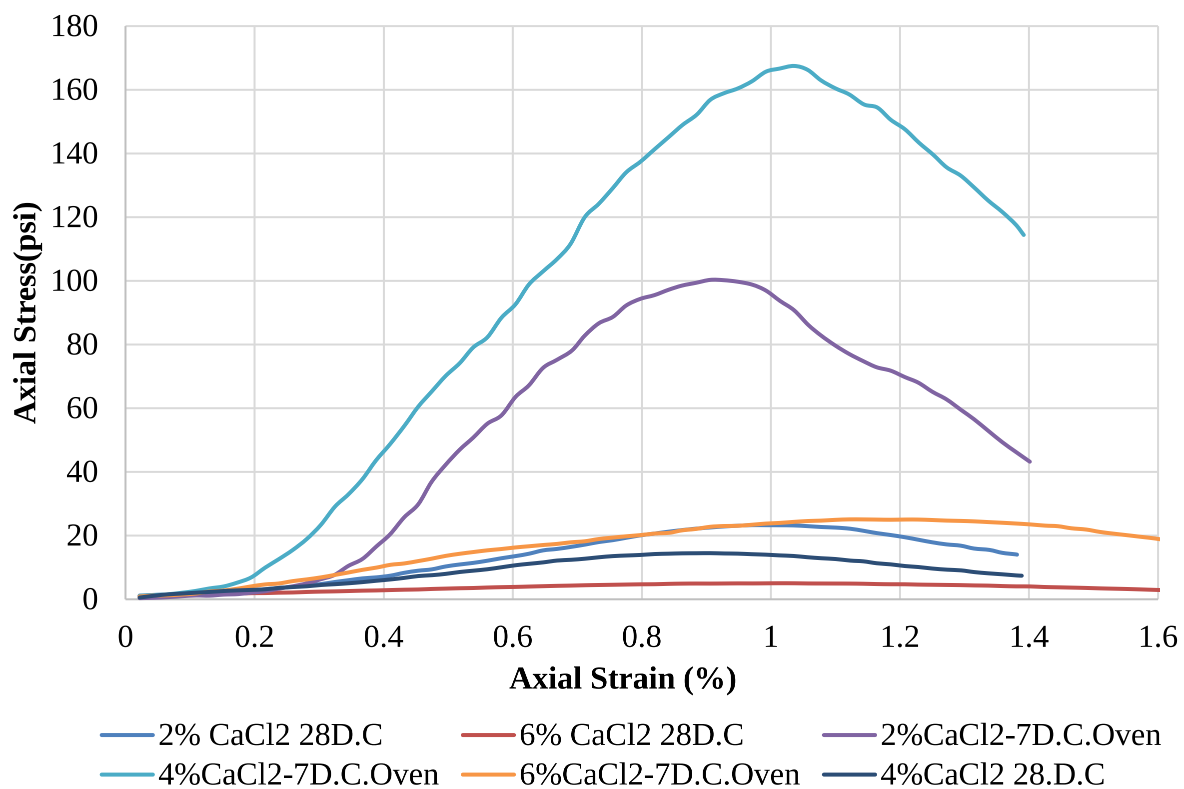


Figure 3.24. Comparing the UCS of oven dried specimens vs room cured specimens.

3.3.3.2. SUBBASE III + RAP

Figure 3.25 compares the plots of UCS of a 4% LCC specimen with pure subbase and 4% LCC specimens containing 30 and 40% RAP.

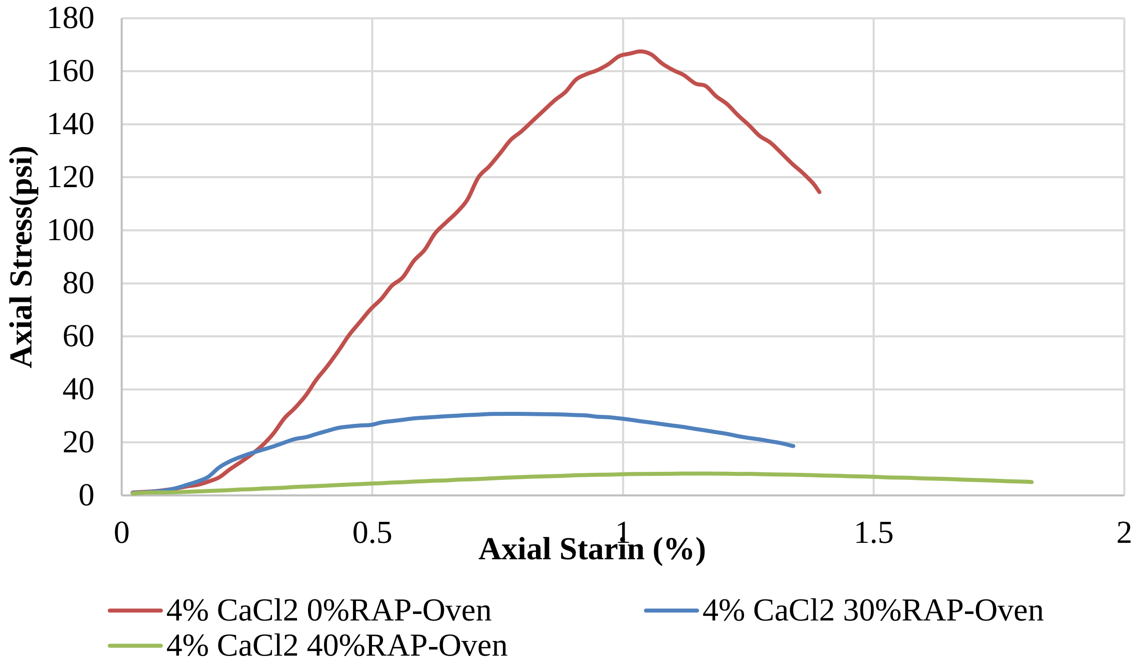


Figure 3.25. Axial Stress VS axial strain of Subbase-RAP specimens.

As it is shown in Figure 3.25, like LCC specimens of Subbase II, the effect of incorporating RAP in reducing the UCS of specimens is significant and even worse for Subbase III.

The UCS values of specimens of Subbase II with and without RAP are summarized in Table 3.16.

Table 3.16. The UCS values of the specimens prepared using Subbase III.

Test#	Mixture	Calcium Chloride %	RAP content (%)	Curing Conditions	UCS (psi)	Avg UCS (psi)
1	Control specimens	0	0	7 days in room	94	94
2	Control specimens	0	0			
3	Soil-Calcium Chloride	2%	0	28 days at room temp.	23	61.5
4	Soil-Calcium Chloride	2%	0	7 days in oven and 104°F (40C)	100	
5	Soil-Calcium Chloride	4%	0	28 days at room temp.	15	91
6	Soil-Calcium Chloride	4%	0	7 days in oven and 104°F (40C)	167	
7	Soil-Calcium Chloride	6%	0	28 days at room temp.	5	15
8	Soil-Calcium Chloride	6%	0	7 days in oven and 104°F (40C)	25	
9	Soil-Calcium Chloride	4%	20	7 days in oven and 104°F (40C)	NA	8.71
10	Soil-Calcium Chloride	4%	20	28 days at room temp.	8.71	
11	Soil-Calcium Chloride	4%	30	7 days in oven and 104°F (40C)	30.77	24.44
12	Soil-Calcium Chloride	4%	30	28 days at room	18.11	
13	Soil-Calcium Chloride	4%	40	7 days in oven and 104°F (40C)	8.26	9.92
14	Soil-Calcium Chloride	4%	40	28 days at room	11.58	

3.3.4. SUMMARY

To compare the range of UCS of LCC stabilized mixtures using Subbase II and III the test data is shown in **Table 3.17**. This data is also graphically presented in Figure 3.26.

Table 3.17. UCS values LCC mixtures of Subbase III.

UCS (psi)	2% CaCl2	4 % CaCl2	6% CaCl2
Subbase III	61	91	15
Subbase II	291	329	161

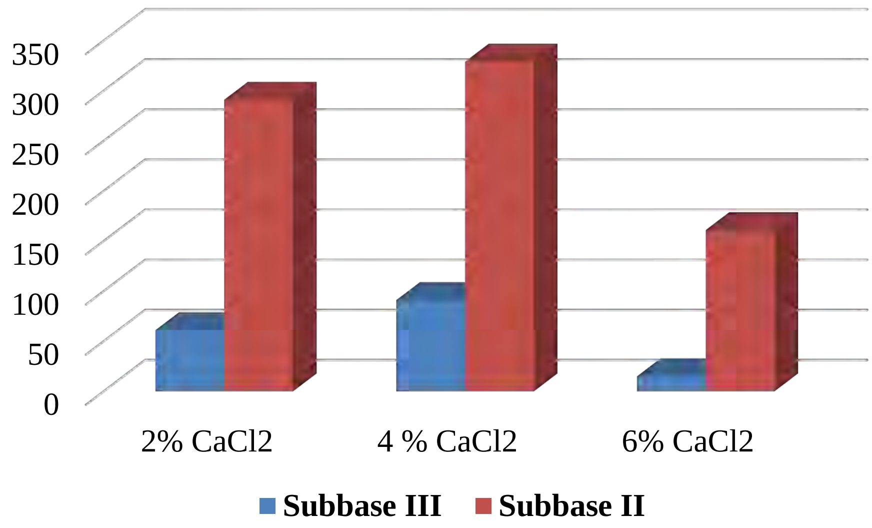


Figure 3.26. UCS of LCC mixtures of Subbase II vs Subbase III

The main findings of the laboratory experiments on LLC-stabilized specimens are summarized below:

Note:

GN2 and GN3 refer to the samples of project site soils passing sieve no. 4 (0% gravel)

Subbase I refers to the manufactured Subbase with 45% gravel content.

Subbase II refers to the manufactured Subbase with 45% gravel content and 5% added Kaolinite clay.

Subbase III refers to the manufactured Subbase with 29% gravel content.

1. Strength gain in LLC stabilization is highly dependent on the proper curing procedure, which is providing conditions under which the drying of the stabilized soil is assisted/expedited.
2. The LLC content of 4% of 35% LLC solution (1.4% residual LLC) by weight of the aggregate was found to be the optimum LLC in terms of UCS for all subbase specimens.
3. The soil gradation significantly contributes to the strength gain of the LLC stabilized specimens, as it does to the cement stabilized mixtures.
4. The higher the portion of the fine aggregate (sand) in the specimen, the drying out will be more difficult and consequently lower strength gain will be expected.
5. The maximum strength gain for Subbase II stabilized with the optimum LLC content is around 330 psi.
6. The maximum strength gain for Subbase III stabilized with optimum LLC content is around 91 psi.
7. RAP significantly decreases the UCS of LCC stabilized specimens.

3.4. ASPHALT EMULSION

Subbase I was the soil that was used for determination of the effect of Asphalt Emulsion (AE) and coming up with the optimum additive content. Samples of Pure subbase and mixtures of 70% pure subbase and 30% RAP were prepared for the AE percentages of 3, 4, 5, 6 and 7. The plots of load vs. deformation from the Marshall tests for all AE stabilized specimens are shown in Figure 3.27. In addition, Table 3.18 summarizes the Marshall stability, Marshall flow and Marshall stiffness values for tested AE specimens.

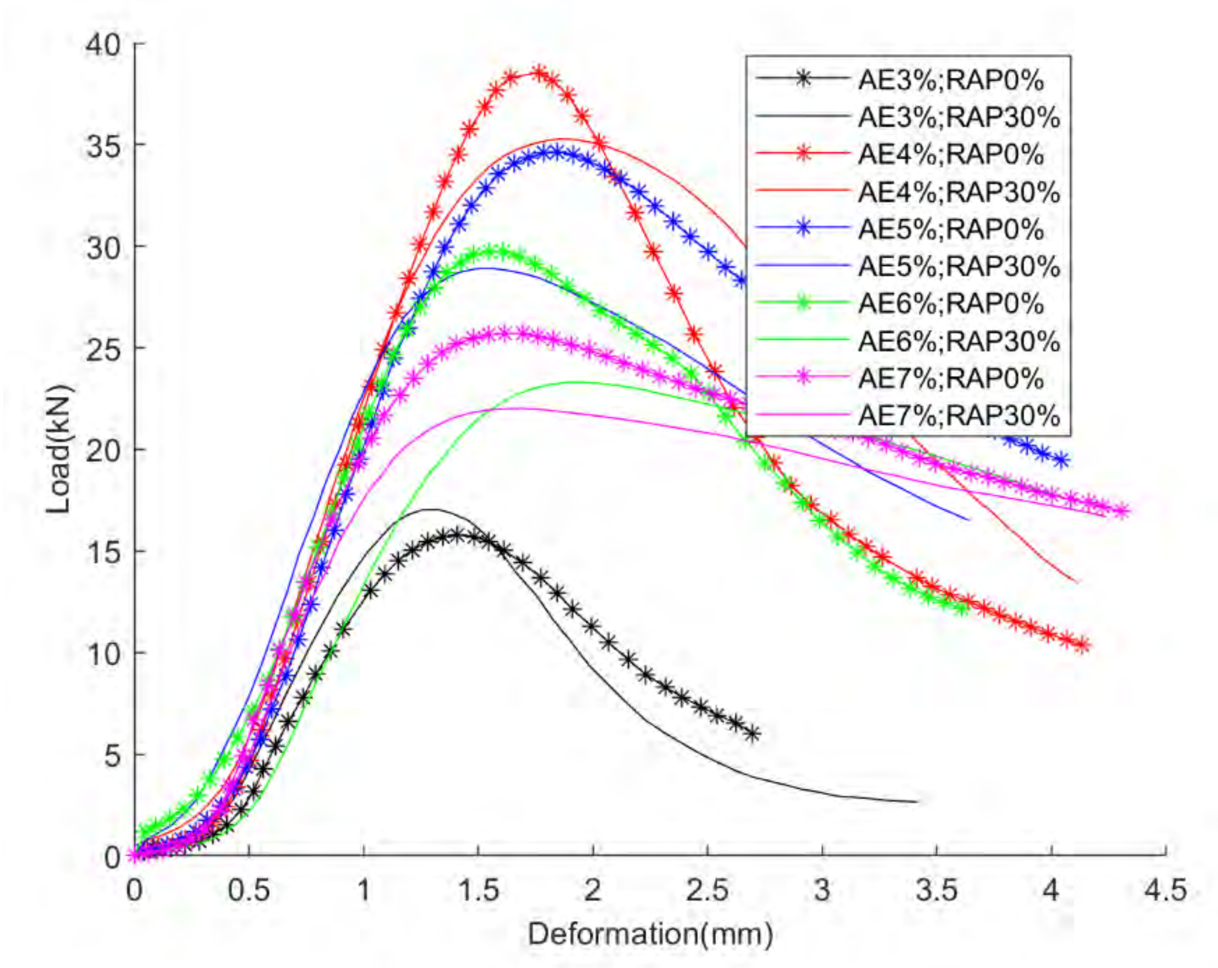


Figure 3.27. Results of Marshall test on asphalt emulsion specimens

Table 3.18. Summary of Marshall stability, Marshall flow and Marshall stiffness values for tested AE specimens.

Asphalt Emulsion Content (%)	Subbase/RAP percentage	Marshall Stability P(kN)	Flow (mm)	Marshall Stiffness (P/F)
3%	100% Subbase	15.79	2.69	5.85
	70% Subbase+ 30% RAP	17.04	3.41	4.98
4%	100% Subbase	38.54	1.77	21.77
	70% Subbase+ 30% RAP	35.28	1.86	18.97
5%	100% Subbase	34.63	1.84	18.82
	70% Subbase+ 30% RAP	28.92	1.55	18.66
6%	100% Subbase	29.73	1.55	19.18
	70% Subbase+ 30% RAP	23.30	1.94	12.01
7%	100% Subbase	25.71	1.62	15.87
	70% Subbase+ 30% RAP	21.94	1.54	14.25

SUMMARY

The highlights of AE tests on Subbase I are as follows:

Note:

Subbase I refers to the manufactured Subbase with 45% gravel content.

1. Asphalt emulsion content above 5% adversely affects the stiffness /Marshall stability value of the AE stabilized specimens.
2. Four percent AE by weight of the aggregate was found to be the optimum AE content with R-S1 AE.
3. Comparing pure-subbase specimens with specimens containing 30% RAP, higher Marshall stability values for specimens of pure subbase were observed.
4. The higher Marshall stability of the 3% AE specimen of pure subbase compared to 3% AE pure subbase containing 30% RAP can be explained by contribution of the binder in the RAP to make up for the low AE content below the optimum AE content.

CHAPTER 4. FINITE ELEMENT ANALYSIS OF THE RSB PAVEMENT

4.1. SCOPE AND OBJECTIVES OF THE FEA MODEL

The main objective of FEA in this project was to perform comparative load/deformation analysis of the RSB pavement structure, where properties of the pavement layers are changed with altering the mix design, and the type and percentage of the stabilizing agent. Construction of a physical model or test section with every single mix design and material property is expensive and impractical. With pavement modeling, however, after determining the model parameters (e.g., elastic modulus, Poisson's ratio, unit weight) a wide range of combinations of pavement structures (e.g., thickness of each layer) and properties (e.g., pre and post-stabilization) can be modeled and simulated in a short time. Sophisticated FEA models usually use the resilient modulus (the modulus of the pavement obtained from cyclic loading). However, the scope of FEA modeling in this research project was limited and therefore resilient modulus was not used in the analysis. Instead, a 3-layer pavement structure loaded with a circular static load within the linear elastic load-deformation region was considered in the modelling effort. The required parameters of this model are the elastic modulus, Poisson's ration, layer thickness and the magnitude of the static load. The comparative analysis of a three-layers system for pre- and post-stabilization cases using 2% cement was performed and the results are reported in this Chapter. This analysis can be performed on any combination of type/percentage of stabilizing agent (modulus change), and layer thickness to gain insight on the optimum range of additive and layer thickness.

4.2. INTRODUCTION TO FINITE ELEMENT ANALYSIS (FEA)

Finite element method of analysis provides an extremely powerful technique for solving problems involving the behavior of structures subjected to accelerations, loads, displacements or changes in temperature. Problems involving the behavior of heterogeneous, anisotropic structures with complex boundary conditions may be handled. If physical property values to simulation of actual boundary and loading conditions are determined to a reliable approximation, the finite element method of analysis gives a good understanding of the behavior of pavement structures under load. (Duncan et al., 1968)

4.2.1. APPLICATION IN RSB

Finite Element Analysis (FEA) of pavement structure is useful in RSB projects as it:

1. Provides comparative analysis of stress/strain on stabilized base/subbase material to predict the improved response of pavement structure after stabilization.
2. Allows performance comparisons of different stabilizers/different properties.
3. Enables prediction of the allowable traffic load, knowing the allowable pavement's deformation range.

4.2.2. NUMERICAL SOLUTIONS

The responses of an object such as pavement structure to load, movement, temperature change, etc. are governed by a series of partial differential equations (PDEs). Analytical solutions to these differential equations are only available under some of the special assumptions e.g., homogeneous materials, regular geometry, and simple boundary conditions. In pavement structure, development of analytical solutions started by the work of Boussinesq 1885 for single layer (Burmister, 1945). He developed solutions for semi-finite elastic homogenous half-space layer. His solution, however, didn't apply directly to flexible pavement structures with different modulus of elasticity and Poisson's ratios (Kim, 2007). Burmister (1943) derived expressions of stress and strains in two- and three-layers systems. Required assumptions to use Burmister's theory are as following which are the required assumptions of multi-layered solutions developed by others as well:

1. Each layer is homogeneous, isotropic, and linearly elastic.
2. Weightless and infinite layers are considered.
3. Layers have a finite thickness except the bottom layer which is infinite.
4. A circular uniform pressure is applied on the surface.
5. Interface between two layers is continuous.
6. Poisson's Ratio of 0.5 for all the layers.

(Kim, 2007).

Fox produced tabular solutions for two-layers system in 1948. Acum and Fox extended tabular solutions to three-layer system (Jones, 1962). A. Jones modified, improved, and more extensively tabulated the three-layer solutions. A. Jones and Peattie produced graphical solutions for the tables of A. Jones in 1962. For the single-Layer pavement, Ahlvin and Ulery developed tabular solutions for stress and strains under uniform circular load that considers the effect of elastic modulus and Poission's ratio in calculation of *strains and displacement*. Their tables were extended and modified after them. Their solution for the *stress* values doesn't consider the effect of Elastic modulus and uses Boussinesq formula of single layer.

For many engineering problems, approximate solutions of PDEs can be obtained using numerical solutions when analytical solutions are not available. Among numerical methods, Finite Element Method (FEM) Finite Difference Method (FDM) and Discrete Element Method (DEM) are the most popular ones. FEM is generally believed to more versatile and sophisticated specially in problems with complex geometry. With advancements in high-speed computer programs, numerical methods, especially finite element methods are increasingly used to provide better simulations of pavement structures compared to analytical methods. (Wu, 2011).

4.2.3. FINITE ELEMENT SIMULATION OF FLEXIBLE PAVEMENT

The basic idea of analysis of the material using FEM is that the body to be analyzed is divided into a set of quadrilateral or triangular elements connected at their joints or nodal points. (See Figure 4.1). Based on an assumed variation of displacements within elements together with the stress-strain characteristics of the element material, the stiffness of each nodal point of each element is computed and to this aim, certain assumptions are made (Duncan, 1968).

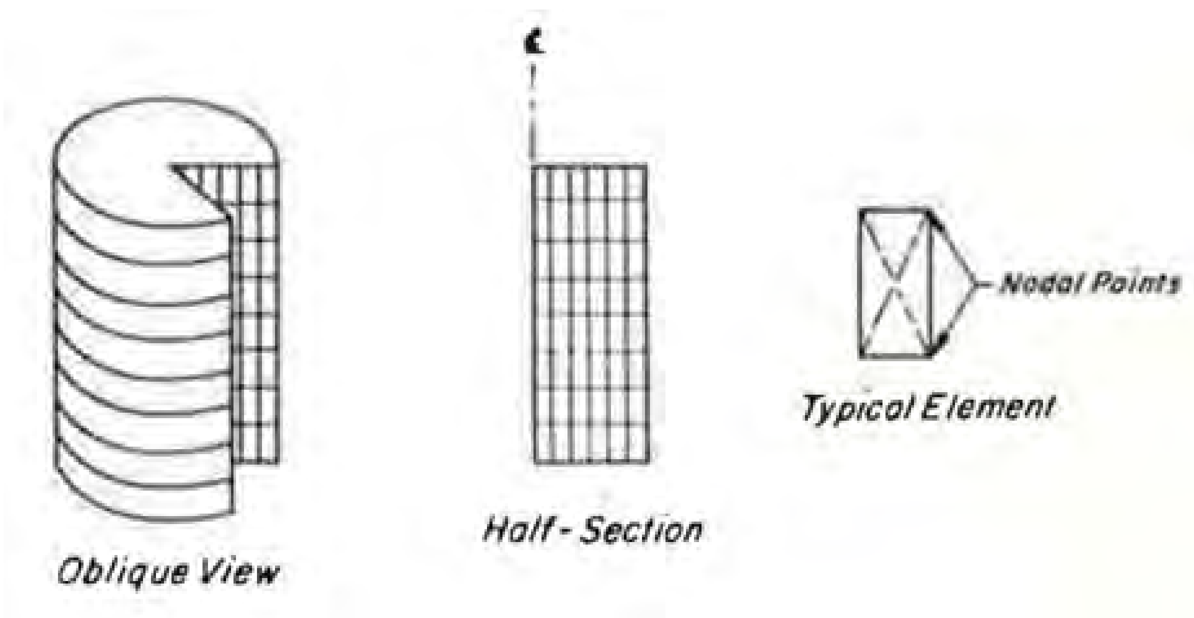


Figure 4.1: Finite Element Idealization of a cylinder. Source: Duncan et al., 1968

Shifley (1967) and Duncan et al (1968) first applied FEM to simulate linear elastic pavement structure models. To account for the linearity of the granular materials, Shifely used interactive procedure, and Duncan et al incorporated the stress-dependent elastic modulus of base and subbase materials. Duncan et al proposed the proper domain size of the axisymmetric model as well. (Kim, 2007). Duncan et al analyzed the pavement structure for winter and summer conditions and reported that large horizontal tensile stresses developed under the load in the granular base, particularly in summer condition. (Duncan et al., 1968).

After Duncan et al many researchers modeled flexible pavements using FEM and from 1970 onward, many special FE programs have been developed, i.e., ILLI_PAVE, MICHI-PAVE, FLEXPASS, yet general-purpose commercial software (eg, ABAQUS, ANSYS, and ADIANA) has been applied enormously by researchers to model flexible pavements. The general purpose software offers ample flexibility to implement various geometry, constitutive models, and boundary conditions. (Wu, Chen, Yang, & Zhang, 2011).

There are three types of numerical models used for FE simulation of pavement structures, (i) three-dimensional (3-D), plain-strain (2-D), and axisymmetric models.

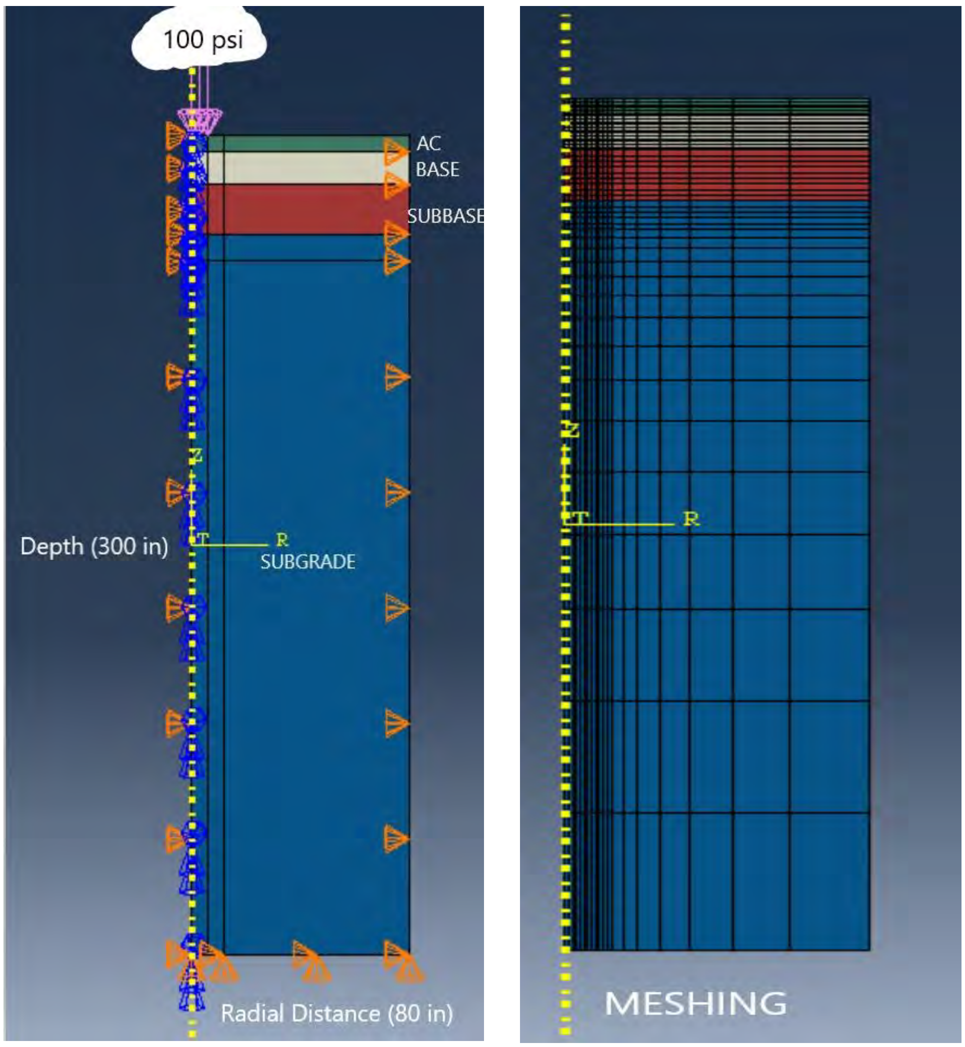
Kim 2007 performed a study of 3-D and axisymmetric models of pavement. He developed nonlinear stress-dependent user material subroutine applicable to general-purpose ABAQUS TM finite element program, studied the effect of dimensionality and mesh domain size and proposed recommendations. (Kim, 2007). He reported that the stress strain results of the 3-D and axisymmetric model don't differ significantly and when the geometric and loading condition limitation is not a matter, axisymmetric model can produce reliable results.

Wu, Chen, et all (2011) developed a 3-D and axisymmetric finite element simulations of pavement under repeated load to investigate the permanent deformation (PD) of the stabilized base and treated subbase materials. Their study included 8 accelerated pavement test (APT) sections. They aimed developing a model capable of predicting the performance of pavement structures with other combinations of stabilized base and subbase materials without running additional APT tests. (Wu, Chen, Yang, et al., 2011). Like Kim 2007, Wu, Chen, et all also emphasized that axisymmetric model has the best computational efficiency compared to 3-D model for the having far lower number of elements. The results of axisymmetric model yet compare favorably to the more "realistic" 3-D model.

4.3. Methodology

4.3.1. MODEL SIZE

In order for the stress/strain results of an axisymmetric linear-elastic model with uniform circular load to compare favorably with numerical solutions, the nodal points at the bottom layer are required be fixed at the depth of about 50 times the circular load radius and restrained from radial movement at a distance of 12 times the radius of the circular load (Duncan et al., 1968), (Kim, 2007). Hence, for 6-in radius circular load a model with the depth of 300 in and radial distance of 80 in was created and partitioned to assign different layer thicknesses. See Figure 4.2.



(a) (b)

Figure 4.2 (a) model geometry and boundary conditions, and (b) element Size

4.3.2. BOUNDARY CONDITIONS

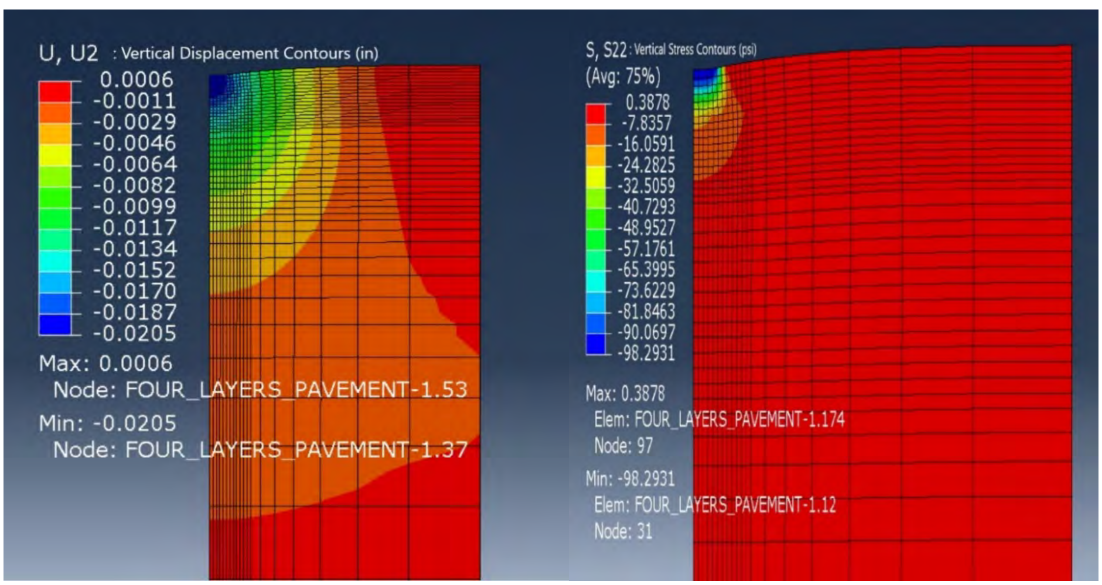
The nodal points at the vertical axis of symmetry (left side boundary) are free to move only vertically. At the righthand vertical boundary are restrained only restrained in x direction and at the bottom boundary the nodal points are restrained in both x and y directions. (Duncan et al., 1968), (Kim, 2007).

4.3.3. GEOMETRY

Various partitions were created in a way that enables the operator to change the number of layers as well as their thicknesses at any stage through the modeling. In the original model four layers were considered. The first layer is Asphalt Concrete (AC), the second layer is Base, the third layer is the Subbase, and the bottom layer is the Subgrade with 6, 12, 18 and 264 in thicknesses, respectively. See Figure 4.2.

4.3.4. ELEMENTS

ABAQUS CAE student version is limited in the number of elements and node to maximum 1000 nodes and elements. This limitation reduces the accuracy and smoothness of the results to a degree compared to professional versions; however, it still provides reliable outputs meeting our comparative purposes. Many different mesh settings were tried, and finally the part to be analyzed was meshed with 960 nodes, 893 linear quadrilateral elements as shown in Figure 4.2. The preliminary outputs of stress and deformation were used for performing the sensitivity analyses and validation of the model. See Figure 4.3 for the output examples of the stress and deformation outputs of the ABAQUS CAE.



(a) (b)

Figure 4.3. (a)Vertical Displacement Contours and (b) vertical Stress Contours

4.3.5. SENSITIVITY ANALYSIS

To validate the results of the model, the model outputs (i.e., stress and strain) were compared to stress and strain values computed using available analytical solutions in literature. To this aim, the stresses and strain values of ABAQUS model for one, two- and three-layers systems were compared to Ahlvin & Ulery analytical solution, Burmister solution and A. Jones and Peattie solution respectively and RMSEs were calculated (Ahlvin, 1962, Miller, 2006, Peattie, 1962).

In all the analytic solutions for the stress and strain values of one, two and three-layer systems, for the feasibility of the problem solving, the Poisson's ratio has been assumed to be 0.5. On the other hand, Abaqus doesn't accept Poisson's ratios greater than 0.48. Therefore, the Poisson ratio of all pavement structures modeled were assumed to be 0.48 to be as close to analytical solutions as possible.

4.3.5.1. SINGLE LAYER

For evaluation of the single layer model, the first and second models to be analyzed were two single-layered systems with highly different elastic moduli. In the first model, all the sections were assigned the modulus elastic of asphalt concrete (400000) and in the second one all the sections were assigned the properties of base layer (20000 psi) and Poisson's ratio of 0.48 for both. Stress and strain values of the second model (base material properties) were compared to Ahlvin & Ulery tabulated solutions and RMSE was calculated. (See the Table 4.2).

Vertical Stress $\sigma z=P[A+B]$

Where P is the pressure load and A, B, C, F, and H are functions of Ahlvin solution derived from Table below.

Vertical Deflection $\omega z = P\ [zA+(1-\nu)\ H]$

Where P is the pressure, ν is the Poisson's ratio, E is the elastic modulus of the Base material (20000 psi), z is the depth, A and H are functions of Ahlvin solution.

Example Values of Coefficients

r/a	z/a	A	B	C	D	E	F	G	H
0	0	1	0	0	0	0.5	0.5	0	2
	0.1	0.9005	0.09852	−0.04926	0.04296	0.45025	0.45025	0	1.80998
	0.5	0.55279	0.35777	−0.17889	0.17889	0.27639	0.27639	0	1.23607
	1	0.29289	0.35355	−0.17678	0.17678	0.14645	0.14645	0	0.82843
	2	0.10557	0.17889	−0.08944	0.08944	0.05279	0.05279	0	0.47214
	5	0.01942	0.03772	−0.01886	0.01886	0.00971	0.00971	0	0.19805
0.2	0	1	0	0	0	0.5	0.5	0	1.97987
	0.1	0.89748	0.10140	−0.05142	0.04998	0.44949	0.44794	0.00315	1.79018
	0.5	0.54403	0.35752	−0.17835	0.17917	0.27407	0.26997	0.04429	1.22176
	1	0.28763	0.34553	−0.17050	0.17503	0.14483	0.14280	0.05266	0.85005
	2	0.10453	0.18144	−0.08491	0.09080	0.05105	0.05348	0.02102	0.47022
	5	0.01938	0.03760	−0.01810	0.01950	0.00927	0.01011	0.00214	0.19785
1	0	0.5	0	0	0	0.5	0	0.31831	1.27319
	0.1	0.43015	0.05388	0.02247	0.07635	0.39198	0.03817	0.31405	1.18107
	0.5	0.28156	0.13591	0.00483	0.14074	0.21119	0.07037	0.26216	0.90298
	1	0.17868	0.15355	−0.02843	0.12513	0.11611	0.06256	0.18198	0.67769
	2	0.08269	0.11331	−0.04144	0.07187	0.04675	0.03593	0.07738	0.43202
	5	0.01835	0.03384	−0.01568	0.01816	0.00929	0.00905	0.00992	0.19455
2	0	0	0	0	0	0.12500	−0.12500	0	0.51671
	0.1	0.00856	−0.00845	0.01536	0.00691	0.11806	−0.10950	0.00159	0.51627
	0.5	0.03701	−0.02651	0.05690	0.03039	0.09180	−0.05479	0.03033	0.49728
	1	0.05185	−0.01005	0.05429	0.04456	0.06552	−0.01367	0.06434	0.45122
	2	0.04496	0.02836	0.01267	0.04103	0.03454	0.01043	0.06275	0.35054
	5	0.01573	0.02474	−0.00939	0.01535	0.00873	0.00700	0.01551	0.18450

Figure 5.3. Example values of Ahlvin & Ulery solution coefficients. Source (CV761-Pavement analysis and design)

4.3.5.2. TWO-LAYER

For the two-layers system, the properties of AC material were assigned to the first layer (E=400000 psi) and the properties of the Base material (20000 psi) was assigned to the bottom layer. stress and displacement values of ABAQUS for the two-Layers system were compared to Burmister' solution. See Figure 4.5 for the Burmister vertical stress chart and Figure 4.6 for surface deflection equation.

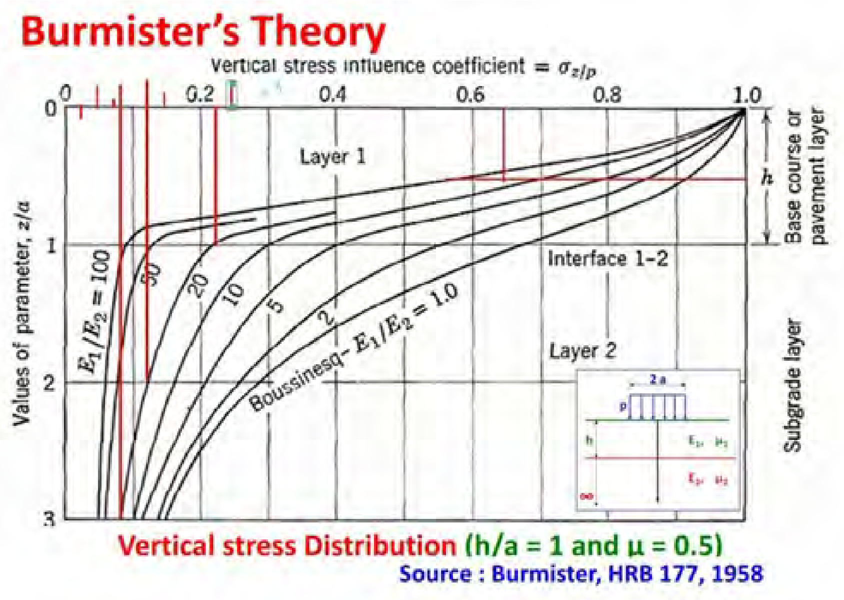


Figure 4.5. Vertical Stress Influence coefficient, Source: CV761-Pavement analysis and design

Surface Deflections - Burmister

Flexible plate

$$\boxed{w = 1.5\,\frac{q\,a}{E_2}F_2 \quad \text{when } \mu = 0.5}$$

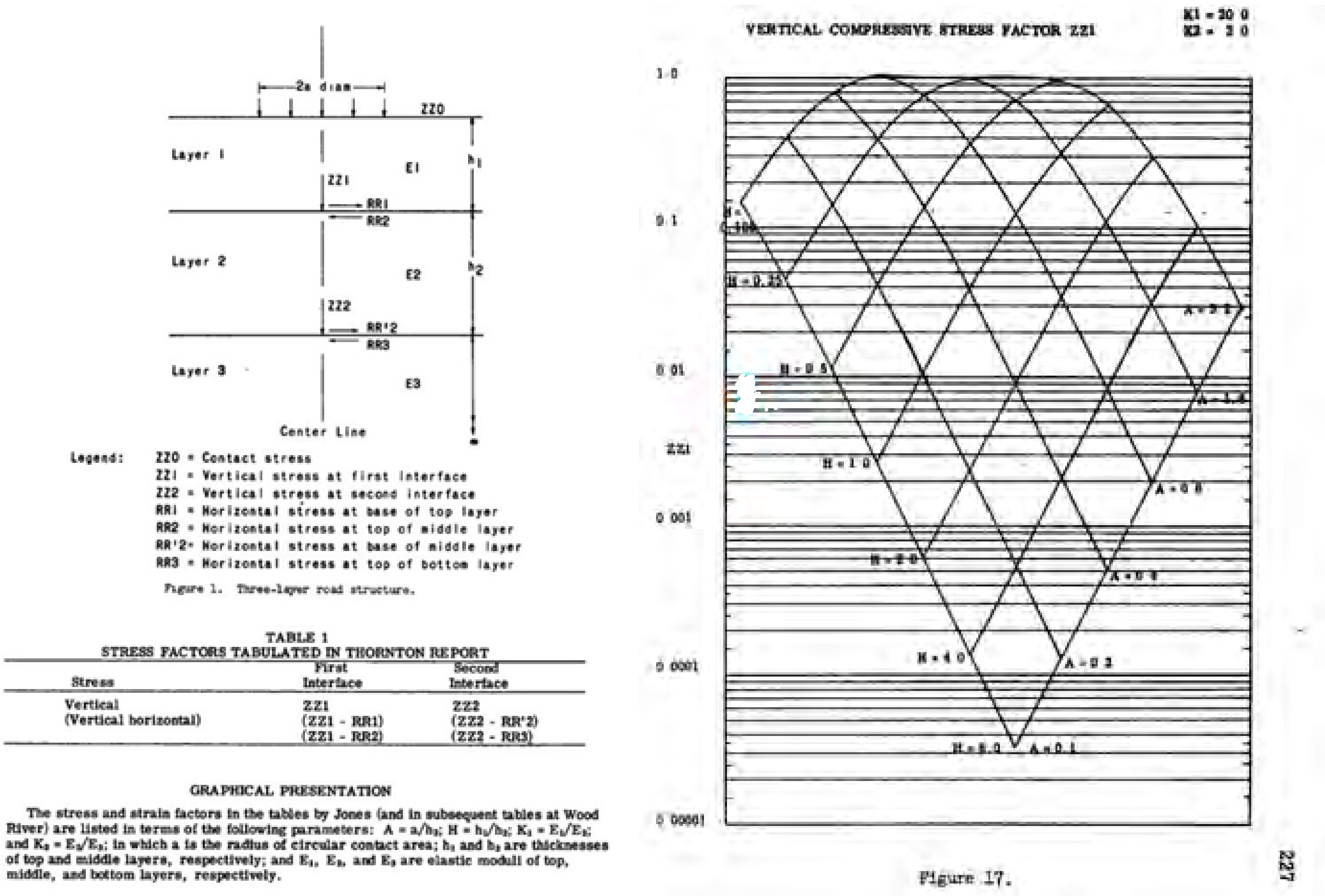
Rigid plate

$$\boxed{w = 1.18\,\frac{q\,a}{E_2}F_2 \quad \text{when } \mu = 0.5}$$

Figure. 4.6. Surface Deflection equations for Burmister solution. Source: CV761-Pavement analysis and design

4.3.5.3. THREE-LAYER SYSTEM

For three-layer system the first, second, and bottom layer were assigned the properties of AC, Base and Subbase (10000 psi) materials. The stress values of the three-layers system were compared to A. Jones and Peattie tabular and graphical solutions. The values of vertical and horizontal stress at the interfaces of the layers and under the center of the loaded area are computable using the ratios of A=r/h2 where r is the radius of the loaded and h2 is the thickness of the second layer and H=h1/h2 is the ratio of the thickness of the first layer to the thickness of the second layer. See Figure 4.7(a). As shown in Figures 4.7 (b) and the Values of stress computed using Peattie analytical solution were compared to ABAQUS values.



a b

Figure 4.7 (a) Table parameters for Peattie solution (b) Vertical stress values of K1=20, K2=2, H=0.5, A=0.5.

Source: Peattie,1962

The model was developed in Abaqus, with input parameters (modulus of elasticity, Poisson's ratio) adapted from literature for the linear elastic model. The load deformation analysis included application of a100 psi, 6-in-radii static circular load (approximation of a single axle load of a truck) on a layered pavement structure as shown in Figure 4.2. Load/deformation analysis was performed, followed by a sensitivity analysis to verify the sensitivity of the model to different input parameters.

The required parameters for a linear elastic stress/strain computation in both FE programs and analytical solutions are (i) modulus of elasticity, (ii) Poisson's ratio, and (iii) thickness of each layer. Table 4.1 summarizes the model parameters (adopted from the literature) used in this study.

Table 4.1. Model Parameters (Kim, 2007)

Layer	Thickness (in)	Elastic Moduli (psi)	Poisson's ratio
ACC	6	400000	0.48
BASE	12	20000	0.48
SUBBASE	18	10000	0.48
SUBGRADE	264	5000	0.48

To validate the model, the outputs (i.e., stress and strain) were compared against stress and strain values computed using available analytical solutions in the literature (i.e., Ahlvin & Ulery solution, Burmister solution, and Jones and Peattie solution for one-, two- and three-layers systems, respectively) and the Root Mean Square Error (RMSE) was calculated at different depths. Table 4.2 summarizes the values of stresses and strains calculated using Ahlvin & Ulery analytical approach and Abaqus predictions. The small RMSE values for both vertical stress and strain indicates good agreement between the model prediction and the analytical solution.

Table 4.2. Comparison of Ahlvin & Ulery analytical solution and Abaqus predictions for single layer pavement.

Single Layer					**Ahlvin Function Values**					**Vertical Stress(psi)**			**Vertical Strain(in)**		
Point	**Depth (in)**	**Poisson**	**r/a**	**z/a**	**A**	**B**	**C**	**F**	**H**	**Abaq.**	**Ahl.**	**RMSE**	**Ahlvin**	**Abaq.**	**RMSE**
1	3	0.5	0	0.5	0.55	0.36	0.18	0.28	1.24	91.33	91.06		0.0110	0.0027	
2	6	0.5	0	1	0.29	0.35	0.18	0.15	0.83	65.14	64.64	**0.37**	0.0070	0.0027	**0.004**
3	12	0.5	0	2	0.11	0.18	0.09	0.05	0.47	28.21	28.45		0.0029	0.0013	
4	18	0.5	0	3	0.05	0.09	0.05	0.03	0.32	14.20	14.62		0.0015	0.0006	

Table 6.3 summarizes the values of stresses and strains calculated using Burmister's analytical approach and Abaqus predictions. The very small RMSE values for both vertical stress and strain indicate (0.07 and 0.02, respectively) indicates good agreement between the model prediction and the analytical solution.

Table 4.3. Comparison of Burmister analytical solution with Abaqus predictions for two-layer pavement.

Elastic Modulus		Point	Depth (in)	z/a	I=σz/σ0	Vertical Stress (Psi)			Vertical Displacement (in)			
						Burmis.	Abaq.	RMSE	Brumis. Factor, F2	Vert. Defl.	Abbaq.	RMSE
E1	20000	1	3	0.5	0.65	65.00	64.63	**0.07**	0.62	0.055	0.015	**0.02**
E2	10000	2	6	1	0.223	22.30	22.32		0.39	0.034	0.014	
E1/E2	2	3	12	2	0.126	12.60	12.70		0.23	0.020	0.010	
		4	18	3	0.082	8.20	8.28		0.19	0.017	0.008	

Table 4.4 summarizes the values of stresses and strains calculated using Jones and Peattie's analytical approach and Abaqus predictions. Although the RMSE values for both vertical stress and strain are not as small as those for single- and two-layer systems, they are in the acceptable range, which indicate reasonable agreement between the model prediction and analytical solution.

Table 4.4 Comparison of Jones and Peattie's analytical solution with Abaqus predictions for two-layer pavement.

Jones and Peattie solutions		Elastic Modulus		Poisson	Points	Vertical Stress Factor (ZZ1)	J. & P. Stress	Abaqus Stress	RMSE
A=r/h2	0.5	E1	40000	0.5	First Interface	0.1890	18.903	20.766	1.38
H=h1/h2	0.5	E2	20000	0.5	Second Interface	0.0660	6.603	6.009	
		E3	10000	0.5					

4.4. RESULTS

The aforementioned analysis (comparing model prediction vs. analytical solutions) validated the model predictions and allowed for FEA of comparative scenarios of pavement structure, materials, and parameters. In order to investigate the improvement in pavement performance gained by RSB, the FE model was employed to analyze the change in the vertical stress and strain for the scenarios of pre- and post-stabilization. Table 4.5 summarizes the model parameters used in the analysis, which represents the case of 2% cement-stabilized base. The model parameters are based on the results from experimental study conducted in this research project as well as estimates from the literature (Kim, 2007).

Table 4.5. The model parameters used in the analysis.

Layer	Thickness (in)	Modulus of Elasticity (psi)	Poisson's Ratio
AC	6	400000	0.35
Un-stabilized base	12	5209	0.3
Stabilized base		36177	0.3
Subgrade	282	5000	0.4

Figure 4.8 compares the distribution of the vertical stress along the depth of the 3-layer model loaded with a 100-psi static load for pre- and post-stabilization cases with 2% cement. As can be seen from Figure 4.8, with improved modulus of elasticity, thanks to RSB treatment, the stress resulting from the static load shifts to a lower depth (transferred from the HMA level to the bound base layer), which will result in higher service life of the HMA.

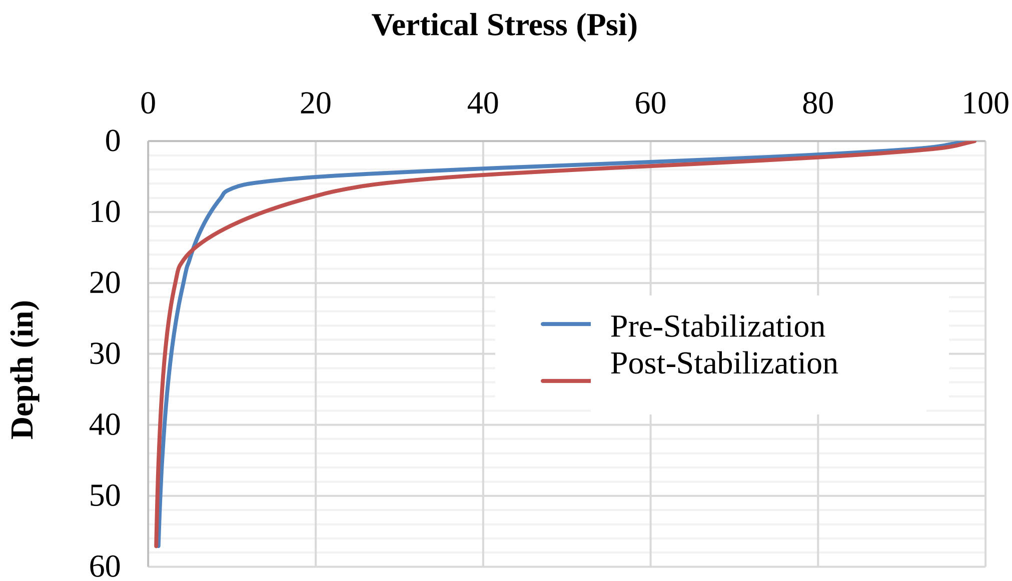


Figure 4.8. Variation of vertical stress

Figure 4.9 compares the horizontal (tensile) stress along the depth of the pavement. As can be seen from the plot, at the depth of 6 inch (bottom of the HMA) the horizontal stress has reduced in the stabilized layer compared to a none-stabilized layer. This will also contribute to a higher service life of the pavement.

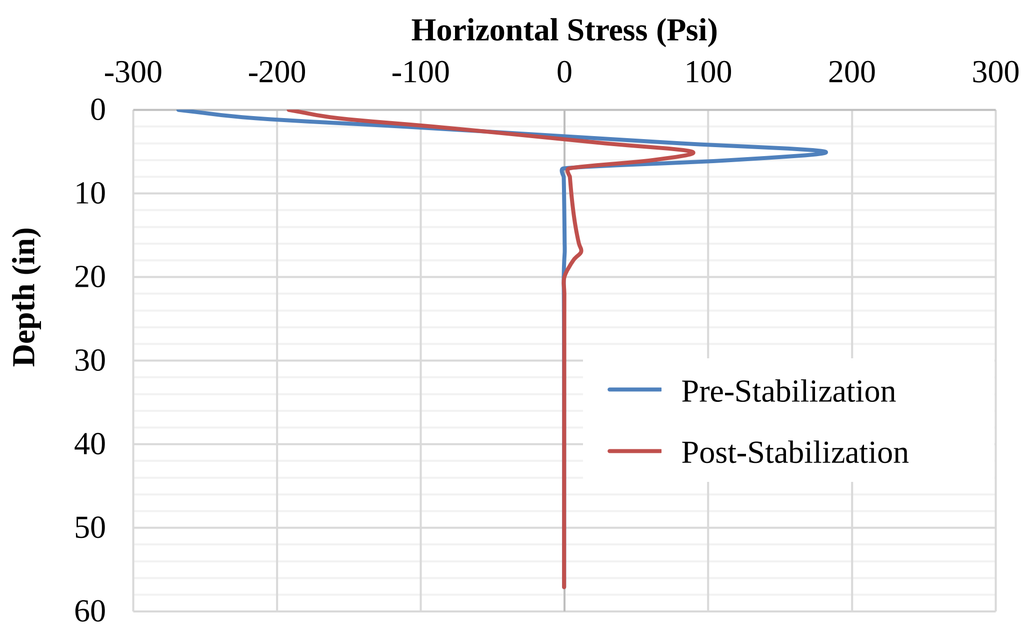
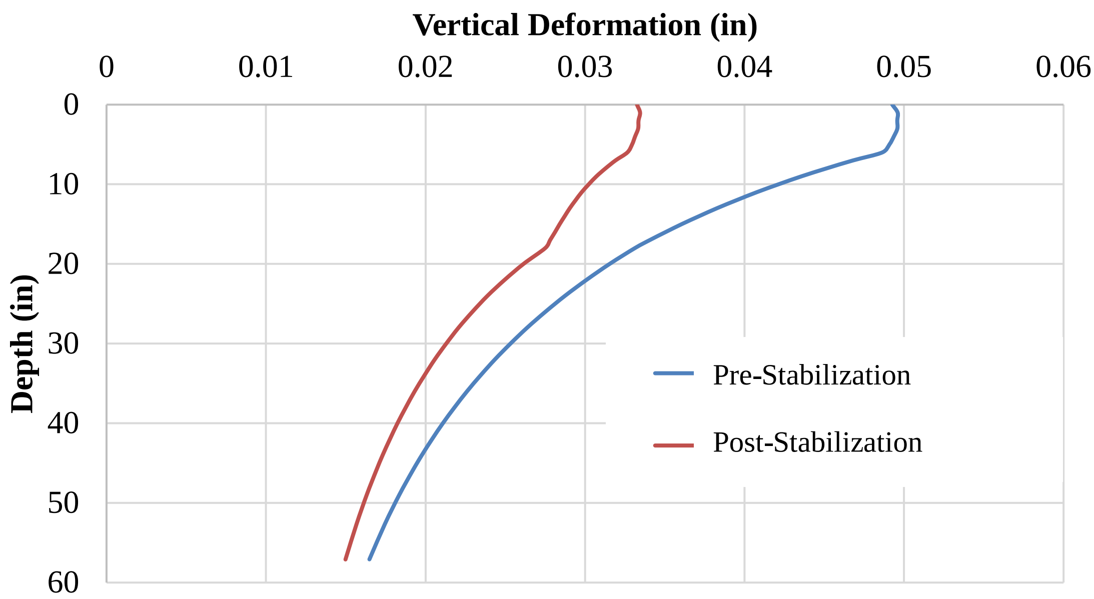


Figure 4.9. Change in horizontal stress with modulus improvement.

Figure 4.10 compares the vertical deformation along the depth of the stabilized vs none-stabilized layer. The vertical deformation along the depth has reduced by approximately 30 %. This will promise less rutting of the pavement.



(c)

Figure 4.10. Vertical deformation for the two scenarios of pre- and post-stabilization

4.5. Conclusions

Base stabilization using cement improved the modulus of elasticity of the base layer which had implications on the pavement structure behavior. The comparative analysis of the three-layer system for pre- and post-stabilization cases using 2% cement described some of these implications as follows:

1. With improved modulus of elasticity, the stress resulting from the static load shifts to a lower depth. This mean the HMA will be subjected to lower distress.
2. In an stabilized base layer compared to none-stabilized (unbound) layer a reduction in the values of horizontal stress along the HMA can be expcted. The reduced tensile stress is resulted in service life improvement of the pavement.
3. In the stabilized base layer, the vertical deformation along the depth is reduced by approximately 30 %. This will result in less rutting of the pavement.

4.6. APPLICATIONS OF THE MODEL

The developed model can be utilized for the primitive evaluation of the pavement to estimate the type and percentage of the additive. In other words, without practically constructing a RSB pavement, the mix design can be evaluated using this model to have an approximate estimation of the optimum required additive, soil type and the layer thickness.Future modeling work can include the followings :

1. Simulation of the pavement structure behavior within the range of possible modulus resulting from various stabilizing agent type and percentage.
2. Comparison of different structures stabilized with different stabilizing agents.
3. Estimation of the optimum stabilizing agent percentage to be used to reach the desired/allowable stress and deformation.
4. Estimation of the effect of load increments on pavement deformation.

CHAPTER 5. CONCLUSIONS

The important research questions intended to be addressed in this project included: the proper mix design, optimum percentage of stabilizing agent, the range of improvement in the properties of the base using the three explored additives, effect of subbase material gradation on the strength properties of the stabilized pavement and the effect of RAP on the properties of the stabilized layer. The literature review along with RSB sites surveys further fine-tuned the scope of the laboratory tests and the FEM analysis. Subbase material collected from existing RSB projects along with manufactured material and three different additives including cement, liquid Calcium Chloride and asphalt emulsion were used in the laboratory experiments. The results of the laboratory tests and their interpretation for each stabilizing agent were presented in Chapter 3. In addition, Finite Element Analysis was performed to demonstrate the improvement in the response of the pavement using RSB. This Chapter summarizes the results of all the laboratory testing and the FEA analysis.

5.1. CEMENT STABILIZATION

The highlights of the laboratory experiments on cement-stabilized specimens are summarized below:

Note:

GN2 and GN3 refer to the samples of project site soils passing sieve no. 4 (0% gravel)

Subbase I refers to the manufactured Subbase with 45% gravel content.

Subbase II refers to the manufactured Subbase with 45% gravel content and 5% added Kaolinite clay.

Subbase III refers to the manufactured Subbase with 29% gravel content.

1. Specimens prepared, cured, and tested following the same procedures, but with different aggregate gradation exhibited substantially different UCS.
2. Fine graded soil with 0% gravel (GN2 and GN3 passing sieve number 4) showed the lowest ranges of UCS. The average UCS of these soils for 2% and 3% cement content were 63 and 145 psi, respectively.
3. A significant increase in the UCS values is evident with increasing the course portion of the soil form 0% (GN) to 45% (Subbase I). The range of UCS values for the 2% cement specimens increased from 63 psi (average value for GN2 and Gn3) to 184 psi (almost three folds). This increase for the 3% cement specimens was from 145 psi to 286 psi (almost two folds).
4. Adding 5% clay to the Subbase I reduced the OMC by 1% and increased the UCS significantly.

5. The Subbase II showed the highest UCS values, compared to other subbase specimens.
6. To obtain a UCS of around 300 psi, with Subbase I, II and III, required cement percentages are around 3%, 2%, and 4 % respectively.
7. Incorporating up to 30% RAP into subbase II stabilized with 2% cement, the UCS will likely stay above 300 psi.
8. The range of UCS for Subbase III is 149 psi for 2% cement and for the 3%-cement specimen is 217 psi. To reach the targeted UCS of around 300 psi, cement contents beyond 3% is required while this target strength is achieved with about 2% and 3% cement additive with subbase II and Subbase I, respectively.
9. For the 2% cement-additive specimens, Subbase III, with higher portions of sand and lower gravel portion compared to Subbases I and II, showed a higher reduction in UCS when RAP was incorporated into this soil.
10. The results of the FEA on a cement stabilized pavement with 2% cement can be summarized as follows:

    - With improved modulus of elasticity, the stress resulting from the static load shifts to a lower depth. This mean the HMA will be subjected to lower distress.
    - In an stabilized base layer compared to none-stabilized (unbound) layer a reduction in the values of horizontal stress along the HMA can be expcted. The reduced tensile stress is resulted in service life improvement of the pavement.
    - In the stabilized base layer, the vertical deformation along the depth is reduced by approximately 30 %. This will result in less rutting of the pavement.

5.2. LIQUID CALCIUM CHLORIDE STABILIZATION

The main findings of the laboratory experiments on LCC-stabilized specimens are summarized below:

Note:

GN2 and GN3 refer to the samples of project site soils passing sieve no. 4 (0% gravel)

Subbase I refers to the manufactured Subbase with 45% gravel content.

Subbase II refers to the manufactured Subbase with 45% gravel content and 5% added Kaolinite clay.

Subbase III refers to the manufactured Subbase with 29% gravel content.

1. Strength gain in LCC stabilization is highly dependent on the proper curing procedure, which is providing conditions under which the drying of the stabilized soil is assisted/expedited.

2. The LCC content of 4% of 35% LLC solution (1.4% residual LCC) by weight of the aggregate was found to be the optimum LLC in terms of UCS for all subbase specimens.
3. The soil gradation significantly contributes to the strength gain of the LCC stabilized specimens, as it does to the cement stabilized mixtures.
4. The higher the portion of the fine aggregate (sand) in the specimen, the drying out will be more difficult and consequently lower strength gain will be expected.
5. The maximum strength gain for Subbase II (representing ***well graded gravel with clay and sand***) stabilized with the optimum LLC content is around 330 psi which is equivalent to 2% cement with the same soil
6. The maximum strength gain for Subbase III (representing a "***well graded sand with silt and gravel.")*** stabilized with optimum LCC content is around 91 psi.
7. RAP significantly decreases the UCS of LCC treated specimens n this reduction is more significant compared to cement treated mixtures.

5.3. ASPHALT EMULSION STABILIZAITON

The main findings of the laboratory experiments on AE stabilized specimens are summarized below:

1. Asphalt emulsion content above 5% adversely affects the stiffness /Marshall stability value of the AE stabilized specimens.
2. 4% AE by weight of the aggregate was found to be the optimum AE content with R-S1 AE.
3. Comparing pure-subbase specimens with specimens containing 30% RAP, higher Marshall stability values for specimens of pure subbase were observed.
4. The higher Marshall stability of the 3% AE specimen of pure subbase compared to 3% AE pure subbase containing 30% RAP can be explained by contribution of the binder in the RAP to make up for the low AE content below the optimum AE content.

5.4. FINITE ELEMENT ANALYSIS

Base stabilization using cement improved the modulus of elasticity of the base layer which had implications on the pavement structure behavior. The comparative analysis of the three-layer system for pre- and post-stabilization cases using 2% cement described some of these implications as follows:

1. With improved modulus of elasticity, the stress resulting from the static load shifts to a lower depth. This mean the HMA will be subjected to lower distress.
2. In an stabilized base layer compared to none-stabilized (unbound) layer a reduction in the values of horizontal stress along the HMA can be expcted. The reduced tensile stress is resulted in service life improvement of the pavement.
3. In the stabilized base layer, the vertical deformation along the depth is reduced by approximately 30 %. This will result in less rutting of the pavement.

CHAPTER 6. RECOMMENDATIONS

Based on the results of the laboratory tests, the following recommendations are made for VTrans to be implemented in future RSB projects:

Note:

GN2 and GN3 refer to the samples of project site soils passing sieve no. 4 (0% gravel)

Subbase I refers to the manufactured Subbase with 45% gravel content.

Subbase II refers to the manufactured Subbase with 45% gravel content and 5% added Kaolinite clay.

Subbase III refers to the manufactured Subbase with 29% gravel content.

1. According to AARA, the gravel content of the soil material used for RSB projects should not be above 55%. In this study, a range of gravel content of 29% to 45% for three Subbase materials were covered. The result of this work indicated that gradations with gravel contents below 45% adversely affects the strength of the bonded layer in (RSB).
2. With both cement and LCC stabilization, Subbase I and Subbase II showed the highest strength values. Therefore, it is highly recommended to screen the gradation of the pure subbase or RAP to ensure that their course portion is not either too high or too low (staying within the range of Subbase I is appropriate).
3. Mixing the RSB materials at the precise optimum moisture content is crucially important for gaining the maximum/desired strength and conducting sufficient and reliable optimum moisture determination tests before approving a mix design is highly recommended.
4. Curing the cement stabilized pavements specially in cold regions where excessive evaporation and moisture lost is not an issue is more convenient compared to curing a pavement stabilized with LCC where the curing procedure calls for drying the compacted base layer in cold regions such as Vermont.
5. Adding 5% clay to the base/subbase material can reduce the optimum cement content by 1%, suggesting that the 5% clay can make up for 1% added cement. This can be potentially considered in future RSB projects, where the subbase/base material contains clay.
6. Adding up to 30% RAP to typical subbase material (e.g., gradation of Subbase I) does not significantly compromise the strength properties of the pavement. Therefore, considering its economic and environmental benefits, it can be considered as a viable option in future RSB projects.
7. The qanitity and rate of strength gain in LCC stabilization is highly dependent on drying out of the mixture. Hence, proper drying out of the stabilized layer is not possible, using this additive is not recommended.

8. The optimum LCC content for different gradations and RAP content was found to be 4% and LLC contents beyond 4% adversely affect the strength gain of the stabilized layer. It is recommended that VTrans follows this range in future RSB projects.
9. Table 6.1 illustrates the trade-off of the strength with adding RAP in cement stabilization vs LCC stabilization, indicating that a more cautious approach needs to be taken while considering adding RAP to the subbase/base material and LCC stabilization.

Table 6.1. Comparison of the reduction in UCS with adding RAP in cement vs LCC stabilization.

Cement mixtures at optimum cement content (2%)		
Soil	**UCS (psi) for 2% Cement**	**Ratio of UCS of SB+RAP /Pure SB**
Subbase III	149	0.74
Subbase III+RAP	110	
Subbase II	362	0.84
Subbase II+RAP	305	
CLC mixtures at optimum cement content (4%)		
Soil	**UCS (psi) for 4 % CaCl2**	**Ratio of UCS of SB+RAP /Pure SB**
Subbase III	91	0.33
Subbase III+RAP	30	
Subbase II	329	0.19
Subbase II+RAP	61	

TECHNOLOGY TRANSFER PLAN

The findings of this study were disseminated in different venues to be incorporated into the work-plan for local contractors for implementation of RSB in Vermont projects. This includes: two factsheets and two poster presentations at VTrans research symposiums in 2020 and 2021, and a poster at the UVM Student Research Conference. In addition, the authors are planning to submit a manuscript to the journal of Automation in Construction.

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