Mechanical and Durability Characteristics of Fly Ash Based Soil-Geopolymer Mixtures for Road Base and Subbase Layers

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Mechanical and Durability Characteristics of Fly Ash Based Soil-Geopolymer Mixtures
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DEDICATION

I would like to dedicate this thesis to my parents, Gopal Krishna Adhikari and Lok Kumari Adhikari and brother, Bikal Adhikari, for their love, support, inspiration, and belief in me.

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LIST OF ABBREVIATIONS

AR	Alkaline ratio
CSD	Cement Stabilized Design
CTD	Cement Treated Design
DOTD	Department of Transportation
EDS	Electron Diffraction Spectroscopy
FA	Fly ash
LA	Louisiana
M	Concentration of Sodium hydroxide
Mr	Resilience Modulus
OPC	Ordinary Portland Cement
SEM	Scanning Electron Microscope
UCS	Unconfined Compressive strength
µm	Nanometer

1. INTRODUCTION

1.1 Introduction and Background

Geopolymers are inorganic, amorphous, and three-dimensional polymeric chain between alumina-silicate materials obtained from alkali activation of silica and alumina rich materials such as; calcinated clays, kaolinitic clays, volcanic rocks, industrial byproducts, and fly ashes (coal, rice husk, sugarcane bagasse, etc.). The class F fly ash contains more than 90% silica and alumina; however, calcium oxide is less than 5%. Because of lack of reactive calcium oxide in Class F fly ash, it cannot react with the addition of water so it is considered as non-self-cementing in nature. Therefore, alkali activator is necessary to trigger the reaction between alkali and silica-alumina (Aydilek and Arora 2003, Abdullah, et al. 2011, Regan 2008). Generally, alkali activator such as; sodium hydroxide, sodium silicate, and potassium hydroxide are used at elevated temperature to initiate the Geopolymer reaction (Fernandez-Jimenez, Garcia-Lodeiro and Palomo 2015). Geopolymer binder has gotten all the attention from the researcher as it has shown enhanced chemical and physical properties in the development of concrete mixes as compared to ordinary Portland cement (OPC) concrete. Geopolymer has also shown improved resistance against fire and heating and exhibited better performance under ultraviolet (UV) rays, highly corrosive environment and acids. In addition, such concrete mixtures consume less energy and produced less greenhouse gas (CO2) and are 10% to 30% cost effective than OPC concrete (Regan 2008). Further, Geopolymers have been extensively used in concrete and modification of construction materials like bricks and mortar (Patimapon Sukmak 2013). However, its use in soil stabilization is limited and only few studies have been conducted.

Recently, researchers have used fly ash based Geopolymers to increase the strength and stiffness of the road base, subbase, and subgrade materials. Traditionally, OPC, lime, and fly ash has been used for stabilization of the road base and subbase materials. In Louisiana, road base and subbase have been constructed with graded aggregate or soil-cement stabilized mixtures with a cement content of 5 to 10%. It has been recognized that OPC is not environmentally friendly and its cost-effectiveness is usually compromised due to its excessive shrinkage strain when mixed with soils. Similarly, lime may be considered as environmentally unfriendly and costly materials as it needs substantial energy to produce, through crushing and burning of limestones rocks in kilns. Therefore, an alternative solution to stabilize the road base and subbase is required that exhibits long-term benefits, cost-effectiveness and environmental friendliness. One such material is the class F-fly ash based Geopolymer binders. Rios and Fonseca conducted an experiment with alkaline activation of soils along with the addition of fly ash, they found that the strength and stiffness of the mixture were significantly improved. The alkali activation of clay and class F fly ash mixtures not only increased the strength at elevated temperature but also exhibited the increase in the long-term performance (Farnandez-Jimenez and Garcia-Lodeiro 2007). Use of such Geopolymer products has several advantages as compared with OPC. First, the production costs of cement were very high and generated the high amount of carbon dioxide gas, a greenhouse gas (Provis and JD 2014). Further, Geopolymer enhances the uses of byproduct such as class F fly ash and reduces the costs of the construction. The alkali activated fly ash Geopolymer soil have also shown high durability and illustrated the continuous increase in strength up to 1 year. Hence the alkali activated fly ash based Geopolymer soil could be used to stabilize soils (Cruz, et al. 2017).

This research was conducted to stabilize the medium and high plastic soils using fly ash based Geopolymer binders. Class F fly ash was used as raw materials to developed Soil-Geopolymer mixtures. The effect of fly ash, alkaline ratio, heat curing conditions, the concentration of sodium hydroxide in strength development of strength Soil-Geopolymers were investigated. In addition, resilient modulus, dynamic modulus and durability characteristics were also evaluated. The scanning electron microscope (SEM) was performed to examine microstructure and morphology of Soil-Geopolymer mixtures, and the growth of Geo-polymerization products as a function of time and heat curing condition.

1.2 Problem Statement

The pavement design guidelines consider the aggregate compacted base for the pavement structure. However, availability of superior quality aggregates may not be available in many locations, like in the state of Louisiana. Likewise, transportation of quality aggregates from long distance is not economically feasible. Some researcher has tried to use the industrial by-products in the road base and subbase for various construction projects, following certain treatment procedures (Aydilek and Arora 2003). Novel road construction materials could be developed by treating natural soil using lime, cement, and fly-ash, or by an addition of certain other cementing materials to soils. Thus, use of stabilized soil product as a road base and sub-base material leads to an economical solution as well as a potential use of the industrial by-products.

The presence of water in the soils or pavement layers is the major problem for the pavement engineers. The strength and stiffness of soils reduce drastically as the moisture content increases beyond the optimum moisture content. Most of the subgrade soils in Louisiana have low strength and stiffness because of in-situ moisture content was found to be higher than the optimum moisture content of soils. In addition, some soils, like expansive soil have great tendency to shrink and swell and often creates serviceability problems during or after the construction of the pavement layers. The shrinkage or swelling characteristic of the soils are a function of in situ moisture content. Moreover, the water from the heavy rainfall penetrates the road base or sub-bases and subgrades, which also decreases the strength and stiffness of these layers. Hence, resulting in the high thickness of the road bases and sub-bases layer, which increases the construction costs. In order to increase pavement stability and durability, soil stabilization has become necessary.

In the state of Louisiana, it has been widespread practice to construct the road bases and subbases using soil-cement mixtures. Usually, two types of soil-cement mixtures design have been used; cement stabilized design (CSD) and cement treated design. In CSD, higher cement content is used and yield higher stiffness and unconfined compressive strength. However, the higher cement content was responsible for high shrinkage cracks due to the heat of hydration of cement over prolonged period. Such shrinkage cracks propagate upwards in the hot mix asphalt (HMA) layer and show as reflective cracks. Hence, the long-term performance of HMA pavements was compromised. On the other hand, CTD mixtures are prepared using lower cement content and hence exhibits less shrinkage. However, the stiffness and strength of such mixtures are lower than the CSD mixtures.

Based on the discussion, there seems to be a pressing need to develop road base and subbase mixtures that are cost-effective and exhibit improved long-term performance of pavement system. In this study, a new class of Soil-Geopolymer mixtures were developed using the class F fly ash and alkaline solutions. It was believed that the soil Geopolymer mixtures would have high compressive strength, adequate stiffness, and enhanced durability.

Such mixtures were not only cost-effective but also environmentally friendly as it promotes the use of industrial by-products and less emission of carbon dioxide.

1.3 Objectives

The main goal of the research is to develop fly ash based Soil-Geopolymer mixtures for road bases or subbase construction. The specific objectives of this research are as follows:

- Develop a mixing, curing and compaction procedure for soils, fly ash based Soil-Geopolymer mixtures, and using alkaline activators such as sodium silicate and sodium hydroxide.
- Evaluate physical and mechanical characteristics; moisture-density curve, unconfined compressive strength, and stiffness of Soil And soil-fly ash mixtures.
- Evaluate the effect of various mixture constituents including soil types, fly ash content, sodium silicate to sodium hydroxide ratio, the concentration of sodium hydroxide, curing period, and curing temperature.
- Develop statistical based regression models to conduct the sensitivity analysis of various Soil-Geopolymer mixtures variables and optimize Soil-Geopolymer mixtures.
- Investigate the effect of repeated loading on the Soil-Geopolymer mixture and compare resilience modulus and dynamic modulus of the optimized Soil-Geopolymer mixture with standard soil-cement stabilized mixtures.
- To determine the durability characteristics of optimized Soil-Geopolymer mixtures and compare it with the standard soil-cement mixtures.
- Evaluate the microstructure and morphology of Soil-Geopolymer mixtures in relation to mix variables and curing conditions.

Using Class F fly ash based Geopolymer binder to stabilize weak subgrade soil will result in enhancing physical as well as mechanical properties of mixtures. It is believed that silicate and alumina present in the Soil Also helps in forming Geopolymer products, which facilitate to bind the soil particles increasing the mixture's compressive strength, resistance to weathering, and improves performance under dynamic loading.

1.4 Research Plan

In order to accomplish the objectives of the study, a research plan was devised and implemented as shown in Figure 1-1. First, the literature review was conducted on Soil stabilization and possible use of fly ash based Geopolymer. Based on literature review mixing and compaction procedure were determined and set eye on physical and mechanical testing of various Soil And soil-fly ash and Soil-Geopolymer mixtures. The research was initiated by collecting the two types of soils, one with medium and the second with high Plasticity Index values. Physical and mechanical characteristics were determined in the laboratory for each type of mixtures. Preliminary experimentations were conducted and experimental design matrix was generated with three variables including fly ash, alkaline ratio, and concentration of sodium hydroxide. Unconfined compressive strength (UCS) test was conducted on selected experimental design matrix and linear regression analysis was conducted. The mixture design was optimized and sensitivity analysis was conducted to understand the impact of each variable on UCS of developed mixtures. Resilience modulus (Mr), Dynamic Modulus (E*), and Durability tests were conducted on optimum Soil-Geopolymer mixtures and compared with recommended soil-cement mixtures based on LA DOTD specifications. Finally, based on the results and analysis various conclusion and recommendation were drawn.

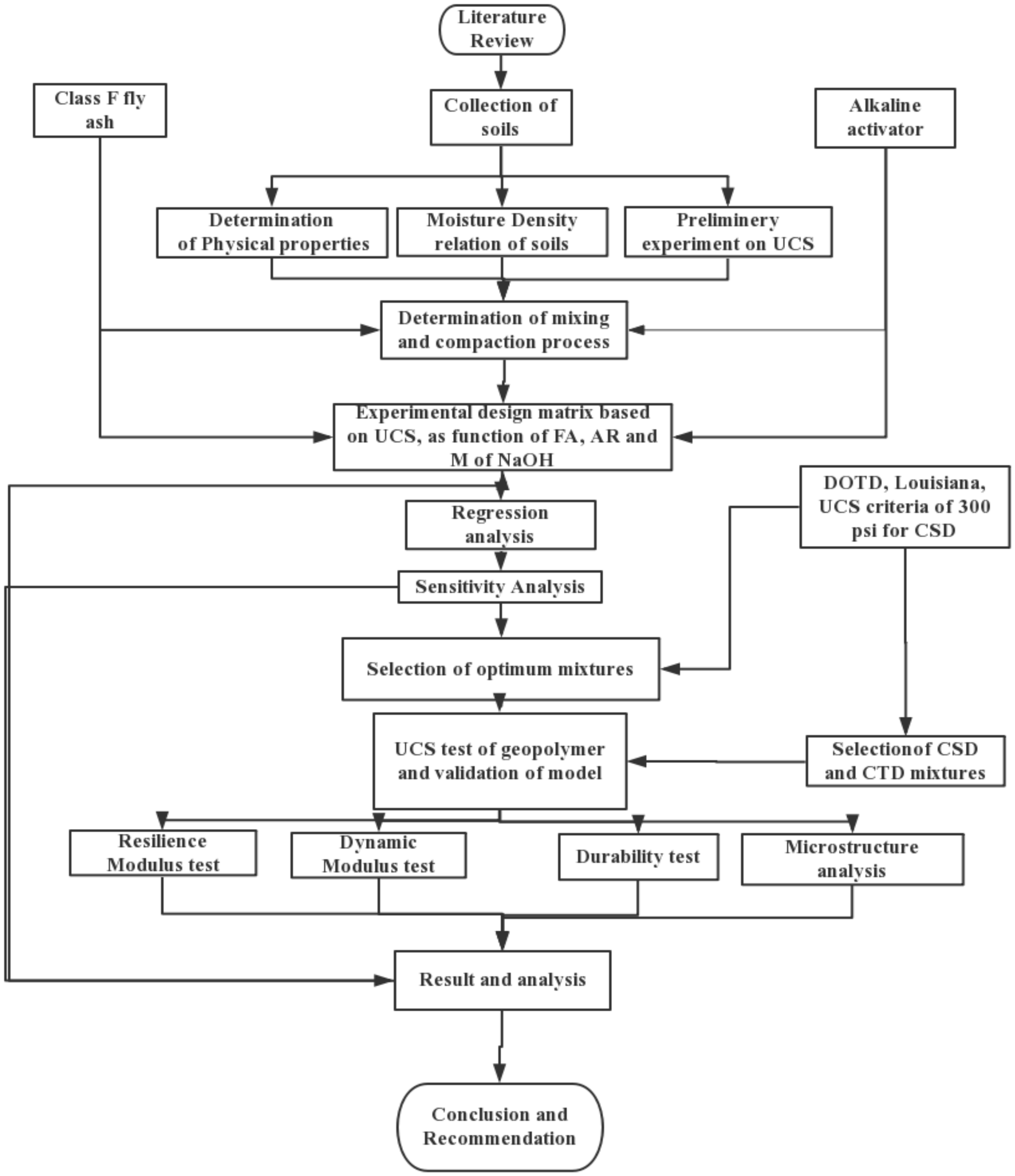


Figure 1-1: Schematic representation of the development of the research

1.5 Thesis Layout

This thesis was organized as explained in this topic. Chapter 2 explains the literature review on soil stabilization and Geopolymer soil stabilization, factor affecting soil stabilization, and various laboratory experiment conducted by various researcher and their conclusions. Chapter 3 explains various laboratory experiment and their detail testing procedure. In Chapter 4, data analysis, sensitivity analysis, and discussion on Unconfined Compressive strength (UCS), Resilience Modulus, Dynamic Modulus, and Durability test were presented. Finally, a conclusion and recommendation are provided in Chapter 5.

2. LITERATURE REVIEW

2.1 Soil stabilization

Soil stabilization is the process of changing the properties of soils to improve the strength and durability. Natural soil contains high clay amount, silt, sand, and gravel, are difficult to use in the construction field so modification and stabilization are necessary. The soils were mixed with additives such as; cement, lime, fly ash, polymer, and compacted in the field to enhance the physical and chemical properties soil (Sherwood 1993). The stabilization of the soils leads to increase in the strength of the soils, resulting in low compressibility and low permeability and high resistance against freeze and thaw. In fact, these properties are the basic requirement for the design of safe and economical highway, airport runways and foundation design (Garciarz 2012). Generally, Stabilization is classified into; mechanical and chemical stabilization process (Little and Nair 2009).

2.1.1 Mechanical stabilization

Mechanical stabilization is conducted by mixing or blending soils of two or more gradations to obtained recommended specification (Little and Nair 2009). In this modification process, the density of the mixtures was increased by compaction, pre-compression, drainage and vibration of the soil-stabilizer mixtures (Makusa 2012). In addition, compaction was conducted with dynamic loading, in which heavy hammer was dropped repeatedly into the ground with an equal interval of time. This mechanical stabilization was very significant in the case of cost but weak in chemical attack (Sherwood 1993).

2.1.2 Chemical stabilization

Sometimes chemical or grouting's were used in the field with a removal of undesirable week top soils. Chemical stabilization was conducted at the construction site, which involved spreading of mixed or blended material and the mixture was compacted to get required density, recommended by Standard or Modified Proctor Test (Makusa 2012). Traditionally, in this method ordinary Portland cement(OPC), lime and fly ash were used for stabilization. The pozzolanic reaction between soils, water, and these chemicals resulted in the formation of cementitious hydration products enhancing mechanical properties (Sherwood 1993).

2.1.2.1 Cement stabilization

Traditionally, cement is used as the stabilizing agent, which showed good strength for the base and sub-base materials. Cement is the primary stabilizing agent or hydraulic binder for the stabilization because of its hydration nature that it can form hydration product immediately with an addition of the water (Gomez S. and Anderson 2012). Based on availability of the cement like ordinary Portland cement, blast furnace cement, sulfate resistant cement and high alumina cement were used according to the requirement of strength and nature of the soil. The high amount of cement along with lime was necessary soils with higher plasticity index for cement stabilization (Sherwood 1993). The hydration process starts just after an addition of water, in which it produced hydration product and makes the soil hard. Eventually, improving the strength of the soil over time as the reaction was a slow process (Gomez S. and Anderson 2012).

2.1.2.2 Lime stabilization

Lime treatment, is also another method of soil stabilization. Lime stabilization was used in high plastic soil to reduce plasticity index, in which certain percentage of lime was added to the soils. (Carmeuse 2007).

2.1.2.3 Fly ash stabilization

Fly ash is the by-product of the pulverized coal in the electric power plant. According to American coal association(ACA) statics, 2007 every year 131 million tons of fly ash was produced in the United State of America. Out of this, only 43 percent were used. The geotechnical properties of FA showed that this fly ash can be used in structural and landfill embankment, filler for mines and queries and pits, along with construction of pavement and airports on soft soils (Vasquez and Alonso 1981).Selection of the stabilizing agent was based on the soil properties, the decision tree for selection of the stabilizing agent is shown in Figure 2-1 (Little and Nair 2009).

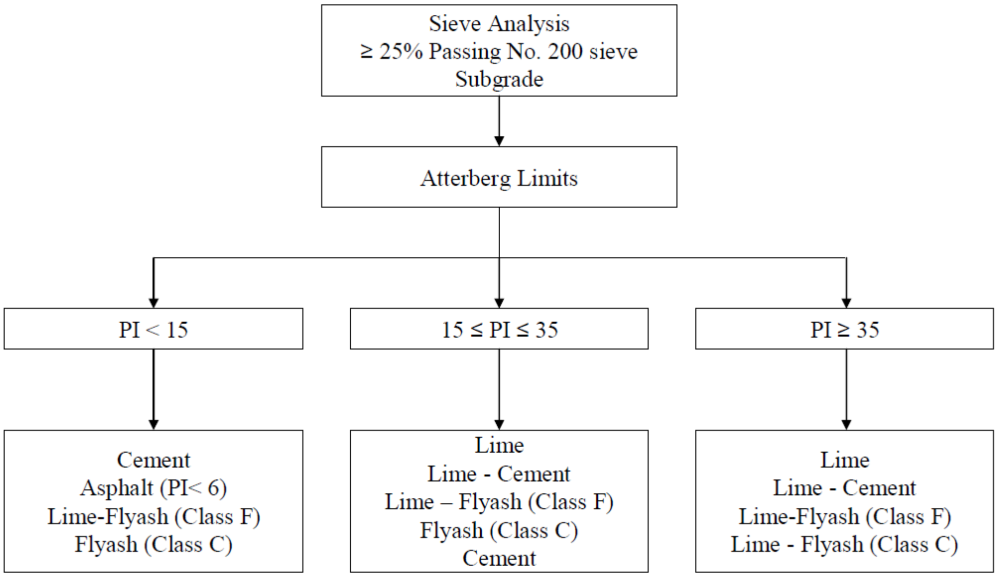


Figure 2-1: Decision tree for selecting the stabilizing agent for subgrade soils (Little and Nair 2009)

Cement is considered essential construction materials and used extensively all over the world. However, it has some issue with environment and costs. The Production of the Ordinary Portland cement was not environment-friendly because it releases significant volume of carbon dioxide (CO2) to the atmosphere (Provis and JD 2014). To produce 1 ton of cement, it produced the same amount of carbon dioxide gas. Moreover, elevated temperature around 1500℃ was used to produced cement. Therefore, arises need further investigation into the cementitious product which decreases environmental impacts and enhances benefits.

Over the last decades, researchers were using class F fly ash to stabilize subbases and bases, they found that it improved the compressive, shear strength of soils, reduce the earth lateral pressure and improve the slope stability (Rios, Cristelo, et al. 2011). The literature review also revealed that various kind of roadbed soil, subbase or base materials including; clay, sands, gravels, crushed stones can be stabilized with fly ash based Geopolymer (Rios, Criste lo, et al. 2011, Kazemian, Vayghan and Rajabipour 2015, Gomez S. and Anderson

2012). This stabilization method might lead to the biggest application of fly ash in the field of transportation and can save millions (Aydilek and Arora 2003). In aggregates, fly ash generally acts as a pozzolana or filler and reduced the void spaces among larger size aggregate particles (Kazemian, Vayghan and Rajabipour 2015). After the addition of appropriate amount of fly ash all the voids were filled with fly ash and increased the density of the mixtures. An activator-like cement, lime, and alkali were used to maximize the pozzolana reaction in the mixture, which enhanced the reaction resulting higher strength (Rios, Cristelo, et al. 2011).

Some of the advantages of the Soil-Fly ash stabilization are as follows,

- Improved the soil or materials properties of like shear strength, plasticity, and density and enhanced the stability and durability of the pavement (Sargent, et al. 2013).
- It was byproduct so cheap and environmental friendly with compared with cement.
- With increasing stiffness and strength of base and sub- bases were increased
- Stabilization improved the permeability of the soil, reduced swell potential, increased resistance to sulfate attack.

2.2 Geopolymer

Geopolymer products are simply an amorphous, complex tetrahedral covalent bond between -Si-O-Al-. Geopolymers are the frameworks structure links between tetra hydra, leading to 3D aluminosilicate network as shown Figure 2-3: Geopolymer structure with silica and alumina contents The aluminosilicates product undergoes a chemical reaction with an alkaline solution, it forms an inorganic amorphous product, called Geopolymer. This Product might form from various source, as specified in Figure 2-2.Basically, this research was conducted on industrial by-product Class F fly ash based Geopolymer cement.

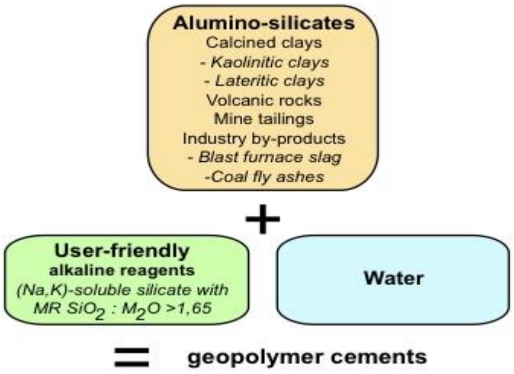


Figure 2-2: Various sources for formation of Geopolymer cement (JDavidovits 2013)

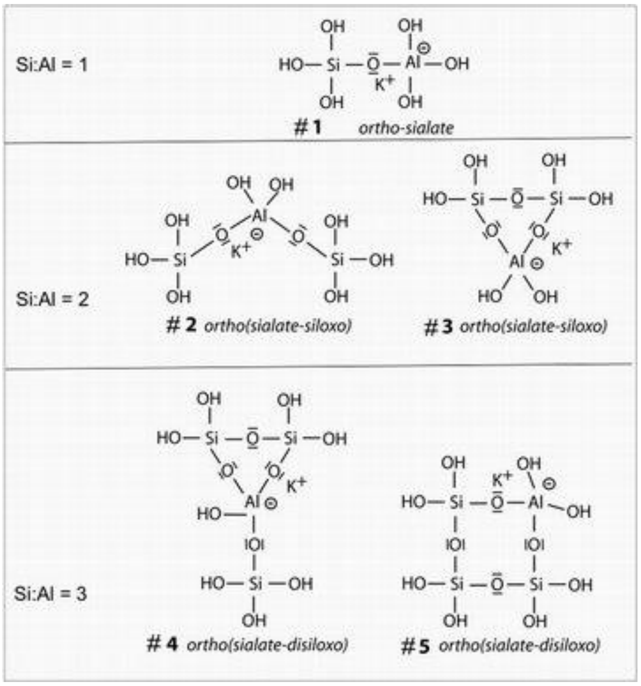


Figure 2-3: Geopolymer structure with silica and alumina contents (JDavidovits 2013)

2.2.1 Fly ash

The fly ash is defined as the byproduct of the electrical power plant and coal power plant. Fly ashes are readily available, cheaper and environmentally friendly materials consisting of high silica and alumina. Depending on the types of power plant, the composition of fly ash differs. Generally, it consists of silicon dioxide, alumina oxide, calcium oxide and ferrous oxide. Based on the ASTM C618 it is classified into two group, Class C and Class F. A Class C fly ash is one which has reactive silica, obtained from burning lignite and sub-bituminous type of coal, which contains more than 10% of calcium oxide. In addition, Class F fly ash contains non-reactive silica and obtained from burning bituminous and anthracite type of coal, which consists of less than 10% of calcium oxide. The main distinction is its self-cementing property, class C fly ash forms cementitious products with the addition of water. However, class F fly ash does not produce the cementitious product in presence of water. It is due to the presence of high amount of calcium oxide in class C fly ash, due to which rapid reaction happens between fly ash and water, providing the better strength, and durability. But in Class F fly ash activator-like; cement, lime, or alkali is necessary to initiate the reaction. The unique spherical shape and particle size distribution of fly ash make its good filler materials (Aydilek and Arora 2003). According to Rios et al (Rios, Cristelo, et al. 2011), the strength of soil-fly ash with alkali activation was higher than the soil-cement mixtures. Moreover, showed the slower, progressive strength development and long-lasting strength (Cruz, et al. 2017). Hence, use of fly in a large amount is possible in various road bases, sub-bases and any other structural as well as geotechnical fill (Aydilek and Arora 2003).

2.2.2 Alkali Activator

The activator quickly caused the precipitation and crystallization of silicate and alumina present in the materials. Hydroxide ion accelerated the reaction and released the silica and alumina into the solution and metal ion act as the structural element (Abdullah, et al. 2011). Generally, Sodium hydroxide (NaOH) and Sodium silicate (Na_2SiO_3) were considered good alkali for the formation of Geopolymer.

NaOH was preferred as an alkali activator rather than KOH because activation level of sodium ion was higher than Potassium ion and has high charge density (Singh, Chowdhury and Mishra 2015). The concentration of sodium hydroxide another factor in the formation of Geopolymer products (Roy, et al. n.d.). Higher the concentration of sodium hydroxide, higher the strength at preliminary stages of the reaction. However, at higher concentration of sodium hydroxide, excessive hydration products were formed resulting formation of undesirable product (Cristelo, G., et al. 2012). According to Rios and Fonseca the strength of Geopolymer was increased with increasing molarity but after 10 M strength decreased (Rios and Fonseca, 2015).

Sodium silicate was used as an activating agent for the Geo-polymerization process along with sodium hydroxide. Generally, soluble silicates were used because it enhances the interparticle bonding between alumina, silica, and aggregates. Per Rashad and Zeedan, the compressive strength of alkali activated fly ash depends on the amount of sodium silicate. With increasing amount of sodium silicate, the strength was increased (AS. and SR. 2011). Based on the experiment on concrete, the strength was decreased with lower soluble silicate. Additionally, the bond between materials was enhanced by the presence of soluble silicates (Khale, Chaudhary and Mater 2007).

2.2.3 Reaction Mechanism

Class F fly ash with soil only acts as the filler materials and increased the mechanical and physical properties. During the hydration process, fly ash was used with the calcium hydroxide (sodium hydroxide or potassium hydroxide) forming calcium silicate hydrate and calcium aluminate hydrates in presence of high alkaline environment, which was provided by sodium silicates and sodium hydroxide or calcium hydroxide or KOH. The reaction process was very fast and generates three- dimensional polymeric chain and ring-like the structure of Si-O-Al-O bonds, as shown in equation 1, 2 and 3. (Abdullah, et al. 2011).

$$Mn\ [\text{-}[SiO_2]\text{z-AlO2]n. w}H_2\text{O} \qquad (1)$$

Where Mn was alkaline element or cations like sodium and calcium and n was degree of poly-condensation or polymerization and z was 1, 2, 3 up to 32,

$$\text{n [Si2O5, Al2O2] + 2nSiO2 + 4nH2O +NaOH/KOH} \rightarrow \text{[Na+, K+ + n[OH]3-Si-O-Al-[OH]\_2-O-Si-[OH]3} \qquad (2)$$

$$n\text{[OH]3-Si-O-Al--Si[OH]3+NaOH} \rightarrow \text{[Na+]-[-[Si-O]-O-[Al-O]-O-[Si-Al-O]-O-] +4nH2O[OH]2} \qquad (3)$$

The chemical reaction occurred in three phases. First, dissolution of Si and Al atoms from fly ash and soil by the action of hydroxide ions, then rearrangement of precursor ions were occurred into monomers and finally Setting or poly-condensation/polymerization of monomers led to the complex polymeric structures (Abdullah, et al. 2011).The reaction happened between alumina, silica, and alkali during the activation period, in which sodium hydroxide or ROH as alkali substance, refer as calcium, potassium etc. The silica and alumina tetra-hydra interconnected and shared the oxygen ions, called Poly-condensation period. This process starts when hydroxide ion concentration helps in breaking the covalent bond between Si-O-Si, AL-O-AL, and Al-O-Si from the vitreous phase, which releases the silica and alumina ions into solution. The cations like NA+, K + compensated the negative charge associated with aluminum during dissolution phase and helped to precipitate the Si and Al ions around the nuclei, such that sharing all oxygen and forming Si-O-AL and Si-O-Si three-dimensional structure, which was the main product of the Geo-polymerization product, has more stiffness, strength and durability properties (Bignozzi, et al. 2014).

2.3 Factor affecting Class F fly ash-soil stabilization

2.3.1 Types of soils

The types of Soil A were major parameter that determined the strength of the soils. Most of the stabilization was conducted on soft soil like organic soil, clayey soil, and silty soils. The fine-grained soil has a large surface area so easier to stabilize but in the case of silty soil, it was difficult because the silty soil was more sensitive toward moisture. Some organic soils hindered the reaction between soil and chemicals so difficult to stabilize. However, the presence of the fines in the soils was a major factor in determining the strength of the soils. With increasing fines, the strength of mixtures was increased (Sherwood 1993). According to Aydilek et at., the strength of the low plasticity soils was not increased significantly with increasing contents of the fly ash (Aydilek and Arora 2003). The plasticity of soils treated with Class F fly ash was influenced by the types of clay minerals present in the soil. Treatment process or chemical used also changes with nature of soils, especially for high plastic soil first its plasticity was reduced with the addition of lime and then treated with cement to achieve desired properties (Sherwood 1993).

2.3.2 Types and amount of the activator

Various activators were used to activate Class F fly ash like cement, lime, and alkali, the result showed that cement activation was effective for mixes containing cohesion-less fines and alkali activator also showed high strength with FA (Aydilek and Arora 2003).In addition, the strength of the soil-fly ash was also affected by the amount of the activator present in the mixtures. With increasing cement activator content from 1% to 5%, the strength increased. However, with further increased in cement contents, the strength of mixtures did not increase significantly (Aydilek and Arora 2003). Similarly, the strength was also increased with increasing concentration of Sodium hydroxide up to optimum (12M) and decreased after increasing concentration of sodium hydroxide (Rahim, et al. 2014).

2.3.3 Fly ash contents

The strength of the soil FA was also affected by the amount of fly ash. The strength of the soil-fly ash mixtures increased with increasing fly ash contents up to optimum FA contents after that increasing FA content was not significant (Kang, et al. 2014). Zumrawi, 2015 suggested that this same final UCS value with increased FA content might be due to the formation of the shrinkage cracks inside the specimen, so the amount of fly ash should be optimized and not more than 15%.

2.4 Temperature and curing time

The alkali activated with fly ash produce low heat of hydration at early ages so the temperature was necessary to accelerate the chemical reaction (Bignozzi, et al. 2014). With increasing the temperature, the rate of the reaction was accelerated and resulted in the formation of Geopolymer product. Eventually enhanced the early strength and durability properties (Fernandez-Jimenez, Garcia-Lodeiro and Palomo 2015, Lizcano 2011). The class F fly ash showed progressive and long-term strength development in the alkaline environment as compared to OPC (Rios, Cristelo, et al. 2011). Rios et al observed that the compressive strength of Soil-Geopolymer mixtures was effected by curing period. The UCS for 30 days and 90 days curing were significantly differenced (Rios, Cristelo, et al. 2011).At similar curing period of 28 days, the specimen with moist cured showed higher compressive strength than dry cured (Kazemian, Vayghan and Rajabipour 2015). Sukmak et al. heated the sample for 24, 48 and 72 hours and kept at room temperature for curing. The result suggested that strength of the specimens was increased with more curing period as the rate of Geo-polymerization increased. Further, concluded that heating of mixtures at-48 hour, resulted in higher strength, as illustrated in Figure 2-4. This figure also illustrates that increased in strength was not significant after 28 days of curing (Patimapon Sukmak 2013).

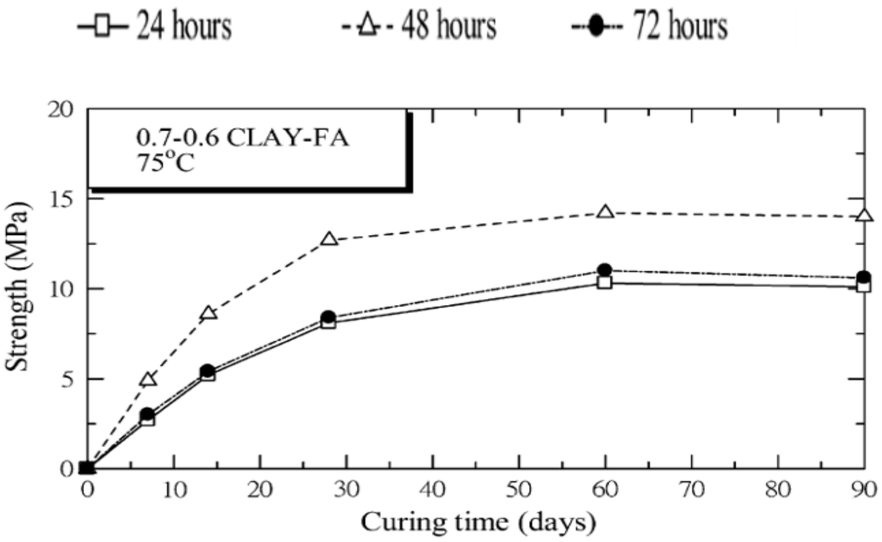


Figure 2-4: Effect of curing after heating for 24, 48 and 72 hours (Patimapon Sukmak 2013)

However, Cruz et al conducted testing up to 350 days, the result showed that the strength was increased significantly. Further, strength versus curing period plot suggested that that the strength was still increasing, as shown in Figure 2-5 (A1C7 was 10%FA, 1 (AR) alkaline ratio and 7.5M sodium hydroxide and A05C7 was 10%FA, 0.5 (AR) alkaline ratio, and 7.5M Sodium hydroxide).

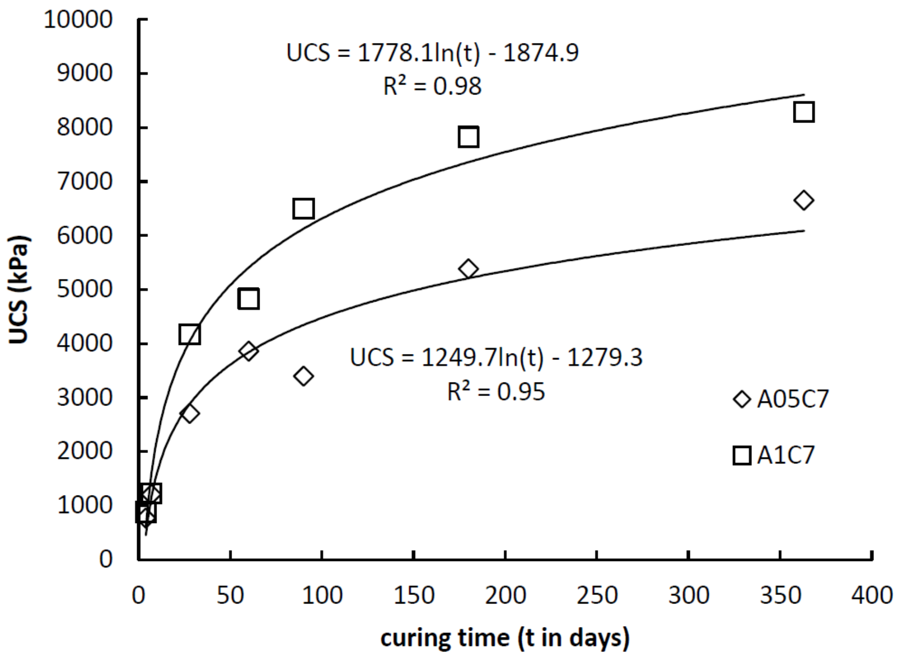


Figure 2-5: Strength and duration of curing for two mixes (Cruz, et al. 2017)

2.4.1 Alkaline Ratio (AR)

It is the ratio between sodium silicate and sodium hydroxide (Na_2SiO_3/NaOH). This was also a major factor in effecting the formation of hydration product. Higher the AR value higher will be the sodium silicate content. According to Bignozzi et al (Bignozzi, et al. 2014), the higher silicate content results in higher strength of the specimen. His works show 200 to 300% increase in strength for the higher amount of sodium silicates. Sukmak et al (Patimapon Sukmak 2013), found that alkaline ratio of 0.7 was optimum for the clay FA Geopolymer mixtures. The compressive strength of the fly ash -Geopolymer increases with increasing alkaline ratio up to an optimum value (1.50) and then decreases, as shown in Figure 2-6.

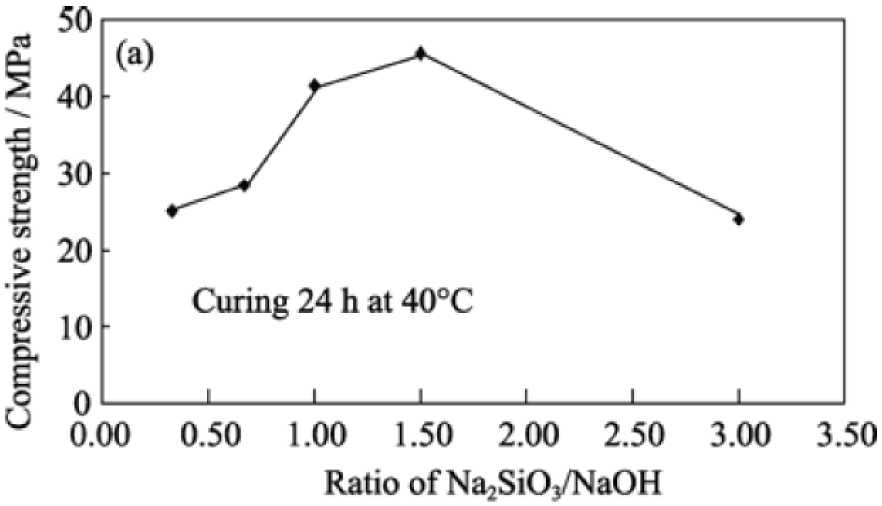


Figure 2-6: Effect of alkaline ratio on UCS for 24-hour curing at 40℃ (Ridtirud and Chindaprasirt 2011)

2.5 Laboratory Test

2.5.1 Atterberg Limits

The research conducted on Atterberg limit test on highly plastic Soil And fly ash mixtures, they found that liquid limit(LL) of the soil decreased with the addition of FA. Similarly, plastic limit of the soil was increased (Aydilek and Arora 2003). This decreased in LL was due to a chemical reaction between clay particles and additive, resulted in the reduction of the size of the diffused double layer and the inter-particle contact was increased (Zumrawi and Mohammed 2016). Aydilek et al conducted laboratory research on two types of fine- grained soil (soil A and soil B) (Aydilek and Arora 2003). The plastic limit, as well as liquid limit, was decreased with increasing fly ash content, also supported by Zumrawi, 2015 (Zumrawi and Mohammed 2016). Vukicevic et al. conducted Atterberg limit test on two types of soils, Soil A (medium plastic soil) and soil B (low plastic soils). The results showed that for Soil A the liquid limit and plasticity index was increased with increased FA content but for soil B the increased content of FA was not affecting the consistency of soils (Vukićević M . 2013).

2.5.2 Compaction curves

With the addition of the FA in soils change in chemical composition of the soil was observed in terms of maximum dry density and optimum moisture content. The maximum dry density decreased with the addition of FA, however, OMC was increased (Aydilek and Arora 2003). But as per Zumrawi, 2015, OMC decreased and maximum dry density increased with increasing fly ash content (Zumrawi and Mohammed 2016). This discrepancy between the dry density and OMC might be due to, varies types of activator, gradation, or types of soils. At 15 % fly ash contents, they found that maximum dry density was 15.3 KN/m^3, which was 15 % higher than untreated soils, and OMC decreased to 15% which was 7% less than untreated soils (Zumrawi and Mohammed 2016).

Various researchers conducted tests on several types of soils, the results of the studies showed that the maximum dry density increased with increased fly ash content (Kang, et al. 2014) (Das and Parhi 2013). In addition, for both soil, the maximum density was observed at 20% FA content. Further, optimum moisture contents of 15% and 20% were found for Atchison Soil and Putnam soil respectively. Similarly, maximum dry density of17 KN/M^3 and 15.2 KN/M^3 was observed for Atchison and Putnam soil respectively (Kang, et al. 2014). However, Vukicevic et al conducted Atterberg limit tests on low and high plastic soils, concluded that maximum dry density decreased and optimum moisture content increased with increasing FA content for both medium and low plastic soils.

2.5.3 Unconfined compressive strength

The unconfined compressive strength (UCS) of FA-soil mixtures was increased with cement, lime or alkaline mixture (Fernandez-Jimenez, Garcia-Lodeiro and Palomo 2015).Aydilek et at conducted experiment with Soil And class F fly ash with cement activator concluded that content was responsible for increased strength of mixtures. In addition, found that the UCS increased with increased content of cement activator but after 5% cement the increased in strength was not significant (Aydilek and Arora 2003). Sergent et al. conducted tests on alkali-activated FA and concluded that strength of the mixtures improved with fly ash content and curing time (Sargent, et al. 2013). In addition, he also conducted testing with various Industrial by-product (Pulverized fly ash (PVA), ground granulated blast furnace slag (GGBS) and red gypsum (RG), with alkali activation and non-alkali activation. The result showed that alkali activated fly ash had higher strength than another industrial by-product, as shown in Figure 2-7. In Figure, the alkali activated FA showed higher than 6000 Kpa UCS (Sargent, et al. 2013).

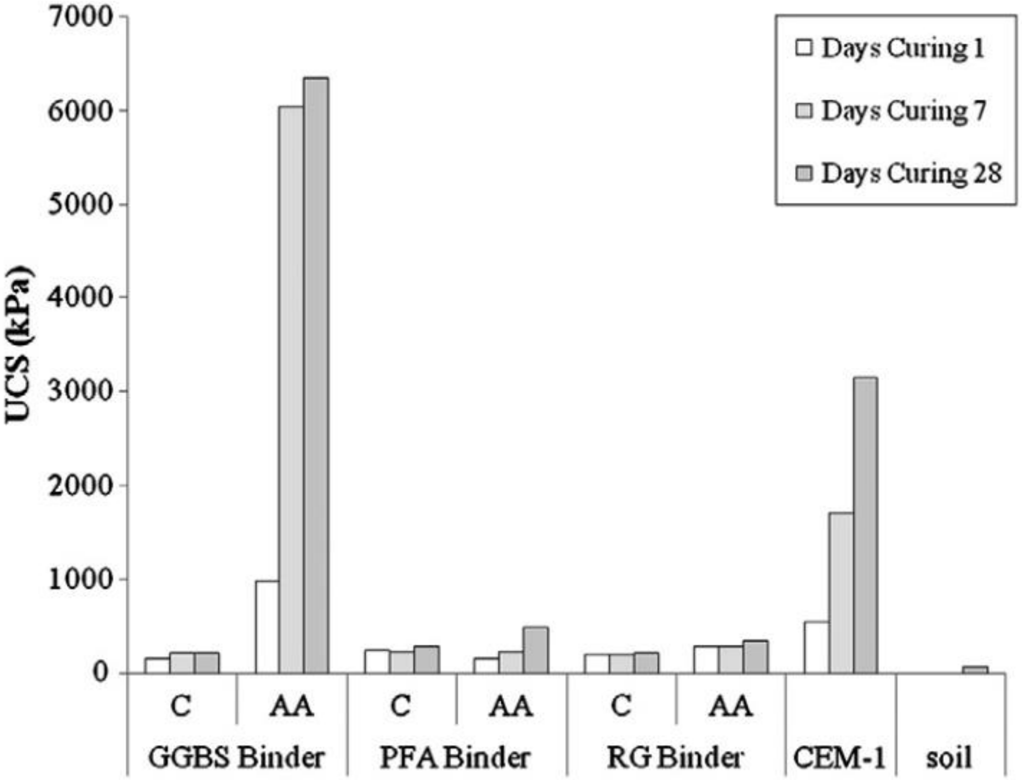


Figure 2-7: Average compressive strength of industrial by-product binder (Sargent, et al. 2013)

Rashad and Zeedan, found that the strength of the Geopolymer increased with the higher concentration of NaOH. Similarly, results showed higher initial strength with the higher concentration of sodium hydroxide, the optimum concentration found to be 12.5 M (AS. and SR. 2011). With a use of small amount Ca(OH)2 activator strength initially increased but with the higher content strength of calcium hydroxide strength of Geopolymer was decreased (Vargas, et al. 2014). Further, Sukmak et al. conducted an experiment on clay fly ash mixtures, the result showed that UCS increased with increasing Alkaline ratio and liquid to FA ratio up to optimum strength. The strength of the alkaline ratio of at 1.5 and 0.5 liquid to fly ash ratio, as shown in Figure 2-8. This figure suggested that at the alkaline ratio of 1.5, the strength was increased with increasing L/FA ratio, after reaching a peak, strength was decreased (Patimapon Sukmak 2013).

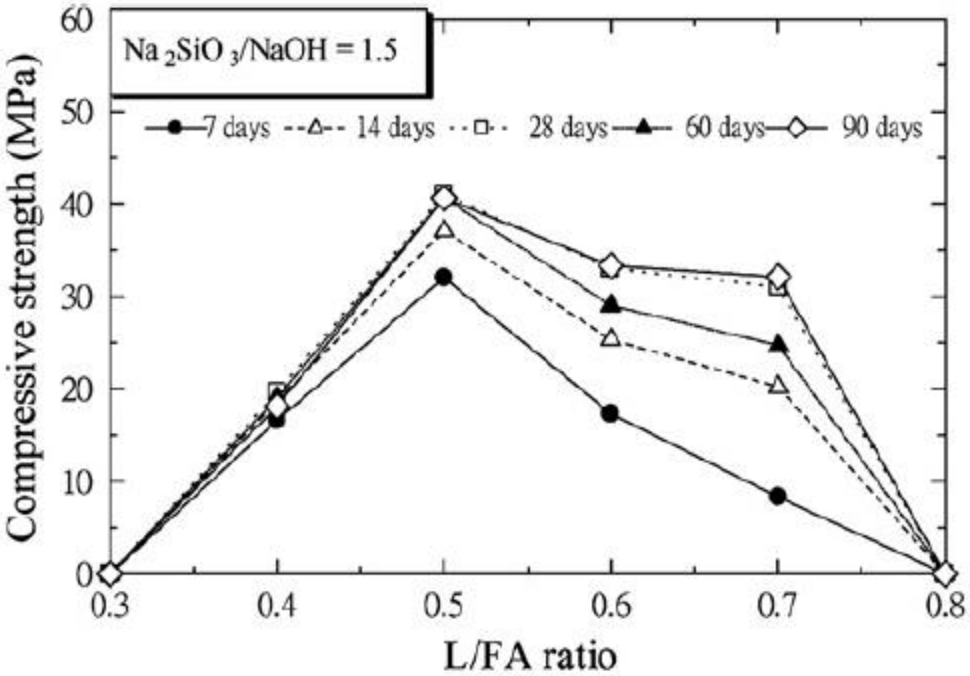


Figure 2-8: Variation of compressive strength of clay fly ash Geopolymer with L/FA ratio, with curing days (Patimapon Sukmak 2013)

Rios and Fonseca conducted tests on class F fly ash with fine soil, passing sieve no. 200 for stabilization. The alkaline ratio of 0.5(sodium silicate to sodium hydroxide) and various combination of the concentration of sodium hydroxide of 4,6,8 and 10 M was considered for laboratory testing at various curing period. The result showed that with increasing molarity the strength of the mixtures increases but at 10M strength decreased. UCS of the alkali activated mixtures showed significantly higher than the soil-cement mixtures, for the longer period of curing time (Rios, Cristelo, et al. 2011). Bignozzzi et al, conducted room temperature curing on several types of fly ash and concluded that strength of the mixtures increased with increased amount of sodium silicate and fineness of the fly ash (Bignozzi, et al. 2014).

The compressive strength of Geopolymer increased with elevated temperature curing as compared with ambient temperature curing, because of the slow rate of the Geo-polymerization process. The result showed that overall strength of Geopolymer was found comparable to the compressive strength of ordinary Portland cement (Ridtirud and Chindaprasirt 2011). Also, strength increased with increased alkaline ratio (silicate to hydroxide ratio) up to 1.5. Meanwhile, the strength was affected by the concentration of NaOH. However, the strength was decreased with increasing liquid to Fly ash ratio (Ridtirud and Chindaprasirt 2011).

Jimenez et al used Blast furnace slag and Industrial Fly ash and produced the Geopolymer products. They suggested that these Geopolymer products showed same hydration Gel provided by cement (Fernandez-Jimenez, Garcia-Lodeiro and Palomo 2015). Moreover, they mentioned that heating at higher temperature accelerated the rate of the reaction in the mixtures. They found that with heating of mixtures at 80 ℃ the strength was higher relative to lower temperature. US Army corps of Engineer recommended unconfined strength for base and subbase materials for rigid and Flexible pavement as shown in Table 2-1.

Table 2-1: Us Army corps minimum 7 days' strength criteria for road materials (Department of Army 1994)

Purposed of Stabilized layer	Minimum 7-day Unconfined compressive strength (psi)	
	Flexible Pavement	Rigid Pavement
Base Course	750	500
Subbase, select material or subgrade	250	200

2.5.4 Resilience Modulus

According to Aydilek et at, resilience modulus of soil- FA increased with the increasing content of cement activator (Aydilek and Arora 2003, Zumrawi and Mohammed 2016, Kang, et al. 2014). In addition, the result indicated that for higher activator resilience modulus was not increased. The resilience modulus increased up to optimum alkaline ratio and decreased after optimum value as shown in Figure 2-9. The MEPDG recommends some values of resilient modulus for the chemically stabilized layers as shown in Table 2-2 (Mechanistic-Empirical design Guidelines 2002).

Table 2-2: Typical resilient modulus for chemically stabilized layers (MEPGD, 2002) (Mechanistic-Empirical design Guidelines 2002)

Chemically Stabilized Materials	E or Mr range (psi)	E or Mr Typical (psi)
Soil-cement	50,000 to 1,000,000	500,000
Lime-Cement-Fly ash	500,000 to 2,000,000	1,500,000
Lime stabilized soils	30,000 to 60,000	45,000

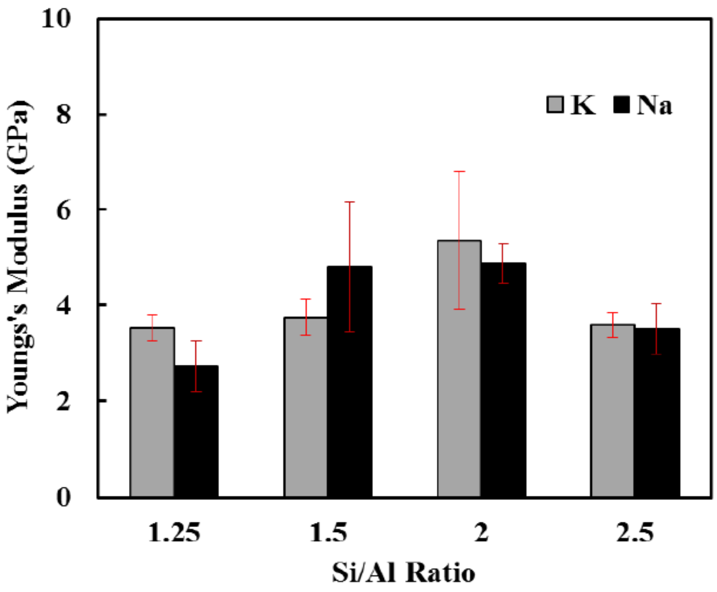


Figure 2-9: Effect of Si/Al ratio in young modulus of soil (Lizcano 2011)

The resilience modulus test was conducted in accordance with AASHTO T307 for highway materials, for which sample was prepared with optimum moisture contents. The resilience modulus test was carried out for different mixture for various curing period, deviator stress and confining pressure. The results showed that the resilience modulus increased with increasing confining pressure and decreased with deviator stress. However, increase and decrease was not significant. Further, they related the resilience modulus with unconfined compressive strength, plotted the graph between UCS and resilience modulus for various fly ash content and soils, as shown in Figure 2-10 (Kang, et al. 2014), they found the linear relationship between UCS and resilience modulus.

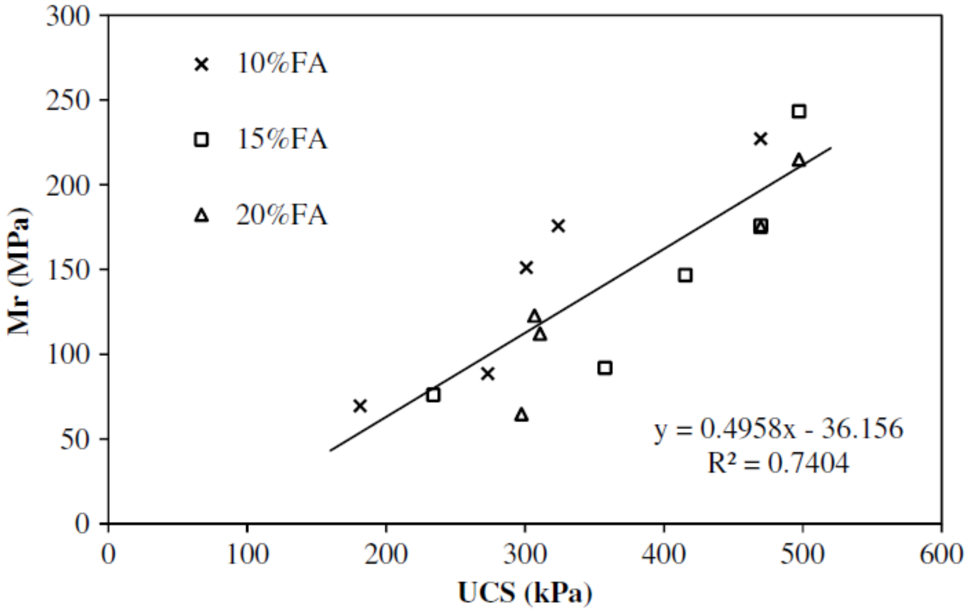


Figure 2-10: Relationship between resilience modulus and UCS of various mixtures (Kang, et al. 2014)

2.5.5 Permanent Deformation

Permanent deformation is another important parameter for determining the quality of base materials, which was related to the rut resistance. Basically, soils are subjected to heavy repeated traffic loads and cause re-arrangement of the soil particles within soils causing deformation to the pavement layer. Each repetition will cause damage in the pavement, which accumulates after the considerable number of cycle. The irrecoverable deformation of the materials at the end of each cycle was called the permanent deformation or the permanent strain. Almost all flexible pavement fails in fails due to excessive rutting or permanent deformation of pavement.

Kang et al (Kang, et al. 2014) related the permanent deformation and resilience modulus of the various mixture, as shown in Figure 2-11, as best fit power curve. Further, they observed that materials with different additives had similar resilience modulus even though they have different permanent deformation (Kang, et al. 2014). The permanent strain of almost 4% was achieved in FA -soil Geopolymer mixture (Cruz, et al. 2017).

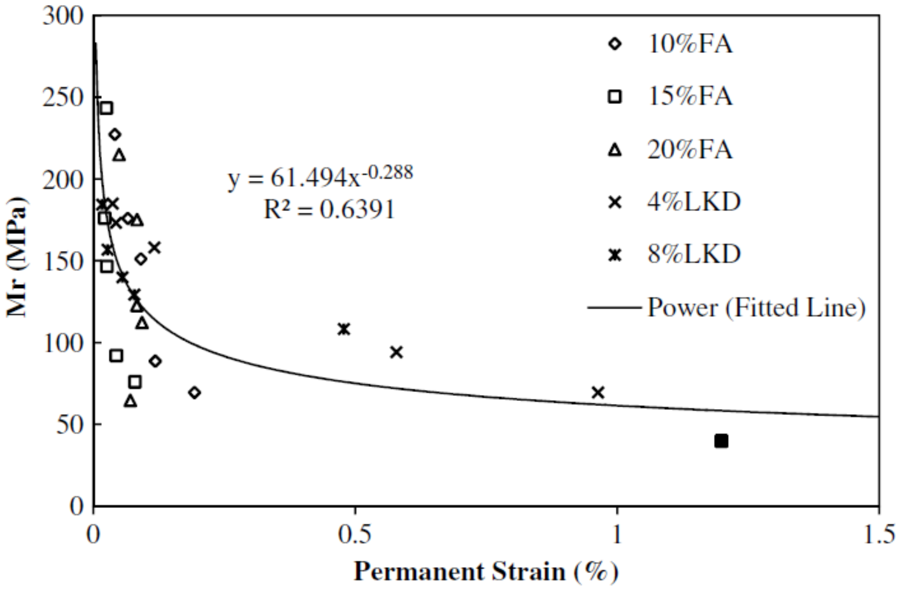


Figure 2-11: Variation of resilience modulus and permanent deformation for various combination of mixture of soils (Kang, et al. 2014)

2.5.6 Shrinkage test and Swell potentials

An increase in alkali concentration was found to increase the initial shrinkage of the pastes during elevated temperature curing. Excessive initial shrinkage and increased temperature internal stresses in the mortar, which could affect the material performance (Farnandez-Jimenez and Garcia-Lodeiro 2007). In addition, it was indicated that Shrinkage of Geopolymers was affected by curing temperature and liquid-to-ash ratio. High NaOH concentration of produced a Geopolymer with high shrinkage comparing to that with a low

NaOH concentration (Ridtirud and Chindaprasirt 2011). The swell potential of the soil was decreased with the addition of fly ash, at 15 % fly ash swelling reduces significantly (Vukićević M . 2013).

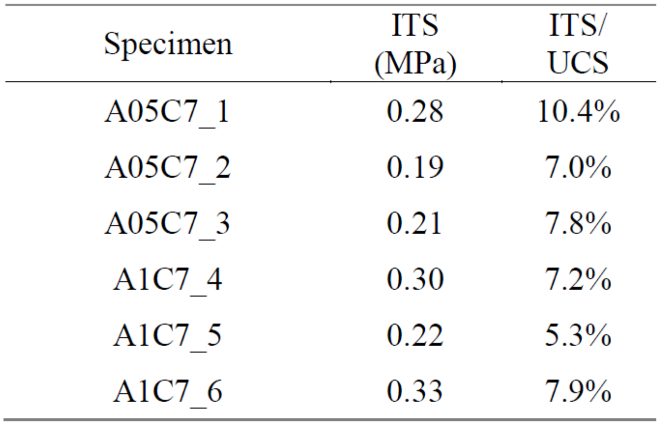
2.5.7 Durability

The sample was prepared according to standard compaction method for compressive strength testing, ASTM D698-91 for durability testing. The durability testing can be done with freeze–thaw and wet–dry tests that follow ASTM procedures for soil stabilization. Cruz et al. (Cruz, et al. 2017) found that soil-FA Geopolymer passed the wet and dry durability test, this Geopolymer sample has less than 1.58% weight loss.

2.5.8 Indirect Tensile Test

Indirect tensile tests were performed alkali activated fly ash-soil mixtures and the ratio between tensile and compressive strength was evaluated. The ratio between indirect tensile strength and unconfined compression strength was evaluated and found that Geopolymer has very high value as compared with cement stabilized soils. Moreover, the tensile strength was 10% of UCS of the mixture (Rios and Fonseca). Similarly, Cruz et al, conducted tensile strength test on two mixture of FA-soil Geopolymer, the result showed that the tensile strength was 7.5% of the UCS of the soils, as shown in Table 2-3 (Cruz, et al. 2017).

Table 2-3: Indirect tensile test result with UCS (Cruz, et al. 2017)



Specimen	ITS (MPa)	ITS/ UCS
A05C7_1	0.28	10.4%
A05C7_2	0.19	7.0%
A05C7_3	0.21	7.8%
A1C7_4	0.30	7.2%
A1C7_5	0.22	5.3%
A1C7_6	0.33	7.9%

3. METHODOLOGY

To achieve the objectives of this research, several laboratory tests were performed and are explained in the following section. Two types of soils, Soil A (medium plastic soil) and Soil B (high plastic soil), were collected and tested in in accordance with American Standard of Testing Materials (ASTM) and American Association of State Highway Transportation Officials (AASHTO). Class F fly ash, sodium hydroxide and sodium silicate were used for the stabilization of these two soils. Several mechanical and durability tests were conducted on soil, soil-cement and Soil-Geopolymer mixtures and details are presented in following sections.

3.1 Materials

Two types of soils were selected for the research, high plastic Soil And medium plastic soil. Commercially available Class F fly ash, sodium hydroxide, and sodium silicate was used in laboratory experiment.

3.1.1 Soils

As mentioned earlier, two types of soil were used. The Soil A was collected from the Lafayette, Louisiana, and Soil B was obtained from Accelerated Loading Facility (ALF) at Baton Rouge, Louisiana. These soils were collected from 1 foot beneath the ground surface in natural form. Then, these soils were transported to the Infrastructure and Materials Laboratory at UL Lafayette, where these were dried and crushed in powder form, using soil crusher, as shown in Figure 3-1. The summary of physical properties, classification of both soil is presented in Table 3-1. Based on AASHTO classification and ASTM standard, Soil A was classified as A-6 and Lean clay whereas Soil B was classified as A-7-6 and Elastic silt, respectively.

The elemental analysis and EDS (energy dispersive spectroscopy) data are shown in Table 3-2 and Figure 3-2, and Figure 3-3. Table 3-2 shows that Soil A consists of more silica than Soil B, but higher alumina content was found in Soil B than Soil A. In addition, Soil B consists of more iron than in Soil A.

Figure 3-1: The soils and fly ash used in the experiment

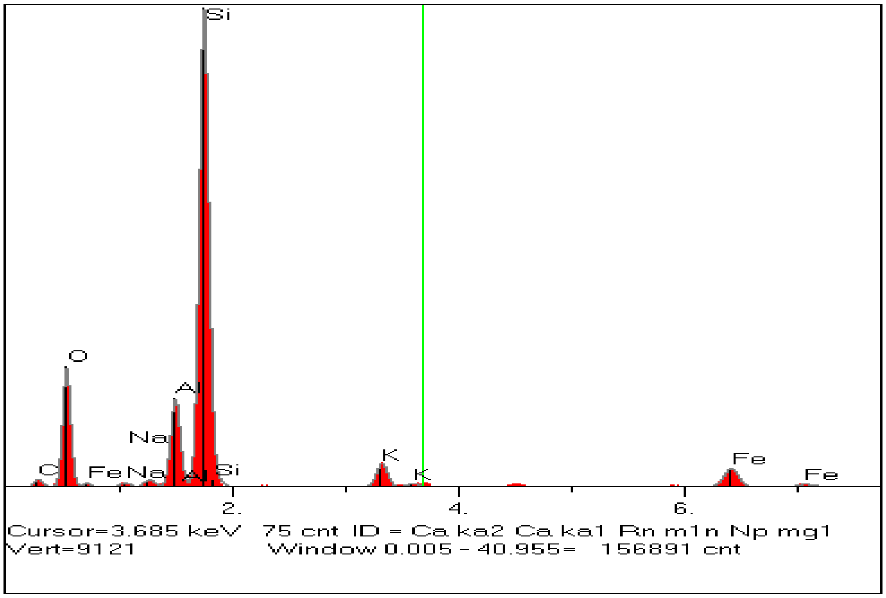


Figure 3-2: EDS Energy Spectrum for Soil A

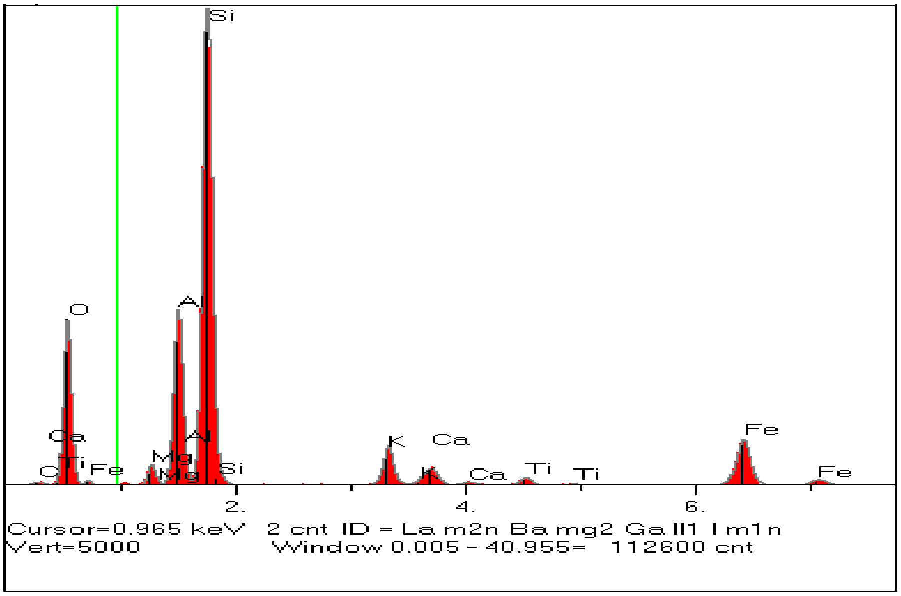


Figure 3-3: EDS data for Soil B

Table 3-1: Summary of properties of Soils

Description	**Soil A**	**Soil B**
Liquid Limit	39	94
Plastic Limit	23	33
Plasticity index	16	61
Color	Brown	Light black
% Passing sieve no. 200	>90%	> 90%
AASHTO	A-6	A-7-6
USC system	Lean clay	Elastic silt

Table 3-2: Composition analysis of Soil A, Soil B and Fly Ash (EDS)

Element	Concentration by % Weight		
	Soil A	Soil B	Fly ash
C	11.182	4.015	0.983
O	42.619	43.487	43.411
Na	0.549	-	-
Mg	0.69	1.358	1.135
Al	6.61	10.5	14.902
Si	33.488	30.747	30.319
K	1.946	2.405	1.116
Ca	-	1.207	5.171
Ti	-	0.611	-
Fe	2.916	5.67	2.963
Total	100%	100%	100%

3.1.2 Fly ash

The commercially available class F fly ash was used for this study, as shown in Figure 3-1. The composition of FA was determined using Energy-dispersive spectroscopy, as illustrated in Table 3-2 and Figure 3-5. High resolution images of FA were also taken using Scanning Electron Microscope (Hitachi s-3000), as shown in Figure 3-4. It can be seen from the SEM micrograph that the FA consists of various particle size of spherical shape.

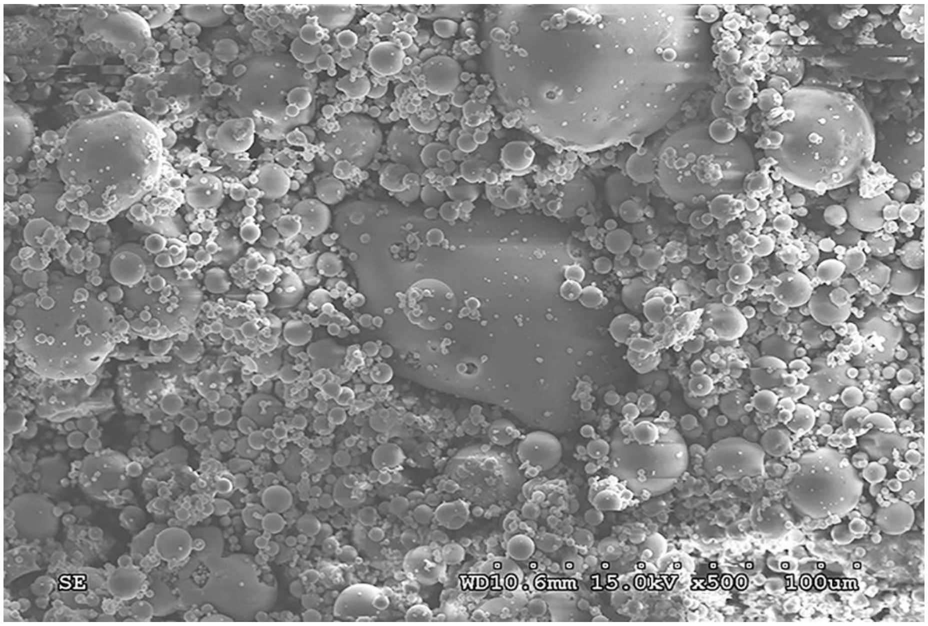
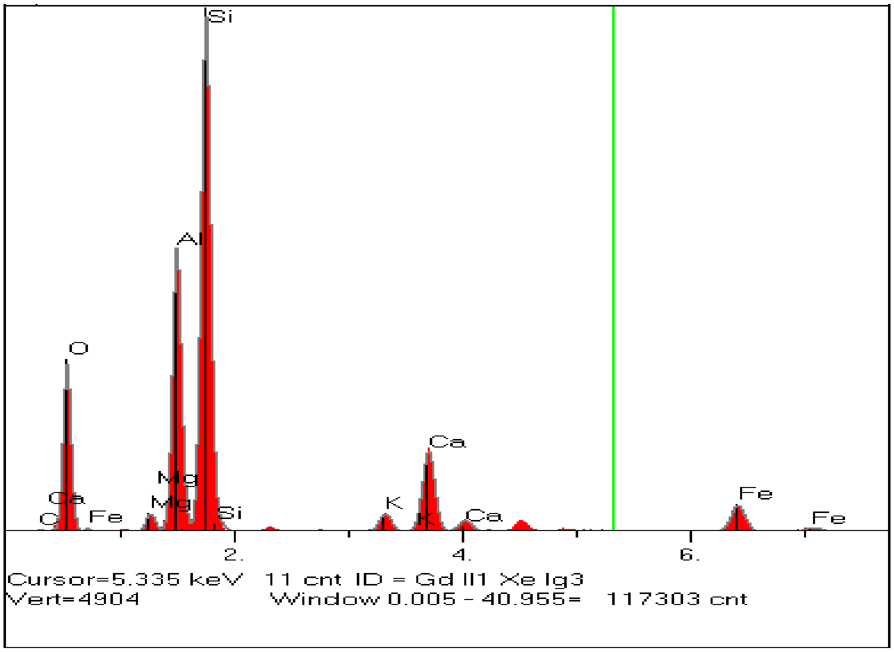


Figure 3-4: Scanning Electron Microscope(SEM) image at 500 magnification



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Cursor=5.335 keV   11 cnt  ID = Gd II1 Xe lg3
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Figure 3-5: EDS data for fly ash

3.1.3 Alkali activator

Commercially available sodium hydroxide (NaOH) in powdered form and sodium silicate ($Na_2Si_2O_3$) were used in this study. The powder NaOH was mixed with distilled water to get required concentration (Molarity) of NaOH for the mixtures. Forty (40) gram of NaOH was added in one (1) liter of distilled water to get one (1) molar solution of NaOH.

Five different molarities 8M,9M,10M,11M and 12M were prepared and mixed with $Na_2Si_2O_3$ solution to achieved required alkaline ratio. Alkaline ratio was prepared by mixing required amount of NaOH and $Na_2Si_2O_3$ and stirred for one minutes and kept in room temperature for another 5 minutes before mixing with fly ash-soil mixture.

3.2 Mixing and Compaction Procedure

3.2.1 Mold Preparation

According to ASTM and AASHTO test standards, UCS test should be conducted on cylindrical sample with the height to diameter ratio of 2 to 2.25. To meet this criterion, 2-inch diameter PVC pipe was selected. This plastic mold was used over metallic mold because plastic mold showed better resistance against alkali mixture. The plastic mold was prepared by cutting 2-inch diameter plastic pipe with 5-inch height instead of 4 in height, this extra inch was provided to facilitate easier compaction. Moreover, the longitudinal slit was provided to facilitate the extraction of the sample, as shown in Figure 3-6.

3.2.2 Mixing Procedure

The two-mixing procedures were used in this experiment, Dry mixing and Wet mixing. For Dry mixing procedure, first, various combination of FA, such as; 5%,10%, 15% 20%, 25% 30% were weighed based on total amount of dry soil. The alkaline solution was prepared by weighing and mixing appropriate amount of NaOH and $Na_2Si_2O_3$ based on corresponding alkaline ratio of the mixtures and concentration of the NaOH. The mixtures were thoroughly mixed by stirring and shaking for about 2 mins. Mixing process generates heat which must be dissipated to the environment otherwise it might create mixing problem, so the mixture of alkaline solution was left in room temperature for about 5 to 10 minutes. In wet mixing process, the mixtures of soil-FA were spread evenly in the tray and mixed with alkaline solution. Hand mixing procedure was used with proper safety and were mixed thoroughly for another 5 to 10 minutes.

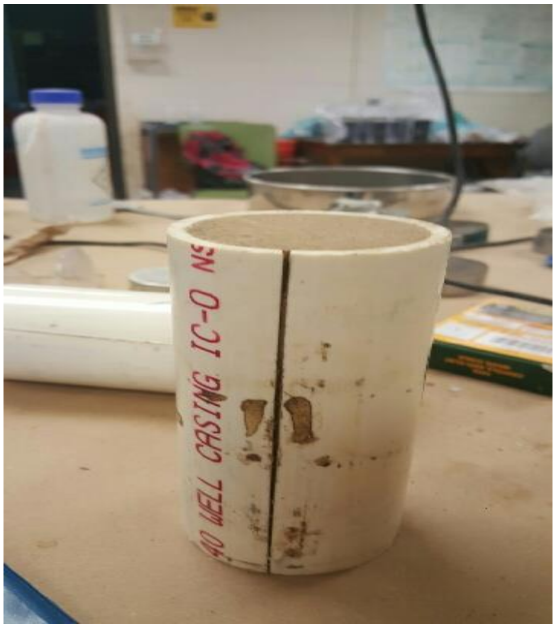

Figure 3-6: Plastic mold with vertical slit

3.2.3 Compaction procedure

The compaction of the soil was conducted on small wooden platform, with metal base. Before compaction, the mold was restrained by stainless steel hose clamp to prevent any volumetric expansion due to compaction and longitudinal slit, as shown in Figure 3-7. Three clamp were used for the restraining of the plastic mold.

Figure 3-7: Restrained of the mold using stainless hose clamp

The compaction was done in three different layers, 10lb hammer, with height of fall of 18 inches and 10 number of blow/layer, which is equivalent to energy per unit volume as specified in Modified Proctor test. The compaction energy was calculated using following equation (4) and comparison with proctor test is shown in Table 3-3. Three samples were prepared for each mixture.

$$Energy = \frac{No.of\ layer * No\ of\ Blow * wt.of\ Rammer * Ht.of fall}{total\ volume} \tag{4}$$

Table 3-3: Comparison of Modified Proctor Test with test procedures

Description	unit	Modified Proctor test	Modified compaction Equipment
Number of layers		5	3
No of blow		25	10
Weight of hammer	lb.	10	10
Height of fall	ft.	1.5	1.5
Total Volume	cu ft.	0.0333	0.0073
Energy	lb. ft./cu ft.	56250	61644

3.3 Heating and Curing of Mixtures

The compacted sample was taken along with mold was sealed with the tape from both sides of mold and as well as longitudinal slit to prevent any loss of moisture during heating, as shown in Figure 3-8. This sealed sample was again kept inside well sealed zip-lock bag and kept in the oven at 60 ℃ for 48 hours of heat curing. The sealing of the sample was assumed to simulate the confining field condition. After the heat curing in the oven the sample was removed and placed at room temperature for 24 hours before remolding and final testing.

The 7-day room curing process was conducted for soil-cement mixtures. The soil-cement mixtures were sealed in mold with tapes in both opening and longitudinal slit. Then specimens were sealed in zip lock bag and kept in room temperature for 7 days and tested in compression and repeated cyclic loading.

Figure 3-8: Sealed sample for before curing

3.4 Laboratory Experiment

3.4.1 Moisture Content

The water content (w) is defined as the ratio of weight of water in soil to weight of solid particles, expressed in percentage. ASTM D2216-10 test procedure was used to calculate the moisture content of any soil mixtures in the laboratory. Moisture can, weighing balance and oven are used in this test.

First, the weight of the moisture can (W_1) was measured and wet soil was placed in it. Again, weight of the Soil And can were measured (W_2) and placed in the oven for uniform temperature of about 110°C for 24 hours. After 24 hours, the samples were removed from oven and the dry weight was measured (W_3), then moisture content of the soil was calculated using the following equation,

$$Water\ content(w\%) = \frac{(\mathrm{W}_3 - \mathrm{W}_2)}{(\mathrm{W}_3 - \mathrm{W}_1)} * 100 \tag{5}$$

Where,

W_1= Weight of moisture can

W_2= Weight of wet Soil And moisture can

W_3 =Weight of dry Soil And moisture can

3.4.2 Atterberg Limit

Atterberg limit tests were conducted to classify the Soil And its consistency behavior. When water was added to the soil, consistency of the soil changes from hard to soft, and with continuous addition of water results in to liquid phase. The tests were conducted based on standard ASTM D-4318 and AASHTO T-89 test procedures.

3.4.2.1 Liquid Limit (LL)

Liquid limit is defined as the water content at which the soil changes from liquid state to plastic state and but has a small shearing strength against flow. In laboratory Casagrande apparatus was used to calculate the liquid limit of the both soils as shown in Figure 3-11. The liquid limit test was conducted by collecting about 250grams of soil sample which passes sieve no. 40. Then, soil was thoroughly mixed with the water. A portion of the soil was placed in the brass cup of the Casagrande device. It was then smoothened with spatula and a groove in the centerline was made using grooving tool. Number of blows was counted until the groove closes through the distance of half inch. Water content of the mixtures was between 15 to 35 number of blow. The test was repeated two more times to get water content corresponding to two number of blow. The water content was calculated by ASTM D 2216-10. The water content and number of blow was plotted in the semi-logarithmic graph. As per definition, liquid limit would be the water content corresponding to 25 number of blow.

The liquid limit of the soil was calculated with water content corresponding to 25 no. of blow from the Figure 3-9 and Figure 3-10 using corresponding equation for the two soils,

Liquid limit (Soil A)=-13.91*Ln(25)+83.408=38.6 (soil A say 39)

Liquid Limit (Soil B)= $-10.51 * Ln(25) + 128.33$ =94.5 (say 95)

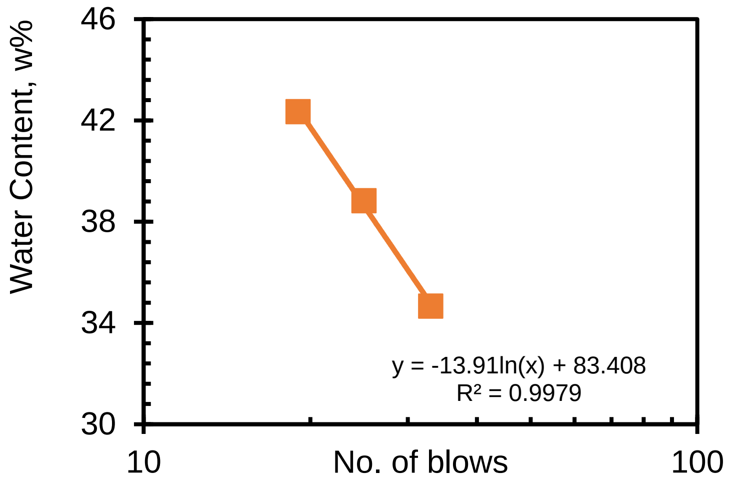

Figure 3-9: Liquid limit test (water content vs no. Of blow) for Soil A

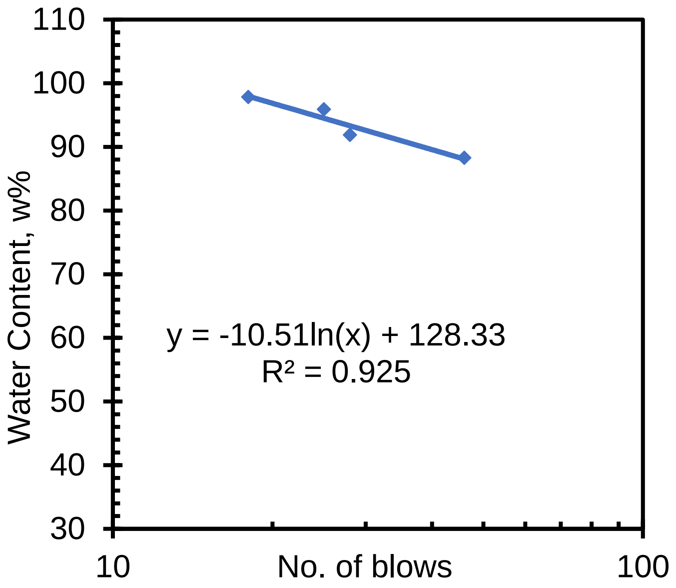

Figure 3-10: Liquid limit test (water content vs no. Of blow) for soil B

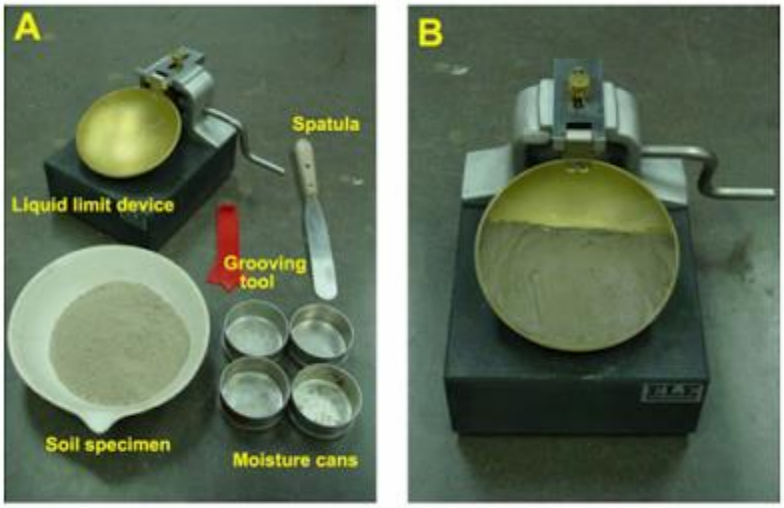

Figure 3-11: Casagrande apparatus for liquid limit of the soil

3.4.2.2 Plastic Limit (PL)

This test was conducted based on standard AASHTO T90 test procedures**.** First, 30 grams of the soil, passing sieve no. 40 was taken and thoroughly mixed with water. Mixed sample was then taken and rolled in the plate with palm of hand until it reached 5mm in diameter. Then the rolled piece was cut into several pieces. Further, the one of several pieces was rolled on the glass plate again until the thread crumbles into several pieces when it reaches 3 mm (1/8”) in diameter. The moisture content of the sample was calculated when sample just crumbled at 3mm diameter.

The plastic limit (PL) of the Soil A and soil B were found to be 23 and 33 respectively. The plasticity index (PI) of the soils were calculated by subtracting plastic limit by liquid limit of soils and found to be 16 and 62 for medium plastic Soil And high plastic soil respectively.

3.4.3 Moisture- Density relationship

The modified proctor test was used for determining the maximum dry density(MDD) and Optimum moisture content (OMC) of the soils. The maximum dry density can be achieved at optimum moisture content, and it is assumed that at this condition soil can easily be compacted and can achieve high density without any voids.

Modified Proctor test was conducted as per the AASHTO T180 and ASTM D 1557 standard procedures. As per the specification, the soils were compacted in five different layers, and each layer was compacted with 25 number of blow. The weight of the hammer was 10 pounds, height of the fall was 18 inches, and total volume of mold was 1/30 cubic feet as per standards.

Generally, clay has optimum moisture content of 11 to 15%. Three Percentage of initial moisture content was selected such that the three values were close to the range of OMC of typical clayey soil. The mixtures were then compacted in five different layer, and then the mold was trimmed to smoothen the surface. The weight of the soil in the mold was measured to get the density of the soil. In addition, corresponding moisture content (w) was calculated for that mixtures by oven dry method (ASTM D2216-10). The bulk density and dry density were determined using following equations,

$$Density\ (\rho) = \frac{weight\ of\ the\ soil\ sample(lb)}{Volume\ of\ the\ mold(\frac{1}{30}Cu\ ft)} * 62.4\ (psf) \tag{6}$$

$$Dry\ density(\rho_d) = \frac{\rho}{1+w} \tag{7}$$

The test was conducted on various combination of soil-fly ash mixtures for three different moisture contents to get three dry densities values. The nonlinear curve was plotted between moisture content as in X- axis and dry density as in Y- axis. Maximum dry density and optimum moisture content were determined using the nonlinear plot, as shown in Figure 3-12.

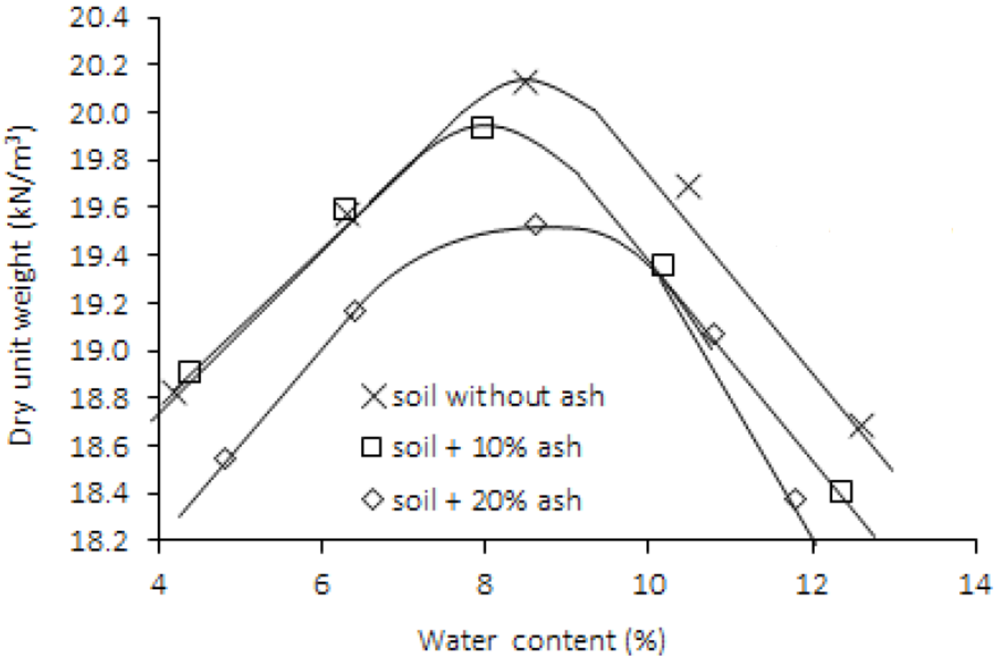

Figure 3-12: Typical Moisture-Density relationship curve (Cruz, et al. 2017)

3.4.4 Unconfined compressive Strength (UCS)

This test was conducted in accordance with ASTM D-2166 standard procedure. The unconfined compressive strength is a type of triaxial compression test, in which confining pressure is zero. The cylindrical sample of height to diameter ratio of 2 to 2.25 was loaded axially by a compressive force until failure took place. This test determines the shearing resistance of cohesive soil.

The UC test was conducted in the material testing system (MTS), as shown in Figure 3-13. This testing equipment was connected to computer based data acquisition system. When test was finished, it records the load, stroke and strain for assigned interval of time. An extensometer was connected to measure the strain in a sample’s middle third portion as shown in Figure 3-13**.**

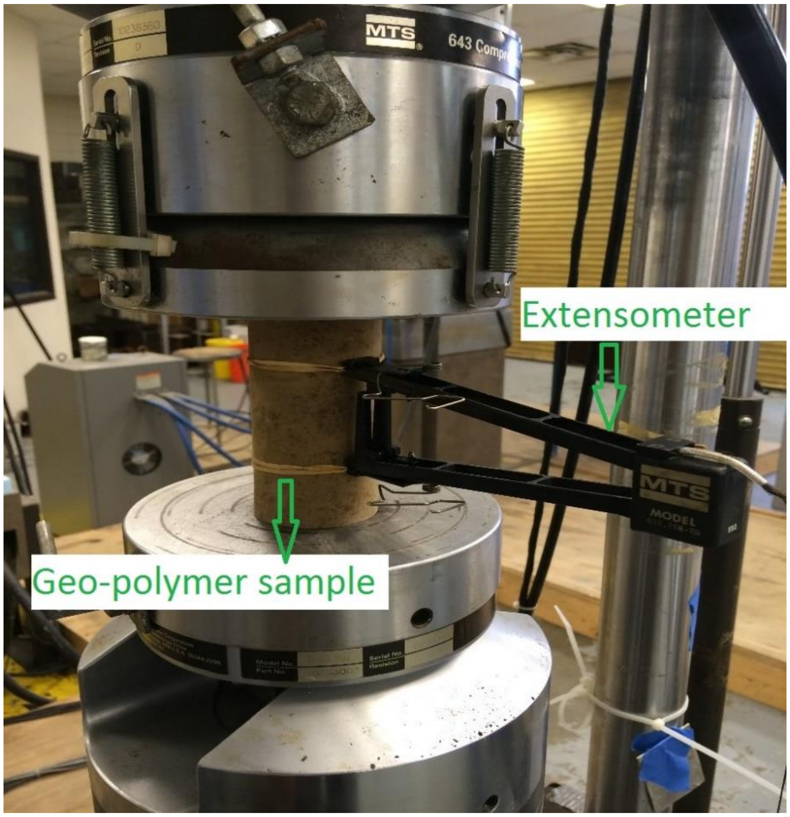

Figure 3-13: MTS testing configuration

In this test, the sample was placed in the lower plate, and an extensometer was connected and attached with sample with the help of rubber band (see Figure 3-13). Then, the small load was applied in the sample so that sample remains fixed between the upper and lower plate. When the sample was placed and ready for testing, the pin of the extensometer was removed, and the strain gauge reading was adjusted to zero.

The test was started following the initial conditioning of the sample. The load was applied at high rate up to 30 pounds, following 1 second of rest period. Then, load was applied at constantly moving lower loading plate with a rate of 0.02 inch/minute until failure. The data was recorded at every 0. 25 second. The shearing failure mode was observed, as shown in Figure 3-14 and stress-strain diagram during failure is shown in Figure 3-15. In this figure, young’s modulus of elasticity was also calculated. Elastic modulus or Young’s modulus is the slope of stress-strain curve. Similarly, fracture energy was calculated from the stress- strain curve. It is the energy associated with the fracture of the sample, and it is equal to the area under stress-strain curve up to the maximum stress level. For a typical soil-cement sample of 10% cement content, Young's modulus was found to be 344,430 psi, as shown in Figure 3-15.

Figure 3-14: Typical shear failure of the specimen in compression

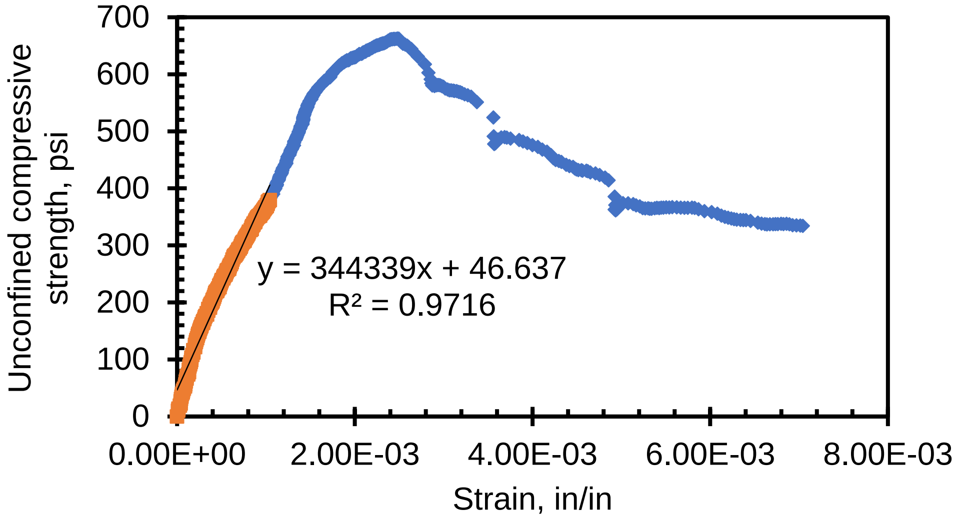

Figure 3-15: Typical stress-strain curve for 10% cement with Soil A along with Young Modulus of elasticity

3.4.5 Resilient Modulus Test

The resilient modulus is the most important parameter in the designing the road pavement. Resilient Modulus defines the elastic behavior of the materials under various loading conditions. Basically, Mr was measured under repeated loading (Haversine wave), which simulates the actual field condition of truck traffic. The AASHTOO 1993 and Mechanistic Empirical Pavement Design Guidelines (MEPDG), both recommended using Mr for analysis and design of the pavements. Figure 3-16, shows the representation of the cycle of repeated loading and unloading on the sample.

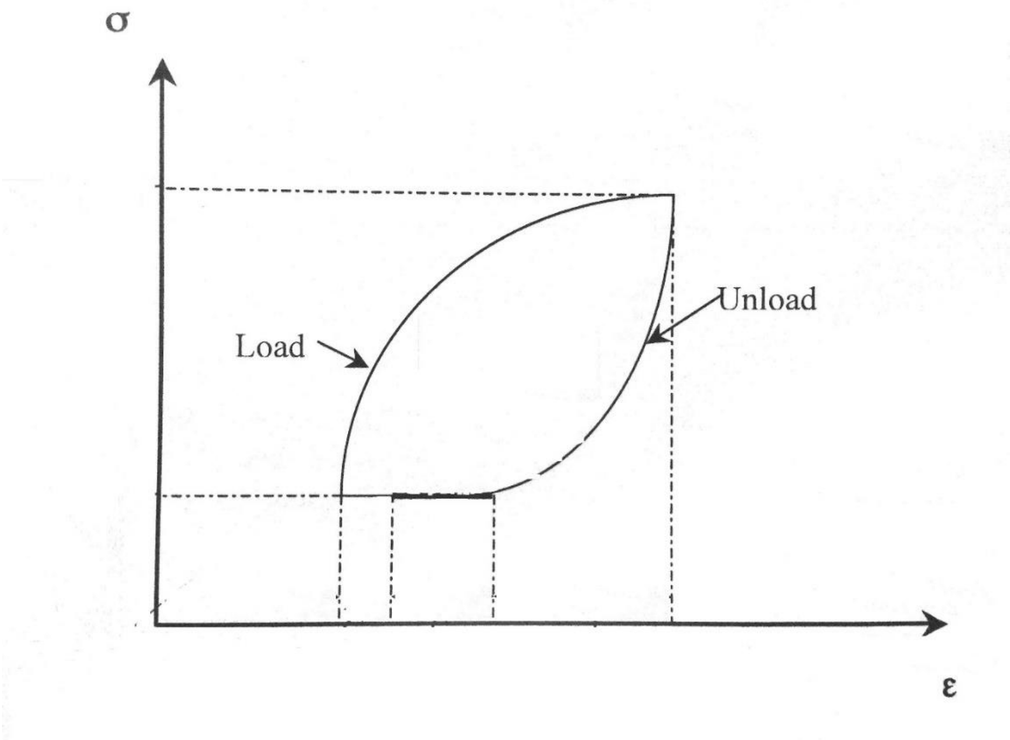

Figure 3-16: Typical loading and unloading of curve of the mixture under repeated loading

This test was conducted under compressive loading in the laboratory. It was conducted by applying cyclic stress of 10% of UCS. Haversine load wave form was used with 0.1 sec of loading-unloading and 0.4 sec of rest period as illustrated in Figure 25. The data was collected at 1,10, 20, 50, 100, 200, 500, 800 and 1000 cycles. The Mr was maximum cyclic stress to resilient strain. The average stress is obtained from the amplitude of the cyclic stress vs time plot as shown in Figure 3-17. Similarly, average amplitude of strain was obtained from variation of cyclic strain against time as shown in Figure 3-18. Resilience Modulus was calculated for each cycle using following equation (8). Note this technique is only valid if there is no viscoelastic strain.

$$Resilience\ Modulus(M_R) = \frac{amplitude\ of\ Cyclic\ stress}{amplitude\ strain} \tag{8}$$

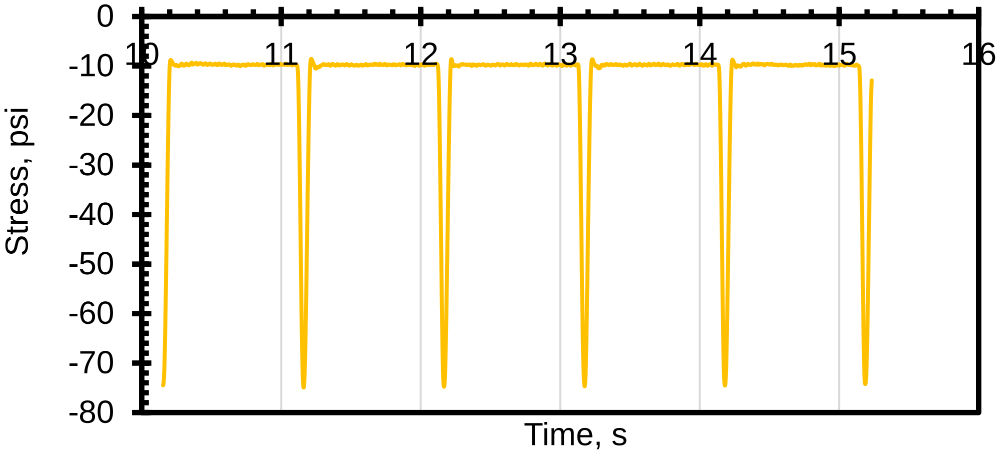

Figure 3-17: Typical time vs load with cyclic stress for 25%FA0.4AR6M Soil A mixture at 10th no. of cycle

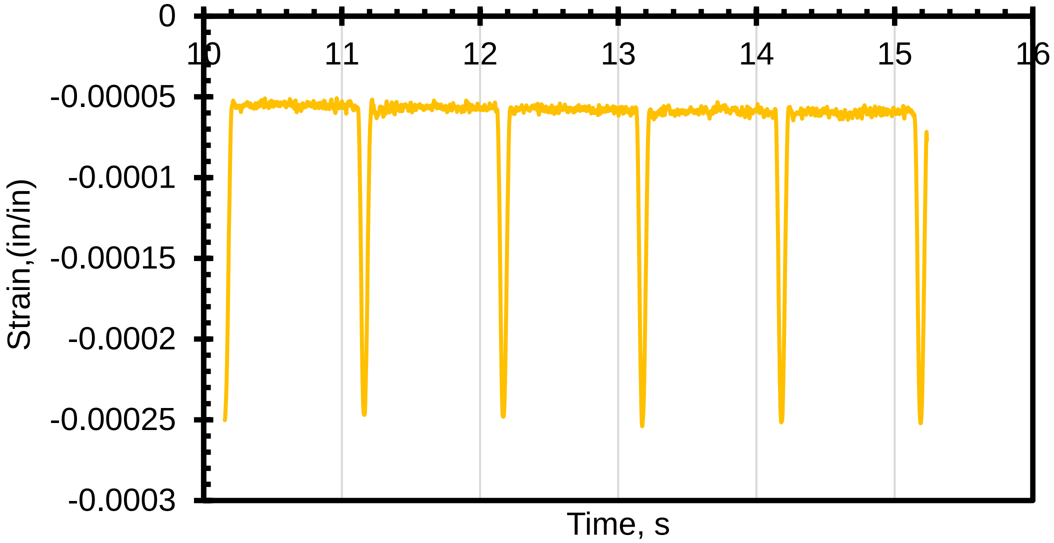

Figure 3-18: Typical time vs strain for 25%FA0.4AR6M Soil A mixture at 10th no. of cycle

3.4.6 Dynamic Modulus Test

Dynamic Modulus (E*) was basically ratio of cyclic stress to cyclic strain under dynamic loading. The test was conducted with modified ASTM C1548 test. In this test, same sample, as unconfined compressive strength sample, was used. Sinusoidal loading wave form was used and the test was conducted at various frequencies of 1, 5, 10 and 25 Hz with cyclic of 20 to 30% of UCS of that mixtures. The amplitude cyclic stress and cyclic strain were used to determine the E*of the soil mixtures. Figure 3-19 and Figure 3-20, shows variation of the typical strain and stress behavior, respectively, as a function of time under dynamic loading at frequency of 25 hz. The following equation was used,

$$Dynamic\ Modulus(E^*) = \frac{Cyclic\ stress\ for\ each\ cycle}{amplitude\ strainfor\ each\ cycle} \tag{9}$$

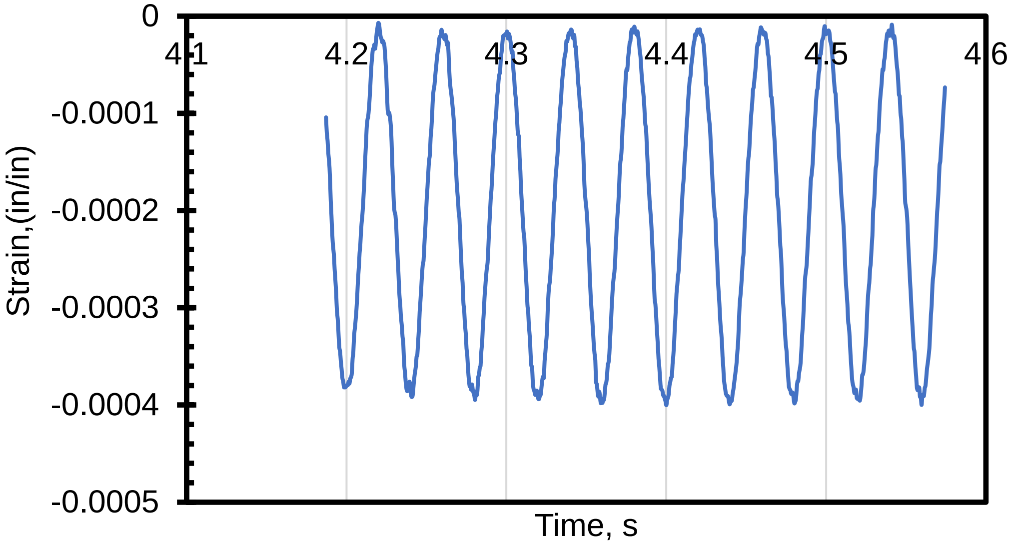

Figure 3-19: Typical time Vs strain with cyclic load for 25%FA0.4AR6M Soil-A Mixture at frequency of 25 Hz

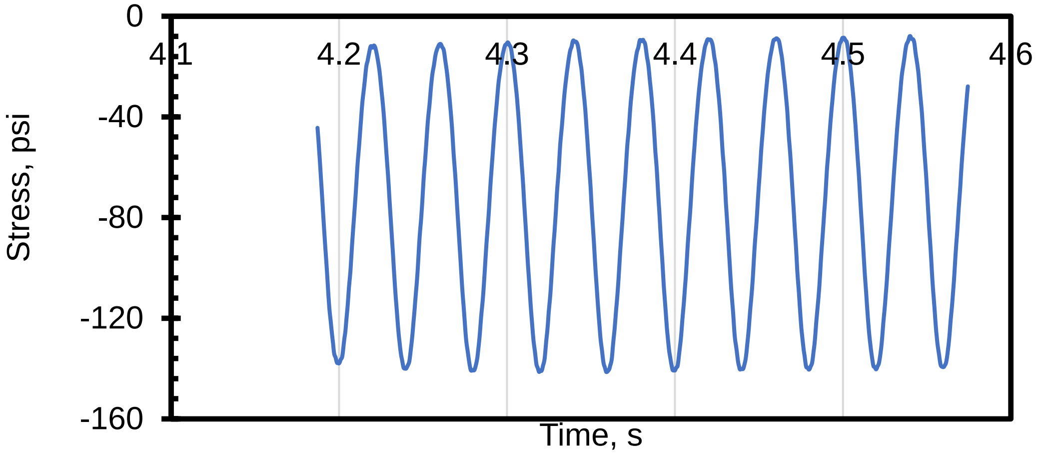

Figure 3-20: Typical time vs stress with dynamic cyclic load for 25%FA0.4AR6M Soil A mixture at frequency of 25 Hz

3.4.7 Durability test

Durability of the base and subbase is the major concern during construction and long-term performances of the pavement structure, especially for the chemically stabilized subgrade and base materials. It defines the ability of the pavement to withstand severe environmental condition during service life of the pavement. Various test like freeze and thaw, wet and dry, California bearing ratio test and tube suction test are used in Durability test for pavements structure. For this research, wet and dry durability test was conducted based on ASTM 559 standard procedure.

The sample was prepared based on ASTM559, which uses same mold as modified proctor test. Then, the sample was compacted in five layers and extracted from the mold using hydraulic Jack. Two sample were prepared for each mixture of soil-cement and soil FA Geopolymer named as sample A (weight loss sample) and sample B (volume change sample). The soil-cement samples were cured in room temperature for 7 days, whereas alkali activated samples were cured in an oven at 60 ℃ for 48 hours, following one day of rest period. After curing, samples were submerged in water at room temperature for 5 hours, and then removed from water. The weight of the sample was measured and recorded for weight loss sample, and dimension of the volume change sample were measured and recorded. Then, samples were placed in an oven for 42 hours at 71℃. The samples were then removed from the oven, and weight and dimension of the sample were measured and recorded again. In this test one cycle was completed in 48 hours.

For weight loss specimen, all the sides of the specimen were scratched with two firm strokes as specified by ASTM 559. The long axis of the brush was held in parallel to the brush to longitudinal axis of the specimen during scratching to cover all area of the sample.

Then, eighteen to twenty strokes with 3-lbf load were applied in all sides of the sample, and four strokes were applied on each end. Further, the weight of the sample was taken again to get the weight loss. The dimension and weight of volume change samples (sample B) were measured and recorded after removing from oven. The dimension was measured three times and averaged to calculate the volume and volume change of the specimen. Above mentioned procedures (submersing, heating, measuring weight and dimensions) were repeated for 12 cycle for both samples.

The submerging of in the water and heating in oven were continue for 12 cycles, sample B (volume change sample) was heated to obtain dry mass, which can be checked for change in overall water content, whereas for specimen A (weight loss specimen) the soil-cement (or, Soil-Geopolymer) loss was calculated using equation (10),

$$Soil\ Cement\ loss, = \frac{Original\ mass - final\ dry\ mass}{original\ dry\ mass} * 100\% \qquad (10)$$

3.5 Design of Experiment

The experimentation was conducted in two phases. During the first phase, physical, consistency, and mechanical characteristics were determined for soil, soil-fly ash and Soil-Geopolymer mixtures, an optimum mixture design was obtained based on UCS of Soil-Geopolymer mixtures. In the second phase Resilience modulus test, Dynamic Modulus test and Durability test were conducted on optimized Soil-Geopolymer mixture and compared with the commonly used soil-cement mixtures.

In order to obtain generality and precision for the developed experimental database, the experiments needed to generate the required data must be conducted in a systematic and organized manner. A three-parameter surface methodology (RSM) with central experimental design is adopted. The design involves a fraction of first-order (2^n) factorial design, two “star points” on the axis of each design variable, and one center point. The parameters were explored at five levels that covered the wide range of variables used in the study. The levels are represented by the commonly used codes (-1, +1, 0) on the factorial portion of the design, and values (-2, +2) on the axial portion.

Table 3-4 displays the parameters, their corresponding levels and values. The variables in the design matrix include: fly ash content, alkaline ratio ($Na_2Si_2O_3$/NaOH) (AR), and NaOH molarity (M). In the table, the variables are set at level of 0 and, levels were increased and decreased for all types of variables. For Soil A, 15% fly ash content, 1 Alkaline ratio and 10 Molarity of NaOH were set at level 0. For increase or decrease in each level variables were increased and decreased correspondingly. For each level of increase or decrease (+1, -1), fly ash of 5%(20%,10%), AR of 0.5 (0.5, 1.5) and molarity of 1(11 M, 9 M) were increased or decreased respectively. However, for Soil B 20% fly ash,0.5AR and 10 M were sets as level 0. For each increase or decrease in level (+1, -1), fly ash of 5% (25%, 15%) and alkaline ratio by 0.25 (0.75, 0.25) were increased or decreased respectively.

Table 3-4: Generalized level for experiment design matrix

Levels	FA, %	Alkaline Ratio, AR	NaOH Molarity
-2	5	0	8
-1	10	0.5	9
0	15	1	10
1	20	1.5	11
2	25	2	12

In the first phase, physical properties of soils such as; color, grain size and classification of the soil were determined. In addition, Atterberg limit tests were conducted and determined the moisture density relationship. Experimental design matrix was generated for compressive strength so that the less sample can have maximum variation in the mixtures. Two mixtures were selected, one from each soil based on minimum compressive strength criteria required by DOTD, Louisiana and maximum compressive strength from the regression and sensitivity analysis. In second phase, the selected two mixtures from Soil A and soil B along with recommended soil-cement mixtures were tested for resilience modulus, dynamic modulus and durability tests. The results of the tests for the soils-FA Geopolymer were compared with soil-cement mixtures.

The detail experimental design matrix for the Soil A is shown in Table 3-5. The fly ash content was varied from 5 % to 25 %, alkaline ratio from 0 to 2 and molarity of NaOH from 8 to 12. These mixtures were denoted as % of FA, AR and M. For example, in 15FA1AR10M mixture, 15FA represented as 15 percentage fly ash by weight of dry soil, 1 AR was alkaline ratio of 1 and 10 M implied 10 molarity solution of NaOH. In this matrix, Table 3-5, amount of alkali was function of fly ash. For each mixture, total alkali content was half the percentage of fly ash and remaining additional water to reach 13% liquid of total weight of solids (FA and soil). For example, for 15FA1AR10M sample, 7.5% was total alkali and 50% sodium silicate and 50% sodium hydroxide of total alkali. For 100 unit of soil 15 units of Fly ash was needed, 3.75 unit of sodium hydroxide of 10 molarity and 3.75 unit of sodium silicate and 7.45 unit of additional water were required.

This matrix was later modified for regression analysis since additional water in the soil FA mixture was reducing the molarity of NaOH and effect the strength characteristics of mixtures. Therefore, it was necessary to calculate new reduced molarity along with new alkaline ratio, shown in Table 3-5. In these mixes, molarity of the mixes varies from 1 molar to 8.45 molar and alkaline ratio varied from 0 to 1.05. A new sample (Sample 19-

15FA0.335AR9M) was also prepared to validate the fact that additional water was effecting the strength of the mixtures. These new molarity and alkaline ratio were used to generate regression model which explained the relationship among these variables. Regression analysis and sensitivity analysis were conducted to find optimum mixture.

Table 3-5: Detailed experimental design matrix for Soil A

				Modified	
Mixture Number	FA, %	AR	M	Molarity of NAOH	AR
1	5	1	10	1.05	0.1
2	15	1	10	3.87	0.33
3	25	1	10	8.33	0.63
4	15	0	10	6.28	0
5	15	2	10	2.79	0.5
6	15	1	8	3	0.33
7	15	1	12	4.79	0.33
8	10	0.5	9	2.62	0.13
9	20	1.5	9	4.41	0.63
10	10	1.5	11	2.11	0.27
11	20	0.5	11	7.86	0.27
12	5	0	8	1.56	0
13	25	2	12	8.45	1.05

Table 3-6, illustrates the experimental design matrix for the soil B, in which the fly ash content was varied from 10% to 30 % with increment of 5%, alkaline ratio was varied from 0 to 1 and molarity was varied from 8 to 12 molars. This matrix was established based on modified mix design of Soil A. Hence, no additional water was added in soil-fly ash Geopolymer mixtures. During the preliminary experiment soil B- FA mixtures showed very different behavior with addition of water. When water was added in the soil-FA mixture, it formed small spherical patches entrapping the water inside it. Even at OMC high plastic soil showed difficulties in workability behavior of mixture. As we know soils shows parallel orientation and has shown high workability property at wet-side of OMC. For high plastic soils 15 % OMC was selected for preparing the Geopolymer mixtures. Even at 15 % liquid content (mixture of sodium hydroxide and sodium silicate), mixtures showed deficiency of the water for workability. However, alkaline solution contents some number of solid particles, which might be responsible for deficiency of water in the mixture. This deficiency was fulfilled by adding more alkaline solution so that total water content in the mixture remains 15%. The total alkaline contents were calculated for midpoint of mixture proportion at 20%FA0.5AR8M. Sodium silicate consist of 37% solid by weight and 10 molar sodium hydroxides contains 32% solid, and in average alkaline solution contains 33.5 % solids. To prepare overall water of 15% by weight of total mixture, 20% alkaline solution was used.

Table 3-6: Detailed experimental design matrix for Soil B

Mixture number	FA, %	AR	Concentration of NaOH
1	10	0.5	10
2	20	0.5	10
3	30	0.5	10
4	20	0	10
5	20	1	10
6	20	0.5	8
7	20	0.5	8
8	10	0	9
9	30	1	12

4. RESULT AND ANALYSIS

4.1 Moisture- density relationships

Moisture-density relationship of the various dosages of fly ash of Soil A and Soil B are presented in Figure 4-1 and Figure 4-2, respectively. From Figure 4-1, it can be observed that with the addition of fly ash, the dry density of the Soil A was decreased. With the addition of 5% fly ash, the maximum dry density (MDD) decreased to 104 pcf, which is 10% less than the MDD of the soil. Similarly, the slight decreased in OMC was observed with the addition of 5% FA, as shown in Table 4-1.However, at 15% FA content MDD and OMC were increased to 114.3 pcf and 13% respectively. This increase in MDD was significant and comparable with the MDD of soil but OMC was almost the same to that of the soil, as shown in Table 4-1. With further increasing FA content of 25%, MDD decreased to 14% with respect to 15% FA content. Similarly, OMC decreased slightly at 25% FA content. This result was different than found in literature as there were no significant changes in OMC values in this study. On other hand, MDD of the soil-fly ash mixtures decreased, which was similar to the finding of Vukićević et al. (Vukićević M . 2013)**.**

Interestingly, the moisture-density relationship for Soil B, with and without FA modification revealed that there are no significant increases in MDD with addition of FA up to 25%, and OMC stayed the same, as shown in Table 4-2. In addition, the peak of the dry density curve for all three mixes exhibit a flat curvature, indicating ranges of OMC, from 12 to 15%, as shown Figure 4-2.

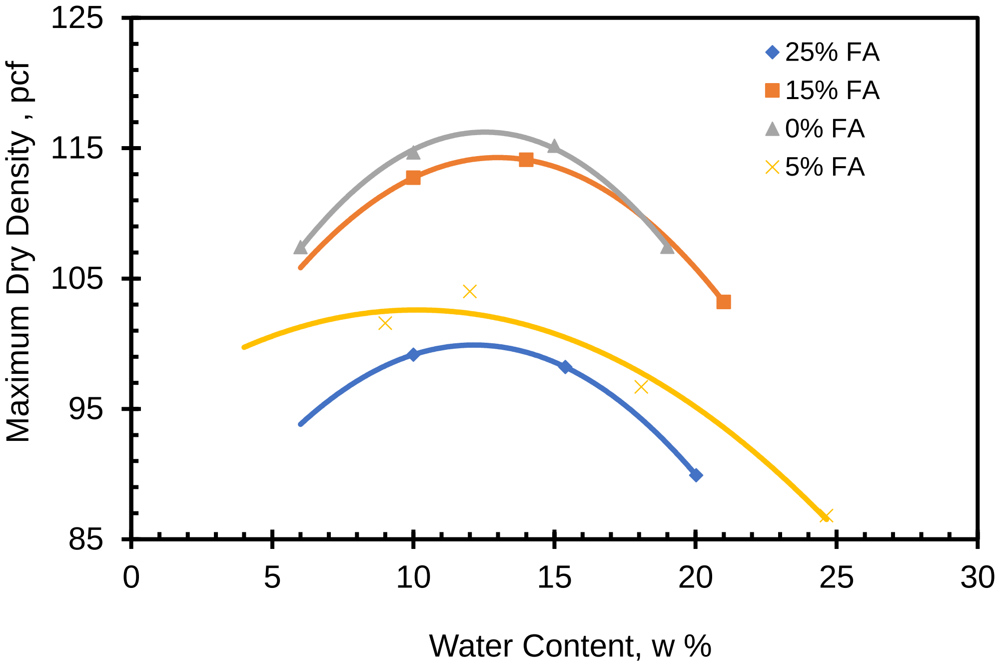

Figure 4-1: Summary of Moisture-density relationship for Soil A-FA mixtures

Table 4-1: Summary Moisture- Density relationship for Soil A -FA mixtures

Mix	OMC	MDD	Variation with soil
	%	PCF	
0 FA	12.5	116.2	
5% FA	12.3	104.0	-10%
15% FA	13.0	114.3	-2%
25% FA	12.1	100.0	-14%

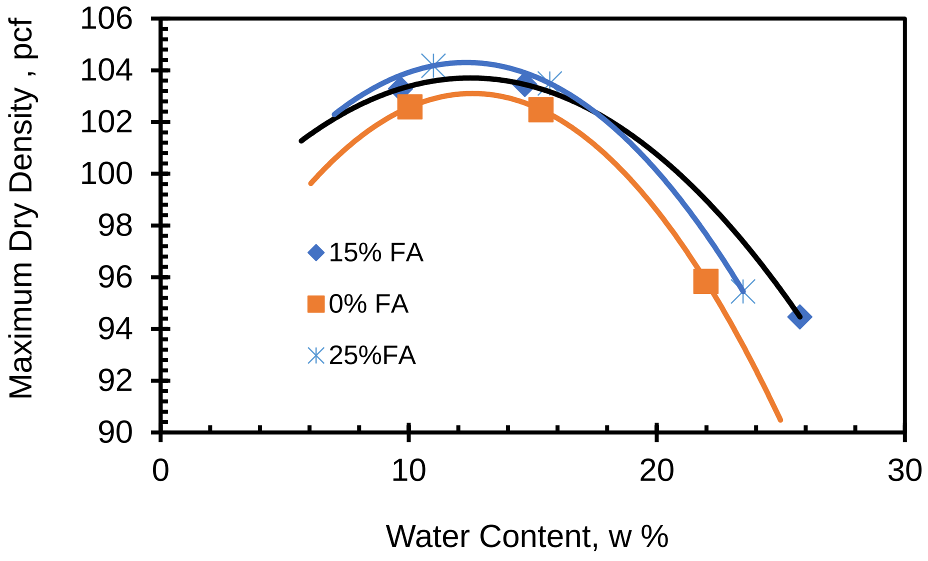

Figure 4-2: Summary of Moisture-density relationship for Soil B-FA mixtures

Table 4-2: Summary of Moisture-Density relationship for Soil B-FA mixtures

SN	Mixture	MDD	OMC	Density	15% water	Diff. with 15%
		PCF	%	PCF	%	%
1	Soil only	103.1	12.6	102.6	15	0.47%
2	15% FA	103.7	12.5	103.4	15	0.32%
3	25% FA	104.3	12.3	103.8	15	0.49%

From the observation, it could be concluded that the MDD and OMC of the soil-FA mixtures largely depend on types of the soils. In the case of Soil A, a medium plastic soil, the addition of FA decreased the density of the mixtures. However, for Soil B, a high plastic soil, MDD increased with the addition of FA content up to 25%. Such increase in dry density might be related to the fineness of the FA particles and plastic nature of the soils. Especially in high plastic soil (Soil B) with the addition of water, it immediately forms small spherical patches, trapped the water inside it, hindering the mixing and compaction procedures, even at OMC level. The compaction curve shows OMC of the soils ranges in between 12% to 15 %.

As we know, soil particles are randomly oriented on the dry side or lower than OMC value however in the wet-side the soil particle are oriented in parallel. In parallel orientation, the extra water forms water film surrounding the soil particles, enhancing workability and contact between the soil particles. Basically, at wet-side of the OMC, soil particles were arranged in parallel direction creating more contact surfaces particles, resulting in easy mixing, compaction and better reaction between soil-FA mixtures. However, the difference between dry density at optimum moisture content and 15% water contents were not significant (less than 0.5%) as shown in Table 4-2. On basis of workability of the soils 15% OMC was chosen for future sample mixing, compaction, and testing.

4.2 Unconfined Compressive Strength (UCS)

4.2.1 Effect of Fly Ash on Unconfined Compressive Strength

The effect of the fly ash on UCS of soil- Geopolymer is shown in Figure 4-3, and Figure 4-4. Figure 4-3 depicts the increase in the UCS of the mixtures with increased FA content. The similar result reported by Kang et al. (Kang, et al. 2014)**.** The UCS of the soil with 5% FA, was found to be 150 psi. Further, with increasing FA content the strength of the mixtures increased up to 378 psi for 25 % FA. This shows the linear relationship between UCS and fly ash content of Soil-Geopolymer for 10M and 1 alkaline ratio.

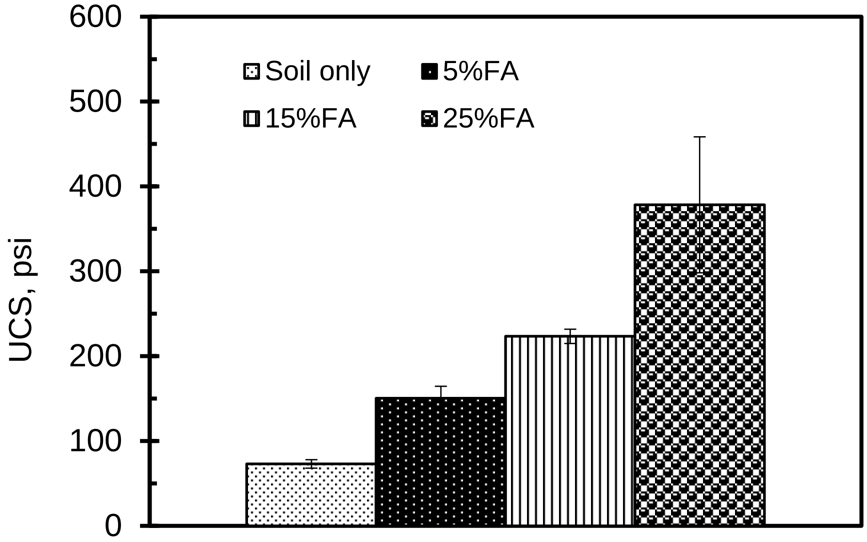

Figure 4-3: Effect of FA on Soil A for 1AR and 10M solution of sodium hydroxide

Similarly, in the case of Soil B, with the addition of FA content, the strength was not affected up to 20 % and is found to be less than the strength of strength of soil. However, at 30% FA, the UCS was rapidly increased by 400 %, as shown in Figure 4-3. This may be associated with plasticity and composition of the soils.

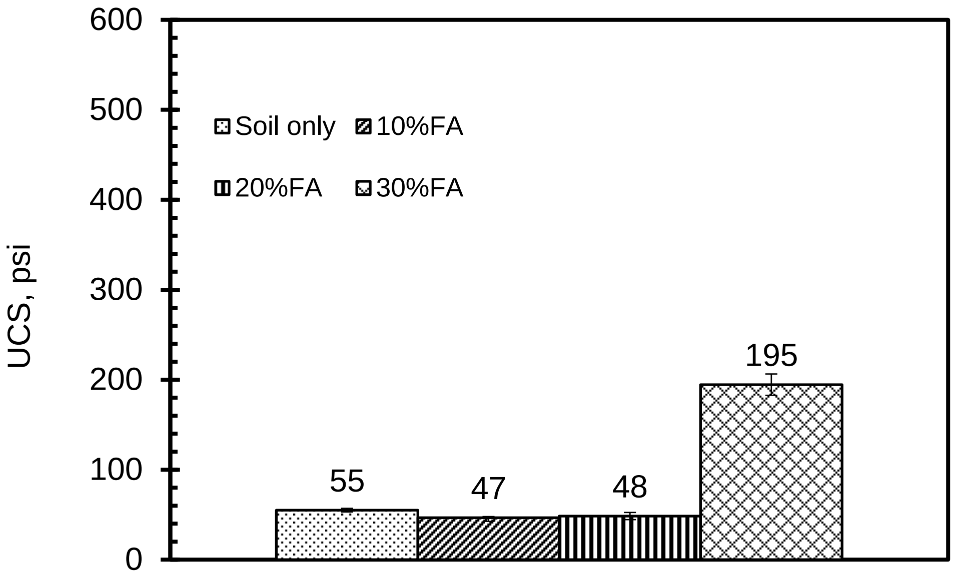

Figure 4-4: Effect of FA on Soil B for alkaline ratio of 0.5AR and 10 M solution of sodium hydroxide

As mention earlier, the Soil B has higher plasticity index and was difficult to compact at OMC because of deficiency of free water to react with mixtures. The soil formed small spherical patches with trapped water inside. Therefore, decreased the free available water needed for mixing and to enhance the Geopolymer reaction. Moreover, Soil B has more Iron content and potassium content than Soil A (Table 3-2), this might be associated with decreased in the strength of FA content up to 20%. However, with the addition of FA, the concentration of iron content and other chemical decreased in the mixture and somehow critical concentration reached so that these chemicals were no more effecting the compaction and strength of the mixture. Hence, a sudden and rapid increase in strength was achieved at FA of 30% as compared with 20% FA.

4.2.2 Effect of Alkaline Ratio with UCS

The effect of alkaline ratio with UCS of the Soil-Geopolymer is illustrated in Figure 4-5, for 15% FA and 10M. The data in the Figure 4-5 reveals, that the UCS of the mixture decreased with increase in alkaline ratio. This result contradicts the results observed by Bignozzi et al. and Sukmak et al., they found that with an increase in the alkaline ratio strength increased up to an optimum value (Bignozzi, et al. 2014, Patimapon Sukmak 2013). This can be explained as, the sodium silicate was reducing the strength of the mixture, implying that strength was mainly due to the sodium hydroxide solution. This discrepancy can only be explained by the chemical composition of the Soil and Fly ash. The modified molarity of NaOH decreases from 6.3 molars to 2.8 molars. Also, the modified AR was recorded from 0 to 0.5, as shown in Table 4-3. On the other hand, the UCS of the Soil-Geopolymer mixtures initially decreased with increasing alkaline ratio for high plastic soil (Soil B), reached a minimum value at 0.5 alkaline ratio, and then started to increase as presented in Figure 4-6. The UCS of Soil-Geopolymer decreased with the addition of sodium silicate indicating similarity in results however one soil was high plastic and another soil was medium plastic. This could further be explained by available free water for mixing and compaction, reaction and composition of soils.

In order to ensure to have same moisture content of 15% (OMC), the mixing and preparation of the sample were conducted at 20 % Alkaline contents. For 0 alkaline ratio of 10 molars about 71.4% of water was present in the alkaline mixture. This decreased the overall water in the mixtures to 14.3%.

However, for 1 alkaline ratio, overall water available for mixing and compaction process is found to be 13.4% of the total mixture by weight. This decreased in moisture content decreased the mixing and processing of the specimen, hindering the reaction between mixtures, resulting in the decrease in UCS with increasing alkaline ratio. In addition, the silica content of the soils and FA might be responsible for the Geopolymer reaction. The literature review suggested that excessive sodium silicate also causes hindrance in the reaction, because of generation of excessive silica in the mixtures. Low plastic soils consist of the same amount of silica as compared with FA, there might be excessive silica so that reaction might have been adverse.

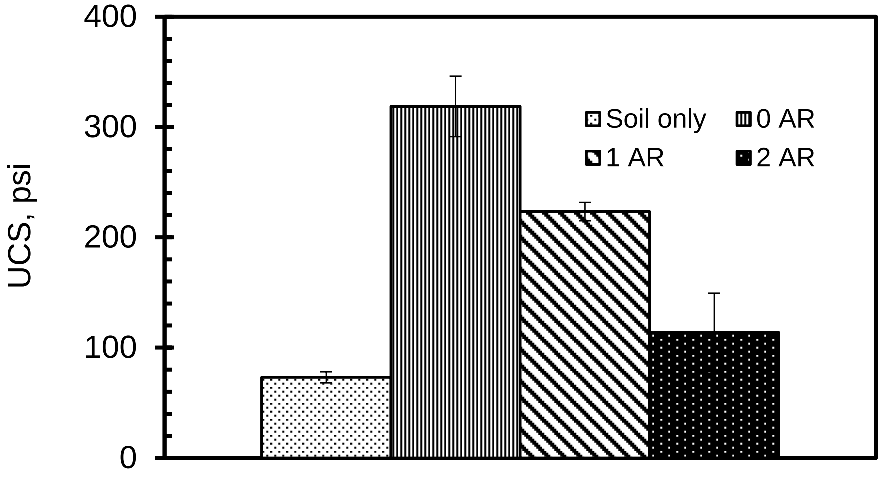

Figure 4-5: Effect of alkaline ratio on Soil A for 15% FA and 10M of sodium hydroxide solution

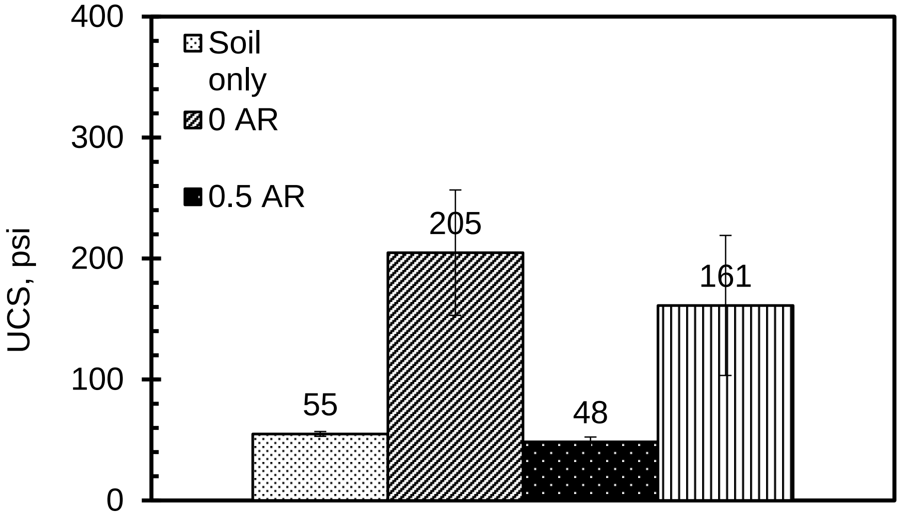

Figure 4-6: Effect of alkaline ratio in Soil B for 20%FA and 10M sodium hydroxide

4.2.3 Effect of concentration of NAOH in UCS

The effect of the molarity with UCS of Soil-Geopolymer for Soil A is shown in Figure 4-7, in which the effect was not significant for 8, 10 and 12 molar solution. It must be noted that the additional water in the mixture reduced the overall molarity of mixture to around 4 M. Therefore, it was difficult to observe the effect of molarity on strength of the mixtures. But in the case of the Soil B, even at 8 to 12 molar concentration of sodium hydroxide the strength was not affected, as shown in Figure 4-8. This decreased in strength was not only depended on molarity of the mix but also on the alkaline ratio and FA contents. For high plastic soils at 20% FA and 0.5 alkaline ratio strength were very low. Hence, it is difficult to conclude molarity was not affecting the strength of Soil-Geopolymer mixture.

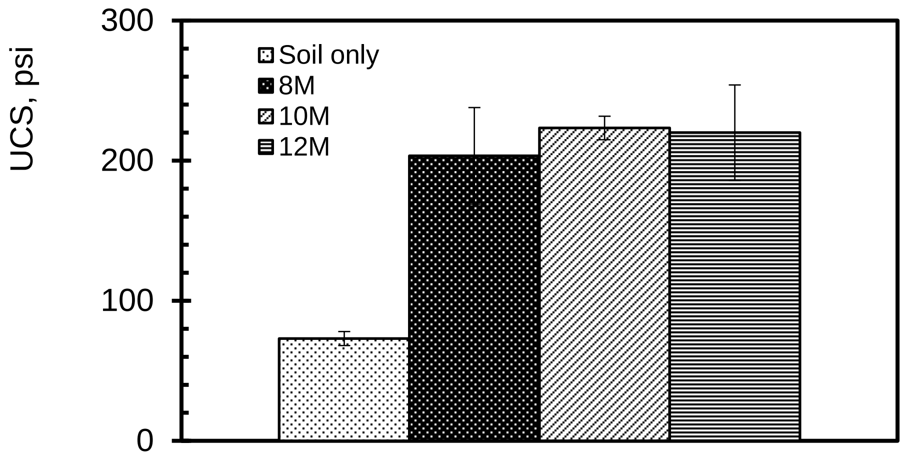

Figure 4-7: Effect of concentration of sodium hydroxide (molarity) for Soil A for 15% FA and 1 AR

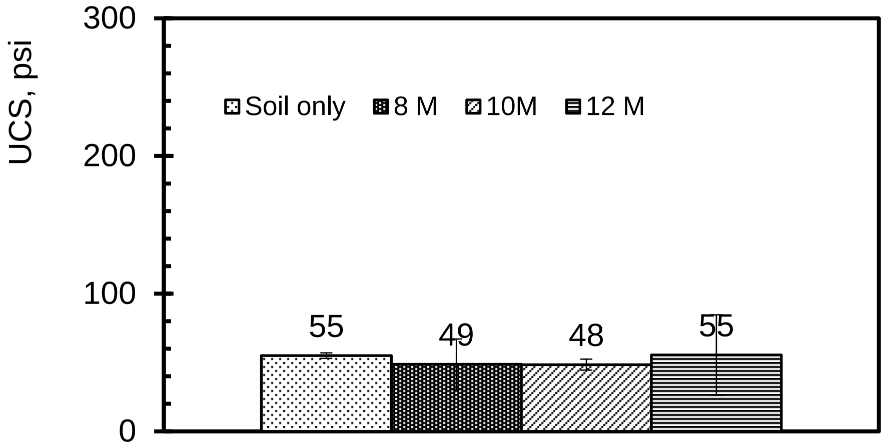

Figure 4-8: Effect of concentration of sodium hydroxide (molarity) for soil B for 20% FA and 0.5 AR

The UCS of Soil B at 20%FA and 0.5 AR were very low as compared to 0AR. As explained above that the overall water available for 0AR is higher than 1 AR in the Soil-Geopolymer mixture, which aided in adequate compaction. To test this hypothesis, an additional experiment was conducted on 30%FA1AR12M mixtures, for which one mixtures was prepared with normal mixing and compaction procedure, however second mixture was prepared with additional 3% of water by total weight of mixture. The comparison between these mixtures are presented in Figure 4-9. With the addition of 3% water, the UCS of the Soil-Geopolymer mixture increased by 89%, depicting additional water during mixing played a significant role in increasing the UCS of the Soil-Geopolymer and also supported the decreased strength with the higher alkaline ratio.

In summary, if all the variables are kept the same, the increase in alkaline ratio will decrease the water content in the mixture and increase the overall solid content. This will result in water content less than the OMC needed for adequate compaction, hence effect the particle-to- particle interlocking, surface area contacts and ultimately less Alkaline, fly ash and soil reaction. This will in turn reduce the UCS of the Soil-Geopolymer mixtures as observed in Figure 4-9.

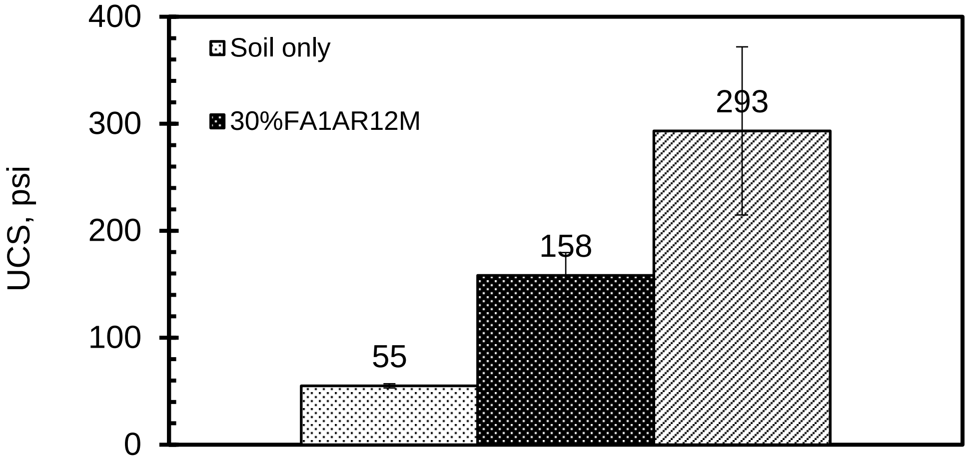

Figure 4-9: Effect of additional water on strength of Soil-Geopolymer mixtures

Table 4-3: Effect of Fly Ash, Alkaline Ratio, and Molarity of NaOH on UCS of Soil A.

Effect of FA							
SN	FA	AR	NaOH	UCS	STD.	Coef. Var.	Increment
	%		M	psi	psi	%	%
1	5	1(0.1)	10(1.1)	150	14	9%	
2	15	1(0.33)	10(3.9)	223	8.4	4%	49%
3	25	1(0.63)	10(8.3)	378	80.1	21%	152%
Effect of silicate							
1	15	0	10(6.3)	319	27.4651	9%	8.6
2	15	1(0.33)	10(3.9)	223	8.3865	4%	-30%
3	15	2(0.5)	10(2.8)	114	35.6493	31%	-64%
Effect of concentration of sodium hydroxide							
1	15	1(0.33)	8(3)	203	34.4351	17%	
2	15	1(0.33)	10(3.9)	223	8.3865	4%	10%
3	15	1(0.33)	12(4.8)	220	34.0441	15%	8%

Note: The values in the parenthesis represents the actual mixture properties for that mixtures.

4.3 Effect of FA, AR, and Concentration of NaOH on Young's Modulus

Young's Modulus of elasticity of FA-based Soil-Geopolymer was obtained from the slope of its stress-strain curve in elastic region. The Young's modulus was calculated for all mixture from the stress- strain curve tabulated in Table 4-5. Table 4-5, illustrates the effect of FA, AR, and concentration of NaOH on Young's modulus of elasticity. The effect of FA content on Young's modulus of elasticity was evaluated using 1 AR and 10M of NaOH and FA varied from 5% to 25%. The Young's modulus of elasticity at 5% FA is found to be 41,429 psi. With further increase in FA content up to 25%, about 217% increase was observed. However, with the increasing AR from 0 to 2, the modulus value was decreased by 72 %. Similarly, it is found that there was not much change in Young's modulus of elasticity with the increase in concentration of NaOH. This illustrates that Young's modulus value was exhibited similar response to UCS as function of FA, AR, and molarity of NaOH, as discussed earlier.

UCS of the various Soil-Geopolymer was plotted with its Young's modulus of elasticity, as shown in Figure 4-10. Figure 4-10, illustrates the young's modulus of elasticity is the linear function of UCS. The best fit line shows good co-relation between UCS and elasticity, with the regression coefficient of 0.68. From this model, the young's modulus of elasticity can be predicted using UCS value of the Soil-Geopolymer mixtures.

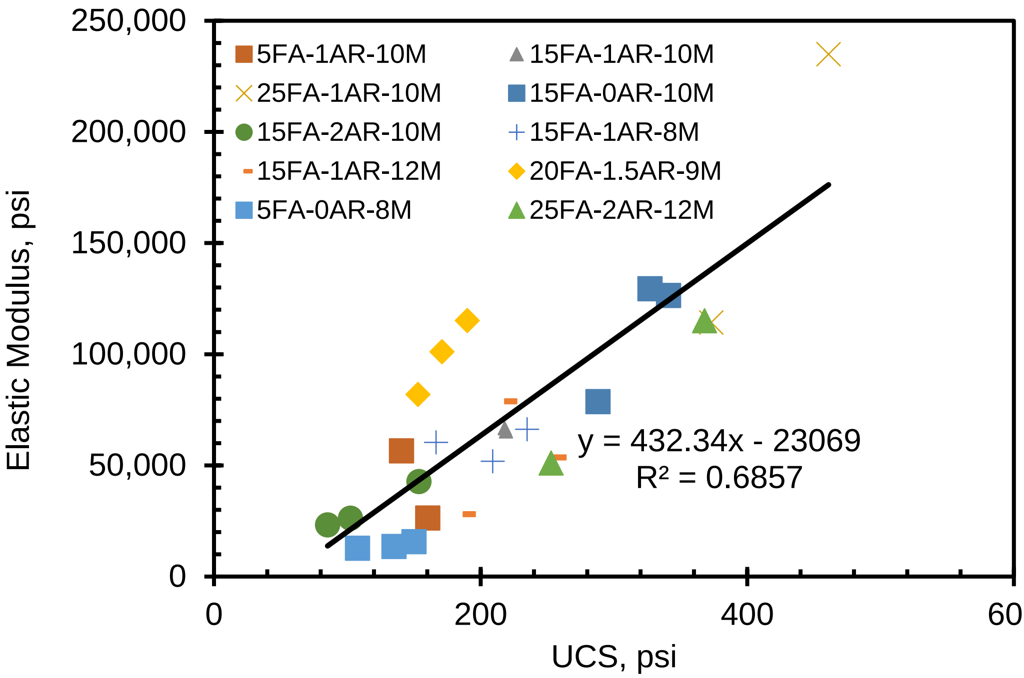

Figure 4-10: Relationship between Young's Modulus and UCS of various Soil-Geopolymer mixtures

Table 4-4: Relationship between FA, AR, And Concentration of NaOH With Young's Modulus of Elasticity of Soil A

Effect of FA							
SN	FA	AR	Molarity, M	Modulus, psi	STD.	Variance, %	% increase
1	5	1(0.1)	10(1.1)	41429	21329	51%	
2	15	1(0.33)	10(3.9)	66017	1014	2%	59%
3	25	1(0.63)	10(8.3)	131509	76755	38%	217%
Effect of silicate							
1	15	0	10(6.3)	111504	28492	26%	
2	15	1(0.33)	10(3.9)	66017	1014	2%	-41%
3	15	2(0.5)	10(2.8)	30769	10497	34%	-72%
Effect of concentration of Sodium hydroxide							
1	15	1(0.33)	8(3)	59478	7212	12%	
2	15	1(0.33)	10(3.9)	66017	1014	2%	11%

Note: The values in the parenthesis represents the actual (modified) mixture properties of that mixtures.

This equation was used to predict the Young's modulus of elasticity for selected mixtures of Soil-Geopolymer. The predicted elastic modulus was found to be very similar with observed modulus value, as shown in Table 4-5, which depicts that young's modulus of FA based Soil-Geopolymer mixture as a function of UCS. This result is also supported by data found in literature. Kang et al also developed the linear relationship between resilience modulus of Soil-FA mixtures for various soils (Kang, et al. 2014).

Table 4-5: Comparison between Predicted Young's Modulus with measured Young's Modulus value of the selected Soil-Geopolymer mixtures using Soil A

Mixture type	UCS, psi	Predicted value for Young's Modulus, psi	Measured value for Young's Modulus, psi	% Difference
25%FA0.4AR6M	564	220,948	271,195	19%
25%FA0AR6M	892	362,627	386,123	6%

4.4 Effect of FA, AR, and Molarity on Fracture Energy

Table 4-6, indicate the effect of the FA, AR, and concentration of NaOH on the fracture energy of the developed Soil-Geopolymer mixture. With increased in FA content, fracture energy was almost similar and exhibited high coefficient of variation amongst the samples. This variation in the results indicate that the fracture energy may not be reliable parameter to fully understand the failure mechanism of the Soil-Geopolymer mixtures. However, with increasing alkaline ratio, fracture energy decreased up to 40% for 2 AR with respect to 0 AR. Recall that the UCS results showed similar trend. On the other hand, increasing molarity of NaOH decreased the fracture energy by 38%. This is may be due to the fact that there was no significant change in the UCS but the strain at failure was reduced due to increase in molarity. The result indicates that due to high variation amongst the samples, fracture energy does not provide a clear understanding of the Soil-Geopolymer fracture behavior.

Table 4-6: Relationship between FA, AR, and concentration of NaOH with fracture energy for Soil A

<table>
<tr><th colspan="8">Effect of FA</th></tr>
<tr><th>SN</th><th>FA</th><th>AR</th><th>M</th><th>Fracture Energy</th><th>STD.</th><th>Coef. Var., %</th><th>% change</th></tr>
<tr><td>1</td><td>5</td><td>1(0.1)</td><td>10(1.1)</td><td>0.553</td><td>0.45</td><td>83%</td><td></td></tr>
<tr><td>2</td><td>15</td><td>1(0.33)</td><td>10(3.9)</td><td>0.53</td><td>0.05</td><td>10%</td><td>-4%</td></tr>
<tr><td>3</td><td>25</td><td>1(0.63)</td><td>10(8.3)</td><td>0.59</td><td>0.34</td><td>58%</td><td>9%</td></tr>
<tr><th colspan="8">Effect of silicate</th></tr>
<tr><th>SN</th><th>FA</th><th>AR</th><th>M</th><th>Fracture Energy</th><th>STD.</th><th>Coef. Var., %</th><th>% change</th></tr>
<tr><td>1</td><td>15</td><td>0</td><td>10(6.3)</td><td>0.52</td><td>0.05</td><td>10%</td><td></td></tr>
<tr><td>2</td><td>15</td><td>1(0.33)</td><td>10(3.9)</td><td>0.53</td><td>0.05</td><td>10%</td><td>0%</td></tr>
<tr><td>3</td><td>15</td><td>2(0.5)</td><td>10(2.8)</td><td>0.31</td><td>0.08</td><td>26%</td><td>-40%</td></tr>
<tr><th colspan="8">Effect of molarity</th></tr>
<tr><th>SN</th><th>FA</th><th>AR</th><th>M</th><th>Fracture Energy</th><th>STD.</th><th>Coef. Var., %</th><th>% change</th></tr>
<tr><td>1</td><td>15</td><td>1(0.33)</td><td>8(3)</td><td>0.57</td><td>0.19</td><td>35%</td><td></td></tr>
<tr><td>2</td><td>15</td><td>1(0.33)</td><td>10(3.9)</td><td>0.53</td><td>0.05</td><td>10%</td><td>-8%</td></tr>
<tr><td>3</td><td>15</td><td>1(0.33)</td><td>12(4.8)</td><td>0.78</td><td>0.48</td><td>62%</td><td>38%</td></tr>
</table>

Note: The values in the parenthesis represents the actual mixture properties for that mixtures.

4.5 Relation Between Failure Strain and UCS

The plot between strain at failure and UCS of the Soil-Geopolymer mixtures were shown in Figure 4-11. Various mixtures were plotted, between the average value of UCS and the average value of maximum strain at failure. Figure 4-11 represents that maximum strain of failure is linearly related with UCS of Soil-Geopolymer mixture, with the correlation coefficient of 0.75. This linear relation depicts that lower the UCS, the strain at failure would be higher.

Soil -Geopolymer with higher strain value suggest that soil is weak and may cause higher plastic deformation under repeated truck traffic. However, Soil-Geopolymer with high UCS value would have low failure strain, suggesting low plastic deformation under dynamic loading system. Low strain and high UCS Soil-Geopolymer mixtures could be suitable for the base, subbase and subgrade materials in Highways.

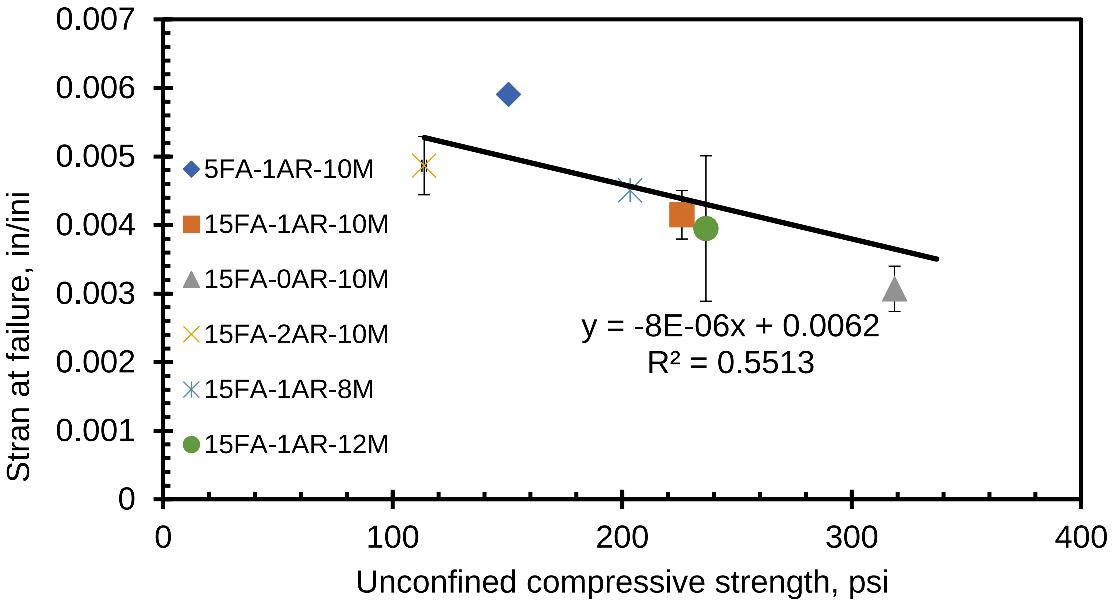

Figure 4-11: Maximum Failure Strain as a function of UCS

4.5 Statistical Analysis

4.5.1 Co-relation analysis and Regression analysis

The co-relation analysis UCS and mixture variables for Soil A and Soil B are shown in Table 4-7 and Table 4-8 respectively. The data in the tables illustrate that UCS of the Soil-Geopolymer exhibit good linear relationship with FA, Molarity of NaOH and AR. However, there is little correlation with AR. Also, strong interdependence among FA, AR and concentration of NaOH was observed (48% to 80%).

Table 4-7: Correlation between UCS, FA, AR and concentration of sodium hydroxide(M), for Soil A

	UCS	*FA*	*M*	*AR*
UCS	1			
FA	0.45	1		
M	0.76	0.79	1	
AR	0.13	0.80	0.48	1

Table 4-8: Correlation between UCS, FA, AR and concentration of sodium hydroxide, M for Soil B

	UCS	*AR*	*M*	*FA*
UCS	1			
AR	0.01	1		
M	0.17	0.52	1	
FA	0.43	0.52	0.55	1

Linear regression analysis was conducted on both soils to develop regression models to predict UCS of the mixtures using FA, AR and M. It was found that the coefficient of determinations was 0.89 and 0.79 for Soil A and Soil B, respectively. This implies that the developed models are highly reliable. The regression model for Soil A and soil B are shown in equation 11 and 12. The detail regression statistics, ANOVA, and regression coefficient and t- tests are tabulated in Table 4-9, Table 4-10, Table 4-11 and Table 4-12, Table 4-13, Table 4-14 for Soil A and soil B respectively. It's clear from these tables that all the variable used in the models are significant as the p-values are less than 10%.

Table 4-9: Regression analysis Statistics for Soil A

Regression Statistics	
Multiple R	0.94
R Square	0.89
Adjusted (R) Square	0.87
Standard Error	39.17
Observations	44

Table 4-10: Regression analysis result (ANOVA) for Soil A

ANOVA					
	df	SS	MS	F	Significance F
Regression	6	472773	78795.5	51.3	1.86E-16
Residual	37	56791.2	1534.9		
Total	43	529564			

Table 4-11: Regression coefficients and t-test for Soil A

	Coefficients	*t Stat*	*P-value*
Intercept	71.3	3.61	0
AR	-464.3	-3.29	0
FA	19.12	3.77	0
M *AR	66.12	2.5	0.02
M^2	3.46	3.9	0
FA*M	-1.84	-2.22	0.03
AR^2	-188.57	-1.8	0.08

Table 4-12: Regression statistics for soil B

Regression Statistics	
Multiple R	0.89
R Square	0.79
Adjusted R Square	0.74
Standard Error	34.13
Observations	24

Table 4-13: Regression analysis result (ANOVA result) soil B

ANOVA					
	df	*SS*	*MS*	*F*	*Significance F*
Regression	5	80741	16148	14	0
Residual	18	20969	1165		
Total	23	101710			

Table 4-14: Regression coefficient and t-test result soil B

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	15.76	31.00	0.51	0.62
FA^2	0.80	0.22	3.60	0.00
AR^2	517.08	77.21	6.70	0.00
AR*M	-56.72	8.13	-6.98	0.00
FA*M	-2.59	0.92	-2.83	0.01
M^2	3.76	1.10	3.42	0.00

$$\begin{aligned} UCS(psi)=&(71.3-464.3*(AR)+19.12*(FA)+66.12*(M)*(AR)\\ &+3.45*(M)^2-1.83*(FA)*(M)-188.57*(AR)^2) \end{aligned} \tag{91}$$

$$\begin{aligned} UCS(psi)=&(15.76+0.80*(FA))^2+517.08*(AR)^2-56.72*(AR)*(M)-\\ &2.59*(FA)*(M)+3.76*(M)^2) \end{aligned} \tag{102}$$

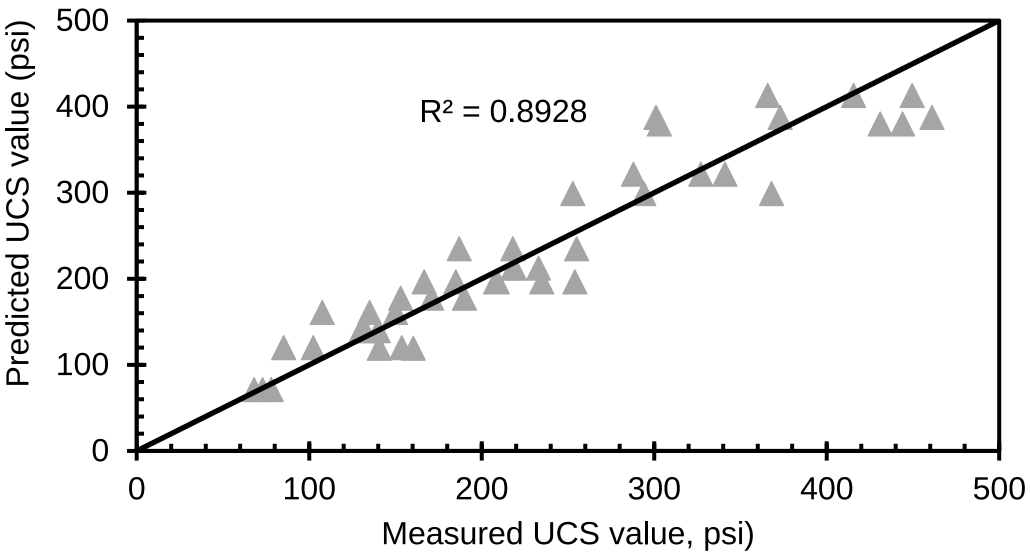

Figure 4-12: Predicted UCS from the equation Vs Actual UCS Value for Soil A

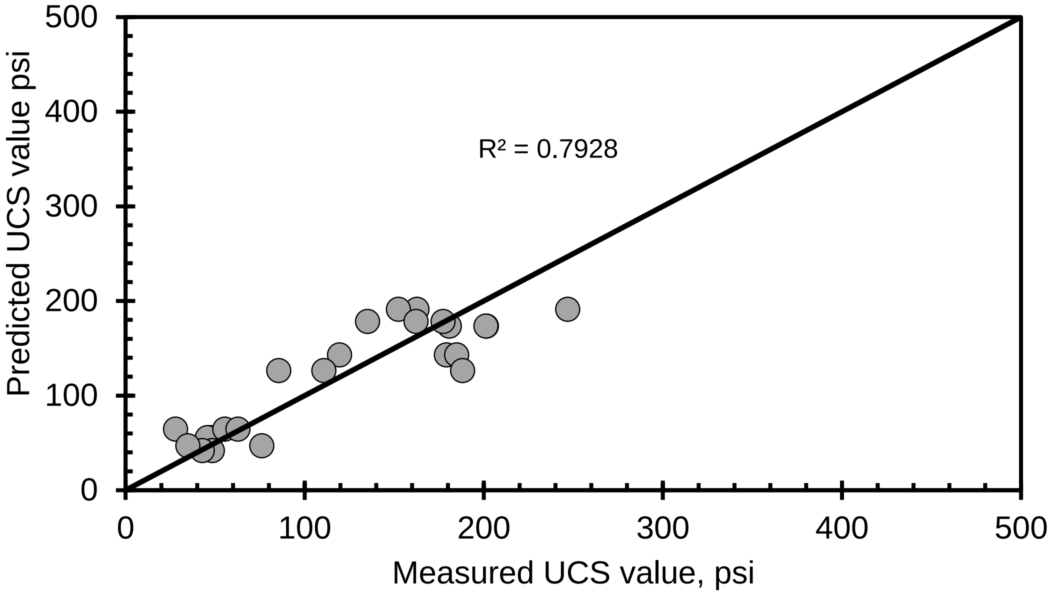

Figure 4-13: Predicted UCS from the equation Vs Actual UCS Value for Soil B

Figure 4-12 and Figure 4-13, depicts the plot between measured and predicted UCS(psi) value. It is clear from the plot that the model predicted the compressive strengths for both soils (Soil A and Soil B) very well, as best fit line lies close to line of equality (45-degree line).

4.5.2 Sensitivity analysis

The developed regression models for both soils were used to investigate the effect of each variable namely, FA content, AR, and NaOH molarity on the UCS of Soil-Geopolymer mixtures. The detailed discussion on sensitivity analysis are presented in the following sections.

4.5.2.1 Effect of Fly ash on UCS

The effect of FA on UCS of Soil A Geopolymer is shown in Figure 4-14, Figure 4-15, and Figure 4-16 for alkaline ratio and concentration of sodium hydroxide. The data if these figures indicate that the UCS of the soil increases with the increase in fly ash content. Higher the fly ash content, more the fine particles of silica and alumina, which are responsible for increasing the reaction of the soil FA mixtures with alkali activator. With the higher fly ash content, the binding properties of the mixture were also increased eventually the strength of the Soil-Geopolymer mixtures was enhanced. The figures also show that, for higher molarity the increase in strength is not significant for all three-alkaline ratio, but for lower molarity of sodium hydroxide, the strength was significantly increased with the increase in FA. At 5%FA0.4AR9M (Figure 4-16) strength was found to be 383 psi, however, at 25% FA0.4AR9M, the strength was 435 psi, indicating strength increase was very low, only 13%. On the other hand, for lower molarity of sodium hydroxide, the strength was increased by more than 50%.

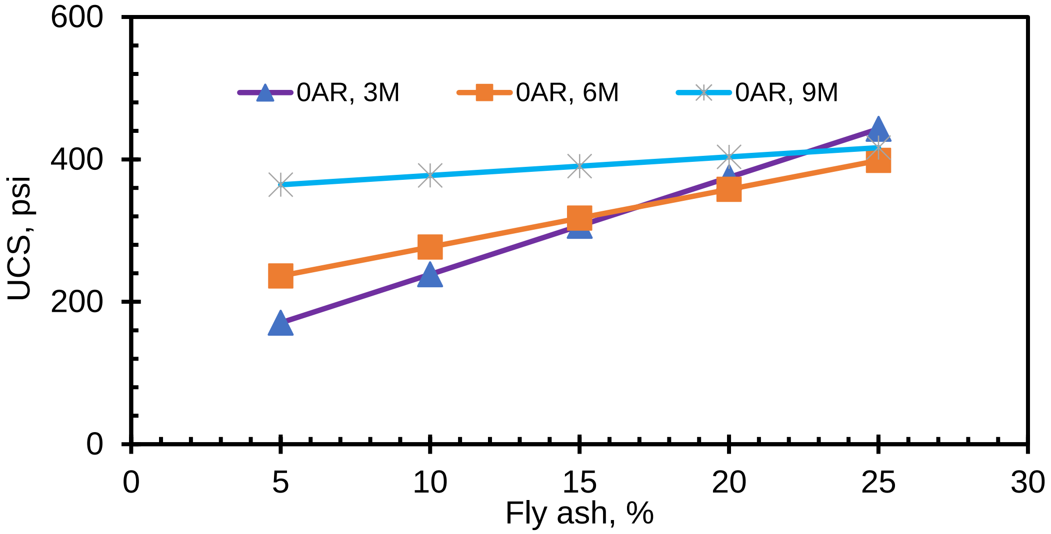

Figure 4-14: Sensitivity Analysis, the effect of Fly Ash, on UCS of Soil A (psi) for the various concentration of sodium hydroxide for 0AR

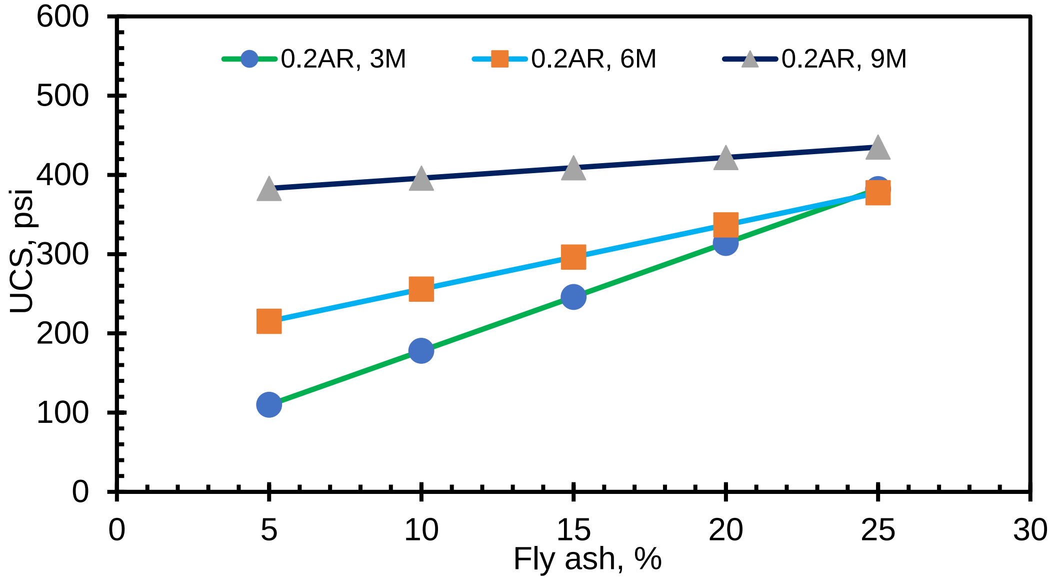

Figure 4-15: Sensitivity Analysis, the effect of Fly Ash, on UCS of Soil A (psi) for the various concentration of sodium hydroxide for 0.2AR

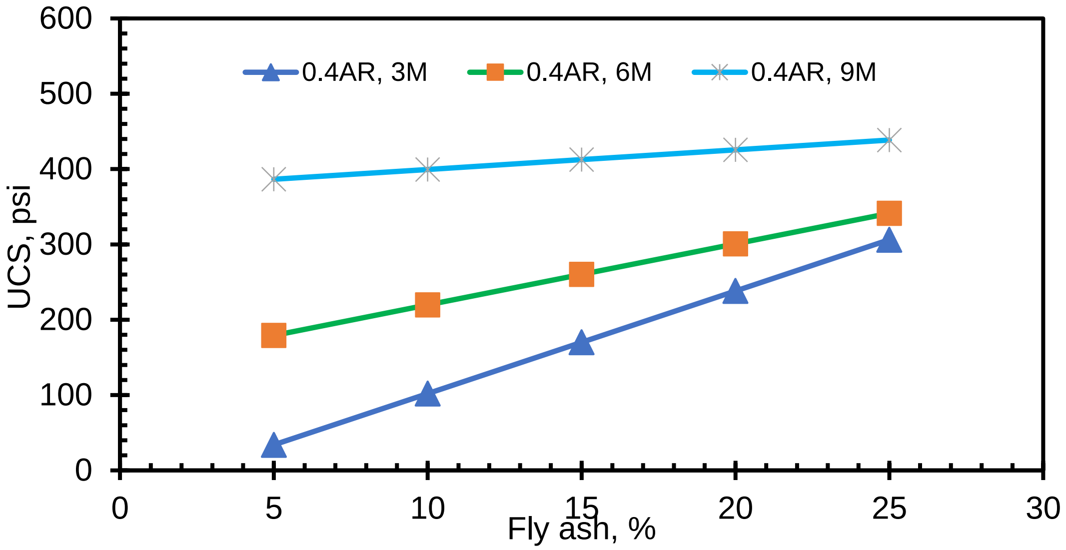

Figure 4-16: Sensitivity Analysis, the effect of Fly Ash, on UCS of Soil A (psi) for the various concentration of sodium hydroxide for 0.4AR

The sensitivity analysis of the effect of FA on compressive strength of Soil-Geopolymer mixtures of soil B is shown in Figure 4-17, Figure 4-18, and Figure 4-19 for various alkaline ratio and molarity of sodium hydroxide. All figure shows that at 30% FA, UCS was found to be maximum. But for lower FA content UCS was depended on other mix properties. Figure 4-17, illustrate that at lower FA content the strength was higher for higher molarity content i.e. at 10%FA0AR12M, strength was 333 psi, Even, with addition of FA strength decreased to 254 psi for 20%FA0AR12M but again strength was increased at FA content of 30%, i.e. at 30%FA strength was 342 psi. A Similar pattern can be found in Figure 4-18, for 12M and 10M of the sample for 0.5 alkaline ratio, strength was decreasing with the addition of FA but increasing after 20%FA. However, at 8M and 10M mixtures the strength was increasing with the addition of FA for all alkaline ratio.

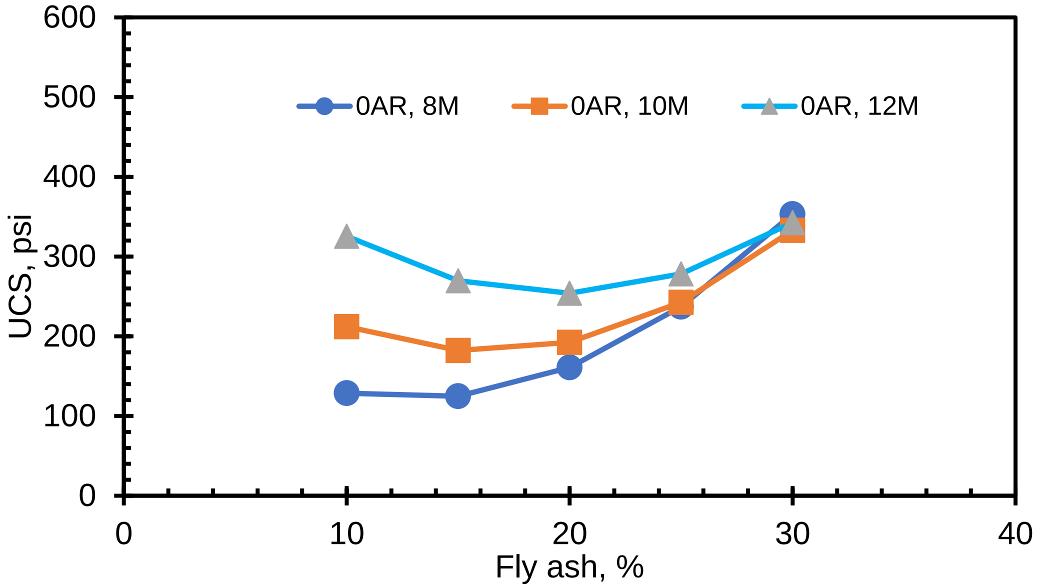

Figure 4-17: Sensitivity Analysis, the effect of Fly Ash on UCS of Soil B (psi) for the various concentration of sodium hydroxide for 0AR

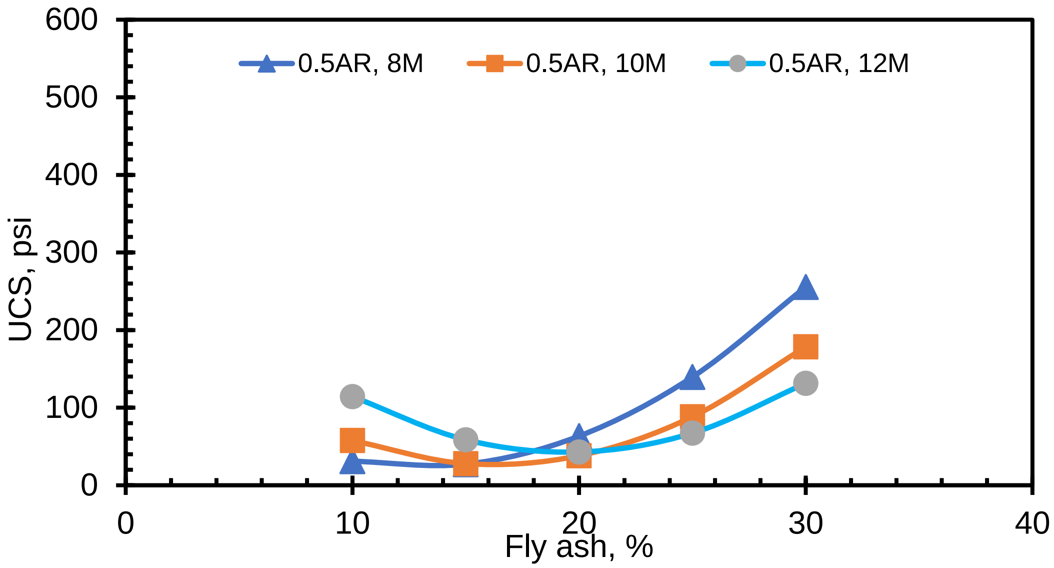

Figure 4-18: Sensitivity Analysis, the effect of Fly Ash on UCS of Soil B (psi) for the various concentration of sodium hydroxide for 0.5AR

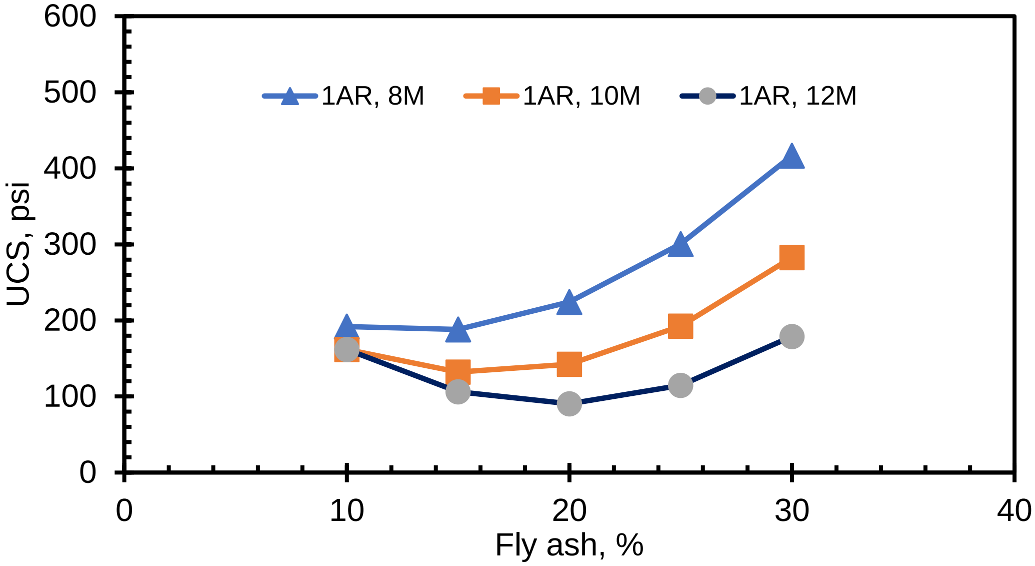

Figure 4-19: Sensitivity Analysis, the effect of Fly Ash on UCS of Soil B (psi) for the various concentration of sodium hydroxide for 1AR

4.5.2.2 Effect of Alkaline ratio on UCS

The effect of alkaline ratio on compressive strength of Soil A-FA Geopolymer mixtures is shown in Figure 4-20, Figure 4-21 and Figure 4-22 for various combination of sodium hydroxide molarity and FA content. The result illustrates that with increased amount of alkaline ratio the strength (psi) decreased. This implies that with increased silicate contents the strength of the Soil-Geopolymer was decreased. The decreased was significant for lower molarity but in case of 9M mixture the curve was almost straight, which means at higher alkaline ratio, the higher concentration sodium hydroxide was not significant.

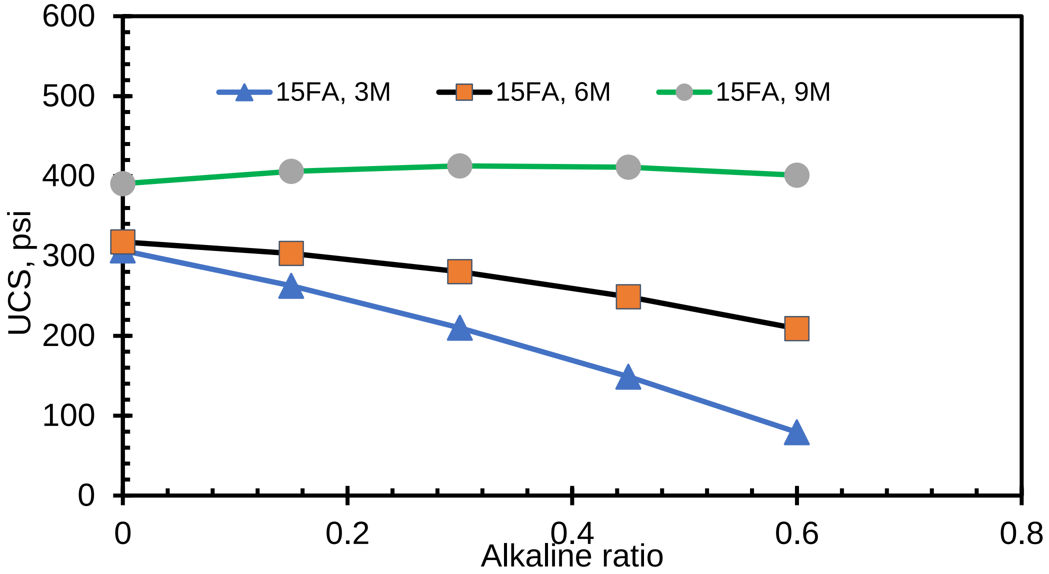

Figure 4-20: Sensitivity analysis, the effect of alkaline ratio, on UCS of Soil A (psi) for various concentration of NaOH for 15% FA

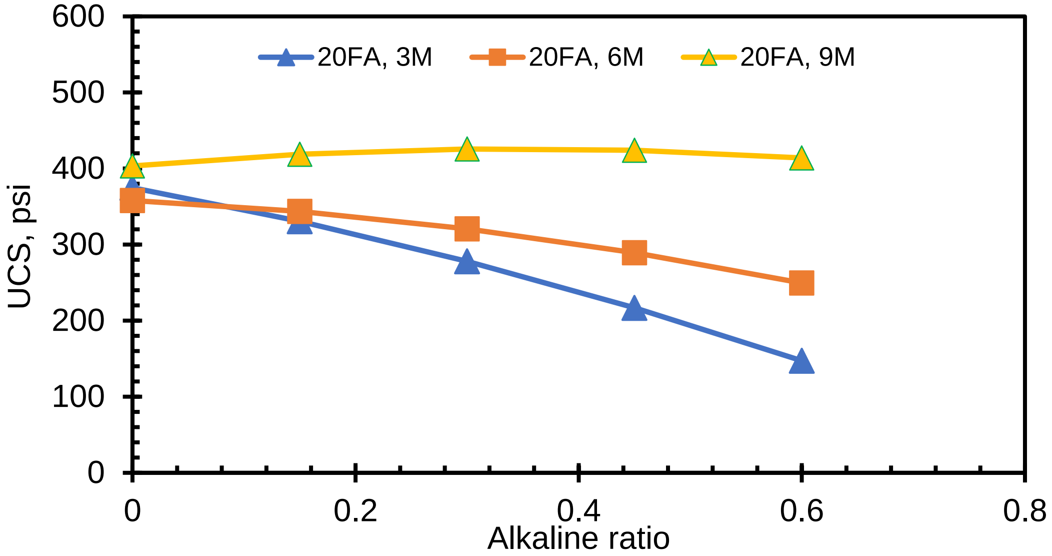

Figure 4-21: Sensitivity analysis, the effect of alkaline ratio, on UCS of Soil A (psi) for various concentration of NaOH for 20% FA

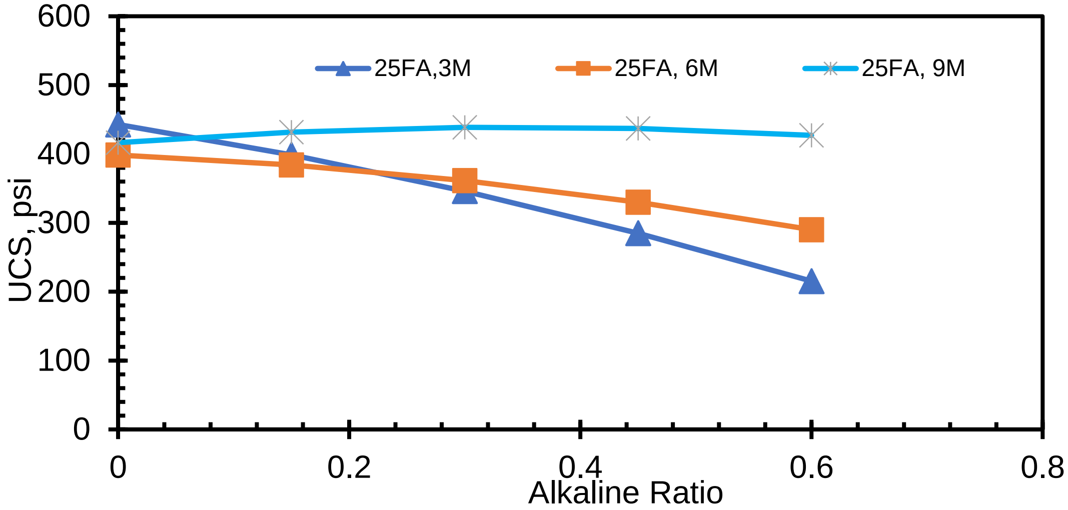

Figure 4-22: Sensitivity analysis, the effect of alkaline ratio, on UCS of Soil A (psi) for various concentration of NaOH for 25% FA

The effect of alkaline ratio on compressive strength of (Soil B) Soil-Geopolymer mixtures is shown in Figure 4-23, Figure 4-24, and Figure 4-25for various combination of sodium hydroxide molarity and FA content. These figures depict that the UCS was decreased with increase in alkaline ratio reached a minimum value and then started to increase. At 10 %

FA, 12M concentration was giving higher strength at 0AR, shown in Figure 4-23. Similarly, in Figure 4-24 and Figure 4-25, for 20%FA0AR12M, strength was higher (254 psi), however, for 30%FA, strength was found to me maximum at 8 M solution and alkaline ratio of 1(416 psi). This maximum strength at 30%FA1AR8M, indicated unusual pattern with respect to other combination of FA and alkaline ratio. For this case, both at 30% FA0AR8M and 30%FA1AR8M were considered to be optimum based on Alkaline ratio.

The result of the study does not follow the same pattern as reported in literature document, where it was suggested that the silicate was responsible for increase in the strength of the mixture. The silicate present in the mixture control the characteristics gel by controlling the rate of silica release with various mix proportion. Also, silica acts as charge balancing agent in the mixture, but in case of excess silica contents the unconfined compressive strength was decreased, because silica hinders the evaporation of water and Geo-polymerization formation (Morsy, et al. 2013)**.**

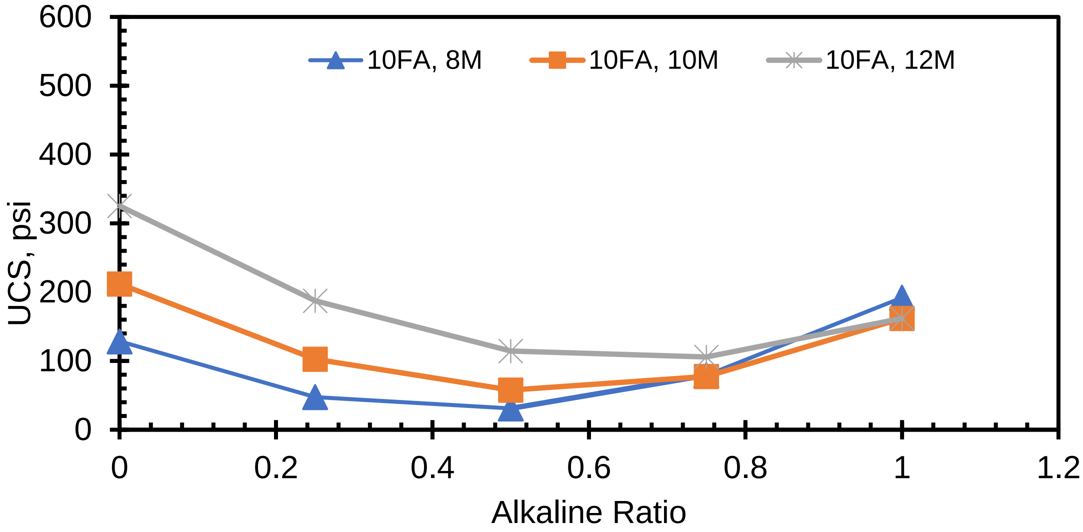

Figure 4-23: Sensitivity analysis, the effect of alkaline ratio, on UCS (psi) of soil B for various concentration of NaOH for 10% FA

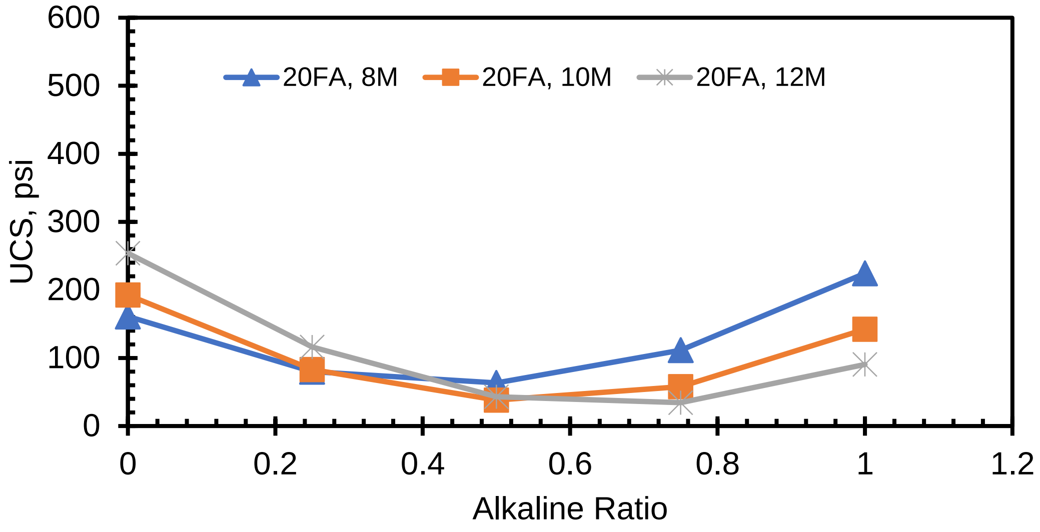

Figure 4-24: Sensitivity analysis, the effect of alkaline ratio, on UCS (psi) of soil B for various concentration of NaOH for 20% FA

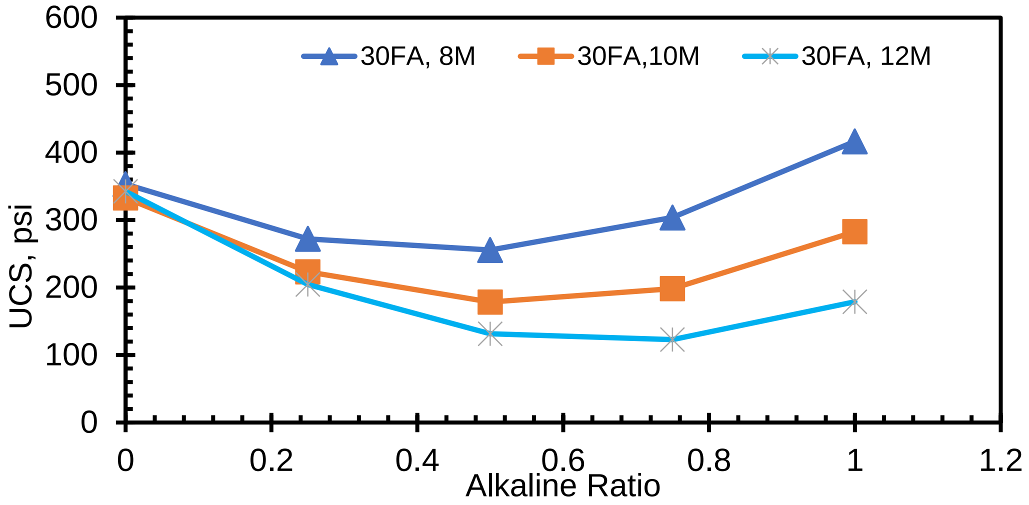

Figure 4-25: Sensitivity analysis, the effect of alkaline ratio, on UCS (psi) of soil B for various concentration of NaOH for 30% FA

4.5.2.3 Effect of Molarity on UCS

The effect of concentration of sodium hydroxide on compressive strength of Soil A Soil-FA Geopolymer mixtures is shown in Figure 4-26, Figure 4-27, and Figure 4-28 for various combination of AR and FA. From the figures, it can be observed that the strength of the Soil-Geopolymer product increased with the increasing molarity of sodium hydroxide for all combination of alkaline ratio. However, at 0AR the strength of mixture was not significantly increased with molarity of NaOH, as shown in Figure 4-26. It showed some controversy that lower molarity showed higher strength, even at 4 M concentration up to 450 psi strength was observed. From this analysis, it can be concluded that lower or medium concentration of sodium hydroxide results in higher strength or fulfilled the CSD criteria for soil- stabilization of DOTD.

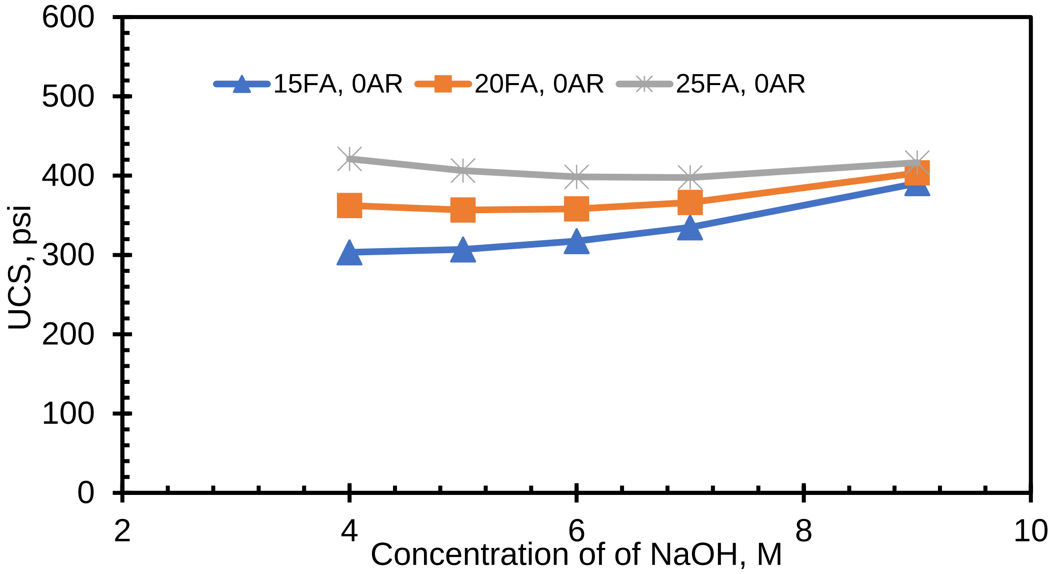

Figure 4-26: *Sensitivity analysis, the effect of concentration of NaOH, on UCS of Soil A (psi) for various content of FA for 0AR*

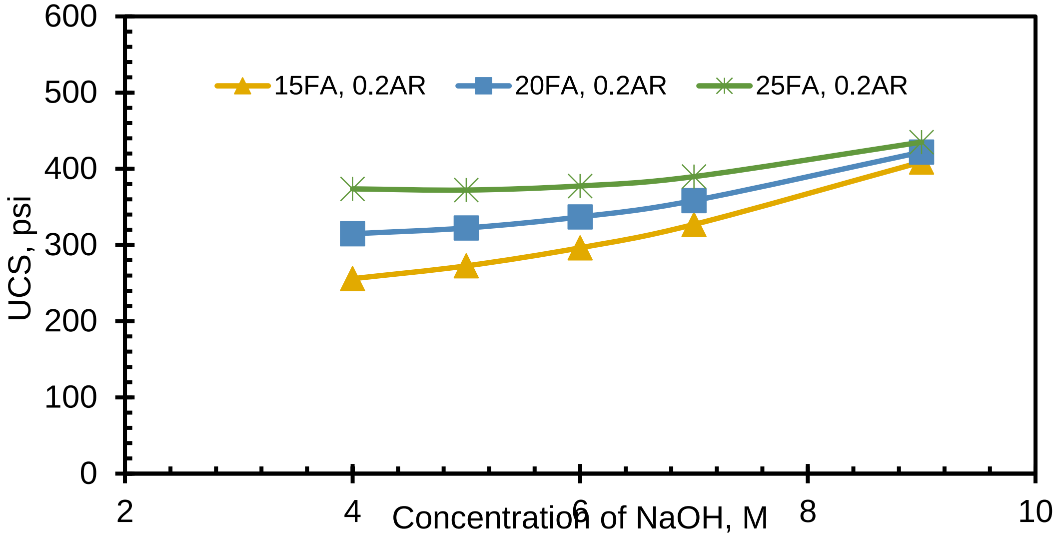

Figure 4-27: Sensitivity analysis, the effect of concentration of NaOH, on UCS of Soil A (psi) for various content of FA for 0.2AR

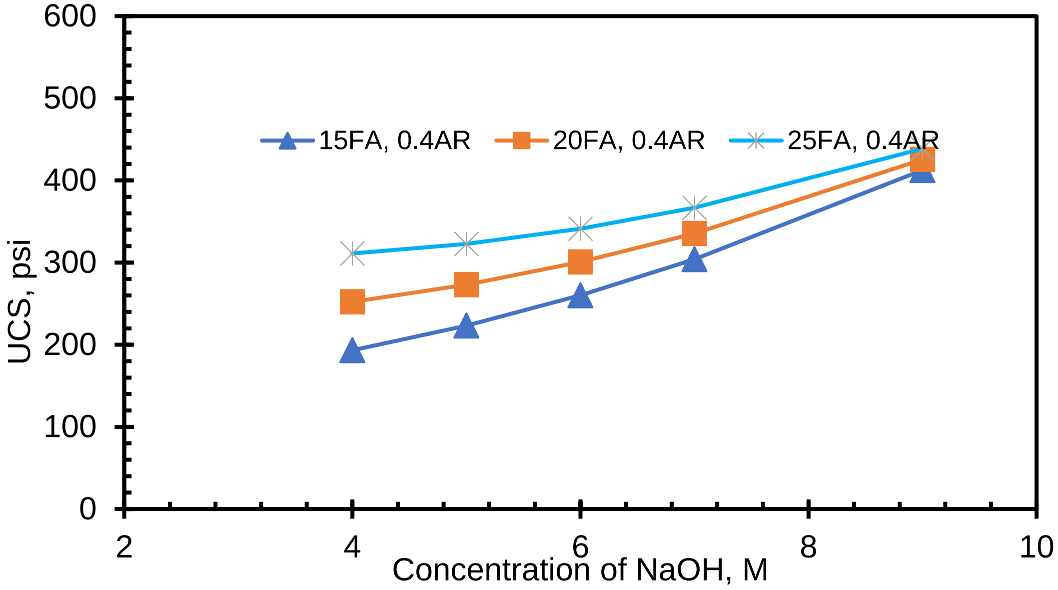

Figure 4-28: Sensitivity analysis, the effect of concentration of NaOH, on UCS of Soil A (psi) for various content of FA for 0.4AR

The sensitivity analysis of effect of concentration of sodium hydroxide on compressive strength of Soil B Soil-FA Geopolymer mixtures is shown in Figure 4-29, Figure 4-30 and Figure 4-31 for various combination of AR and FA. Figure 4-29 depicts that at 30%FA and 0AR strength was almost constant but strength was increased with lower FA contents. On other hand, Figure 4-30 and Figure 4-31 shows that, with increased concentration of NaOH, UCS was decreased. But at 10% FA strength was increased for increasing concentration of sodium hydroxide for 0AR and 0.5AR. However, for 30%FA1AR strength was decreased for increasing molarity of sodium hydroxide. At 8M concentration of NaOH was showing high strength with 30% FA and was considered optimum mixture for high plastic soil for the analysis.

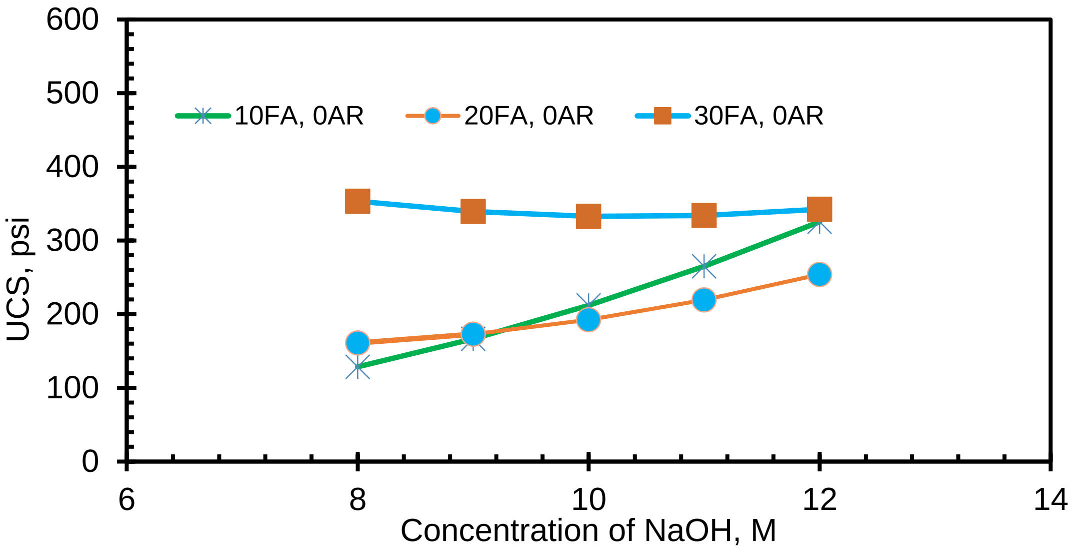

Figure 4-29: Sensitivity analysis, the effect of concentration of sodium hydroxide with UCS of soil B-FA Geopolymer for 0 AR and various content of FA

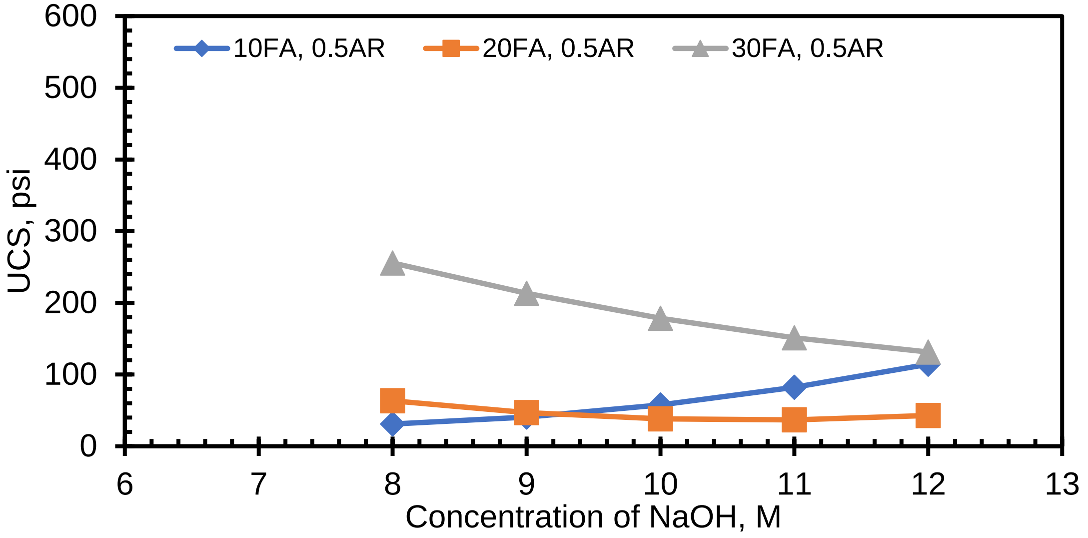

Figure 4-30: Sensitivity analysis, the effect of concentration of sodium hydroxide with UCS of soil B-FA Geopolymer for 0.5 AR and various content of FA

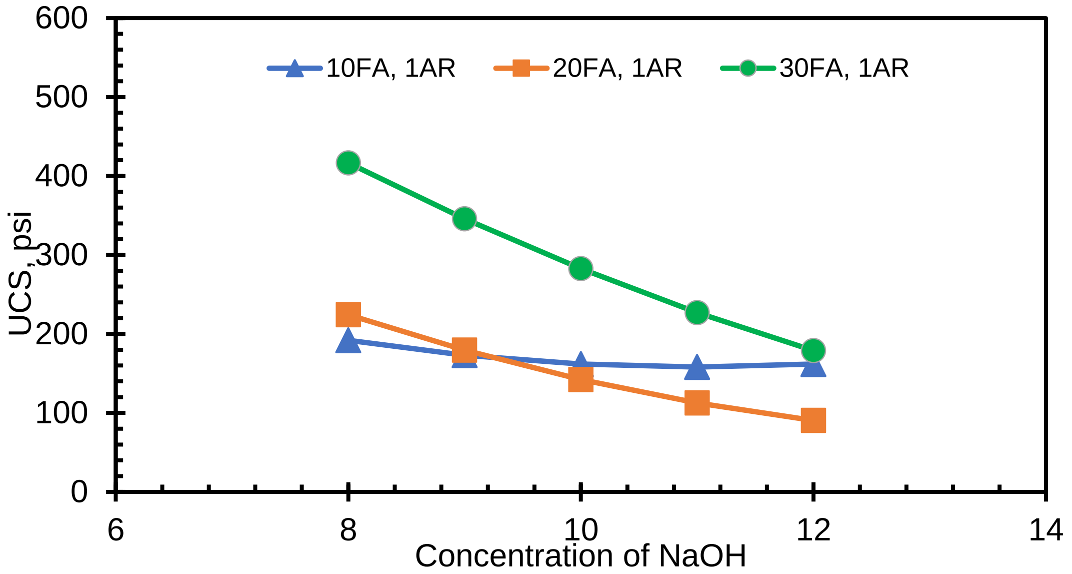

Figure 4-31: Sensitivity analysis, effect of concentration of sodium hydroxide with UCS of soil B-FA Geopolymer for 1AR and various content of FA

From the above analysis for medium plastic soil, it can be concluded maximum strength was obtained at higher content of FA, 0 alkaline ratio and high molarity. However, higher concentration of NaOH is generally considered unsafe be used in construction, therefore medium concentration (6 molar) of NaOH was selected. In addition, 25%FA0.4AR6M also satisfied the minimum strength criteria of CSD. Based on these parameter, two mixtures, 25%FA0AR6M and 25%FA0.4AR6M were selected from the medium plastic soils such that mixture fulfilled the minimum strength criteria for CSD. Similarly, for high plastic soils 30%FA0AR8M, and 30%FA1AR8M mixtures were selected for further laboratory experiment and analysis.

4.5.3 Validation of statistical model and sensitivity analysis.

Figure 4-32, displays the comparison between the regression model and measured sample verification for both mixtures of Geopolymer of 25%FA0AR6M and 25%FA0.4AR6M. Both the Figure indicate that the measured UCS of the mixtures were higher as compared to the predicted UCS using the regression models. From the Figure 4-32, it can be explained that both selected mixture satisfied the 300-psi strength of CSD, therefore both mixtures were selected for further investigation.

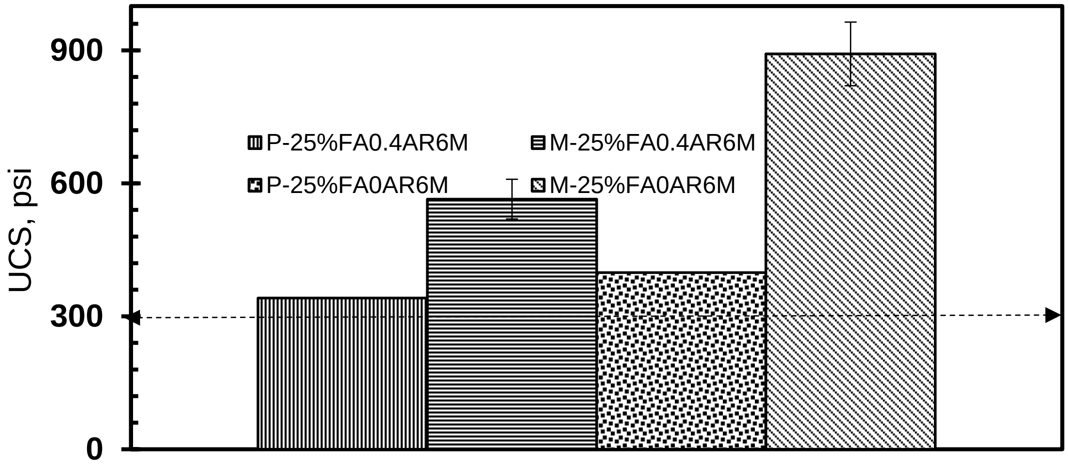

Figure 4-32: Comparison of predicted (psi) UCS and actual UCS value for Soil A-Geopolymer mixtures

Similarly, validation of compressive strength was conducted on 30%FA0AR8M mixture and 30%FA1AR8M, for soil B as shown in Figure 4-33. The figure illustrates that the predicted value from the model was similar to the observed value from the experiment for 30%FA0AR8M mixtures. However, 30%FA1AR8M mixture exhibits 178% higher predicted strength than measured one. Further, it did not pass the cement stabilized design(CSD) criteria of 300 psi for road base and subbase. Therefore, 30%FA0AR8M mixture was selected for further experimentation for soil B. Further CSD and CTD mixture were selected to determine their UCS and compare with geopolymer mixtures for both soils.

These selected mixtures from both soils were further conducted in laboratory experiment such as; resilience modulus, dynamic modulus and durability test. In addition, these tests were conducted on CSD and CTD mixture as specified by DOTD, LA.

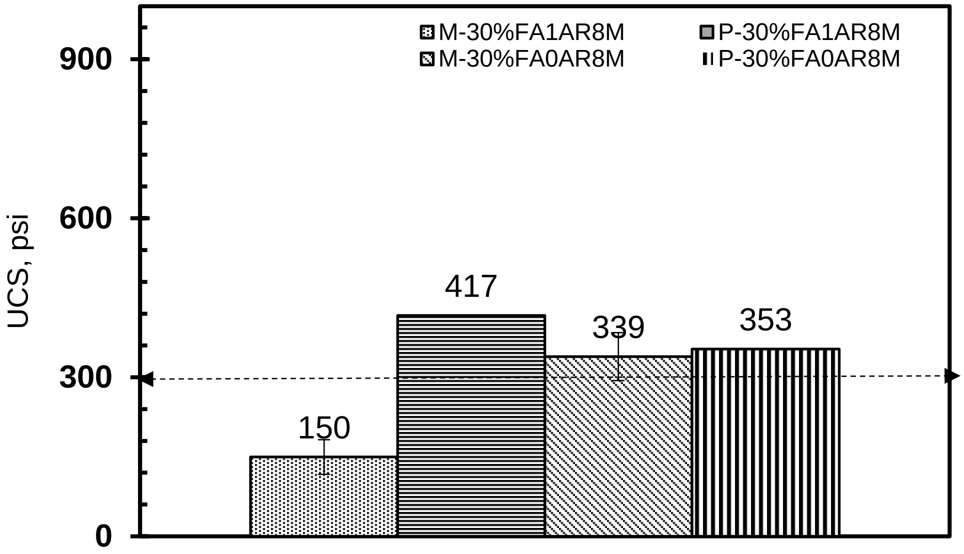

Figure 4-33: Comparison between model predicted UCS with observed UCS for Soil B-Geopolymer mixture

4.6 Comparison of UCS with soil- cement stabilization

The comparison of UCS for soil-cement stabilization and fly ash based Soil-Geopolymer for Soil A are shown in Figure 4-33. It can be observed that 5% soil-cement mixtures have low strength as compared with other mixes. At 10% cement content, the UCS increased by 200% relative to 5% cement content in soil-cement mixtures. In addition, 25%FA0.4AR6M Soil-Geopolymer mixtures illustrated similar UCS values as compared with 10% soil-cement. Moreover, the unconfined compressive strength of Soil-Geopolymer with 0 AR has higher compressive strength than 10% cement content and 0.4 AR mixtures. These results clearly reinforce the similar conclusion as discussed in sensitivity analysis based on the regression model, that is with the addition of silicate the strength of Geopolymer-soil decreases. Figure 4-35 illustrates that stress-strain curve and stiffness properties of the various mixtures. The stiffness of 5% cement is found to be less 10% cement. Further, the stiffness of the 25%FA0AR6M, 25%FA0.4AR6M and 10% Cement shows similar stiffness properties.

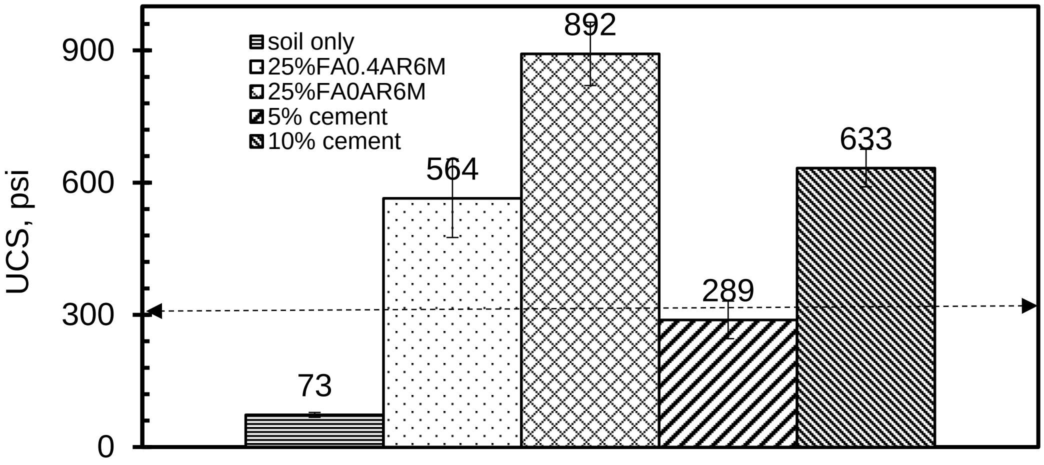

Figure 4-34: Comparison of the UCS between various method of soil stabilization and UCS of control mix, for Soil A

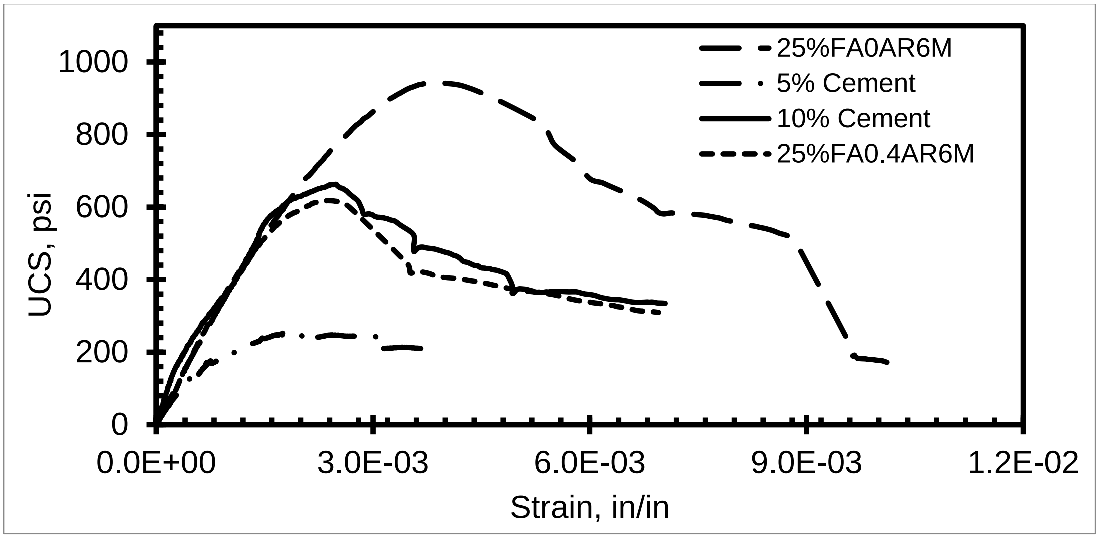

Figure 4-35: Stress- strain diagram for various mixtures of Soil-Geopolymer and soil-cement for Soil A

The strength of the selected Soil-Geopolymer mixtures were then compared with the strengths of the Soil B stabilized with cement, cement with lime soil, as shown in Figure 4-36. This figure depicts that 10% cement content, 9%cement-9% lime and Soil-Geopolymer mixture have similar UCS values, and passed the criteria of 300 psi. The result represents that even for high plastic soil, FA based Geopolymer soil stabilization exhibited improved compressive behavior. With the use of soil-FA Geopolymer, 500% increase in strength was achieved as compared with the strength of soils. Similar improved were also observed with other stabilization method as discussed.

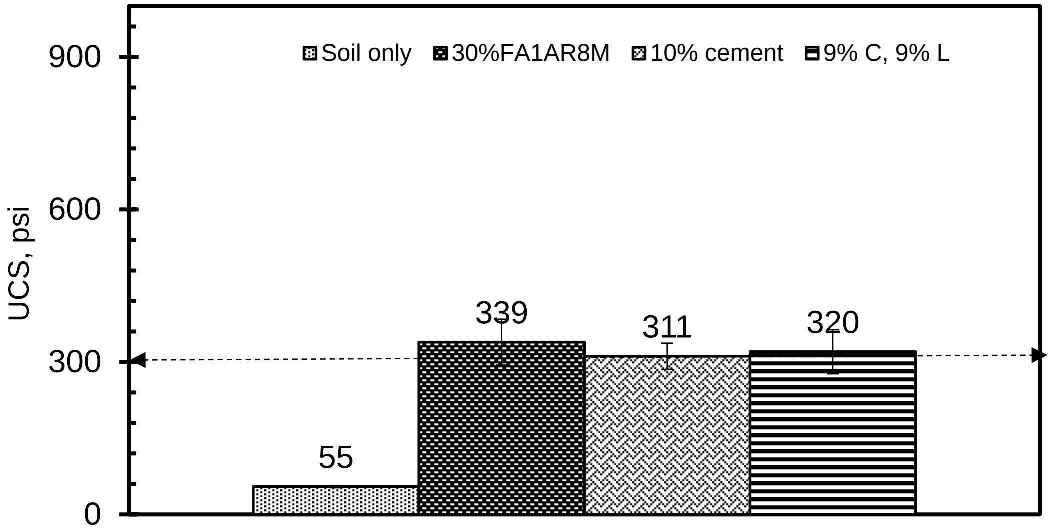

Figure 4-36: Comparison of the UCS between various method of soil stabilization and UCS of control mix, for Soil B

4.7 Durability Test

The durability test results for Soil A are summarized in Table 4-15. The data in the table depicts that the 10% soil-cement mixture, 25% FA0.4AR6M and 25%FA0AR6M Soil-Geopolymer mixtures have passed the durability test criteria. Whereas soil-cement of 5% soil-cement content has failed in the 11th cycle of wet and drying durability test. The volume change of the 5% soil-cement mixture was 16.74%. Moreover, the soil-cement loss was about 3.68% of dry weight of soil-cement, indicating that 5% soil-cement has more shrinkage and swelling than other mixture. On the other hand, the soil-cement mixtures and Soil-Geopolymer mixtures experienced less than 1.5% weight loss. This result indicates that alkali activated fly ash based Soil-Geopolymer mixtures passed the wet and dry durability test and can be used to stabilize the soil, as explained by Cruz et al. (Cruz, et al. 2017). Moreover, these passed mixtures showed a similar pattern in volume change. Soil with 10% cement exhibited slightly less volume change, than Soil-Geopolymer, however, 25%FA0AR6M and 25%FA0.4AR6M showed almost similar volume change. This result indicates that mixtures with odium silicate and sodium hydroxide and specimen with only sodium hydroxide shows similar result in durability. Even though, 25%FA0,4AR6M has relatively less strength than 25%FA0AR6M, both passed the wet and dry durability test.

In addition, the sample was placed in the oven for drying to calculate moisture content, after completing 12th cycle or even for failed sample. Moisture content was calculated and compared with original moisture content (while preparing sample). The result shows that the moisture content was decreased for 10% and 5% soil-cement sample with respect to moisture at compaction. This result illustrates that soil-cement are less susceptible to moisture content. However, final moisture content of Geopolymer soil was found to be higher than optimum moisture content. Figure 4-37 depicts the images of various mixture after completing the cycle (for 5% soil-cement mixture, 11th cycle images was taken). In the figure, it can be observed that soil-cement mixtures showed more cracks than Soil-Geopolymer and 10% cement content showed more crack as compared with 5% cement. Indicating 10% soil-cement mixtures can be replaced with Soil-Geopolymer mixtures, as Soil-Geopolymer showed very few cracks, as shown in Figure 4-37.

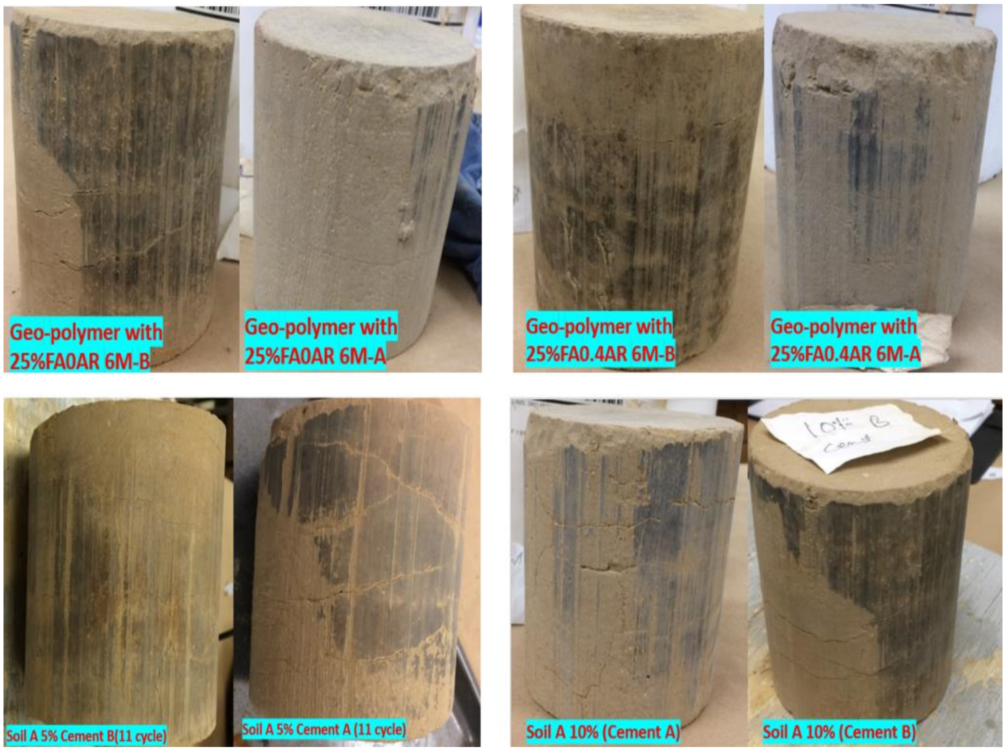

Figure 4-37: The image of the durability test of the mixture after completing 12 cycles for Soil A. (in case of 5% cement sample was just before breaking-11 cycle)

Table 4-15: Summary of durability test Soil A

Mixture type	Dry density	Soil loss	Volume change	Final moisture content	Original moisture content	Diff. in moisture content	Remarks
	pcf	%	%	%	%	%	
10% Cement	116.71	1.19	6.08	12.95	13.17	0.21	Passed
5% Cement	117.84	3.68	16.74	11.41	12.87	1.46	Failed
25FA0AR6M	113.56	1.41	9.54	15.95	13.13	-2.82	Passed
25FA0.4AR6M	112.02	1.44	8.55	16.81	13.18	-3.63	Passed

4.8 Resilience Modulus test

The resilience modulus tests results for medium plastic soils are shown in Figure 4-38, which depicts that the resilience modulus for 10 % soil-cement mixtures was higher than the 5% soil-cement, and Soil-Geopolymer mixtures at same condition of cyclic stress. The resilience modulus of 10 % soil-cement was increased by 50% than 5% soil-cement. In addition, the resilience modulus of FA based Soil-Geopolymer was about 33% to 50% less than 10% soil-cement. Recall that the Soil-Geopolymer mixtures have either similar of high compressive strengths. Hence lower resilient modulus indicate that the mixtures have more strain carrying capacity before failure, leading towards ductile behavior and low potential of tensile and shrinkage cracking

Also, 25%FA0AR6M exhibited 36% higher resilience modulus 25%FA0.4AR6M. Indicate that the mixture with only sodium hydroxide(0AR) showed more resilience modulus than the mixture of silicate and sodium hydroxide (0.4AR). This means with the addition of silicate the resilience modulus was decreased in Soil-Geopolymer mixtures, hence depicted similar result with UCS.

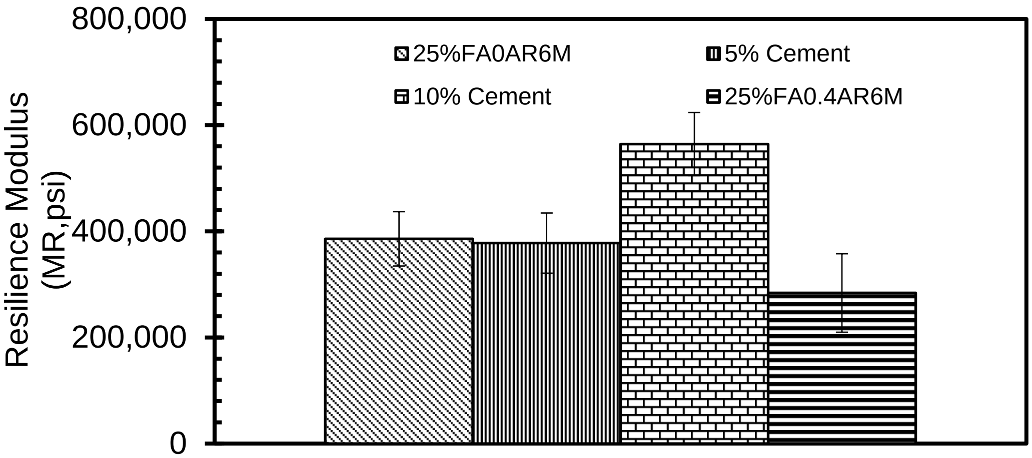

Figure 4-38: Comparison of resilience modulus for soil-cement and Soil-Geopolymer for Soil A

Figure 4-39, represents the resilience modulus of FA based Soil-Geopolymer mixture for optimum combination for B, 10% soil-cement, and 9%cement-9%lime-soil mixtures stabilization. It was observed that all mixtures have similar resilience modulus. Additionally, the moduli values were much lower than the medium plastic Soil A. Interestingly, the UCS of all these mixtures were same. This implies that Soil-Geopolymer mixtures perform the same as the soil-cement and soil-lime-cement stabilized mixtures.

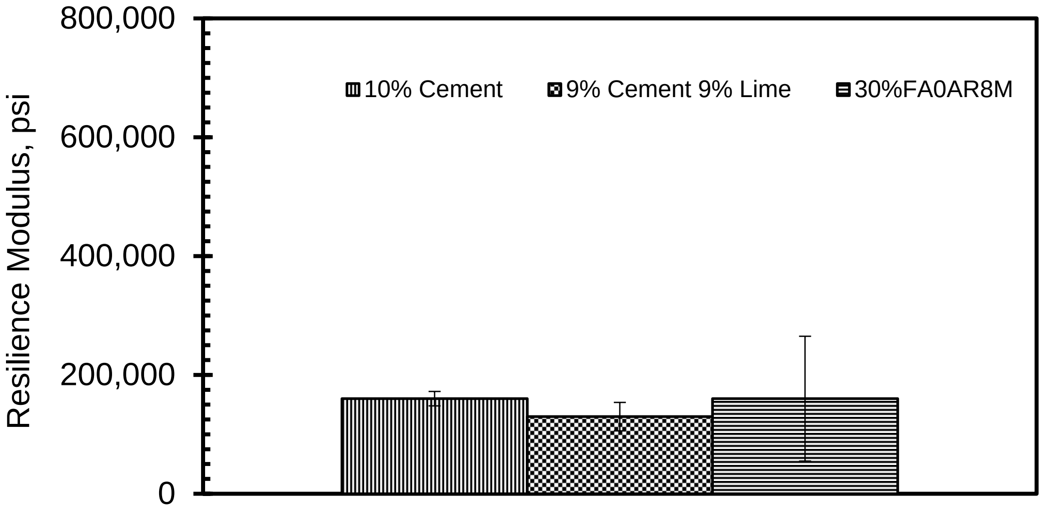

Figure 4-39: Comparision of resilience modulus of the soil- Geopolymer with recommended stabilization process for soil B

From the result, it is observed that resilience modulus of Soil A is higher than Soil B. This might be due to the high plastic nature of the soil B and formation of Geopolymer products. The resilience modulus of the Geopolymer products of Soils A and B were comparable with the corresponding standard recommended soil-cement stabilization. From this experiment, it can be concluded that soil FA Geopolymer can replace the soil-cement stabilization method.

4.9 Permanent Strain

The permanent strain was calculated for the soil-cement of 5% and 10 % cement content and Soil-Geopolymer 25%FA0.4AR6M and 25%FA0AR6M for 1000 cycle of constant cyclic stress of 63.66 psi. The variation of permanent deformation with number of cycles for Soil-Geopolymer and soil-cement for medium plastic soils is shown in Figure 4-40 and Figure 4-41 respectively. It shows that for all sample, the rate of deformation was increased rapidly up to 200 to 400 cycles but after 400 cycles the rate of permanent deformation was decreased and remained almost constant up to 1000 cycles.

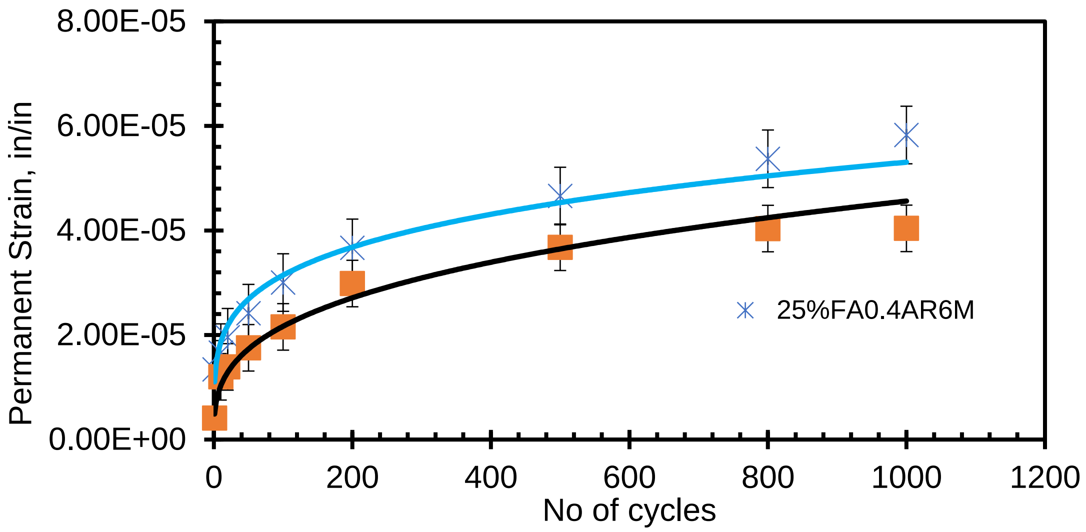

Figure 4-40: Variation of permanent strain for Soil-Geopolymer specimen with no of cycle for Soil A

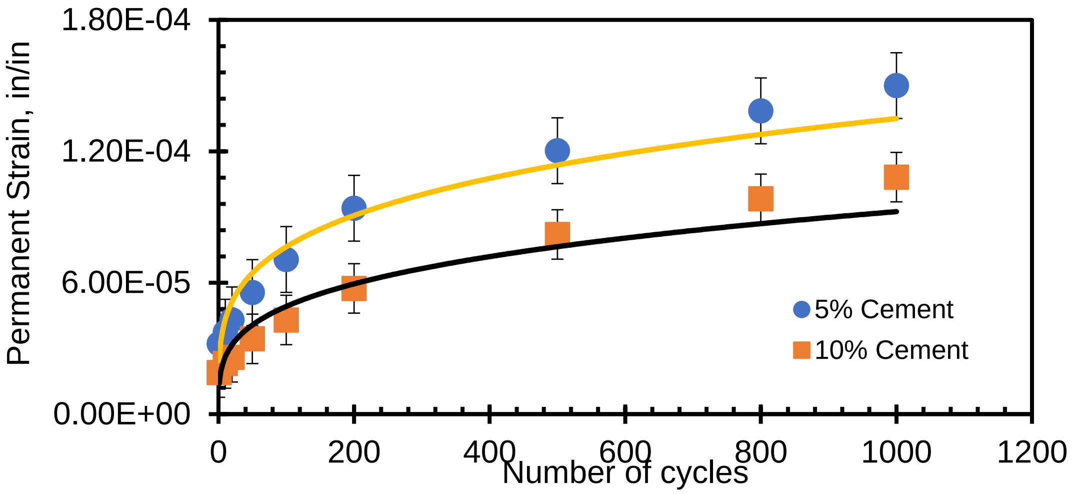

Figure 4-41: Variation of permanent strain for soil-cement specimen with no of cycle for Soil A

The cumulative permanent strain at 1000th cycle of repeated loading test is shown in Figure 4-42. Result indicated that 5% soil-cement was very week in cyclic load as it has higher permanent deformation (1.5e-4 in/in), as compared with other mixtures. In addition, 25%FA0AR6M showed minimum permanent deformation than 25%FA0.4AAR6M and 10% cement. The permanent deformation of 25%FA0AR6M Geopolymer soil was found to have 44%, 168%, and 271% higher than 25%FA0.4AR6M, 10% cement and 5% cement mixtures respectively. This result indicates that soil-Geopolymer performed exceptionally well in stabilizing the medium plastic soils in the alkaline environment, using industrial byproduct fly ash.

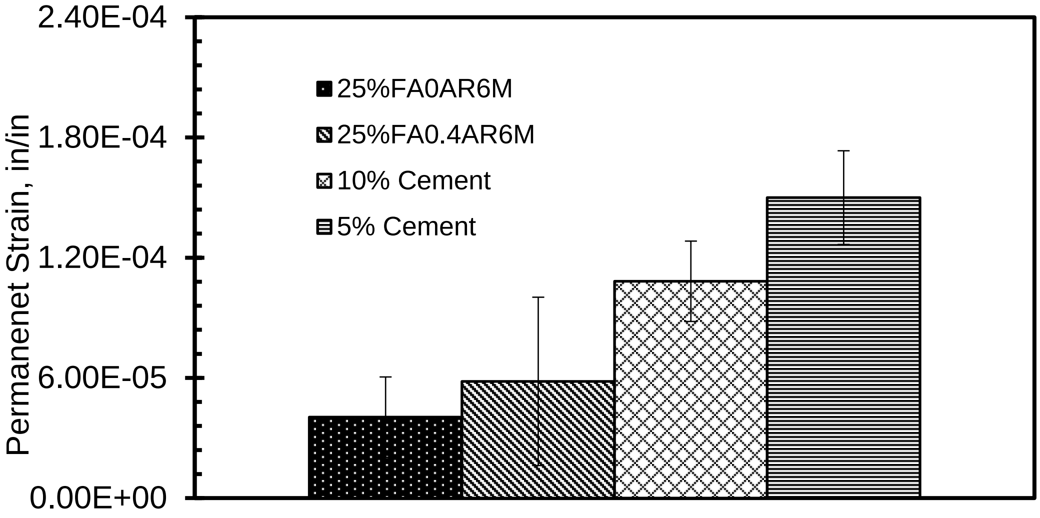

Figure 4-42: Permanent strain for the various mixtures for 1000 cycle of dynamic load for Soil A

The variation of the cumulative permanent strain of the various mixture with no. of cycles for Soil B is presented in Figure 4-43. Figure represents that cumulative permanent strain was increasing with increased number of cycles for all mixture. However, 10% soil-cement mixture indicated less permanent deformation. The permanent deformation of the 10% cement, 9%cement-9%lime and 30%FA0AR8M Geopolymer at 1000th cycle of repeated loading for high plastic soils is represented in Figure 4-44. The figure shows that 30%FA0AR8M Geopolymer showed less permanent deformation than 9% cement-9%lime but showed 130% higher deformation than 10% soil-cement mixture. This implies that 10% soil-cement performed better against repeated loading (i.e. have less permanent deformation). It was observed that the cement with high plastic soil exhibited excessive cracking. In general, it has been recommended that to use lime along with cement, to reduce the plasticity of the soils (Little and Nair 2009, Department of Army 1994). Based on this discussion, it is obvious that the Soil-Geopolymer mixtures performed better than the recommended soil-cement-lime mixtures for high plastic soils.

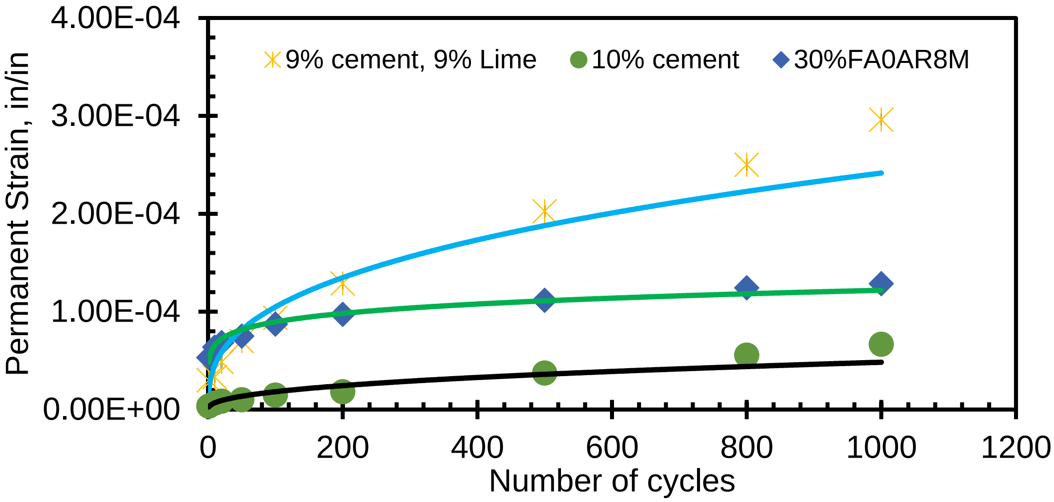

Figure 4-43: Variation of the permanent strain of various mixtures with Soil B

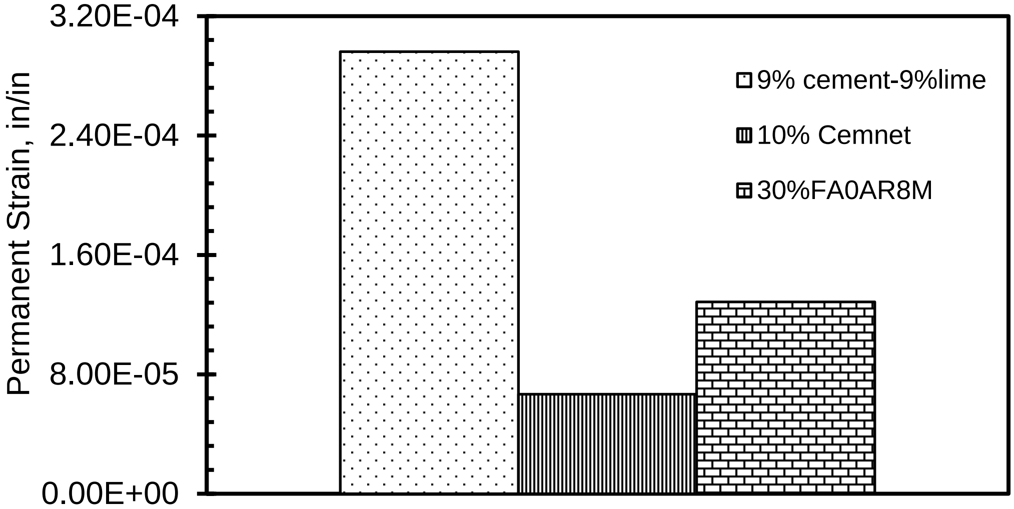

Figure 4-44: Permanent deformation of the various mixture at 1000 cycle of repeated load for Soil B

4.10 Dynamic Modulus

Variation of the dynamic modulus (psi) of the various mixes of Soil A-Geopolymer with load frequency, hz are shown in Figure 4-45. The results show that dynamic modulus of the soil-cement and Soil-Geopolymer mixtures were independent of loading frequency thus exhibited elastic response. It can be seen from the figures that, the dynamic modulus is higher for 25%FA0AR6M mixture than 25%FA0.4AR6M mixtures, indicating that with the addition of sodium silicate the dynamic modulus of Soil-Geopolymer mixture decreases. On the other hand, the dynamic modulus of the Soil-Geopolymer mixtures were between the 10% and 5% soil-cement mixtures.

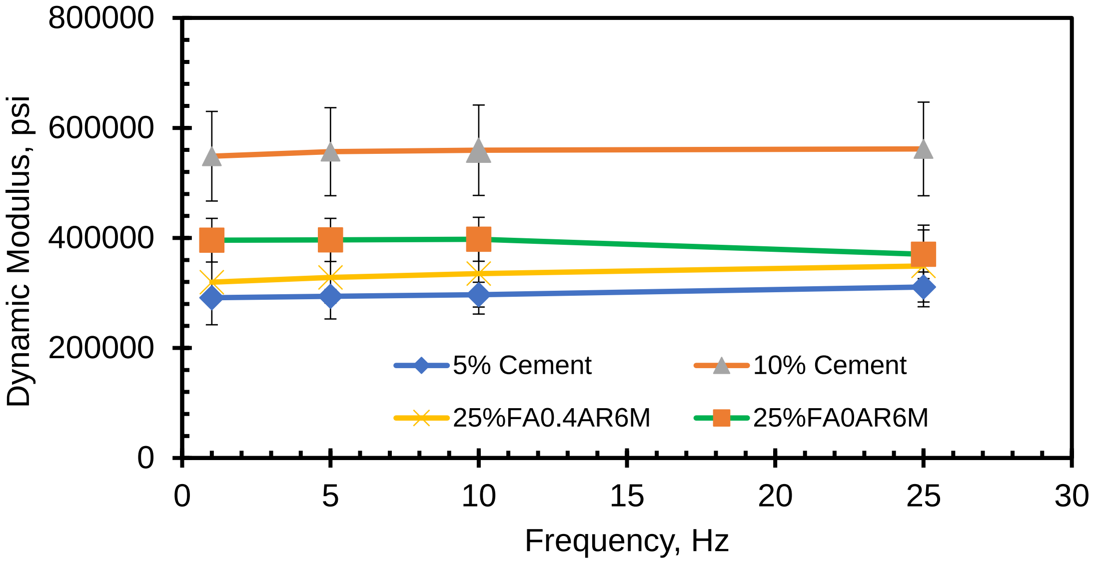

Figure 4-45: Variation of dynamic modulus, psi with test frequency, H. for soil -cement and soil-FA-Geopolymer

The variation of dynamic modulus of the10% soil-cement, soil- 9%cement-9% lime and 30%FA0AR8M mixtures with various loading frequency for soil B are also shown in Figure 4-46. Figure 4-46 depicts that dynamic modulus of the all three mixtures were increased with the loading frequency, Hz. Geopolymer mixtures have lower dynamic modulus value than soil- cement mixtures, even though both mixtures have similar strength. Soil A-Geopolymer has more dynamic modulus than Soil B-Geopolymer, as the strength for Soil A was higher than soil B, it was obvious that dynamic modulus of the Soil A-Geopolymer showed higher dynamic modulus value.

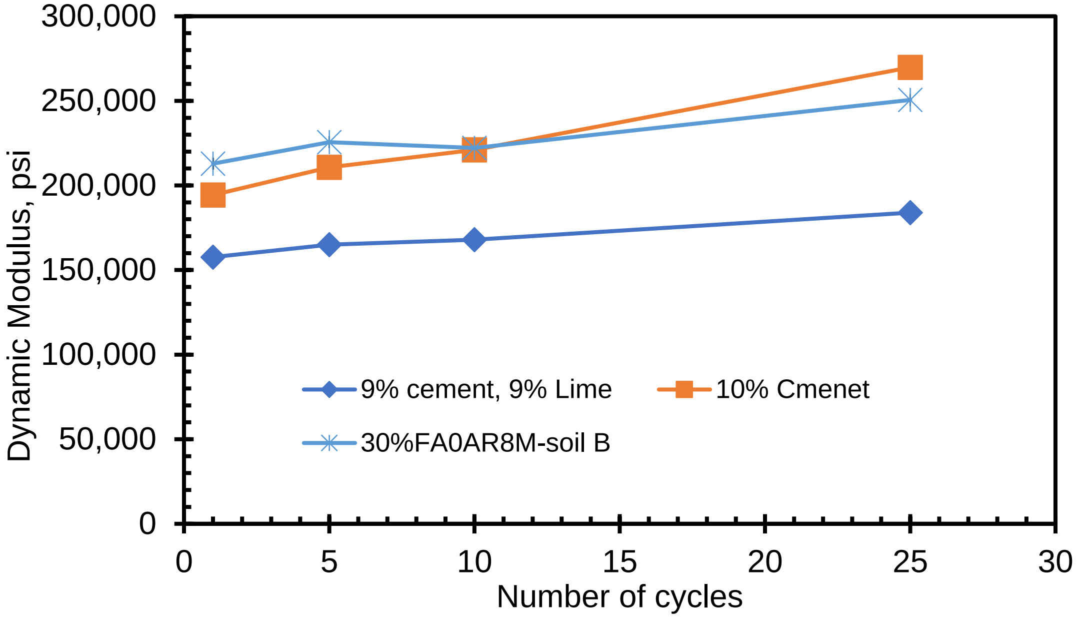

Figure 4-46: Variation of dynamic modulus, psi with test frequency, Hz for soil-cement and soil-FA-Geopolymer for soil B

The response of mixture under dynamic loading system (cyclic stress) is shown in Figure 4-47, which depicts that strain followed the same path as followed by stress, i.e. there was no considerable time lag between strain response with respect to cyclic stress. Moreover, indicating that Soil-Geopolymer mixtures showed only elastic behavior. This result suggests that for Soil-Geopolymer dynamic modulus was similar test as resilience modulus in cyclic loads. Since specimen was indicating immediate response in strain with cyclic stress, it can be assumed that Soil-Geopolymer has no or very low viscous properties. The relationship between dynamic modulus under cyclic loads and resilience modulus under repeated load as shown in Figure 4-48. This figure depicts the relationship between resilience modulus and dynamic modulus of the various mixture of soil-cement stabilization and optimized Soil-Geopolymer mixtures for both soils. Result depicted that resilience modulus was highly related with the dynamic modulus with 0.96 correlation coefficient.

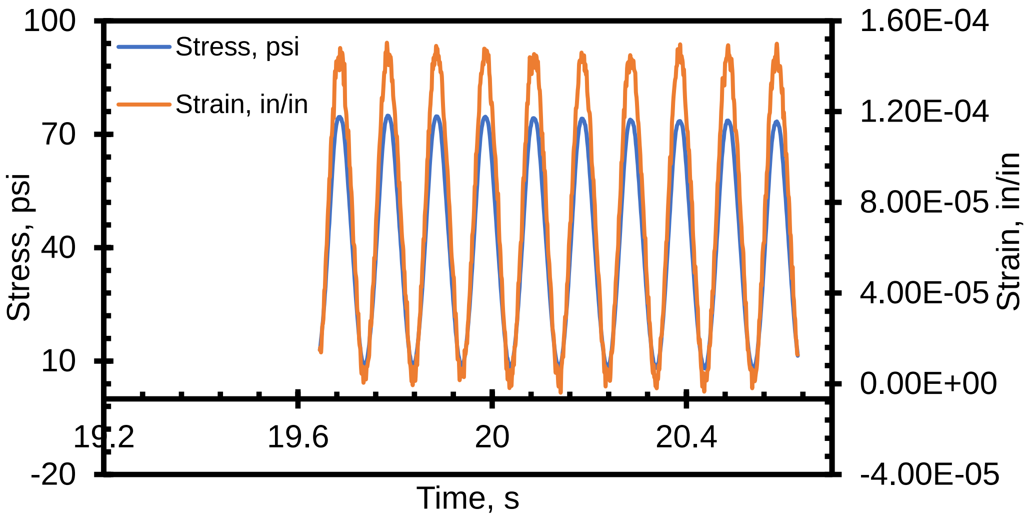

Figure 4-47: Variation of stress and strain with time for 25 Hz frequency for 25%FA0AR6M

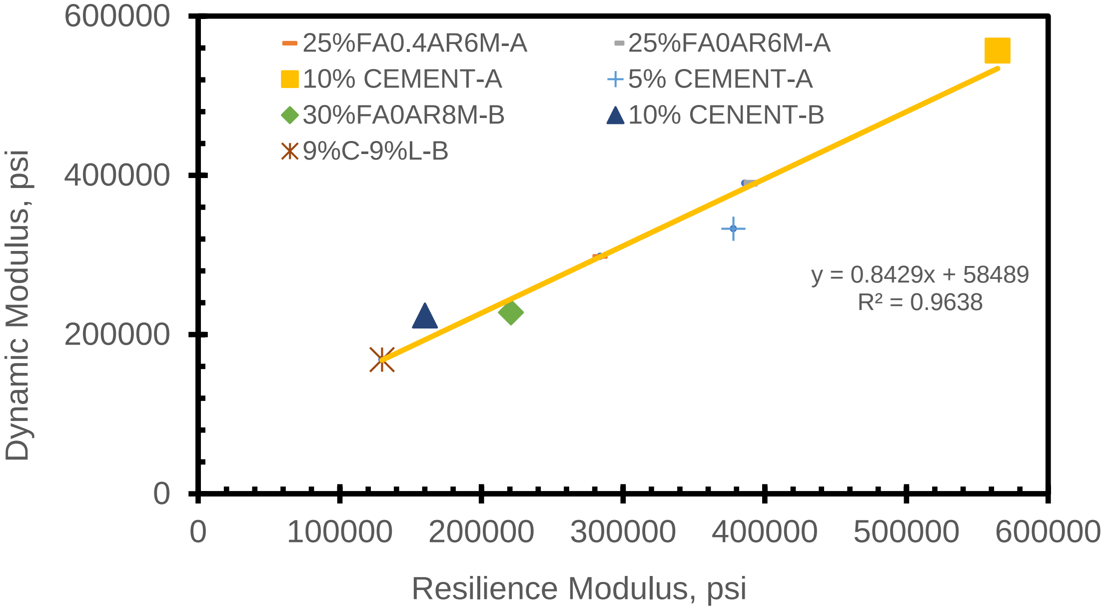

Figure 4-48: Relationship between resilience modulus and dynamic modulus of Soil-Geopolymer mixture and soil-cement for both soils

4.11 Scanning Electron Microscope analysis

The scanning electron microscope (SEM) analysis were conducted on the soils, FA, and Soil-Geopolymer mixtures Various Soil-Geopolymer mixtures at different curing condition were studied under SEM including 3 days' room temperature, 12-hour heat curing at 60℃ , 48-hour heat curing at 60℃. Figure 4-49(a, b c) show the SEM micrographs of Soil A, Soil B and FA respectively. This figure depicts that soil particles are flaky particles in nature. However, FA are rounded particle with various size distribution from 1 µm to 40 µm.

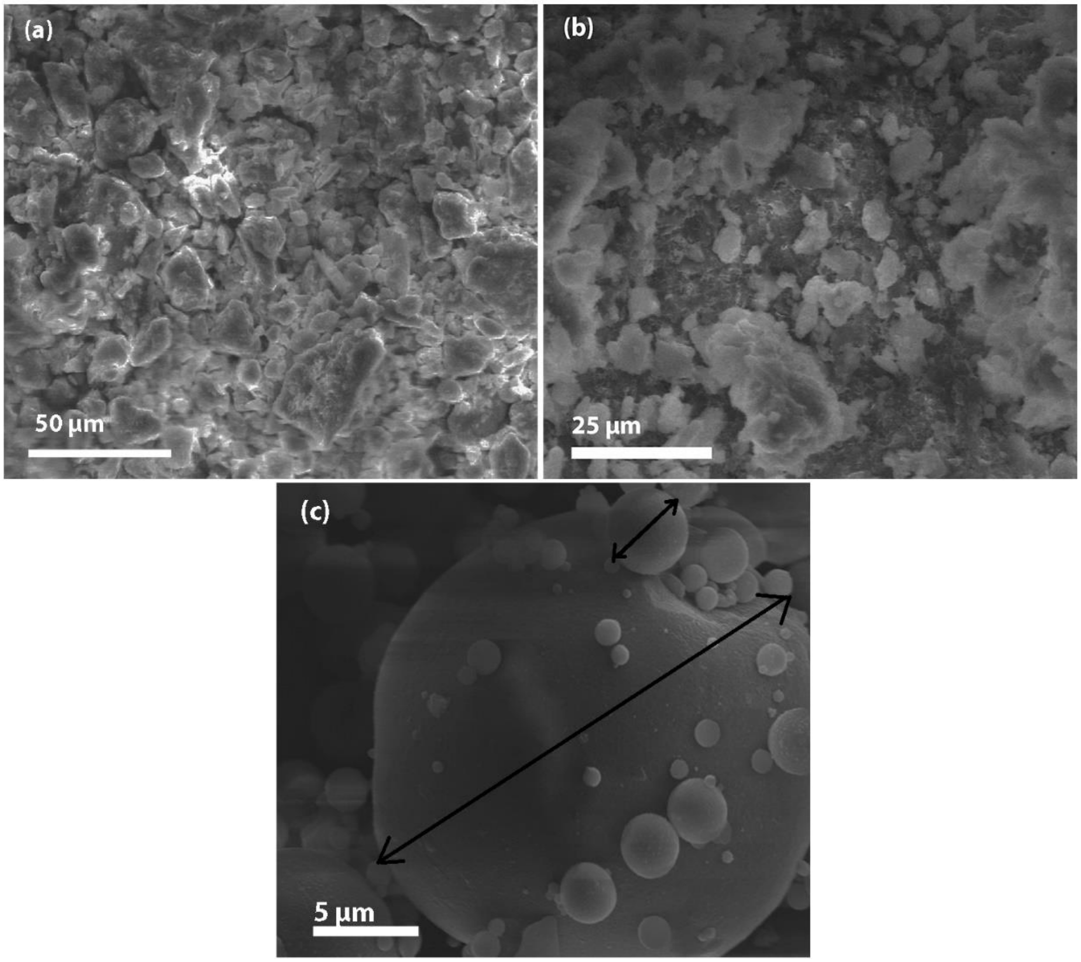

Figure 4-49: SEM Micrographs of Soil A, Soil B and Fly ash

Figure 4-50 (a, b, c, d) represents the SEM micrographs of 25%FA0AR6M Soil-Geopolymer at various curing conditions. Figure 4-50(a) depicts SEM images of 25%FA0AR6M at 3-days of room temperature curing conditions. The early age micrographs are significant in terms of analyzing its mixing and compaction behavior. From the observation of Figure 4-50(a), spherical FA and flaky soil particles coated with alkaline solutions were observed, suggesting FA are uniformly distributed in the mixture. Also, unreacted fly ash was detected in the images depicting that reaction might have initiated however no significant changes are observed in morphology of Soil-Geopolymer mixtures.

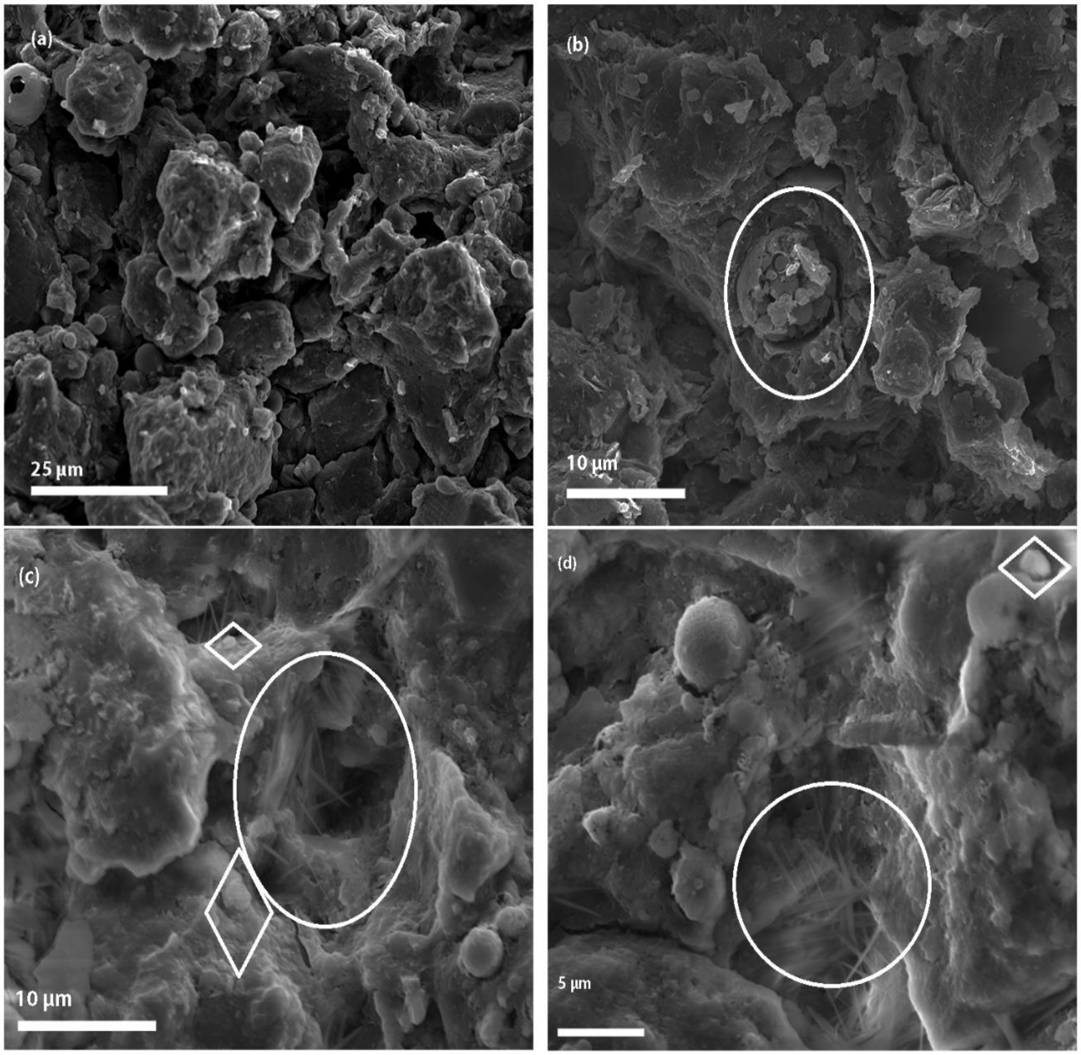

Figure 4-50: SEM analysis, the evolution of geopolymer product for25%FA0AR6M at various heat curing time(a) 3-days of room temperature, (b) 12 hour at 60℃ curing, (c)48 hour at 60℃ curing at 10µm scale and (d) 48 hour at 60℃ curing at 5µm. (diffused FA is indicated with diamond shape)

Figure 4-50 (b) represent SEM images of NaOH activated Soil A Soil-Geopolymer mixtures oven cured at 60℃ for 12 hours. From the figure, it can be seen that, the Soil And fly ash particles are adhering in the a geopolymer matrix, as well as some other unreacted FA particles are detected. Figure 4-50 (c at 10 µm scale and d at 5 µm scale) represent 48-hour heat curing at oven at 60℃ 25%FA0AR6M mixtures. In the figure, glassy feature and diffused FA were observed. In addition, hydration shell, alkali coated fly ash, are noticed suggesting the reaction is still happening and will continue further. This analysis suggests that the microstructure of the Soil-Geopolymer are changing over increased heat curing periods for NaOH activated Soil A-Geopolymer.

Additionally, to observe the reaction mechanism of the Soil A Soil-Geopolymer with NaOH only and alkaline mixtures of sodium silicate and sodium hydroxide activator, another sample was prepared with alkaline ratio(AR) of 0.4, and analyzed under SEM analysis as presented in Figure 4-50 (a, b c and d). Figure 4-50(a)depicts the SEM images at 3 days' room temperature curing for the mixture of 0.4 AR. This figure shows unreacted FA, cluster of FA and flaky soil particle along with geopolymer matrix. The FA seems uniformly distributed mixture however cluster of FA also can be observed. These clusters of FA are coated with alkaline solutions and trying to form hydration products in 3 days of room temperature curing. This implies that reaction has been initiated.

The SEM image at 12 hours of heat curing in oven at 60℃ as in Figure 4-51 depict that the amount of unreacted FA has decrease, as compared with 3-days room temperature curing Figure 4-51(a). Also, some FA and soils particles were adhering and trying to form a bond between them, as shown in Figure 4-51(b). Figure 4-51(c) exhibits the SEM micrograph at 10 nanometer scales for 0.4AR Soil A Soil-geopolymer mixtures at 48 hours curing in oven at 60°C. Sufficient amount of glassy feature and diffusing FA can be observed. The zoomed image of rectangular part of Figure 4-51 (c) is also shown in Figure 4-51(d), which depicts geo-polymerization reaction is happening with formation of gel like morphology and some pointed glassy features.

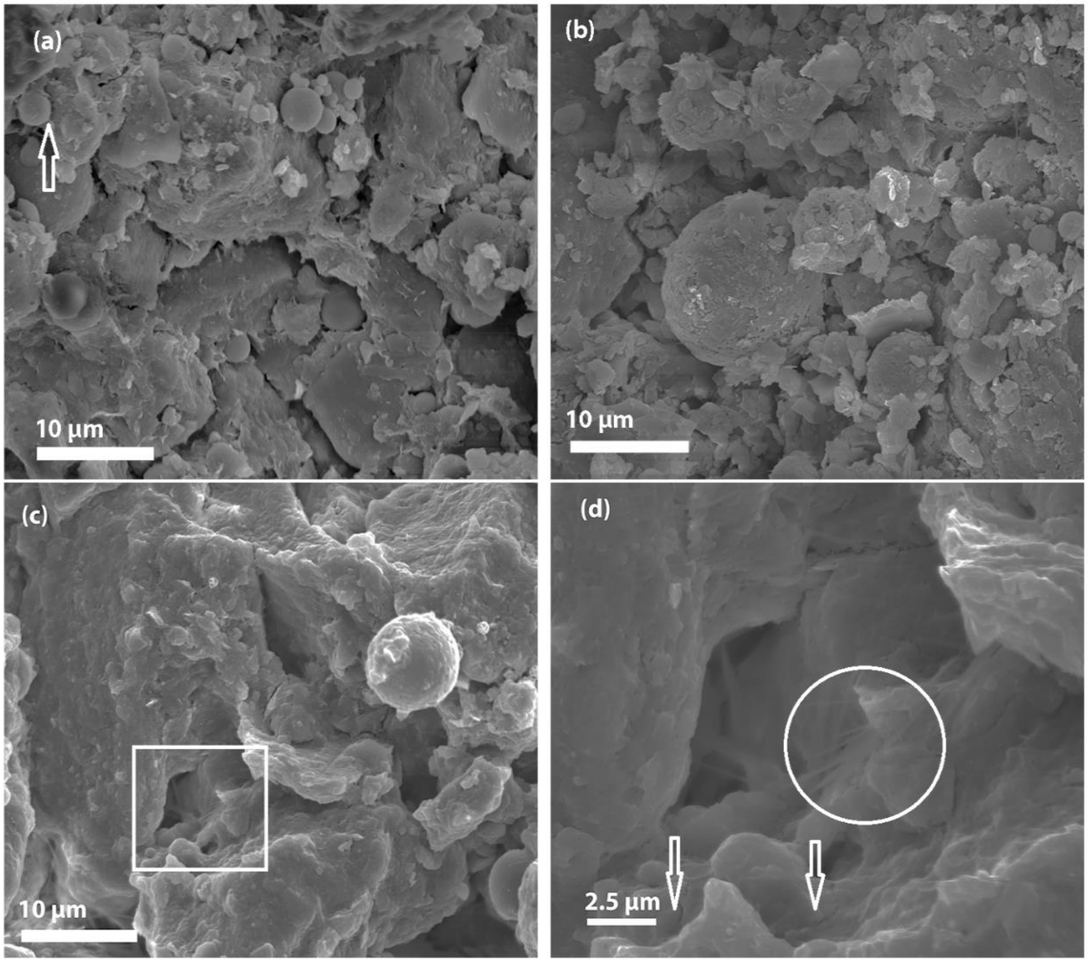

Figure 4-51: SEM analysis, the evolution of geopolymer product for25%FA0.4AR6M at various heat curing periods. a) 3-days of room temperature, (b) 12 hour at 60°C curing, (c)48 hour at 60°C curing at 10µm scale and (d) 48 hour at 60°C curing at 2.5µm at rectangle in (c) (FA and diffused is indicated with arrow and pointed features ere indicated with circle

Furthermore, some FA particles with spherical shape are diffusing in alkali medium, suggesting that the reaction between soil-FA alkali and Soil Alkali are in progress. This diffused FA might have formed geopolymer gel leading to formation of pointed feature like structure (may be geopolymer). Also, presence of unreacted FA and hydration shell in the process of diffusing. This suggest that the reaction will continue for long period of time, enhancing the mechanical and durability characteristics of Soil-Geopolymer mixtures. It is also found that glassy like feature was more convincing in 48 hours of heat curing mixtures. In addition, small cracks were also observed in the matrix, indicating shrinkage cracks is happening in the samples. From SEM analysis, 0.4 AR mixture (addition of sodium silicate) showed similar microstructure as 0AR mixture, indicating that sodium silicate does not seem effective in the formation of geo-polymerization.

The microstructure and morphology of Soil-Geopolymer mixtures made from high plastic soil (Soil B) was also investigated under SEM, as presented in Figure 4-52 (a, b, c, d, e, f). The formation of geopolymer for Soil B(30%FA0AR8M) at 3 days of curing at room temperature is illustrated in Figure 4-52 (a), it shows that the fly ash seems uniformly distributed in the mixtures but most of them are in unreacted form. This indicates that reaction is not significant in first 3 days of room temperature curing. This trend is similar to that of Soil A 0AR and 0.4 AR mixtures.

Figure 4-52Figure 4-52 (b), depicts the SEM image of 25 um scale at 12-hours of heat curing. In this image, very few fly ash particles were observed as unreacted, however compacted flaky soils particle can easily be observed. The microstructure of the mixtures is also changing with increased temperature oven curing of mixture for 12-hour at 60℃.The Figure 4-52 (c), shows the 5um scale of Figure 4-52 (b) in selected rectangle region. The zoomed figure indicate that the FA was coated with geopolymer gel and small elongated particles were also observed in image along with glassy geo-polymer gel. Figure 4-52 (d) exhibit the 10-um scale micrograph of 48-hour heat cured sample. It can be observed that it consists of unreacted FA but appears to have smooth and stronger geopolymer matrix. Similarly, few small elongated particles are observed along with some cluster of FA. This decreased number of elongated particles suggest that these particles might have interconnected to form continuous gel like features as shown in Figure 4-52(e). Similarly, the presence of unreacted FA and small hydration cell in the image suggest that the reaction is still in progress. This implies that the performance of Soil-Geopolymer can increase over time due to enhanced mechanical and durability characteristics. The scale is reduced to 5um to observe the gel and elongated particle shown in Figure 4-52 (f). The figure illustrates that the hydration shell was surrounded by geopolymer gel. However, in the 5um scale image could not observed any instead of micro cracks but consisted of small voids.

Soil A with 0AR and 0.4 AR showed almost similar reaction progress and variation in microstructure. Indicating formation of glassy and pointed features, as shown in Figure 4-50(d) and Figure 4-51(d). However, for Soil B we observed few glassy structures but could not observed pointed structure at 48 hours of heating, as illustrated in Figure 4-52(e and f). However, in soil we observed small elongated white particles, and assumed that these elongated particles interconnected and diffused to formed glassy products. This variation of reaction products might be associated with composition of soils, indicting soils also took part in formation of geopolymer products. From these analysis, it can be concluded that the geopolymer reaction is continuous and progressive process which might happen for extended period.

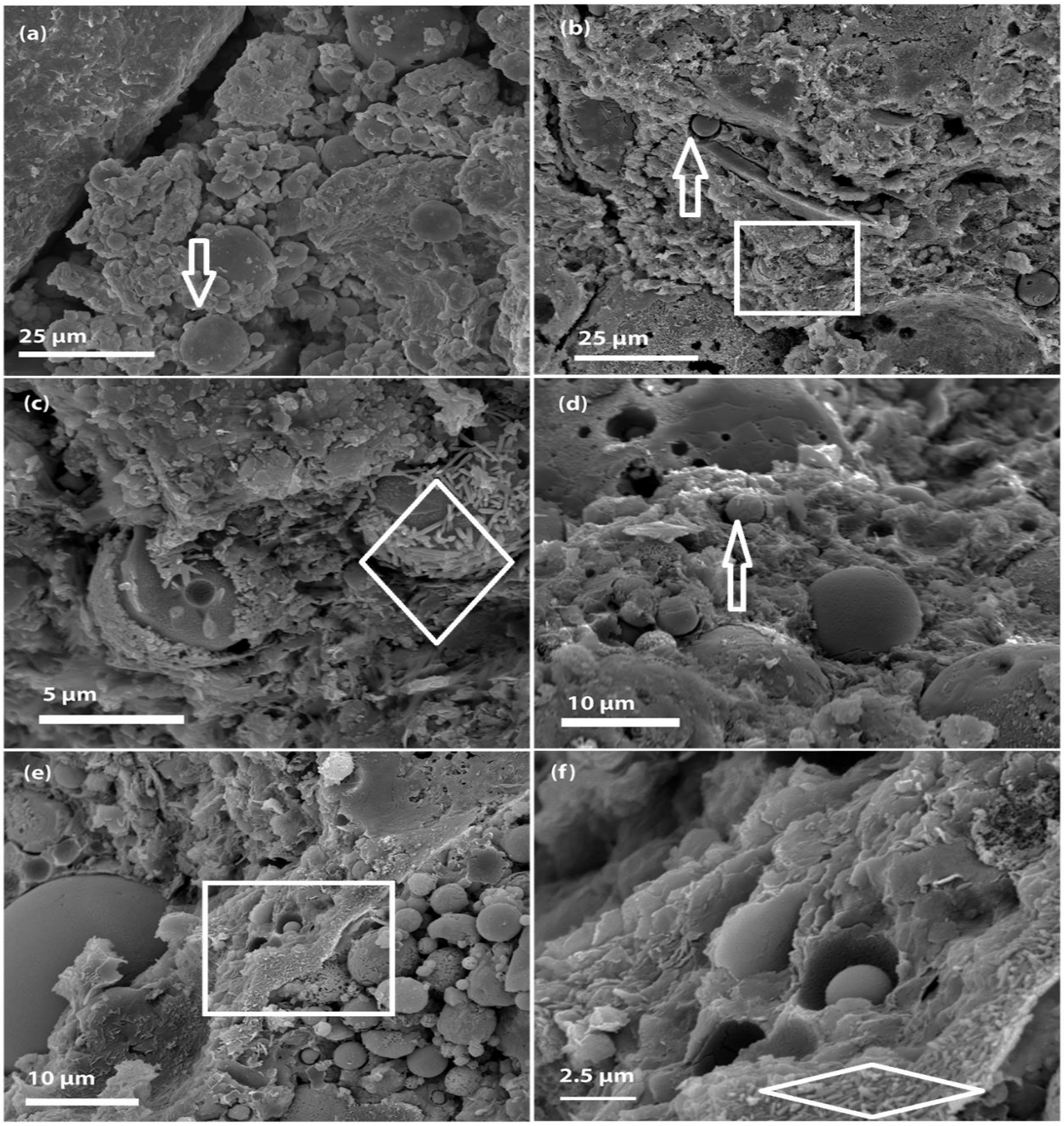

Figure 4-52: SEM image for the 30%FA0AR8M for soil B to analysis effect of heat curing or formation of geo-polymer. a) 3-days of room temperature, (b) 12 hour at 60℃ curing at 25 µm, (c)12 hour at 60℃ curing at 5µm scale, (d) 48 hour at 60℃ curing at 10µm (e)48 hour at 60℃ curing at 10µm scale and (f) 48 hour at 60℃ curing at 2.5µm (FA, elongated particles are indicated by arrow and diamond respectively)

5. CONCLUSION AND RECOMMENDATION

5.1 Summary

The medium plastic soil and high plastic soils were collected from Lafayette, LA and ALF (Accelerated Loading Facility) facility in Baton Rouge, LA. The soils were mixed with FA, and Atterberg limit test and moisture-density relationship tests were conducted in the laboratory. The mixing and compaction procedure were defined based on modified proctor test. Based on literature review and preliminary experiment, the experimental design matrix was developed for Unconfined Compressive strength (UCS). Regression analysis was conducted to develop the UCS regression model as a function of FA, AR and Concentration of NaOH. Sensitivity analysis was conducted and the optimum mixture for both soils were determined, based on minimum strength criteria of CTD and CSD, recommended by LA DOTD. The selected mixtures from both soils and recommended CTD and CSD mixture were prepared and tested for performance validation. The resilience modulus test and dynamic modulus test and durability tests were conducted on the selected Soil-Geopolymer mixtures and recommended soil-cement mixtures. The morphology and microstructure of the selected Soil-Geopolymer mixtures were studied for various period of heat curing at oven at 60℃, using Scanning electron microscope (SEM) and electron diffractive spectroscopy (EDS). The results were compared with the recommended practice of soil stabilization and conclusions and recommendations were drawn based on result and analysis.

5.2 Conclusion

Based on the result and analysis, the following conclusions were established:

5.2.1 Mechanical Characteristics

- The moisture-density relationship was established for soil-fly ash mixtures. MDD of medium plastic soil-fly ash mixtures were decreased with the addition of fly ash, however, for high plastic soil, MDD was increased. In addition, optimum moisture content was not affected for both the medium and the high plastic soils with the addition of FA.
- The UCS of the medium plastic soil was increased with the addition of FA. But for high plastic soil, strength was higher at high FA content of 30%, whereas at a low FA content the increase in strength was not significant.
- It was found that with the increase of AR, UCS of the Soil-Geopolymer mixtures decreased for both soils, suggesting sodium silicate is not effective in the formation of Soil-Geopolymer.
- The strength of the Soil-Geopolymer mixture increased with increased concentration of NaOH for the medium plastic soils. However, high plastic soils did not show clear increase and decrease in strength with increasing concentration of NaOH.
- The linear relationship between Young’s modulus of elasticity and UCS of Soil-Geopolymer were established. It was shown that the Young’s modulus of elasticity increased with higher UCS value of the mixtures.
- The Fracture energy showed little or no effect with FA, AR and concentration of NaoH. In addition, it exhibited high variation within the triplicate sample.
- Maximum failure strain of the mixture showed linear relationship with UCS. With the increasing UCS of mixture, the failure strain decreased.

5.2.2 Statistical analysis

- The unconfined compressive strength of Soil-Geopolymer was predicted using regression model as a function of FA, AR and molarity of NaOH for both soils. The coefficient of determination for Soil A and Soil B were found to be 0.89 and 0.79 respectively.
- Sensitivity analysis was conducted using the developed regression models. The result illustrated that, UCS increased with increased content of FA. However, UCS decreased with increasing AR for both soils used in the study. In the case of concentration of NaOH, Soil A showed increased in UCS with increasing concentration of NaOH.
- The optimum mixtures for Soil A and Soil B were found to be 25%FA, 0AR, and 6M and for soil B 30%FA, 0AR, and 8M respectively.

5.2.3 Comparison between soil-cement and Soil-Geopolymer mixtures

Based on various test conducted and comparison between selected Soil-Geopolymer mixtures, CSD and CTD for both soils, following conclusions were drawn for each test.

5.2.3.1 Unconfined compressive strength (UCS)

- For low plastic soil, Soil-Geopolymer mixture exhibited higher compressive strength than 10% soil-cement and 5% soil-cement mixtures.
- The UCS of Soil-Geopolymer mixtures was similar with 10% cement and 9%cement-9%lime mixtures, for high plastic soil.
- The Soil-Geopolymer mixtures passed the criteria as established by LA-DOTD for cement stabilized design (CSD, 300 psi) and cement treated design (CTD,150 psi).

5.2.3.2 Resilience Modulus and Dynamic modulus

- The resilience modulus of Soil-Geopolymer mixtures were found to be less than 10% soil-cement, even though 10% soil-cement mixture has lower strength. In addition, the addition of sodium silicate was not significant for repeated loading, as compared with sodium hydroxide only mixtures

- Soil-Geopolymer mixture made using low plastic soil showed less cumulative permanent strain at 1000 cycle of repeated load, than 10% soil-cement mixture, however, resilience modulus of 10% soil-cement was higher than Soil-Geopolymer mixtures.
- For high plastic soil, selected Soil-Geopolymer mixture exhibited higher cumulative permanent strain than 10% soil-cement, but shows less permanent strain than 9% cement-9% lime mixtures.
- The dynamic modulus of the Soil-Geopolymer mixtures increases with load frequency, for high plastic soil. However, for the medium plastic soil the dynamic modulus did not show any relation with load frequencies.
- Dynamic modulus of Soil A is found to be higher than soil B. Also, linear relationship between the dynamic modulus and the resilience modulus was established for various mixtures.

5.2.3.3 Durability Characteristics

- The 5% soil-cement made with Soil A failed in durability test. However,10% soil-cement mixture and optimum Soil-Geopolymer mixtures, 25%FA0.4AR6M and 25%FA0AR6M passed the wet and dry durability test.
- The mass loss of the Soil-Geopolymer mixtures was found to be comparable to that of 10% cement. The final moisture content after 12 cycles of soil-cement mixtures were found to be less than OMC but for Soil-Geopolymer mixture the final moisture content was higher than OMC of mixtures.

- The soil-cement mixtures showed less volume change then Soil-Geopolymer, indicating Soil-Geopolymers mixtures have more swell potential.

5.2.4 Morphology and Microstructure of Soil-Geopolymer mixtures

- From the morphology study, it is concluded that spherical FA and flaky soil particles were uniformly distributed in the mixtures.
- The microstructure of the Soil-Geopolymers mixtures are changing significantly with increased curing duration at elevated temperature.
- In the SEM microstructure study, glassy gel, small elongated particles and pointed glassy features were detected at 48 hours of heat curing condition, suggesting these features might be Geopolymers products.
- The unreacted FA at 48 hours of heat curing at 60°C condition suggested that reaction was still in progression, therefore enhanced mechanical and durability characteristics with time is expected in Soil-Geopolymer mixtures.

From this study, weak subgrade soils were successfully stabilized using alkali activated fly ash. The UCS of the soils were significantly increased for Soil-Geopolymer mixtures, and fulfilled the CSD and CTD criteria recommended by DOTD, Louisiana. The result showed that Soil-Geopolymer mixtures performed satisfactorily in repeated loading and wet and dry durability test as compared with soil-cement mixture as recommended by DOTD, LA. The morphology and microstructure study show the increased geopolymer product with increased heat curing duration at elevated temperature and possible improved long-term performance. This research derived the alternative method for stabilization of subgrade and base materials, using environment friendly, cost effective FA based Soil-Geopolymer products.

5.3 Recommendation

Based on laboratory data and conclusion, following recommendation are made;

- First, hand mixing procedure may not be effective to make the homogeneous mixture, especially for high plastic soils. In such case the soil forms small spherical patches, entrapping water. This makes hand mixing very difficult so it was recommended to use mechanical mixing process.
- It is recommended to investigate the effect of mixing and processing time, on unconfined strength of the mixtures.
- It is recommended to use optimum content of the fly ash, AR and concentration of sodium hydroxide for stabilization of soils for comparable properties of the soils.
- Strength was decreased with addition of silicate; further investigation is necessary to validate effect of alkaline ratio on formation of Soil-Geopolymer mixtures. This study concluded sodium silicate was not necessary to produce Geopolymer products in soils.
- It is suggested to study effect of silica and alumina content in soils in formation of Geopolymer products in alkaline environment. This study will validate reaction behavior of soils in alkaline medium.
- It is recommended to conduct SEM analysis at different elevated temperature to determine the effect of temperature in the Soil-Geopolymer.
- It is recommended to conduct EDS on glassy and elongated feature to validate formation of Soil-Geopolymer.

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APPENDIX

Table 1: Variation of moisture content various sample of Geopolymer and soil-cement for Soil A

Moisture content for each cycle				
Cycle	10%Cement	5%cement	25FA0AR6M	25FA0.4AR6M
1	12.95	11.43	15.90	16.91
2	13.07	11.38	15.85	17.06
3	12.74	11.28	15.87	17.06
4	12.74	11.36	15.92	17.04
5	12.66	11.36	15.98	17.10
6	12.72	11.35	16.01	17.01
7	12.80	11.39	15.55	16.81
8	12.77	11.37	16.03	16.21
9	12.79	11.40	16.02	16.45
10	13.06	11.45	16.09	15.89
11	13.47	11.53	16.10	17.06
12	13.68	11.66	16.13	17.16
Mean	12.95	11.41	15.95	16.81
% diff with OMC	-0.05	-1.59	2.95	3.81

Table 2: Volume change in each first cycle of test for various mixture for Soil A

Cycle	10% Cement	5% cement	25FA0AR6 M	25FA0.4AR6M
	cu in	cu in	cu in	cu in
1	0.209	0.138	0.042	0.034
2	0.209	0.138	0.042	0
3	0.126	0.275	0.235	0
4	0.371	0.727	0.72	0.194
5	0.062	0.096	1.266	0.097
6	0.479	0.373	0.195	0.194
7	0.441	0.297	0.485	0.277
8	0.31	0.564	0.222	0.277
9	0.337	0.268	0.084	0.277
10	0.57	0.096	0.09	0.291
11	1.403	0.93	0.929	1.15
12	1.641	1.827	0.929	1.388
Average	0.513	0.478	0.437	0.348

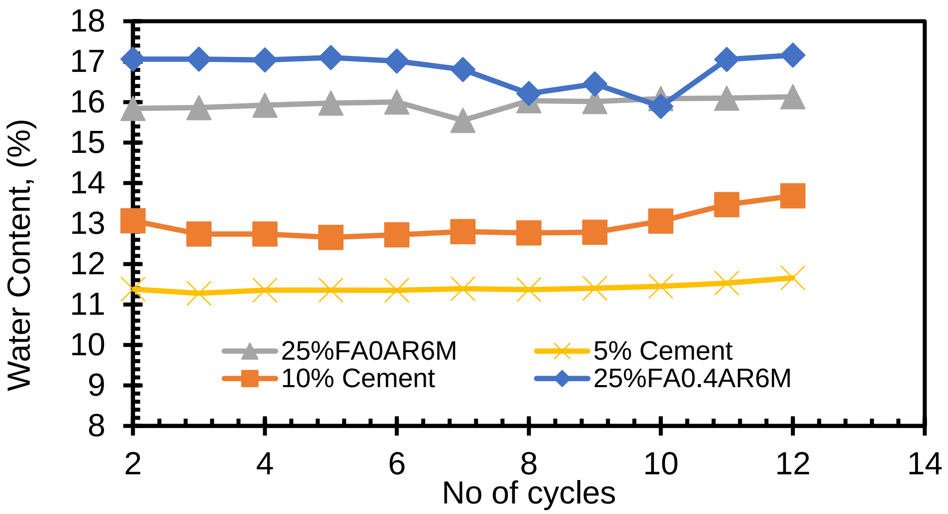

Figure 1: Variation of moisture content in durability test form various sample

Table 3: Soil loss by mixtures during durability test for Soil A

Weight loss Sample				
Cycle	10% Cement	5% cement	25FA0AR6M	25FA0.4AR6M
1	1.6	1.4	1.9	1.8
2	0.7	2.8	0.7	1.2
3	1.5	1	0.9	1
4	1.7	2.3	1.3	2.1
5	0.8	2.9	1.5	2.2
6	1.3	1.8	1.1	1.9
7	1.2	4.9	2.4	0.8
8	1.7	2.7	1.3	1.2
9	1	4.4	1.5	1
10	2	7.1	3.3	1.6
11	3.5	12.6	3	4.6
12	4	21	6	6
Total	21	64.9	24.9	25.4
%	1.19	3.68	1.41	1.44

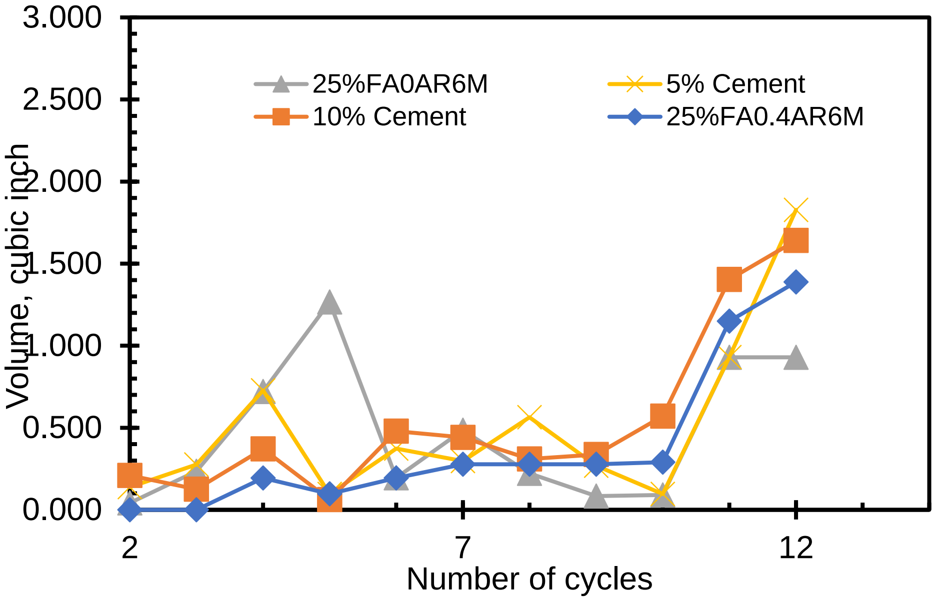

Figure 2: Volume change due to water absorption in each cycle in durability test for Soil A.

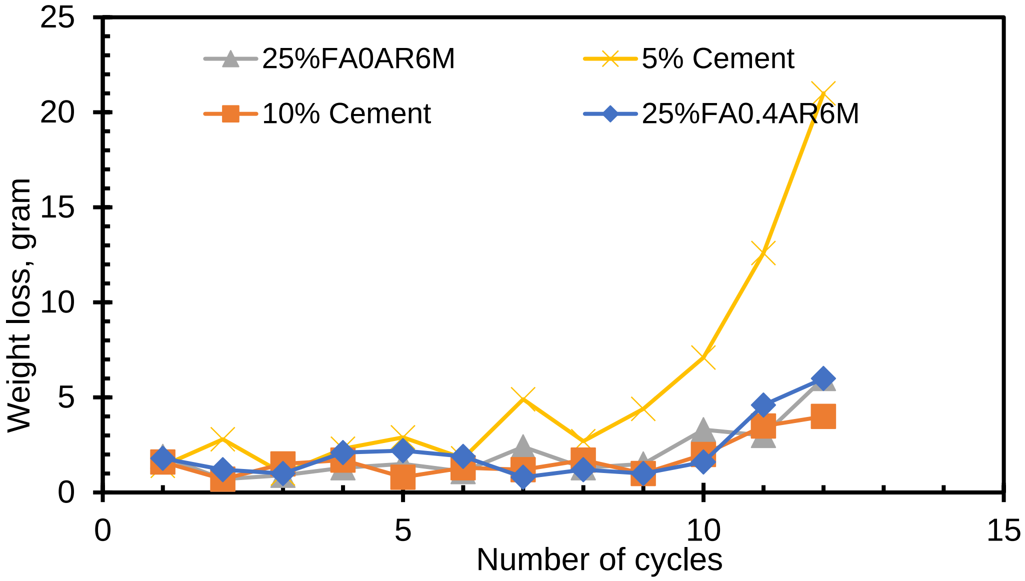

Figure 3: Cumulative weight loss in durability test for Soil A-cement and Soil-Geopolymer

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Thesis Director: Dr. Mohammad Jamal Khattak
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ABSTRACT

In this research study, high plastic and medium plastic soils were used in evaluating the physical and mechanical properties of fly ash-soil Geopolymer mixtures. Sieve analysis and Atterberg limit tests were conducted to classify the soil. Class F fly ash, an industrial byproduct, was used in the alkaline environment of a mixture of sodium silicate and various concentration of sodium hydroxide to prepare fly ash based Soil-Geopolymer mixtures. The modified proctor test was conducted to investigate the moisture-density characteristics of mixtures. Based on literature search and preliminary experiments, the experimental design matrix of was developed by using various combination of fly ash content, alkaline ratio, and concentration of sodium hydroxide. Several mixtures based on the experimental design matrix were compacted, cured at elevated temperature and tested for unconfined compressive strength.

Regression analysis was conducted to develop regression models to optimize and conduct sensitivity of the compressive strength in relation to mix variables of Soil-Geopolymer mixtures. Based on sensitivity analysis and UCS criteria recommended by DOTD Louisiana, two mixtures were selected for further evaluation along with Standard soil-cement mixtures. The resilience modulus, dynamic modulus, and durability test were conducted for selected Soil-Geopolymer mixtures and standard soil-cement mixtures, as recommended by LA DOTD.

The result of the study showed that the alkali activated soil-fly ash mixtures fulfilled the compressive strength criteria of cement treated design (CTD) and cement stabilize design (CSD) of 150 psi and 300 psi, as recommended by LA DOTD. Also, the results indicated that Soil-Geopolymer showed satisfactory performance under compressive strength test and dynamic loading tests. Further, the Soil-Geopolymer mixtures passed wet and dry durability test criteria. Based on mechanical and durability characteristics evaluated it can be recommended that Soil-Geopolymer mixtures studied in this study has an immense potential to be used as pavement subgrade, subbase, and bases.

BIOGRAPHICAL SKETCH

Bikash Adhikari was born in January 1991 in Nepal. He received his bachelor's degree in Civil Engineering from Institute of Engineering, Paschamanchal Campus, Pokhara, Nepal in 2013. He worked as Site Engineer and Environmental Engineer for two years and came to the United States to pursue a Master's degree in Civil Engineering at the University of Louisiana at Lafayette in 2015. He graduated with Masters of Science degree in Civil Engineering in August 2017.