GUIDELINES FOR CHEMICALLY STABILIZING PROBLEMATIC SOILS

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Final Report

prepared for
THE STATE OF MONTANA
DEPARTMENT OF TRANSPORTATION

in cooperation with
THE U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

April 2020

prepared by
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Boise, ID

RESEARCH PROGRAMS

MDT

Guidelines for Chemically Stabilizing Problematic Soils

Final Report

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16. Abstract

The main goal of this project was to establish protocols for conducting efficient chemical stabilization design for problematic soils with and without soluble sulfates within the state of Montana. A major portion of chemical stabilization protocols involves the selection of type and amount of the additives. The proper selection of type and concentration of additive for a given soil should consider, the complex interactions between the mineralogy of the materials and additives, the presence or absence of moisture, and the method of construction and curing. Keeping this in mind several tasks were undertaken to meet the project goal. The study focused on lime and cement as additives and six different soils with varying geology were selected from different parts of Montana. This report contains an overview of the tasks, followed by a summary of the results obtained, and a discussion on the results. The results indicate that chemical stabilization can be a viable alternative for all soils tested in this research especially for short-term applications. However, for long term applications, only three out of six soils showed satisfactory performance.

17. Key Words
Problematic soils, soil stabilization, soil tests, sandy soils, clay soils, soil types, limestone soils, soil remediation, additives, calcium oxide, fly ash, soil samplers, mix design, cement, accelerating agent, curing and setting agents, freeze thaw durability, durability tests, wetting and drying tests, life cycle analysis

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1 INTRODUCTION

Thorough understanding of the subgrade soils is key to designing an appropriate pavement section. Depending on the strength of in-situ soils and project requirements, the properties of the subgrade may need to be improved using ground improvement techniques such as mechanical compaction, chemical stabilization, geogrid reinforcement, and moisture control barriers (Zornberg et al. 2008). Subgrade stabilization using any of these techniques is critical to providing a pavement that is resilient and sustainable over its design life. Subgrade stabilization can increase shear strength, minimize permanent deformation and fatigue cracking, along with limiting expansion and contraction. The process of chemical stabilization avoids the need to dispose the in-situ soil, consequently reducing the costs and corresponding environmental impacts associated with transportation of subgrade materials.

Montana has a diverse geology and a wide variety of subgrade soils are present across the state. These subgrades include soils that have low bearing capacities such as soft clays and loose sands as well as soils with high bearing capacity such as dense sands and stiff clays. In addition, the state also has expansive soils that are capable of volumetric strains due to moisture fluctuations. The pavement subgrade section of the geotechnical manual of the Montana Department of Transportation (MDT) lists geosynthetic reinforcement as well as chemical treatment as choices for subgrade stabilization. The practice has been inclined towards geosynthetic usage due to the familiarity with the method. Hence, through this project, MDT is researching chemical methods for subgrade stabilization and develop stabilization guidelines tailored to the needs of MDT, thus, providing an alternative for subgrade stabilization other than geosynthetics.

In the process of developing this guideline, several tasks were performed.

- Task 1 (Practices Survey) dealt with the literature review of current chemical stabilization guidelines of several state and federal agencies along with the stabilization practices of the Department of Transportation (DOTs) of the state neighboring the State of Montana.
- Task 2 (Material Selection) involved the sample collection from various problematic areas, encountered by MDT, within the State of Montana.
- Task 3 (Evaluate Chemical Stabilizer) primarily dealt with establishing the baseline data and determining the type and amount of additive needed for stabilization – based on the existing guidelines.
- Task 4 (Establish Chemical and Mineralogical Changes) was performed to study changes between treated and untreated soil samples.
- Task 5 (Establish Curing Protocols) was accomplished to establish curing and moisture conditioning protocols that can help minimize the time taken for curing soil samples.
- Task 6 (Durability Studies) was a study of the permanency of the stabilized soil against freezing/thawing and wetting/drying.
- Task 7 (Life Cycle Cost Analysis) was done for an economic analysis and cost comparison of various pavement sections on untreated and treated subgrade soils.

1.1 Study Objectives

The goal of this project is to develop a comprehensive guideline to efficiently evaluate the suitability and concentration of additives. Specific research objectives include:

1. Determine the effectiveness of common soil stabilizing agents for mitigating problematic Montana soils
2. Develop protocols and specifications for the selection of additive type and dosage.
3. Understand the factors of sulfate heaving issues, such as soil fabric, additive types, and reactivity of alumina and silica.
4. Examine the scope and impact of using stabilizing agents to mitigate problematic soils against current MDT practice.

2 REVIEW OF LITERATURE

This chapter presents a review of chemical stabilization techniques, an overview of the current chemical stabilization guidelines of various state and federal agencies, and findings from a survey of stabilization practices carried out in the neighboring states.

2.1 Chemical Stabilization

Chemical soil stabilization has been a topic of interest and discussion for several decades due to a potential reduction in the construction and maintenance costs of pavement infrastructure built on problematic grounds (Tayabji et al. 1982). Chemical stabilization is achieved either through bonding of fine particles (and forming new compounds) or through waterproofing of particles by coating particle surface with the additive or through a combination of the both. Chemical stabilization generally involves mixing or injecting the soil with a chemical stabilizer. A good chemical stabilizer should be capable of stabilizing a wide range of soils, have a permanent stabilizing effect, be readily available at low cost in large quantities, present no serious storage or transport problems, be relatively nontoxic, and preferably non-corrosive (Sherwood 1993). Commonly used stabilizers like lime, cement, and fly ash meet these criteria (Puppala et al. 2013) and the following discussions will focus on the mechanisms responsible for stabilizing subgrade soils using these stabilizers.

2.1.1 Lime stabilization

Lime is a product of decomposition of limestone at elevated temperatures. Lime stabilization is a widely used means of chemically transforming unstable soils into structurally sound construction materials. Lime stabilization enhances engineering properties in soils, including improved strength; improved resistance to fracture, fatigue, and permanent deformation; improved resilient properties; reduced swelling; and resistance to the damaging effects of moisture (Little 2000).

When lime, water, and soil are intermixed, two types of reactions occur: (1) *flocculation and agglomeration* and (2) *pozzolanic compound formation*.

> *Flocculation and Agglomeration*: After initial mixing, the calcium ions (Ca^{++}) from hydrated lime migrate to the surface of the clay particles and displace water and other ions. As a result, the ion density around the clay particles changes due to the modification of diffuse-double layer. This change in double layer thickness causes the particles to come close to each other – this phenomenon is termed as *flocculation*. Due to this phenomenon, the soil becomes friable and granular, making it easier to work and compact (NLA 2004). At this stage, the Plasticity Index of the soil decreases dramatically, as does its tendency to swell and shrink. This process generally occurs in a matter of hours to days depending on the type of clay present in the soil (Al-Rawas et al. 2005; Bell 1996b; Chittoori et al. 2009; Puppala 2017; Sherwood 1993). For this reason, 1 to 4 day mellowing period is generally suggested. The amount of lime required to complete this reaction is typically known as lime fixation point (LFP) or initial consumption lime (ICL).
>
> *Pozzolanic Compound Formation*: After the initial flocculation reactions are complete, the soil continues to react with the remaining lime, forming chemical compounds such as calcium-silicate-hydrates (CSH) and calcium-aluminate-hydrates (CAH). These compounds are known as pozzolanic compounds and are responsible for the strength increase in treated soils. They form the matrix that contributes to the strength of lime-

> stabilized soil layers. As this matrix forms, the soil is transformed from a weak/soft material to a hard, relatively impermeable layer with significant load bearing capacity. The matrix formed is permanent, durable, and significantly impermeable, producing a structural layer that is both strong and flexible. The strength gain is directly proportional to the curing time and temperature (Bell 1996a).

As per Chen (1988), the amount of lime required for stabilization of expansive soils lies between 2% to 8%. In the case of collapsible soils, lime has been used for loess stabilization, particularly in relation to road constructions (Bell 1993). In a study conducted by Akpokodje (1985), it was noted that 7-day unconfined compressive strength after lime treatment using 2% to 12% had very little to no improvement in unconfined compressive strength (UCS) strength for sandy soil (SM) with miniscule amount of clay. Since the lime stabilization mechanism is dependent on the presence of clay particles, lime stabilization does not seem viable for such geomaterials. Lime stabilization is also not preferred when sulfate-laden soils are encountered, due to the issues with ettringite-induced swelling after lime stabilization.

2.1.2 Cement Stabilization

Cement is a product manufactured to meet a variety of performance criteria by controlling the relative proportions of calcium, silica, alumina, and iron compounds. When combined with water, hydration occurs, resulting in the formation of new compounds, most of which have strength-producing properties. The two most important compounds formed after hydration of cement are calcium silicate hydrate (CSH) and calcium hydroxide (CH). The CSH forms a hard structure around the soil particles while the CH initiates the ion exchange process (Prusinski and Bhattacharja 1999). In the case of clayey soils, the ion exchange results in flocculation of clay particles and reaction of calcium hydroxide with silicates and aluminates of clay results in long term stabilization which are similar to lime. As per Herzog and Mitchell (1963), the calcium hydroxide from the hydration of cement is finer and well dispersed through the soil as compared to that of lime.

Water to cement ratio (w_c/c) plays a vital role in the strength of cement treated clay (Geiman 2005; Rafalko et al. 2007). Higher values of w_c/c, result in lower strength of the stabilized mix. For soil with high specific surface area (i.e. montmorillonite dominant clay), use of finer cement is more effective whereas for clay dominated by kaolinite, fineness of cement doesn't play a vital role in strength gain (Rafalko et al. 2007).

Cement is effective in stabilizing a wide variety of soils, including granular materials, silts, and clays. It is generally more effective and economical for use with granular soils due to the ease of mixing, and reduced consumption of cement. Fine-grained soils of low to medium plasticity have been stabilized with cement, but not as effectively as coarse-grained soils. If the PI exceeds 30, cement becomes difficult to mix with the soil. In these cases, it is recommended that lime be added as an initial additive before adding cement, as lime will reduce PI and improve workability (Hicks 2002).

In the case of high sulfate soils, where the chance of ettringite induced swelling is likely, it is recommended to use Type I/II and Type V cements as these were known to resist sulfate attack. (Puppala et al. 2004).

2.1.3 Fly Ash

Fly ash is a non-plastic fine silt-sized residue that results from burning coal and is considered a waste material. The exact composition of fly ash varies depending on the type of coal burnt (Sharma et al. 2012). There are mainly two types of fly ashes, namely Class F and Class C fly ash. Class F fly ash is obtained from burning bituminous coal containing low concentrations of calcium and thus does not have self-cementing characteristics. This type of fly ash needs lime/cement to initiate the cementation process and is typically used to partially replace lime or cement. Class C fly ash, on the other hand, is obtained from burning sub-bituminous coal, which is rich in calcium carbonate and is self-cementing. Although Class C fly ash is used in the presence of lime and cement, it has the ability to form cementitious bonds similar to Portland cement and can be used as a standalone additive (Ferguson 1993; Mackiewicz and Ferguson 2005)**.** The stabilization mechanism for clays using fly ash is cementation resulting from the hydration of tricalcium aluminate and the reaction of free lime (CaO) with silica and alumina (Acosta et al. 2003).

Fergusson (1993) showed that fly ash increases California Bearing Ratio (CBR) while reducing the optimum moisture content of fine-grained soils. However, it was noted that decreases in plasticity and swell potential were generally less than that of lime because fly ash did not provide as many calcium ions to modify the surface charge of clay particles. White et al. (2005) noted that fly ash could also dry wet soils effectively and provide an initial rapid strength gain, which is useful during construction in wet, unstable ground conditions. Sulfate contents can cause the formation of expansive minerals such as ettringite and thaumasite in soil-fly ash mixtures, which severely reduce the long-term strength and durability (Puppala et al. 2004).

2.2 Review of Chemical Stabilization Practices

The success of the chemical stabilization of subgrades depends on the type of material present in the subgrade layers, type of stabilizer, and the construction method (Jones et al. 2012). The following generalized flowchart presented in Figure 2.1 assists in understanding the process involved in chemical soil stabilization.

Texas Department of Transportation (TxDOT), the US Army and Air Force (U.S. Army TM 5-882-14/AFM 32-1019), Portland Cement Association (PCA), National Lime Association (NLA), Federal Highway Administration (FHWA), and other relevant agencies, have developed stabilization guidelines for pavement subgrade stabilization. The research team conducted a thorough review of these stabilization guidelines with a purpose to understand and adapt these guidelines for Montana specific soils. Table 2.1 lists all the guidelines reviewed by the research team and highlights the major aspects of each stabilization process. The table shows similarities when addressing sulfates and organics present in the soils. All stabilization guidelines address strength improvements of the soil.

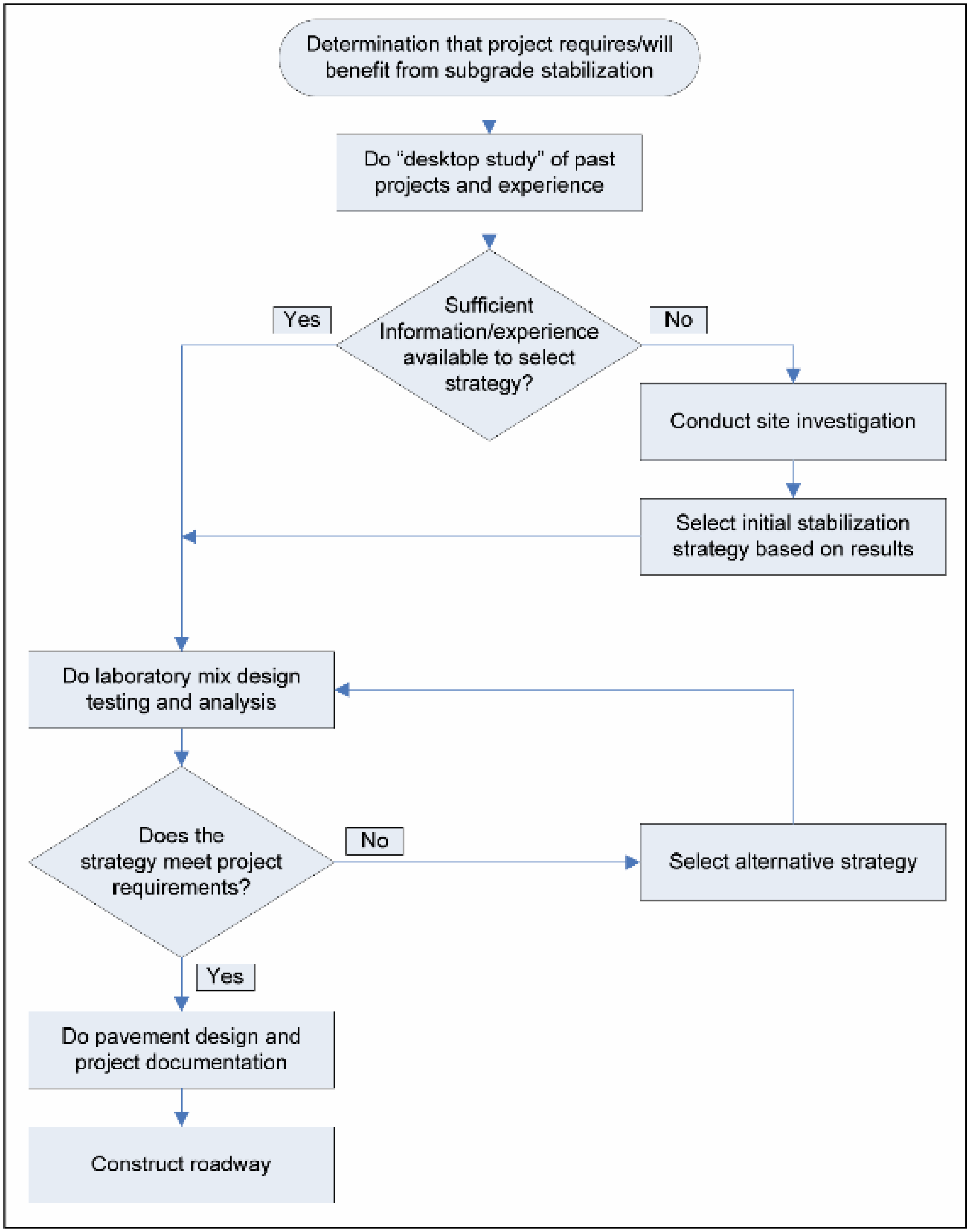


Figure 2.1: Overview of stabilization design procedure (Jones et al. 2012)

Table 2.1: Overview of stabilization guidelines reviewed in this research

Important aspects of the guideline	TxDOT (2005a)	CALTRANS (2012)	INDOT (2015)	NCHRP (2009)	U.S. Army / Air Force	NLA (2006)	PCA (1992)	SHT (1980)
Does the guideline specify procedures for all aspects of the stabilization process listed in Figure 2.1	YES	YES	YES	YES	YES	NO	YES	NO
Does the guideline specify target strength values?	YES	YES (Follows U.S. Army / Air Force Guidelines)	YES	YES U.S. Army / Air Force Guidelines for Cement NLA Guideline for Lime	YES	YES	YES But supplementary and used by shortcut method	YES
Does the guideline require durability studies as part of the stabilizer evaluation process?	NO Curing, drying and moisture condition is done before strength testing	YES (Follows U.S. Army/ Air Force Guidelines)	NO	YES U.S. Army/ Air Force Guideline for Cement Fly ash – Freeze/thaw durability as required	YES pH test to determine the interference on hardening	NO Gives UCS values for expected no. of cycles during the first winter of exposure	YES	YES For Cement Stabilization only
Does the guideline specify separate procedures for sulfate rich soils?	YES	YES >3000 ppm and <8000 ppm modified procedure	YES No Modified design methods soils having sulfate just above 1000 ppm	YES Follow TxDOT(2005b) guideline	YES – Cement Stabilization NO - Lime stabilization	YES Modified procedures for various level of sulfates present	NO	YES- Lime stabilization No Modified design methods soils having sulfate above 0.2-0.5% NO – Cement Stabilization
Does the guideline specify separate procedures for organic soils?	YES	YES >1% and <5% Analysis for stabilization economics	YES Loss of Ignition >6 % • Unacceptable	YES	YES	YES	NO	YES
What type of chemicals does this guideline address?	Lime, Cement, Lime-Fly Ash	Lime, cement, Fly ash (Class F and Class C)	Lime, Cement, Fly-ash, Lime-Fly ash	Lime, Cement, Fly-Ash, Lime-Fly Ash	Lime, Cement, Lime-Fly ash	Lime	Cement	Lime and Cement
What stabilization purposes does this guideline address?	• Strength increase	• Moisture control, • Plasticity reduction, • Strength increase	• Plasticity reduction • Strength increase • Drying	• Strength increase	• Plasticity reduction • Strength increase	• Strength increase	• Strength increase	• Drying • Strength increase

Note: TxDOT: Texas Department of Transportation ; CALTRANS: California Department of Transportation (Jones et al. 2012) ; INDOT: Indiana Department of Transportation ; NCHRP: National Cooperative Highway Research Program - (Syam and Little 2009); U.S. Army/Air Force : U.S. Army TM 5-882-14/AFM 32-1019 NLA: National Lime Association ; PCA: Portland Cement Association; SHT : Saskatchewan Highway and Trasportation , Canada

Excluding the desktop study mentioned in Figure 2.1, the stabilization design procedure can be divided into three phases: (1) *Soil Exploration, Material Sampling, and Soil Classification*; (2) *Additive Selection*, (3) *Mix Design*. Figure 2.2 presents a flowchart of these phases as per TxDOT (2005a) guidelines for subgrade stabilization. In an attempt to describe the steps involved in these phases, stabilization guidelines from U.S. Army TM 5-882-14/AFM 32-1019 and TxDOT (2005a) manuals are discussed in the following sections.

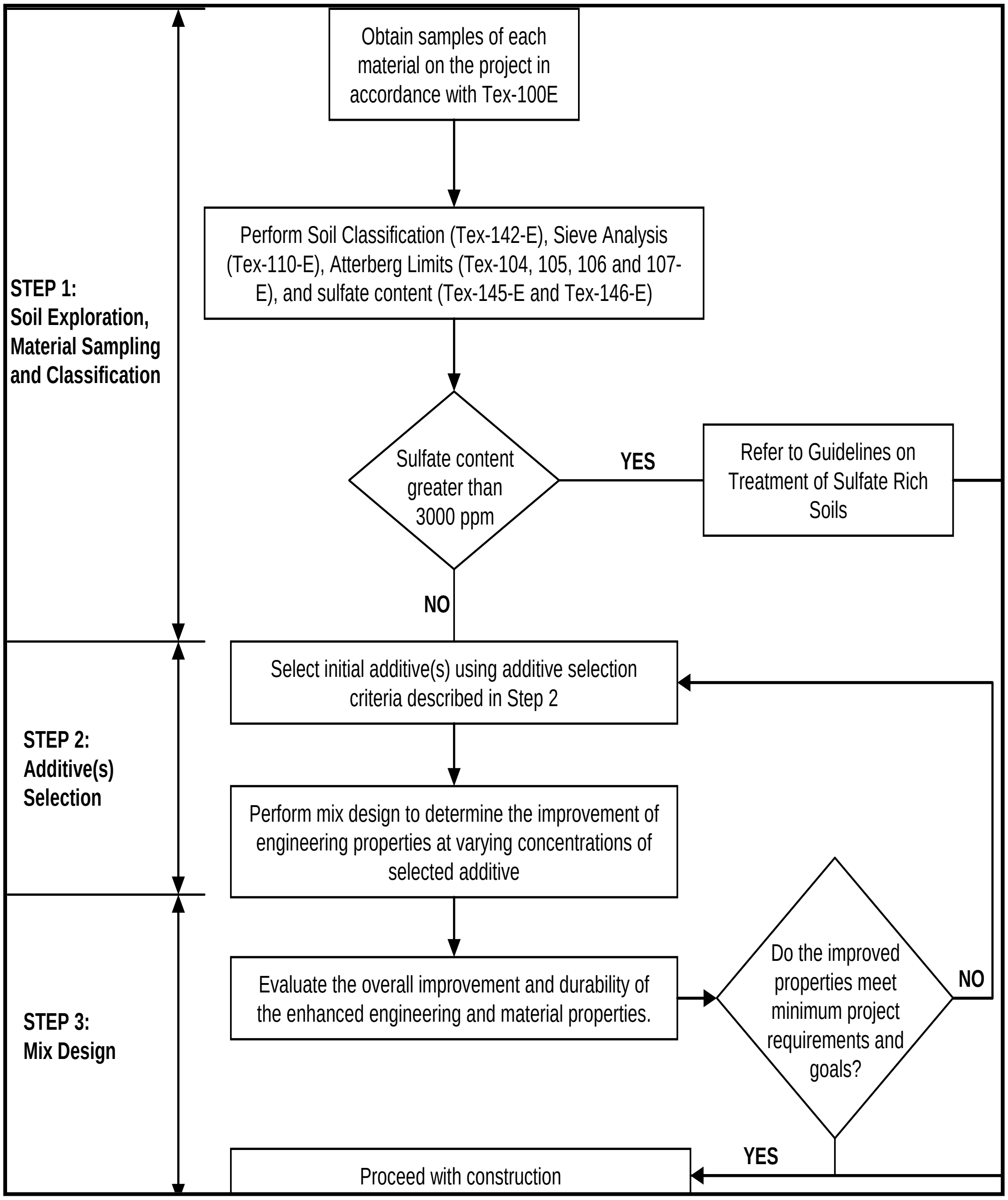


Figure 2.2: Flowchart for subgrade soil treatment (from TxDOT (2005a) Guidelines)

2.2.1 Subsurface Exploration, Material Sampling, Soil Classification

As per the studies conducted by Little and Nair (2009) and TxDOT (2005a), subsurface exploration is vital for preliminary engineering survey. It provides material for testing and also reveals soil stratigraphy, water table location, and other aspects of the underlying strata that can affect the performance of the pavement structure and treated layers. In case of soil stabilization, subsurface exploration aids in deciding if stabilization is warranted for a particular subgrade and if the stabilized material can be used as subgrade or a base or both. For example, soils generally need stabilization when the fraction passing a No. 200 sieve is greater than 25%. On the other hand, when the fine fraction is lower than 25%, the same soil can serve as a base layer with proper stabilization techniques. Soil containing an appreciable amount of sulfate salts cause a potential problem when stabilized with calcium based stabilizers (Hunter 1988). As per Little and Nair (2009) and TxDOT (2005b), sulfate contents greater than 3000 ppm require sulfate specific treatment.

2.2.2 Additive selection

According to the U.S. Army TM 5-882-14/AFM 32-1019, the selection of candidate stabilizers is made using the soil gradation triangle shown in Figure 2.3. This triangle is based on percentage of soil particles retained and passing #200 sieve (0.075 mm). The triangle is divided into three broad areas 1, 2, and 3 representing sands, gravels, and clays. Areas 1 and 2 are further subdivided into subgroups based on the amount of fines present in the soil. The selection process continues with Table 2.2 which recommends candidate stabilizers and restrictions based on grain size and/or PI. For example, successful lime stabilization is expected for soils with PI greater than 12, except for soils in Area 1B while cement stabilization is suitable for sandy soils with PI less than 30 and fine-grained soils with PI less than 20 and LL less than 40.

As per TxDOT (2005a), the selection of an appropriate additive(s) depends on factors including soil type, soil mineralogy, content, soil classification, goals of treatment, mechanisms of stabilization, desired engineering properties, design life, environmental conditions, and engineering economics. A chart for selecting an appropriate additive based on gradation and plasticity index is presented in Figure 2.4. As per this figure, subgrade stabilization is recommended for soils containing more than 25% fines. The selection of additive is based on the PI of the soil. Cement stabilization is preferred if PI is less than 15 while lime stabilization is preferred for soils with PI greater than 15. A lime-fly ash combination could work for all three PI ranges.

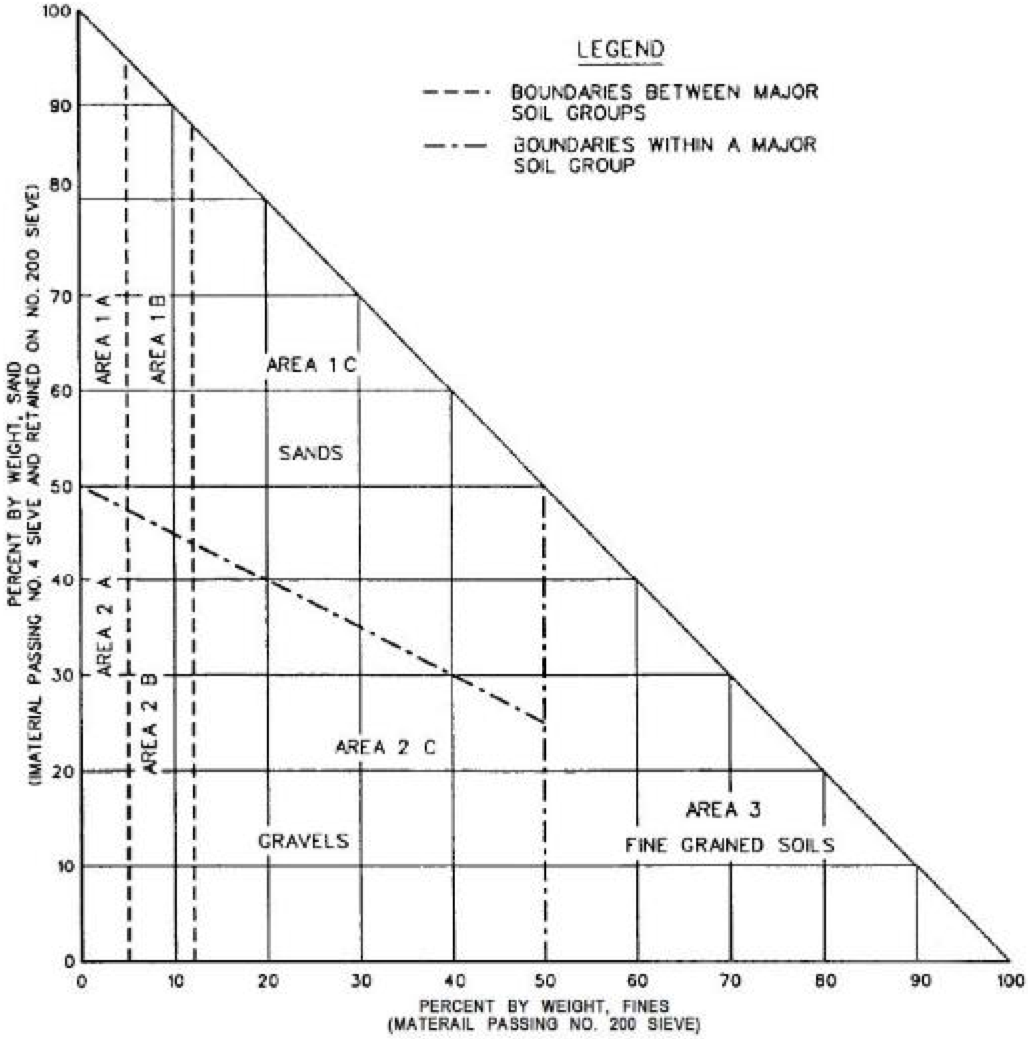


Figure 2.3: Gradation triangle for aid in selecting a commercial stabilizing agent (U.S. Army TM 5-882-14/AFM 32-1019)

Table 2.2: Guide for selecting a stabilizing additive (U.S. Army TM 5-882-14/AFM 32-1019)

Area	Soil Classification	Type of Stabilizing Additive Recommended	Restriction on LL and PI of Soil	Restriction of % Passing No. 200 sieve	Remarks
1A	SW or SP	(1) Bituminous (2) Portland -Cement			
		(3) Lime-Cement-Fly Ash	PI not to exceed 25		
1B	SW-SM or SP-SM or SW-SC or SP-SC	(1) Bituminous	PI not to exceed 10		
		(2) Portland -Cement	PI not to exceed 30		
		(3) Lime	PI not to exceed 12		
		(4) Lime-Cement-Fly Ash	PI not to exceed 25		
1C	SM or SC or SM-SC	(1) Bituminous	PI not to exceed 10	Not to exceed 30% by weight	
		(2) Portland -Cement	*		
		(3) Lime	PI not less than 12		
		(4) Lime-Cement-Fly Ash	PI not to exceed 25		
2A	GW or GP	(1) Bituminous			Well-graded material only
		(2) Portland -Cement			The material should contain at least 45% by weight of material passing No.4 sieve
		(3) Lime-Cement-Fly Ash	PI not to exceed 25		
2B	GW-GM or GP-GM or GW-GC or GP-GC	(1) Bituminous	PI not to exceed 10		Well-graded material only
		(2) Portland -Cement	PI not to exceed 30		The material should contain at least 45% by weight of material passing No.4 sieve
		(3) Lime	PI not less than 12		
		(4) Lime-Cement-Fly Ash	PI not to exceed 25		
2C	GM or GC or GM-GC	(1) Bituminous	PI not to exceed 10	Not to exceed 30% by weight	Well-graded material only
		(2) Portland -Cement	*		The material should contain at least 45% by weight of material passing No.4 sieve
		(3) Lime	PI not less than 12		
		(4) Lime-Cement-Fly Ash	PI not to exceed 25		
3	GH or CL or MH or ML or OH or OL or ML-CL	(1) Portland	LL less than 40 and PI less than 20		Organic and strongly acid soils falling within this area are not susceptible to stabilization by ordinary means
		(2) Lime	PI not less than 12		

* PI ≤ 20 + [(50-percent passing No. 200 sieve) / 4]

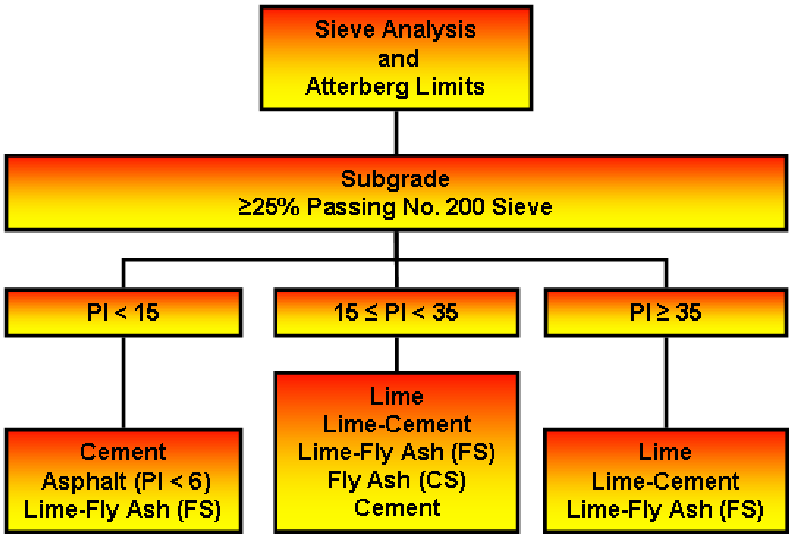


Figure 2.4: Additive selection criteria for subgrade material using soil classification

2.2.3 Mix design

The main goal of developing a mix design is to determine the optimal additive content that achieves the targeted material properties. Mix design procedures also include testing the durability of the mix (soil/additive/water) to verify its effectiveness over long term enviromental conditions. At the end of the mix-design, the amount of additive required to achieve the targeted properties is established along with the density and moisture control parameters for construction.

The following sections discuss the mix design procedures using cement, lime and fly ash stabilizers as per U.S. Army TM 5-882-14/AFM 32-1019 and TxDOT (2005a) stabilization guidelines. This procedure is similar to the various stabilization guidelines reviewed.

2.2.3.1 Determination of additive content for Cement

According to U.S. Army TM 5-882-14/AFM 32-1019 guidelines, the cement content is initially estimated based on the soil classification (see Table 2.3). Using this cement content, maximum dry unit weight, and optimum water content of the soil-cement mixture is calculated. Three specimens are prepared: one at initial estimated cement content and two at ±2% of the estimated cement content. Unconfined compressive strength and durability tests are performed on these specimens and the lowest cement content which meets the strength requirement and demonstrates the required durability is chosen as the design cement content.

TxDOT (2005a) guidelines are primarily based on exceeding a minimum unconfined compressive strength and attaining a minimum strength after moisture conditioning in the laboratory. Minimum strength requirements for stabilized mixes are based on the class specified in the plans as summarized in Table 2.4 (Veisi et al. 2010a). As per Table 4, 2068 kPa (300 psi) should be the target strength for cement stabilized bases. Higher strengths are not recommended because they can lead to brittle cracking.

Table 2.3: Cement requirements for soils (U.S. Army TM 5-882-14/AFM 32-1019)

Soil Classification	Initial Estimated Cement Content (% dry weight)
GW, SW	5
GP, GW-GC, GW-GM, SW-SC, SW-SM	6
GC, GM, GP-GC, GP-GM, GM-GC, SC, SM, SP-SC, SP-SM, SM-SC, SP	7
CL. ML, MH	9
CH	11

Table 2.4: Soil-cement strength requirements as per Veisi et al. (2010a)

Class		7-Day Unconfined Compressive Strength, Min. kPa(psi)
L	Flexible pavements	2068(300)
M		1206(175)
N	Rigid pavements	As shown on the plans

Table 2.5 Ranges of UCS for soil-cement (ACI 230.1R-90)

Soil Type	7-Day Soaked Compressive Strength, kPa(psi)	28-Day Soaked Compressive Strength, kPa(psi)
Sandy and gravelly soils	2068(300)-4137(600)	2758(400)-6895(1000)
Silty soils	1724(250)-3447(500)	2068(300)-6205(900)
Clayey soils	1379(200)-2758(400)	1724(250)-4137(600)

UCS is the most widely referenced property of soil cement. UCS serves as a criterion for determining the minimum cement content requirements. Typical ranges of UCS after 7 and 28 days of curing for soaked soil-cement mixtures are presented in Table 2.5, classified into soil groups (ACI 230.1R-90 1997)

2.2.3.2 Determination of additive content for Lime

As discussed in Section 2.1.1, there are two stages in the determination of the design lime content. The first step is to determine the Initial Consumption Lime (ICL) to satisfy the cation exchange and the second step is to determine the total lime required to achieve a target strength value by adding additional lime beyond the ICL. U.S. Army TM 5-882-14/AFM 32-1019 prefers that the ICL be established using the pH test or Eades & Grim test (ASTM D6276). The ICL is the lowest lime content at which a pH of 12.4 (the pH of free lime) is reached.

An alternate method for determining ICL is by the PI wet method (AASHTO T-220), as shown in the example in Figure 2.5. In this method, the ICL is established by using PI and % soil binder (passing #40). After the ICL is obtained, proctor curves are established for soils mixed with ICL lime content and with increments in lime content by 1 to 2%. The lowest lime content that meets the targeted UCS strength is selected as the required lime content.

U.S. Army TM 5-882-14/AFM 32-1019 also recommends that durability tests be performed at the final lime content to ensure that the stabilization effects are lasting. If results of the specimens tested do not meet both the strength and durability requirements, a higher lime content may be selected and the mix design is reevaluated.

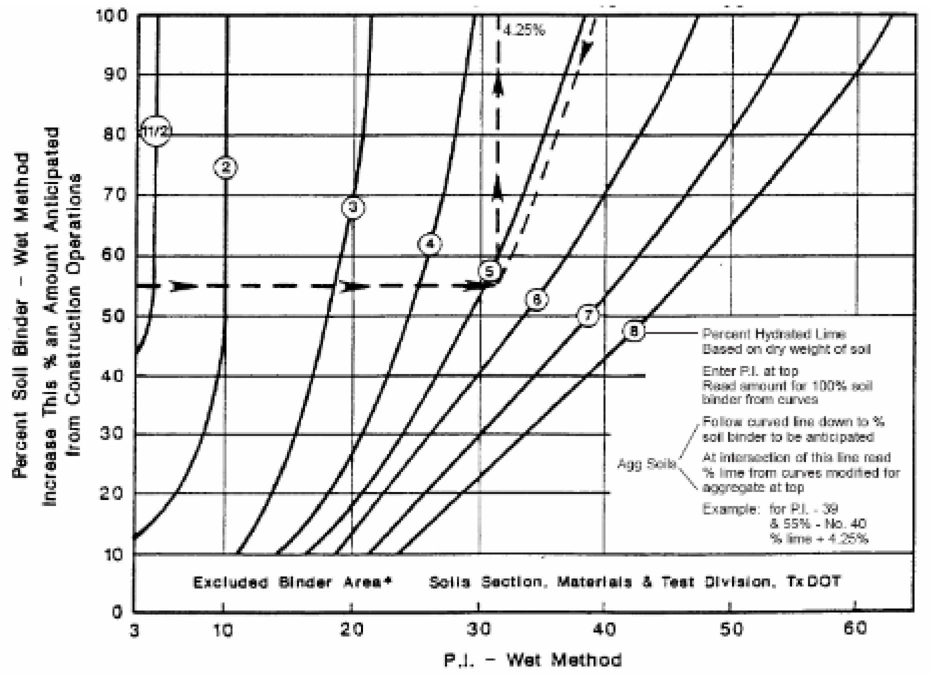


Figure 2.5: PI wet method to calculate the amount of lime for stabilization (Tex-121-E 2002)

TxDOT (2005a) follows Eades & Grim test to establish the ICL. After establishing the ICL, a series of specimens with lime percentages ranging from ICL to 10% in 1 or 2% increments are tested in the lab to determine the required amount of lime. The required lime content is the lowest amount of lime that satisfies the strength criterion. The minimum strength criterion for lime content is based on an unconfined compressive strength of 1034 kPa (150 psi) for the base and 345kPa (50 psi) for subgrade.

2.2.3.3 Determination of additive content for Fly Ash (FA) and Lime-Fly Ash (LFA)

Like cement, the UCS is used as an index to determine the suitable amount of additive. A minimum unconfined compressive strength of 150 psi is suggested as adequate for FA or LFA stabilized soils. Unconfined compressive strengths for FA or LFA base courses should approach the strength requirements of soil cement presented in Figure 2.6 Minimum compressive strength requirements for soil cement as per PCA (1992). According to U.S. Army TM 5-882-14/AFM 32-1019 guidelines, design with LFA is different from that with lime or cement. For a given combination of materials (aggregate, fly ash, lime), a number of factors can be varied in the mix design process such as the percentage of lime-fly ash, the moisture content, the ratio of lime to fly ash and the percentage of aggregate fines. The fly ash, lime, and minus No. 4 aggregate fines are referred to as matrix materials. To establish the amount of additives, the first step is to determine the optimum fines content by targeting the maximum dry density of the matrix. In the case of LFA mixtures, it is recommended that this optimum value be increased by 2% since, higher strength and improved durability are achieved when matrix material is able to “float” the coarse-aggregate particle after filling the available void spaces.

The initial fly ash content should be about 10% based on the dry weight of the mix. Tests are run at 2% increments of fly ash, up to a total of about 20%. Compaction characteristics using standard proctor are determined for these fly ash contents and the final content is selected at 2% above the mix that yields maximum unit weight. The ratio of lime to fly ash that will yield the highest strength and durability is determined, by using lime to fly ash ratios of 1:3, 1:4, and 1:5. Three specimens are prepared and tested for unconfined compression strength and wet-dry or free-thaw cycles are conducted. The lowest LFA content that meets the strength criteria from Figure 2.6 is selected as the design LFA content.

2.3 Stabilization Practices in the Neighboring States

A thorough literature search was performed to collect soil stabilization literature published by agencies in the states neighboring Montana as the conditions in these regions are similar to that of Montana. This search included DOTs of North Dakota, South Dakota, Wyoming, and Idaho in the US along with the Saskatchewan Province in Canada. The search yielded very limited information as most of the nearby states except for the Saskatchewan Province, do not have established guidelines to perform chemical stabilization of soils. NDDOT (2008) and WYDOT (2010) included stabilization construction protocols after the mix designs are determined, but the actual protocols for the mix design procedures are not specified. In the case of Idaho Department of Transportation and South Dakota DOT, there were no specific guidelines for either laboratory mix design protocols or field construction procedures.

Saskatchewan Highway and Transportation (SHT 1980) specifies laboratory protocols for both modification and stabilization. It defines modification as the addition of chemicals for the purpose of minimizing swelling and making the soil drier to have stable working ground. Stabilization is defined as the addition of chemicals with the intent of improving the strength characteristics of the subgrade soil. For modification, the lime content purpose is selected by mixing various amounts of lime and testing for PI. The amount of lime beyond which there is no change in PI is chosen as design lime content for modification. For stabilization, the amount of lime is chosen based on the 28-day UCS value with a targeted of 1320 kPa (191 psi) for high plastic clays and 2070 kPa (300 psi) for silty soils. Lime stabilization is recommended for soils with PI greater than 10 except when these soils have pH less than 7 (acidic), organic content more than one percent, or sulfate content higher than 2000 to 5000 ppm (0.2 to 0.5 percent).

As per SHT (1980), cement stabilization is recommended for well graded sands and gravels and the required cement content is typically less than 6% by dry weight of the soil. The mix design procedure for soil cement is based on the target UCS value and durability performance of soil cement specimen cured 7 days. The durability tests include wet-dry and freeze-thaw tests as recommended by PCA (1992). The maximum allowable loss in weight after 12 cycles of freeze-thaw and wet-dry is 14 percent.

The minimum compressive strength requirement for soil-cement is based on the percentage of 50 µm and 5 mm sizes contained in the soil sample as per Figure 2.6.

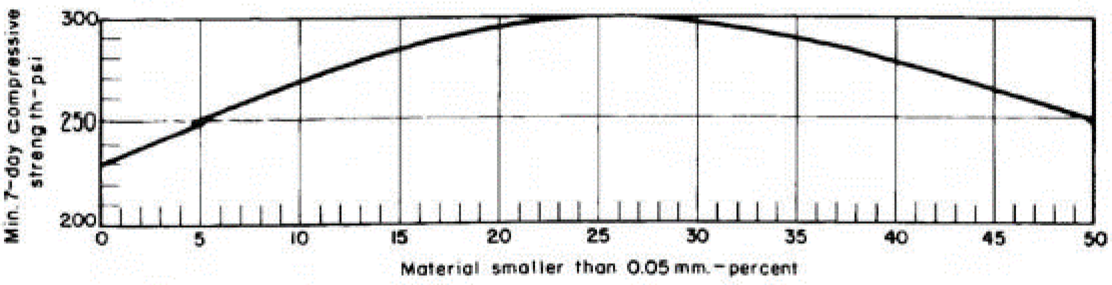


Figure 2.6 Minimum compressive strength requirements for soil cement as per PCA (1992)

The minimum percentage of cement that satisfies both compressive strength and durability criteria is chosen as the final design content. Generally, the percentage of cement is controlled by freeze-thaw results. Approximate cement content can be determined by using correlations developed from the results of Saskatchewan soil cement designs over the years (SHT 1980).

2.3.1 Survey of Practices

In order to ensure that the research team did not miss any unpublished data from the neighboring states and province, a survey of stabilization practices was conducted. The results of this survey are presented and discussed in this section.

A survey questionnaire was modeled after the surveys by Carpenter et al. (1992) and Sargand et al. (2014) and was formulated to obtain information on whether the participants use chemicals as a stabilizer for problematic soils. The survey was conducted using an online survey tool, Qualtrics, to automatically collect the responses and analyze them. The survey questionnaire is presented in Table 2.6.

Table 2.6: Survey questionnaire

Q. No.	Question	Options
1	Has your Department dealt with problematic subgrade soils? (i.e. weak, unstable, excessive rutting, or failure to support construction)	Yes No
2	What is the magnitude of the problematic soil issues (cost and frequency)?	High Medium Low N/A
3	What types of problematic soils has your Department dealt with? (please check all that apply)	Swelling Soils Collapsible Soils Soft Clay Soils Loose Sands Others (Please Specify)

Q. No.	Question	Options
4	Please state the encountered issues with the selected problematic soils.	[Description]
5	Please list general locations where problematic soils have been encountered. (City, county, the area of the state, road name/number, etc...)	[Description]
6	Does your Department use chemical treatments on soils for the purpose of stabilization of problematic subgrade soils? (if no is answered, please go to question 10)	Yes No
7	If yes, is the stabilization meant for the short term, or long term?	Short term (allowing construction or drying out) Long term (stabilized strength integral to pavement design)
8	What chemicals has your department used for the purpose of stabilization? (please check all that apply)	Cement Lime Fly Ash Kiln Dust Calcium Chloride Magnesium Chloride Asphalt Emulsion Other (please specify) N/A
9	What is the deciding factor in the chemicals used? (please check all that apply)	Cost Effectiveness Availability Recommendations (please specify) Previous Experience Others (please specify) N/A
10	If chemicals are not permitted or used, please state why. (please check all that apply)	Environmental concerns Legal concerns Equipment limitations Lack of research Not cost effective Unavailable in area Others (please specify) N/A
11	If chemicals are not used to stabilize soils, please identify the current method(s) used.	[Description]

Q. No.	Question	Options
12	What factors are used to determine the type of stabilizer? (please check all that apply)	Pervious experiences Compressive strength Laboratory/field testing Mineralogy Plasticity index Depth of base material Triaxial testing Others (please specify)
13	What are the typical layer types and thicknesses of a flexible highway pavement in your jurisdiction? (please answer numerically in inches)	Hot Mix Asphalt (0-9<) = Base (0-24<) = Subbase (0-24<) =
14	What are the typical layer types and thicknesses of a rigid highway pavement in your jurisdiction? (please answer numerically in inches)	Portland Cement Concrete (0-12<) Base (0-12<) Subbase (0-12<)
15	Please provide the following contact information:	Name: Title: Office or Section: State: Phone No.: Email address:

2.3.1.1 Participation and Roles

A total of 5 participants started the survey, with only 3 continuing to the completion. The participants of the survey were involved in the engineering department of their respective organizations. Two identified themselves as Geotechnical Engineers, with the remaining participant listed as the Chief Engineering Geologist. All participants were in positions to handle the design and management of the specifications for stabilizing problematic soils.

2.3.1.2 Overall Experience with Problematic Soils

All of the participants who completed the survey stated that their department has dealt with problematic soils. When asked about the severity of the problematic soils in terms of cost and frequency, all participants stated that the issue was of medium severity. The types of problematic soils encountered by agencies are summarized in Table 2.7.

Table 2.7: Type of Problematic Soils dealt by surrounding states

States	Problematic Soils
North Dakota	Soft Clay Soils
South Dakota	Swelling Soils, Soft Clay Soils, Loess
Wyoming	Swelling Soils, Collapsible Soils

All participants indicated that the problems with soft clay soils affected their base and subbase design. NDDOT participant stated that where soft clay soils are encountered, it is "not competent to place base". SDDOT participant stated that the soft clay soils do not support construction traffic and are unsuitable for embankment designs. WY has a similar issue, but WYDOT typically excavates the problematic soils, replaces them with a more suitable subbase material, and uses geosynthetics in order to stabilize the remainder of the soil.

When swelling soils are encountered in WY, an impermeable membrane is spread over the subgrade in order to mitigate the effects of differential swelling. In order to mitigate swelling soils in SD, SDDOT controls density and water content during construction to achieve stability during the design life.

WYDOT is the only DOT surveyed, which stated having dealt with collapsible soils. Previously, WYDOT dealt with collapsible soil through dynamic compaction but found the costs to be excessive. The current practice is to excavate the collapsible soils, and compact using a geosynthetic material as reinforcement in the upper two to three feet.

2.3.1.3 Use of Chemical Stabilizers

The practice of chemical stabilization, type of chemical stabilizer and factors for stabilizer type determination by different DOT's are summarized in Table 2.8.

Table 2.8: Use of chemical stabilization by states

States	Use of Chemical stabilization	Type of Chemical Stabilizer	Factor for stabilizer type determination
North Dakota	No	N/A	N/A
South Dakota	Yes	Lime, Fly ash, Cement	Previous Experiences Laboratory/field testing Gradation Plasticity index CBR
Wyoming	Yes	Lime, Fly ash, Cement	Previous Experiences Industry recommendations

SDDOT and WYDOT participants confirmed that they used chemical stabilization on soils, while NDDOT participants did not. Both responses indicated that this form of stabilization had been used in only a short-term capacity (allowing for construction or drying out of soils). The stabilizers used in both SD and WY are lime, fly ash, and cement. The participants did not have an established guideline for chemical stabilization and relied mostly on past experiences in selecting the stabilizer type.

2.3.1.4 Roadway Specifications

The final questions dealt with the thicknesses of pavement designs. Typical pavement sections adopted by the respondents of the survey for Flexible and Rigid Highway Pavement are shown in Figure 2.7 and 2.8.

Table 2.9 Typical flexible pavement sections reported by the survey respondents

Agency	HMA (in.)	Base (in.)	Subbase (in.)
WYDOT	4	6	18
NDDOT	5	15	N/A
SDDOT	6	12	N/A

Table 2.10 Typical rigid pavement sections reported by the survey respondents

Agency	PCC (in.)	Base (in.)	Subbase (in.)
WYDOT	10	6	18
NDDOT	10	8	N/A
SDDOT	9	5	N/A

2.4 Current MDT Stabilization Guidelines

As per the MDT geotechnical manual, when poor subgrades are encountered, it is recommended that the soil be stabilized using geosynthetics or chemical additives. In the case of geosynthetics, it is recommended that a thorough geotechnical evaluation be performed before using them in the field. This evaluation should include factors such as soil conditions, design loads, service life, geosynthetic type, and life cycle costs.

In the case of chemical stabilization, the manual recommends the use of cementing agents such as Portland cement, lime, and lime fly ash as chemical additives. Cement stabilization is recommended for granular soils, silty soils, and lean clays, but it is not recommended for organic materials. The cement percentages for lean clays range from 9 to 15% while that for granular soils is 5 to 9%. Lime stabilization is currently recommended for granular soils and lean clays. This recommendation can be extended to high plastic clays as lime stabilization works best in these soil types. Fly ash is recommended as a stabilizer in the presence of lime to provide the required calcium for pozzolanic compound formation. However, there are no mix design procedures established for these additives.

2.5 Summary

Based on the literature search and survey of practices, it is evident that many states nearby Montana, except for Saskatchewan Province in Canada, do not have established protocols to perform chemical stabilization of soils. Stabilization guidelines adopted in the Saskatchewan Province, SHT (1980), reveal that freezing-thawing is going to be the controlling factor for this region and hence the research team will focus on freezing/thawing while evaluating the durability of stabilization.

3 MATERIALS, METHODS, AND SCOPE OF WORK

This chapter presents the soils used in testing, the sample preparation methods, and test procedures used to complete this research.

3.1 Material Selection

Since a wide variety of problematic soils exist across the state of Montana including low bearing capacity soils, expansive soils, high sulfate bearing soils and high organic content soils, it is important that the selected soils represent these problematic soils. Hence, six different locations were chosen from different regions of Montana. The goal was to obtain different problematic soil types from various geological conditions to ensure that stabilization guidelines will address diverse problematic soils. These soils consisted of two high-plasticity clays, two low plasticity clays, one low plasticity silt, and one sandy subgrade. Chemical additives used in this study were lime and cement. These were procured from different manufacturing plants in Montana. The sampling locations along with the nearest highway and reference post information are presented in Table 3.1.

Table 3.1: Summary of soil sampling locations

Soil Name	Nearest City/Town	Nearest Highway	Nearest Reference Post
Great Falls (GF)	Great Falls	US87	RP 9.3
Dry Creek (DC)	Jordan	MT200	RP 243.8
Bad Route (BR)	Fallon	I-94	RP 193.3
US2/Chinook (CNK)	Chinook	US2	RP 400.2
North Three Forks Low Plastic (NTF_LP)	Three Forks	US287	RP 106.5
North Three Forks High Plastic (NTF_HP)	Three Forks	US287	RP 105.9

3.1.1 Drilling and Sampling

MDT personnel performed the drilling and sampling of the subsurface soils. Table 3.2 presents the soil sampling depths for each location. About 200-300 lb. of soil was collected from each of the sites and transferred to Boise State laboratories for further testing.

Table 3.2: Sampling depth for soils collected

Soil	Sampling Depth
GF	2 – 9 ft
DC	1 – 5 ft
BR	1 – 2 ft
CNK	2 – 5 ft
NTF_LP	2 – 5 ft
NTF_HP	2 – 5 ft

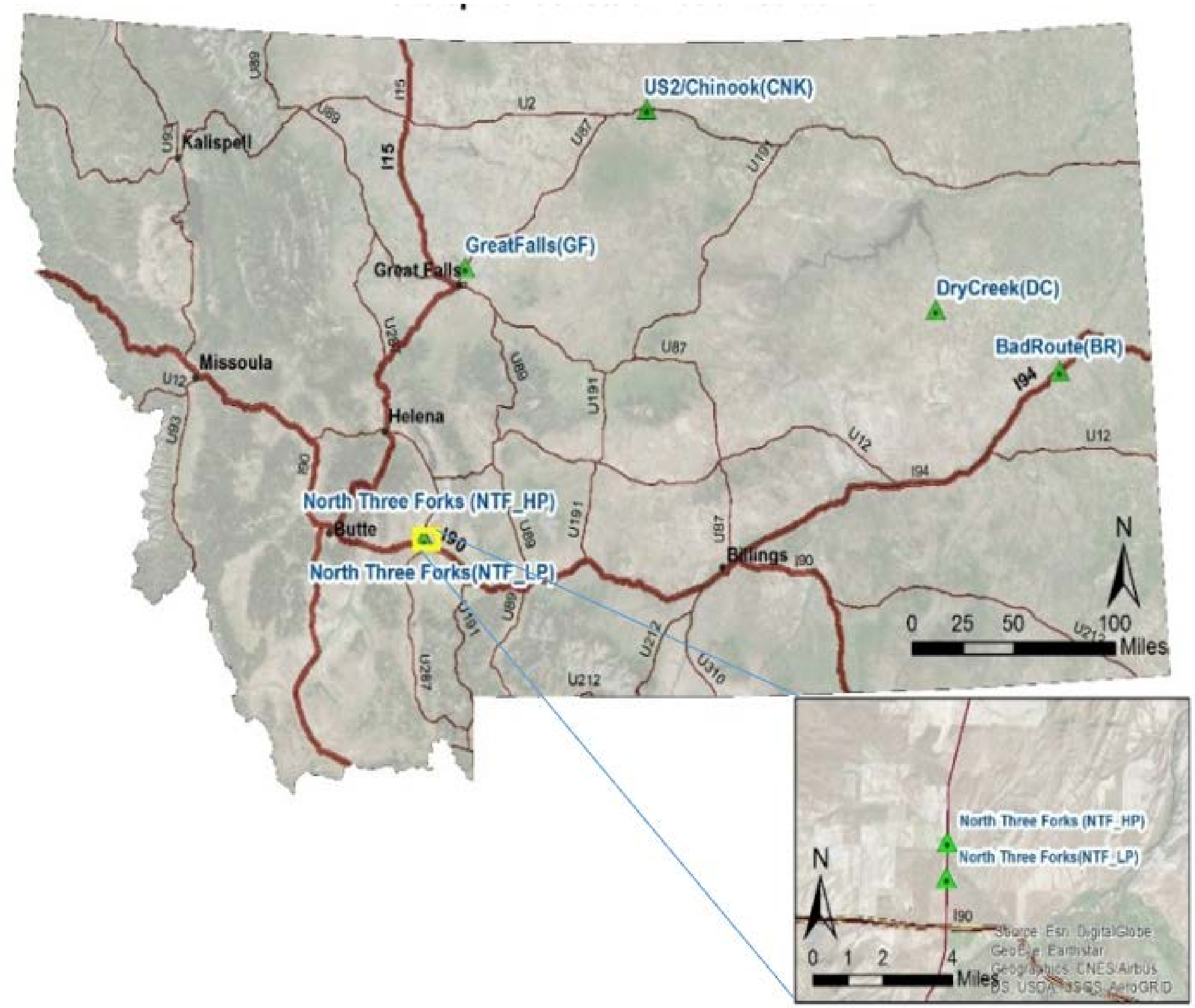


Figure 3.1: Locations of selected sites for this study

3.1.2 Chemical Additives

The additives used in this study were lime and cement. The lime used for stabilization was high Calcium Quick Lime from Graymont Western US Inc., Townsend, Montana. The cement used for stabilization was Type II/V, from Ash Grove Cement, Montana City Plant, Clancy, Montana.

3.2 Establish Baseline Data

To establish this database, soil samples from all six locations were tested. Table 3.3 lists the various tests performed and the corresponding test standards followed in this research. It should be noted that, where applicable, MDT standards took priority, except for the soluble sulfate test due to the accuracy of the procedure followed. ASTM standards were followed when MDT standards did not specify any procedure. The test procedures followed in this study are described in Appendix A.

Table 3.3: Standards used to establish physical parameter

Test Category	Tests	ASTM Standard	Equivalent AASHTO Standard	Equivalent MDT Standard
INDEX TESTS	Specific Gravity	ASTM D854	AASHTO T100	Use AASHTO standard
	Sieve Analysis	ASTM D6913	AASHTO T27	MT 202-11
	Hydrometer	ASTM D7928	AASHTO T88	N/A
	Atterberg Limits	ASTM D4318	AASHTO T89 & T90	Use AASHTO standard
PHYSICAL TESTS	Standard Proctor Compaction	ASTM D698	AASHTO T99	MT 210
	Unconfined Compression Strength	ASTM D2166	AASHTO T208	Use ASTM Standard
	1-D Swell Test	ASTM D4546 (Method A)	AASHTO T216	N/A
	Permeability	ASTM D5084 (Method B)	Use ASTM Standard	N/A
CHEMICAL TESTS	Organic Content	ASTM D2974 (Method C)	AASHTO T267	N/A
	Soluble Sulfates	Puppala et al. (2002)	ASTM C1580	MT 232-16
	Cation Exchange Capacity	ASTM D7503	N/A	N/A
	Specific Surface Area	Cerato and Lutenegger (2002) +		
	Reactive Alumina and Silica	Veisi et al. (2010a)+		

+No established AASTHO or ASTM standard

3.3 Summary of Results

Table 3.4 presents the results from index tests discussed in the previous section. It can be observed that the samples collected consisted of non-plastic soils like silty sand, and low plastic clays and silt, along with highly plastic clays. The clay content in these soils ranged from 2% (CNK) to 73.5 % (GF). Similarly, fines content ranged from 14.15% (CNK) to 97.47% (GF). The liquid limit values ranged from 37 (DC) to 103 (GF) while the plasticity index ranged from 8 to 62. The plasticity and gradation characteristics indicate that the issues with these soils could range from the low bearing capacity to high volumetric changes.

Table 3.5 presents the results of the physical property measurements performed in this study. The optimum moisture content ranges from 13.9 to 35.5% while the maximum dry unit weight ranges from 82.6 to 109.8 pcf. The UCS strengths ranged from 1.7 (CNK) to 53.7 psi (BR). The CNK soil had a very low UCS because it is primarily sand and has very little cohesion.

The swell pressures ranged from 5.80 to 30.46 psi while the swell strains ranged from 0.96 to 10.27%. The high plastic clay from Great Falls and Three Forks area showed significant swelling potential while the remaining low plastic clay and silt did not show significant swell characteristics. The permeability values ranged from 1.66E-03 in/s (CNK) to 5.43E-08 (GF) in/s.

Table 3.4: Summary of index properties of soils

Soil ID	Atterberg Limits			Classification		Gradation			Specific Gravity
	LL (%)	PL (%)	PI (%)	USCS	AASHTO	% Sand	% Silt	% Clay	
GF	103	41	62	CH	A-7-5	2.53	23.97	73.50	2.60
DC	37	20	17	CL	A-6	16.43	54.07	29.50	2.75
BR	42	26	16	CL	A-7-6	5.73	60.77	33.50	2.90
CNK	*			SM	A-2-4	85.85	12.15	2.00	2.65
NTF_LP	43	35	8	ML	A-5	32.60	55.40	12.00	2.40
NTF_HP	58	26	32	CH	A-7-6	6.00	59.00	34.00	2.65

Note: *Non-Plastic Soil; LL-Liquid limit; PL – Plastic limit; PI – Plasticity Index; USCS – Unified Soil Classification System; AASTHO – American Association of State Highway and Transportation Officials

Table 3.5: Summary of physical properties of soils

Soil ID	OMC (%)	MDUW (pcf)	UCS (psi)	Modulus (psi)	Swell Test		Permeability (in/s)
					Swell Pressure (psi)	Swell Strain (%)	
GF	35.5	82.6	24.1	494.43	30.46	10.27	5.43E-08
DC	16.9	109.8	24.1	352.20	5.80	1.15	4.46E-05
BR	19.8	105.5	53.7	1521.90	7.25	1.38	3.59E-06
CNK	13.9	100.5	1.7	44.07	*		1.66E-03
NTF_LP	25.0	87.9	47.9	1536.15	7.25	0.96	1.73E-05
NTF_HP	24.2	94.8	35.0	621.5	15.23	2.49	2.94E-07

Note: *Non-Plastic Soil; OMC – Optimum Moisture Content; MDUW – Maximum Dry Unit Weight; UCS – Unconfined Compressive Strength

Table 3.6 presents the results of the chemical property measurements performed in this study. The sulfate content ranged from 29 to 14500 ppm. CNK has the lowest sulfate content whereas NTF_HP has the highest. Both NTF_HP and BR have sulfate concentrations well above the threshold for using calcium-based chemical stabilizer as per established guidelines, while the rest of the soils could potentially be treated with these stabilizers.

Organic content ranged from 0.7% (for CNK) to 4.2% (for NTF_HP). All soils except CNK have crossed the 1% threshold for organic content, which, as per Jones et al. (2012), requires additional consideration for the economics of stabilization as these types of soils may need higher amounts of stabilizer amounts than soils with lower organic contents.

Reactive alumina measurements ranged from 13 ppm (for CNK) to 118 ppm (for GF), and reactive silica measurements ranged from 35 ppm (for BR) to 1135 ppm (NTF_LP). These measurements were later used in Task 4 for comparing the changes in reactive alumina and silica due to chemical treatments. As for CEC, CNK has the lowest, i.e., 8.29 meq/100g as it is primarily sand and GF has highest, i.e. 241.60 meq/100g as it is primarily clay. A similar trend can be seen in the case of SSA where it ranged from 33 m^2/g (for CNK) to 400 m^2/g (for GF).

Table 3.6: Summary of chemical properties of soils

Soil ID	Sulfate (ppm)	Organic Content (%)	Reactive Alumina (ppm)	Reactive Silica (ppm)	CEC (meq/100gm)	SSA (m^2/g)
GF	3107	3.3^+	118	492	241	400
DC	722	3.5^+	106	241	75	111
BR	13635	1.2^+	50	35	19	52
CNK	29	0.7	13	59	8	33
NTF_LP	2450	3.7^+	107.8	1135	86	85
NTF_HP	14500	4.2^+	91.07	519	120	146

Note: $^+$higher organic content (>1%) – economics of stabilization needs to be considered (Jones et al. 2012); CEC – Cation Exchange Capacity; SSA – Specific Surface Area

3.4 Selection of the additive type

For selecting the additive type, guidelines (Little and Nair 2009) provided by NCHRP were used in this study. It should be noted here that NCHRP guidelines closely follow TxDOT (2005a) guidelines. These criteria for selection of stabilizer types are mainly dependent on the gradation and plasticity characteristics of the soil along with soluble sulfates. Before selecting the additive type for stabilization, the soils need to be assessed for use as base or subgrade. As per the guideline, soils with less than 25% passing #200 sieve are suitable for base materials while those with more than 25% passing #200 sieve are suitable for subgrade improvements only. In this study, five out of six soils used have percentage passing #200 sieve more than 25% hence we focused on subgrade treatments only. Once this determination is made, stabilizer selection is made using Figure 3.2. Table 3.7 presents the stabilizers selected for each of the six soils. It can be noted from the figure that all soils could be treated using cement while only GF and DC soils could be treated using lime. BR and NTF_HP soils had medium to high plasticity indices qualifying them for lime stabilization, however, the presence of high sulfates in these soils disqualified them for lime stabilization and were subsequently treated with cement.

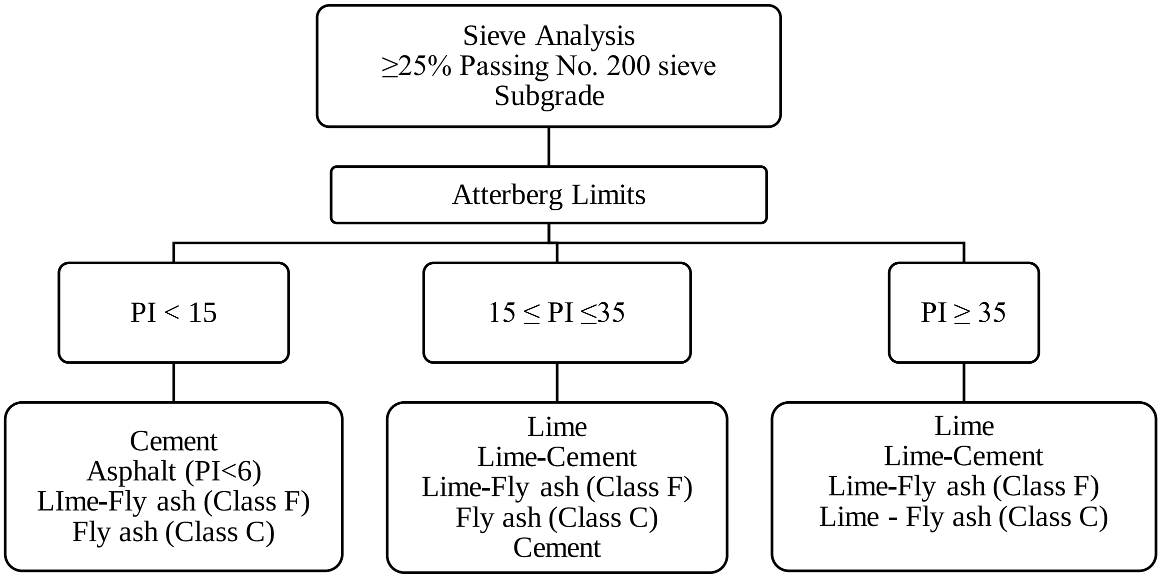


Figure 3.2: Stabilizer selection criteria based on TxDOT (2005a) method

Table 3.7: Stabilizer recommend for soils under consideration

SOIL ID	SELECTED ADDITIVE(S)
GF	Lime and Cement
DC	Lime and Cement
BR	Cement only*
CNK	Cement
NTF_LP	Cement
NTF_HP	Cement only*

*Soluble sulfate concentrations beyond threshold levels recommended for conventional treatment methods

3.5 Selection of additive content

The objective of the stabilizer mix design was to determine the amount of additive and evaluate the improvement of engineering properties with varying concentrations. Performing a mix design is essential in order to:

- ✓ Ensure the optimum percentage of additive(s)
- ✓ Optimize the engineering properties
- ✓ Measure the effectiveness of these engineering properties using moisture conditioning
- ✓ Observe the effectiveness of the additive(s) with specific soil and its inherent mineralogy
- ✓ Provide density and moisture control parameters for construction
- ✓ Mitigate cracking and other distresses associated with material behavior

The procedures for selecting additive contents differ between lime and cement additives. The following section will detail the steps involved in this process, and the guidelines followed for each of these additive types.

3.5.1 Determination of Lime Content

The following step by step procedure was used to obtain the optimum lime content for the two soils that are selected for lime stabilization:

1. Establish optimum lime content
2. Establish moisture-density relationship at this lime content
3. Determine the unconfined compressive strength of the mix
4. Verify if the UCS meets the standard
5. If the UCS meets the standard, select the percentage as required lime content otherwise increase lime content and repeat steps 2 to 4

The optimum lime content is established using a procedure developed by Eades and Grim (1966) which targets a pH of 12.4 or higher. This test is specified by most of the current design guidelines (INDOT 2015; Jones et al. 2012; NLA 2006; TxDOT 2005a; U.S. Army TM 5-882-14/AFM 32-1019 1994). The pH versus lime content variation for the GF and DC soils is shown in Figure 3.3. Based on these results, the minimum amount of lime necessary to stabilize both soils is 2%.

The appropriateness of the design mix is established by strength requirements. NCHRP guidelines (Little and Nair 2009) suggest a UCS increase of 50 psi as adequate for soils treated with lime for subgrade applications. In order to determine the UCS of the treated mix, samples need to be prepared at OMC and MDUW and cured before testing for UCS. Standard Proctor's compaction was used to determine the OMC and MDUW for the soil-lime mix. After establishing the OMC and MDUW, soil specimens, 2.8 in. (diameter) and 5.6 in. (height) were prepared and cured for seven days at ~100% humidity conditions. The soil samples were sealed to ensure there is no moisture ingress or egress from the soil sample. After the curing time is completed, UCS tests were conducted.

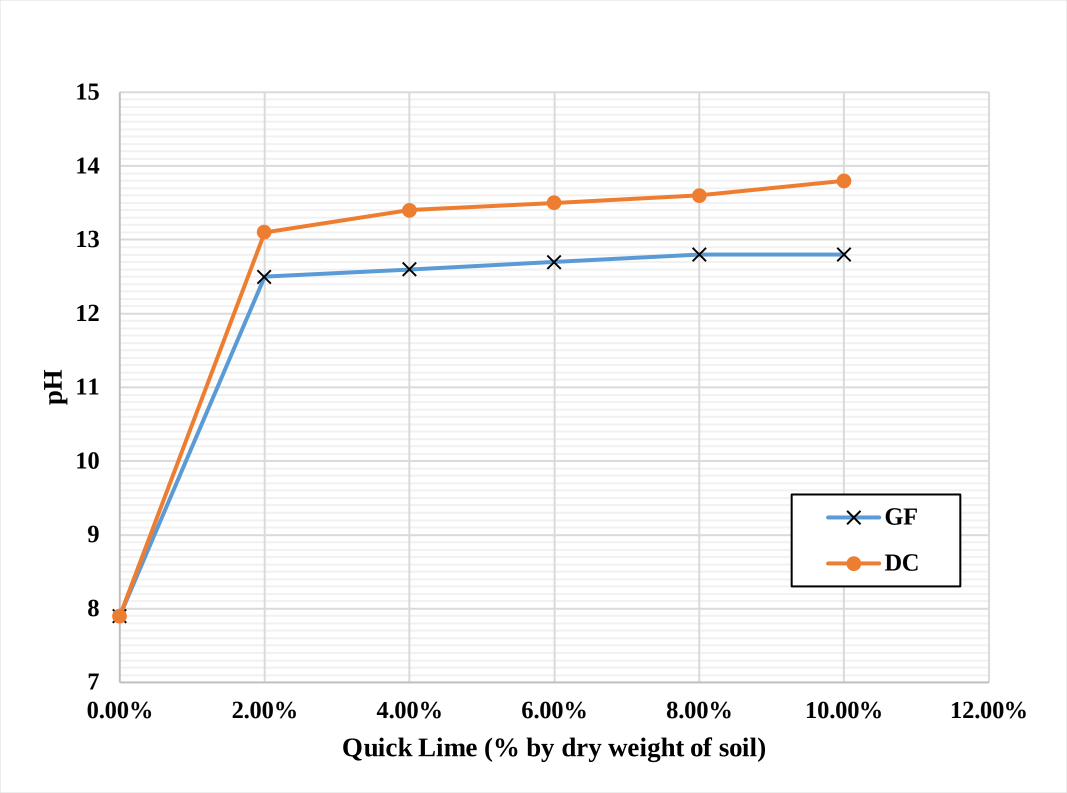


Figure 3.3: Change in pH with % of lime treatment

Table 3.8 presents the UCS values for GF and DC soils before and after treatment with 2% lime. This data is plotted in Figure 3.4. It can be observed from this plot that both soils showed UCS values higher than 50 psi at 2% lime indicating that 2% lime is the design lime content. However, it should be noted here that this percentage needs to be finalized after performing durability studies to ensure the long-term performance of the stabilization (chapter 6). Figure 3.4 also shows sulfate concentrations. An observation from the figure is that despite GF and DC soils have similar untreated strengths and lime content, the strength improvements after treatments differed considerably. This could be attributed to the high plastic nature of GF soil and the presence of mineral montmorillonite.

Table 3.8: Initial Stabilizer content for lime and UCS strength

Soil ID	USCS Classification	Stabilizer content	UCS (psi) Treated	UCS (psi) Untreated
GF	CH	2%	58.79	24.08
DC	CL	2%	99.27	24.13

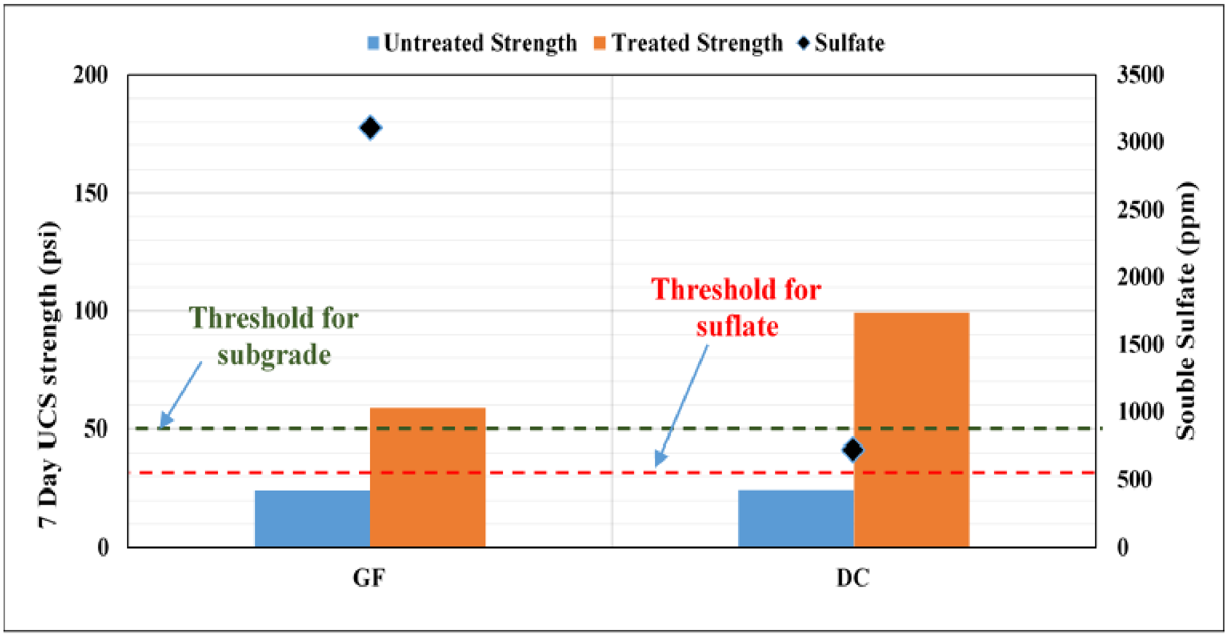


Figure 3.4: Plot showing lime stabilization results for the six soils

3.5.2 Determination of cement content

As for cement stabilization, U.S. Army TM 5-882-14/AFM 32-1019 (1994) guideline was followed. The initial amount of cement recommended was based on USCS classification of the sample which is shown in Figure 3.5. The additive content was changed by 2% above and below the initial stabilizer content and the samples were studied for strength and durability. UCS samples for cement treatment were prepared for the initial additive amount as per NLA (2006) and PCA (1992), respectively. 7-days UCS strengths for cement treated samples are reported in Table 3.9 and Figure 3.5 along with their untreated strength. The lower amount of cement treatment for DC resulted in higher UCS strength in comparison to GF with a higher amount of cement treatment. Likewise, NTF_LP was treated with a lower amount of cement than NTF_HP but resulted in a higher strength increase for NTF_LP. Overall, it can be observed that cement stabilization resulted in UCS strengths considerably higher than that (50 psi) required for subgrade stabilization. It should be noted that the durability of the stabilization is discussed in Chapter 6.

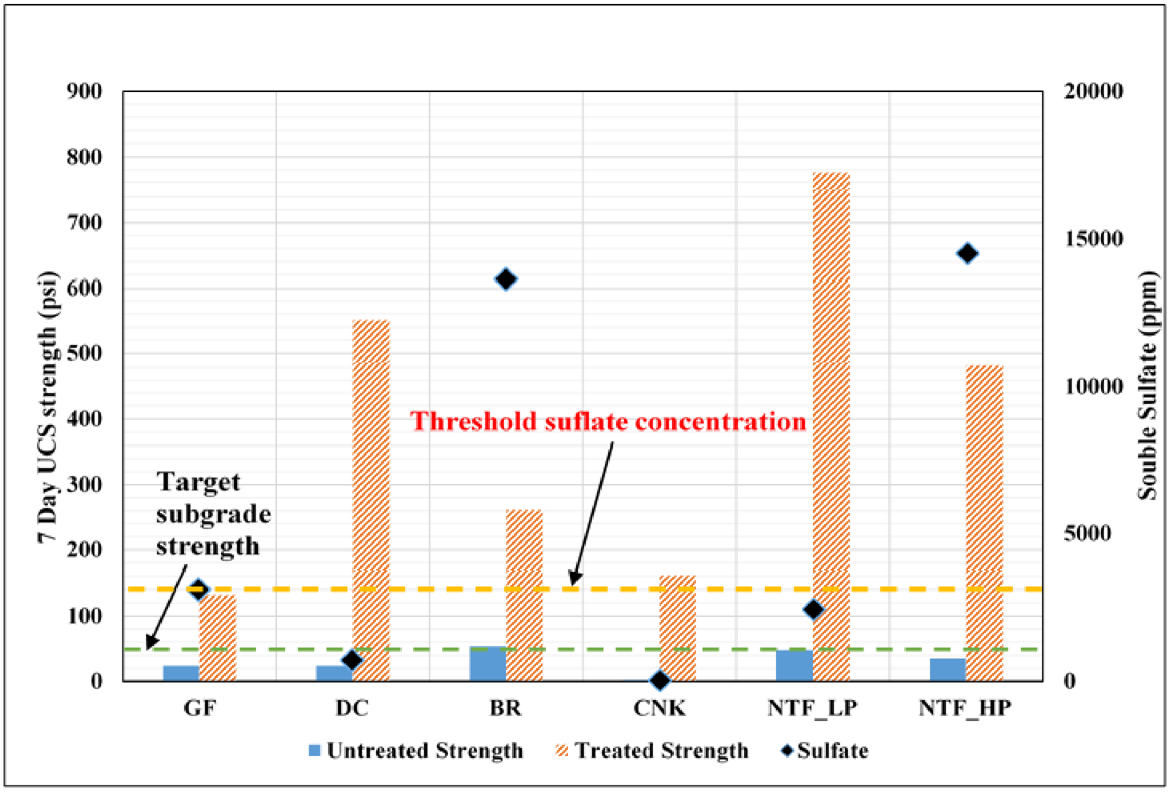


Figure 3.5: Plot showing cement stabilization results for the six soils

Table 3.9: Initial Stabilizer content for cement and UCS strength

Soil ID	USCS Classification	AASHTO Classification	Stabilizer content	UCS (psi) Treated	UCS (psi) Untreated
GF	CH	A-7	11%	131.87	24.08
DC	CL	A-6	9%	551.19	24.13
BR	CL	A-7-6	9%	260.78	53.69
CNK	SM	A-2-4	7%	161.904	1.67
NTF_LP	ML	A-5	9%	775.74	47.93
NTF_HP	CH	A-7-6	11%	483.027	35.03

3.6 Summary and Findings

Out of the six soils collected, there were two high plastic clays, two low plastic clays, one low plastic silt, and, one silty sand. Two of the soils (BR and NTF_HP) contained soluble sulfates in excess of 10,000 ppm which require special stabilization methods. Soils from BR, CNK, NTF_LP, and NTF_HP were excluded from lime treatment due to low plasticity index or high sulfate concentrations. Only GF and DC soils were selected for lime stabilization whereas all six soils were chosen for cement stabilization.

The lime treated soils (GF and DC) were able to achieve UCS strengths above the target of 50 psi. It was later observed upon further testing that the optimum percentage of the line for GF and DC was 4% and further increasing the amount of lime decreases the strength for these soils (section 5.2). When treated with cement, all soils except DC had UCS values higher than target strength. However, the strength values obtained in this chapter only reflect the short-term strength improvements and need to be tested for durability (chapter 6).

4 ESTABLISH CHEMICAL AND MINERALOGICAL CHANGES

4.1 Introduction

Chemical and mineralogical changes due to chemical treatments were studied for observations that could later help in establishing stabilization guidelines tailored specifically to Montana soils. The evaluation was performed by comparing the following properties between untreated and treated soils:

1. Cation Exchange Capacity
2. Specific Surface Area
3. Reactive alumina and silica
4. X-ray diffraction (XRD)
5. Scanning electron microscopy (SEM)

Please note that these comparisons were made on soil samples treated for the UCS test and the chemical and mineralogical measurements were made after the UCS test is complete. The testing procedures were identical to those of untreated soil samples as described in Chapter 3. Table 4.1 and Table 4.2 summarize the CEC, SSA and Reactive alumina/silica data for lime and cement treatments. The following sections discuss the results.

Table 4.1: Change in chemical properties for lime treated soils

Soil ID	CEC (meq/100gm)		SSA (m^2/g)		Reactive Alumina (ppm)		Reactive Silica (ppm)	
	Untreated	Treated	Untreated	Treated	Untreated	Treated	Untreated	Treated
GF	242	258	400	380	118	158	492	131
DC	75	124	111	110	106	117	241	95

Table 4.2: Change in chemical properties for cement treated soils

Soil ID	CEC (meq/100gm)		SSA (m^2/g)		Reactive Alumina (ppm)		Reactive Silica (ppm)	
	Untreated	Treated	Untreated	Treated	Untreated	Treated	Untreated	Treated
GF	241.6	281.4	400	309	118	180	492	63
DC	75.54	98.3	111	101	106	93	241	32
BR	19.03	46.69	52	44	50	81	35	16
CNK	8.29	38.06	33	16	13	92.68	59	14.71
NTF_LP	86	124.38	85	126.82	107.8	83.8	1135	629.2
NTF_HP	120	145.52	146.29	136.76	91.07	69.19	519	94.48

4.1 Cation Exchange Capacity

Figure 4.1 presents the CEC results before and after treatments for lime and cement additives. It can be observed from both Figures that CEC values increased after both lime and cement treatments. This change is due to the presence of free calcium present in the soil due to the addition of lime and cement. An increase in CEC after treatment is a common occurrence after stabilization.

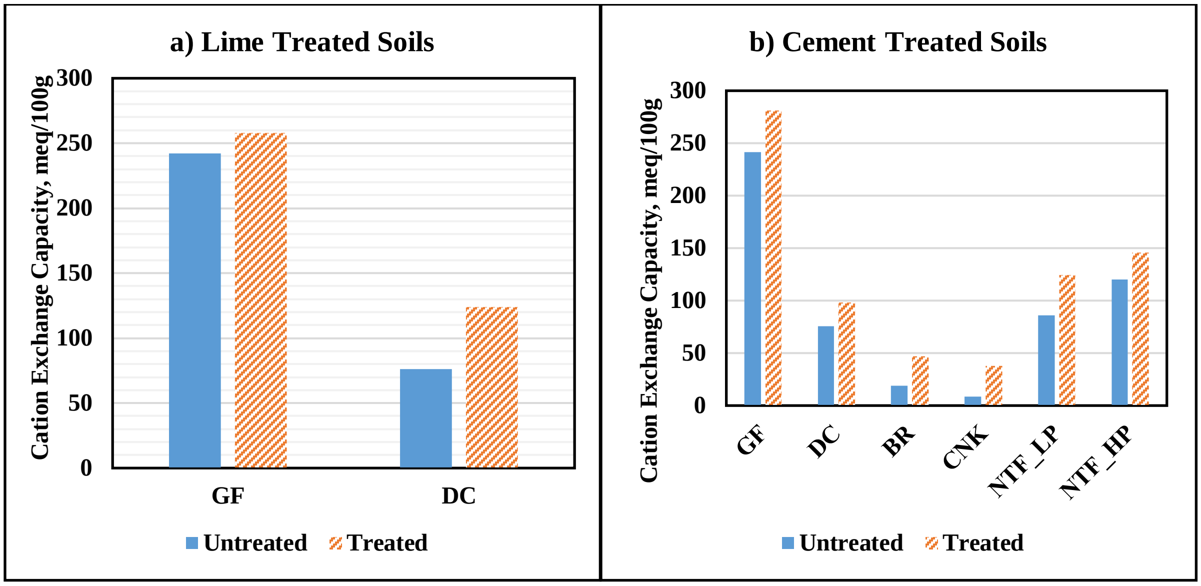


Figure 4.1: Comparisons of CEC test results before and after treatments with both lime and cement

4.2 Specific Surface Area

Figure 4.2 presents the comparisons of SSA data before and after treatments. It can be noted from the figures that the treated SSA values were generally less than untreated, indicating the formation of newer compounds with larger particle size. Because lime stabilization results in flocculation and agglomeration, the particles become coarser resulting in a reduction in SSA. A similar observation can be made after cement stabilization for all soils except for NTF_LP. One reason for this non-typical behavior could be that some of the cement added to this soil did not react with the soil, potentially increasing the portion of smaller particles and resulting in an increase in SSA.

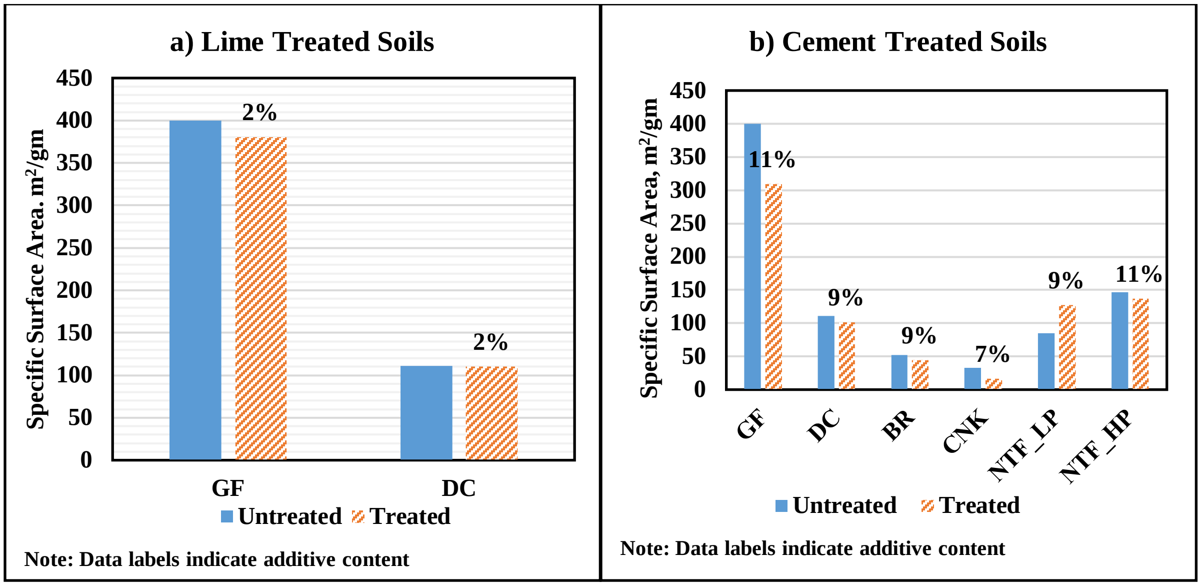


Figure 4.2: Comparisons of SSA test results before and after treatments with both lime and cement

4.3 Reactive Alumina and Silica

Figure 4.3 and Figure 4.4 present the before and after treatment comparisons for reactive alumina and reactive silica data, respectively. In the case of lime stabilization, reactive alumina increased after treatment whereas reactive silica decreased substantially for both the soils. The reduction in reactive silica indicates that some of the silica has participated in the pozzolanic compound formation. The increase in the reactive alumina could be attributed to the excess release of alumina at high pH resulting from the addition of lime.

In the case of cement stabilization, the reduction in reactive silica is consistent among soils, while the increase in reactive alumina was not. Reactive alumina increased after treatment for GF, BR and CNK soils (by 53% to 613%) while it reduced for DC, NTF_LP, and NTF_HP (by 12% to 24%).

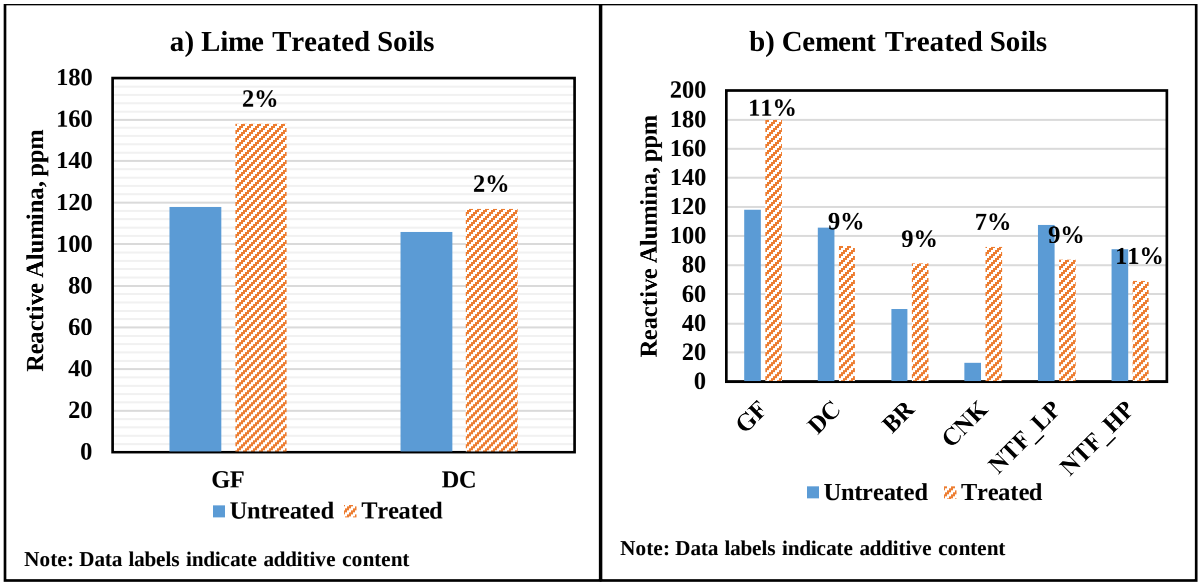


Figure 4.3: Comparisons of reactive alumina test results before and after treatments with both lime and cement

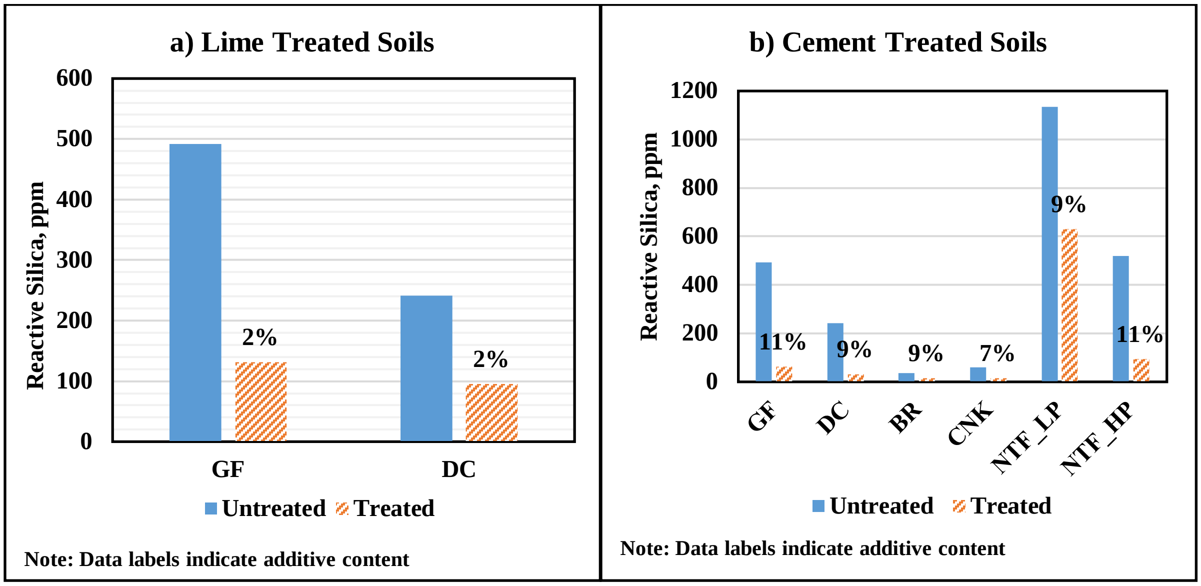


Figure 4.4: Comparisons of reactive silica test results before and after treatments with both lime and cement

4.4 X-ray Diffraction Studies

Crystalline clay minerals can be identified by X-ray diffraction studies since each crystal contains planes of atoms separated by a constant distance (Whittig and Allardice 1986). The X-rays that are diffracted from the crystalline mineral structure are converted to digital data. This data can be used to measure the distance between the planes of the atom using Bragg's law. It is assumed that no two minerals have the same interatomic distance in three dimensions for the identification of mineral crystals. Powder diffraction methods, in which a sample of particles containing all possible orientation is kept under X-ray, is generally used for soils as a large number of particles ensures that some will be oriented in the right way to produce the desired reflection (Whittig and Allardice 1986). Air dried and hand crushed samples passing #200 sieve were placed on the sample holder as shown in Figure 4.5. The diffraction studies were conducted by using a CuKα Rigaku Miniflex 600 X-ray diffractometer with an input voltage of 40kV and a current of 15mA which is shown in Figure 4.6. A step scan mode with a step size of 0.02° of 2Θ from 5°-80° was selected.

XRD results for untreated and treated soils are presented in Figure 4.7 and Figure 4.8 respectively. Peaks associated with natural minerals such as Montmorillonite (M), Kaolinite (K), Illite (I) and Quartz (Q) are identified in both the plots. In this analysis, it is assumed that only these minerals were present in the finer fraction of the soil. In the case of untreated soil, only GF showed a distinct peak for Montmorillonite (M). Excessive swelling characteristics of GF in comparison to other soils could be attributed to the presence of this mineral. GF and DC had peaked for Illite (I) in between 10°-20°. As for Kaolinite (K) and Quartz (Q), almost all the soils showed their presence. Peaks associated with Montmorillonite and Illite minerals in untreated soils have reduced or vanished after treatment as seen in Figure 4.8.

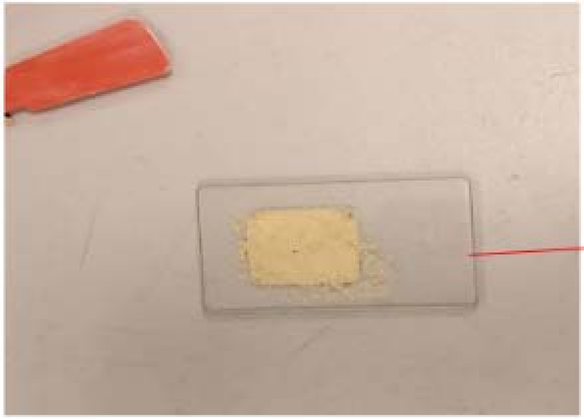


Figure 4.5: Sample holder and soil sample

Figure 4.6: Sample loading in Rigaku Miniflex 600 X-ray diffractometer

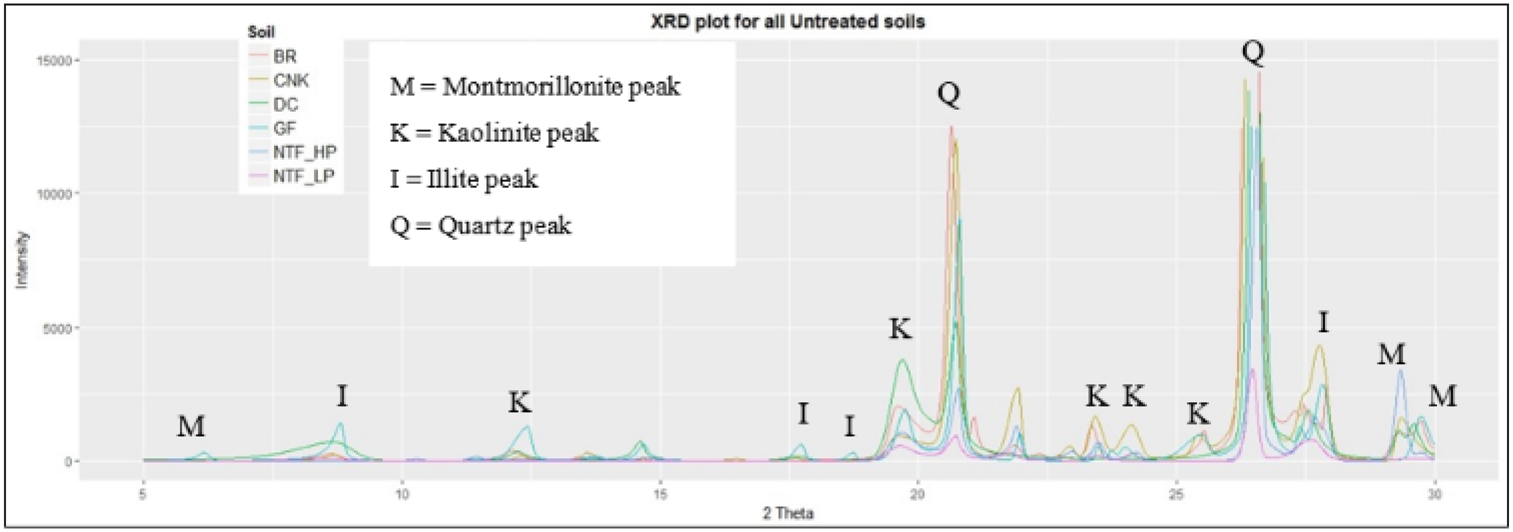


Figure 4.7: XRD plot for all untreated soils

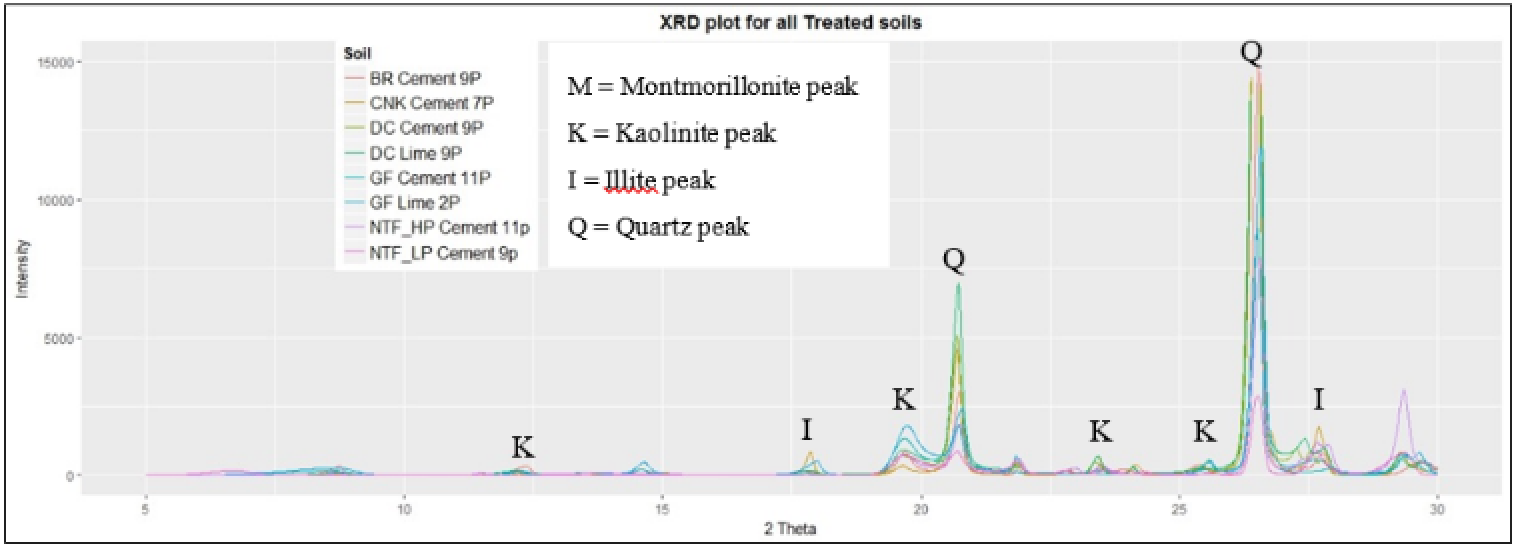


Figure 4.8: XRD plot for all treated soils

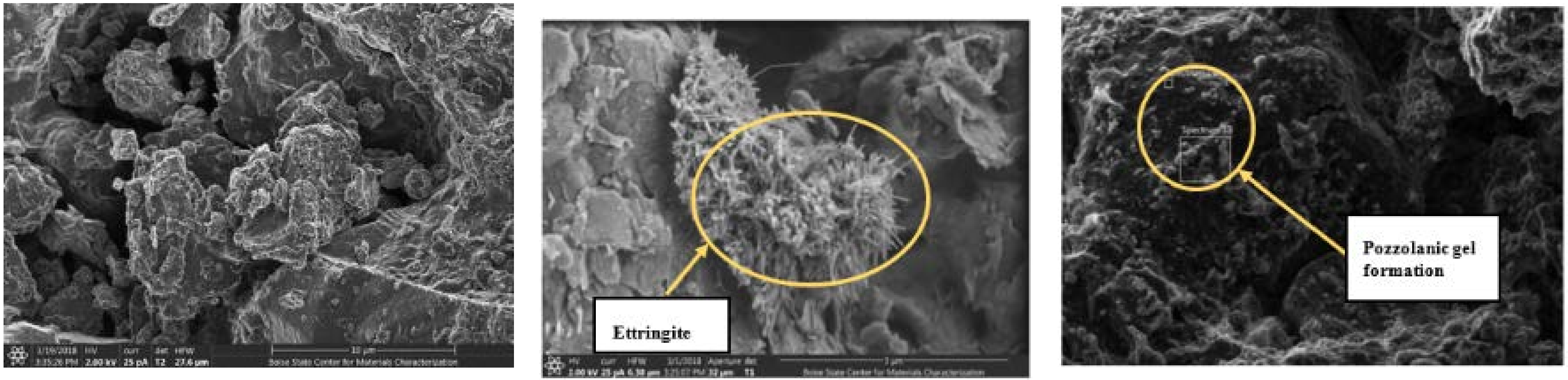
4.5 Scanning Electron Microscopy (SEM)

Mineralogical studies including Scanning Electron Microscopy (SEM) and Electron Dispersive X-ray analysis (EDS or EDX) were performed on both treated and untreated soil samples. These tests were carried out in Idaho Microfabrication Laboratory (IML) facility at Boise State. SEM analysis allows a closer examination of the soil matrix, which helps detect matrix changes after treatments. The magnification range of well over 100,000 times and large 3-D depth field, yield substantial information on the specimen surface structures and topography. Most of SEM instruments are equipped with energy dispersive X-ray spectrometry (EDS) which provide information on compositional characteristics in addition to the visual characteristics. EDS helps in determining the elements/compounds formed at the particle level and thereby the formation of cementitious and pozzolanic compounds. FEI Tenso – a Field Emission Scanning Electron Microscope (FESEM) was used for this study and is shown in Figure 4.10. The high vacuum was applied to the sample with an accelerating voltage of 2kV and a current of 25μA with T2 secondary electron detector. Since clay particles are charged particles, the samples were coated with carbon to reduce charge interference. Coating the clay particles with a thin layer of carbon resulted in a conductive surface that reduces the charging in the particles. A carbon-coated sample is shown in Figure 4.9. This sample is inserted into the vacuum chamber as shown in Figure 4.10.

Figure 4.9: Carbon coated sample for SEM imaging

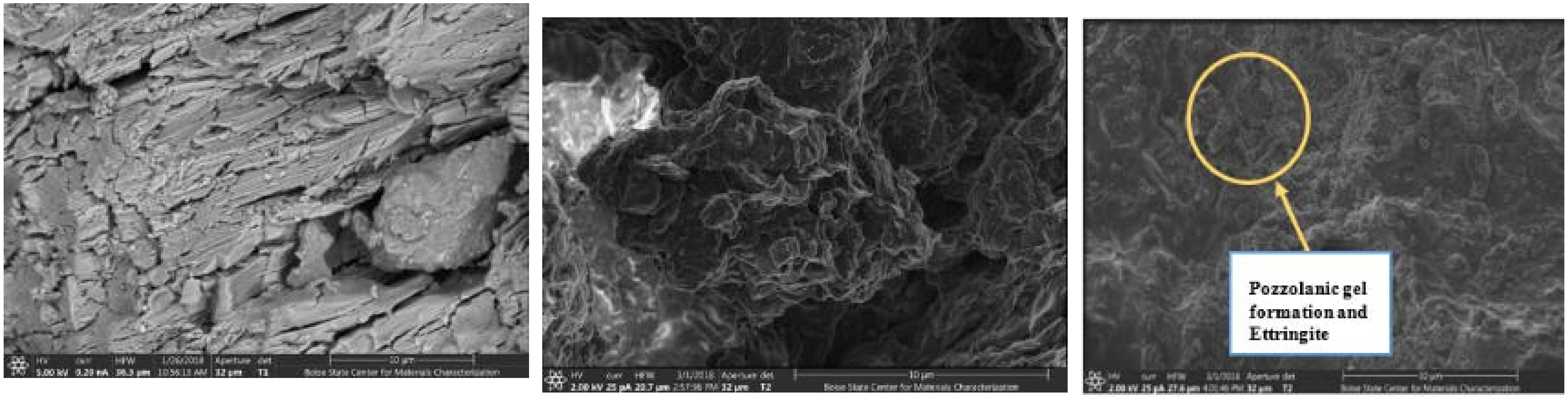
Figure 4.10: FESEM used for imaging

Figure 4.11 and Figure 4.12 show images for GF and DC soil, respectively. Each of these figures has images for untreated, lime treated and cement treated soils labeled (a), (b) and (c), respectively. Significant changes/presence of cementing gel in lime-treated GF and DC couldn't be observed for lime treated samples of these soils as seen in Figure 4.11 (b) and Figure 4.12 (b), respectively. Cement treated samples of these soils showed pozzolanic compounds were visible, as seen in Figure 4.11 (c) and Figure 4.12 (c). Figure 4.13 to Figure 4.16 present images for BR, CNK, NTF_LP, and NTF_HP, respectively. Since these soils were treated only with cement, each of these figures has images for untreated and cement treated soils only, with distinct characteristics highlighted in each figure. Typical ettringite formation can be observed in GF (Figure 4.11 (b)), BR (Figure 4.13 (b)), and, NTF_HP (Figure 4.16(b)).



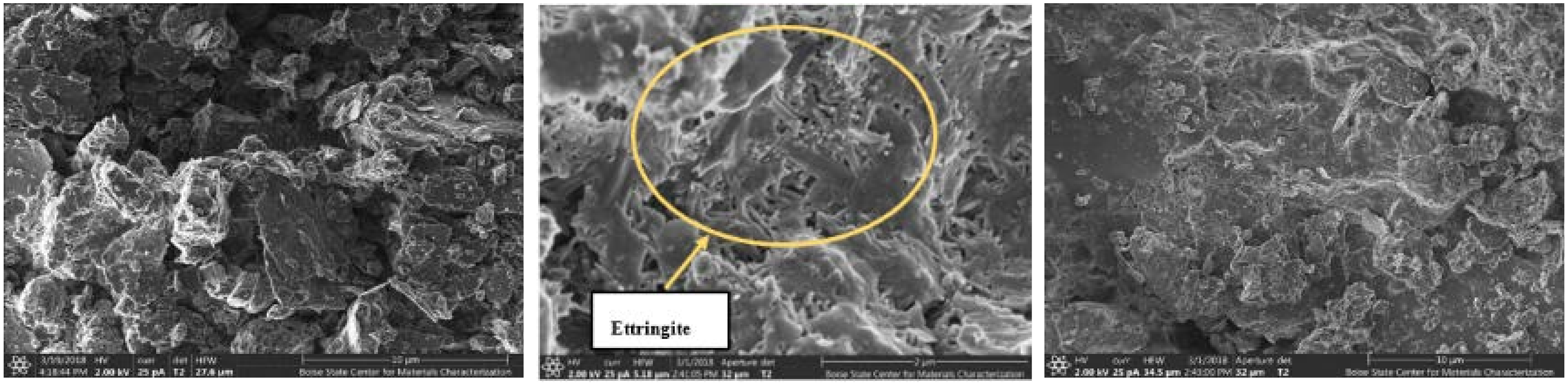
(a) (b) (c)

Figure 4.11: SEM Images of GF soil before and after treatments (a) Untreated (b) 2% Lime treated (c) 11% Cement treated



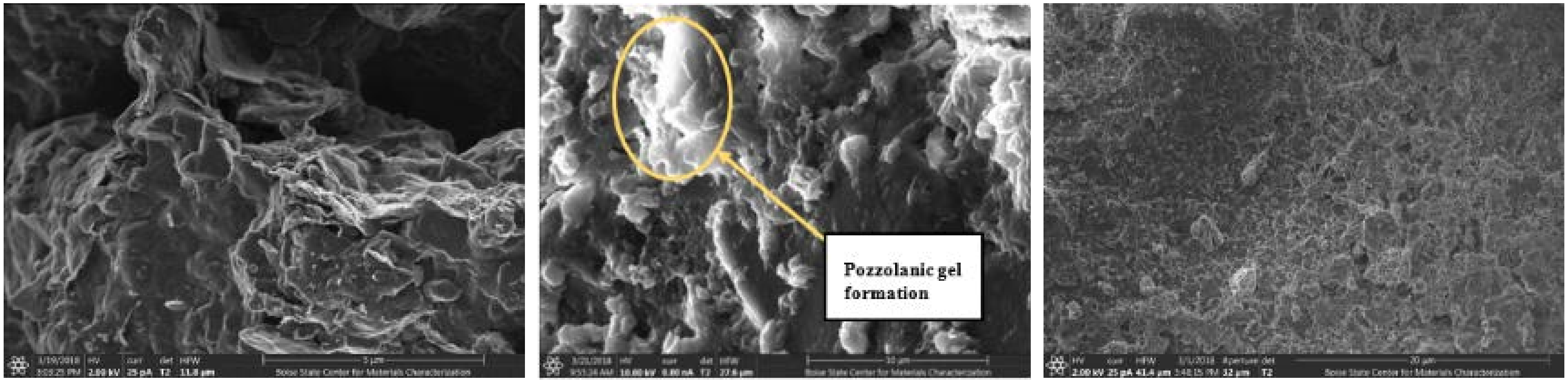
(a) (b) (c)

Figure 4.12: SEM Images of DC soil before and after treatments (a) Untreated (b) 2% Lime treated (c) 9% Cement treated



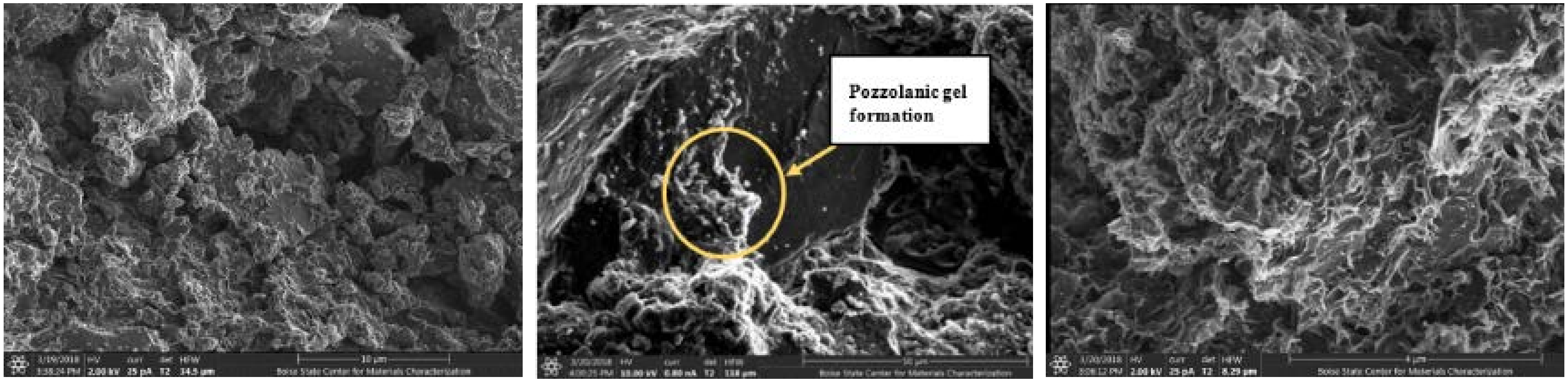
(a) (b) (c)

Figure 4.13: SEM Images of BR soil before and after treatment (a) Untreated (b) 9% cement with ettringite (c) 9% cement



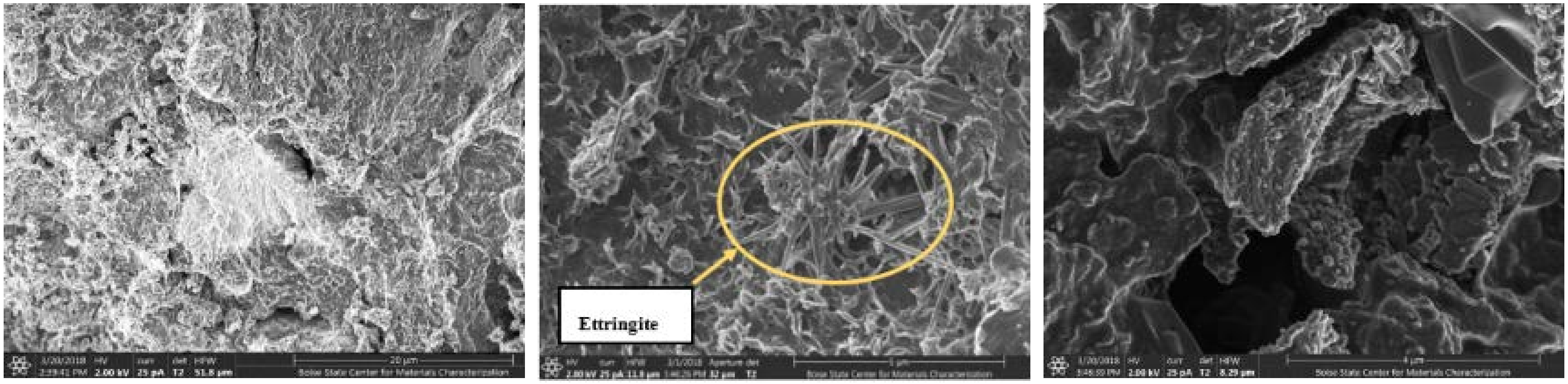
(a) (b) (c)

Figure 4.14: SEM Images of CNK soil before and after treatment (a) Untreated (b) 7% cement pozzolanic gel (c) 7% cement



(a) (b) (c)

Figure 4.15: SEM Images of NTF_LP soil before and after treatment (a) Untreated (b) 9% cement with visible cementation (c) 9% cement



(a) (b) (c)

Figure 4.16: SEM Images of NTF_HP soil before and after treatment (a) Untreated (b) 11% cement with ettringite (c) 11% cement

4.6 Summary and Findings

Chemical and mineralogical changes due to chemical treatments were studied as a part of task 4 of this project. The following observations were made:

1. Reactive alumina measurements increased after lime stabilization while results were mixed after cement stabilization.
2. Reactive silica measurements reduced after both lime and cement stabilizations.
3. CEC measurements showed an increase after treatments due to the presence of free calcium in the soil system.
4. The specific surface area generally reduced after both lime and cement stabilization except for NTF_LP soil.
5. X-ray diffraction studies indicated the presence of swelling minerals in case of high plastic soils which vanished after both lime and cement stabilizations.
6. SEM images confirmed the formation of pozzolanic compounds in the soil matrix for both treatments.
7. SEM images also showed Ettringite crystals in soils with high sulfate concentrations.

The goal of the tests performed in the section was to study if the trends in chemical and mineralogical changes after chemical treatments are consistent with general observations made in other studies and there were no surprises in the way the chemicals are reacting with Montana soils. The tests confirm the physical, chemical and mineralogical changes that are consistent with expected changes.

5 ESTABLISHING CURING PROTOCOLS

5.1 Introduction

The success of soil stabilization is dependent on the proper selection of type/amount of stabilizer with appropriate curing and moisture conditioning that yield acceptable strength and durability characteristics (Mitchell, J K, Hooper 1961 & Celaya et al. 2011). These characteristics are dependent on the duration and temperature of curing of the specimen as well as the type of moisture conditioning of the specimen. The goal of this section is to develop curing and moisture conditioning methods that accelerate the laboratory mix design process while maintaining similar strength characteristics. Therefore, an attempt was made to shorten the duration of curing by elevating the curing temperature to produce similar Unconfined Compressive Strength (UCS) as that obtained after typical 7-day curing. In the following section, some of the commonly used curing methods, as well as novel methods proposed by this study, are discussed.

5.2 Conventional curing method

Conventionally cement treated soil samples are cured at 25°C at 100% humidity. This procedure removes the need for moisture conditioning as the moisture is not allowed to escape from the sample and hence no need to test its susceptibility to it. All soils tested in this research were first subjected to the conventional curing method to establish a baseline of target strengths to be compared to the newly developed curing protocols. For this purpose, cylindrical samples of height 5.6 in. and diameter 2.8 in. were prepared at optimum moisture content (OMC) and maximum dry unit weight (MDUW) for various percentages of lime and cement. The samples were then wrapped with impermeable wrapping material making it airtight to avoid the loss of moisture during curing (See Figure 5.1). In addition to wrapping, the samples were placed inside a chamber with 95%±5% humidity and temperature of 23°C (±2°C). This insured a minimum loss of moisture during curing. The setup used for this process is shown in Figure 5.2.

The percentages of lime and cement targeted a cut-off strength of 50 psi – as discussed earlier in this paper. All the soil samples except CNK, which is a silty sand, were treated with minimal quantities of lime (2%). Although, NTF_LP and NTF_HP soils do not qualify for lime treatment, they were added to the testing matrix to evaluate short-term strength gains.

It should be noted that CNK soil is a Silty Sand with 14% fines. As a result, conducting UCS tests on these samples was challenging and may not be representative of the true strength improvements in this soil. Hence, California Bearing Ratio (CBR) test as per ASTM D1883-07 was conducted on these soil samples to better represent the strength characteristics. The untreated and treated CNK specimens were tested in the un-soaked condition with standard energy of compaction. For the treated sample, the sample was cured with CBR mold inside a chamber with 95% (±5%) humidity and temperature of 23^0C ($\pm 2^0$C).

Figure 5.1: Sample wrapped in cellophane sheet to ensure no loss of moisture

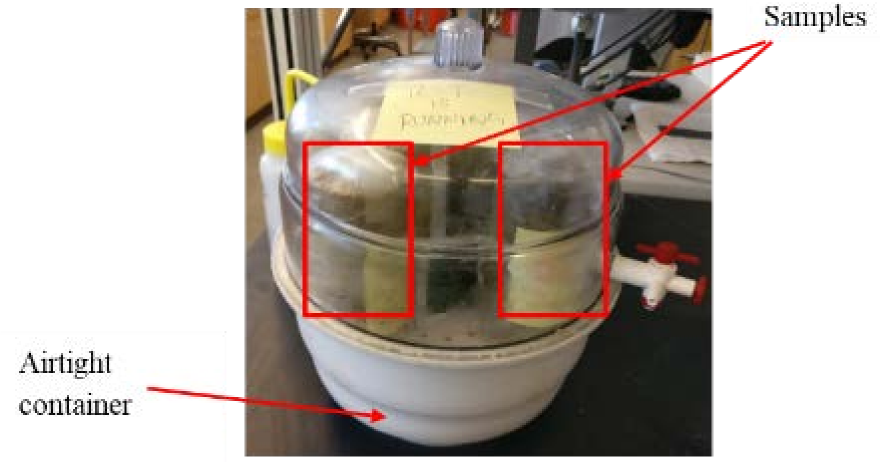


Figure 5.2: Samples wrapped in cellophane (red boxes) being cured in an airtight container at 95% (± 5%) humidity and 23°C (± 2°C).

5.2.1 Results from the conventional curing method

5.2.1.1 Cement treated Samples

The initial amount of stabilizer used was per recommendations of various standard guidelines from federal and state agencies. These recommended amounts correspond to the maximum additive percentage used for each soil in this study. Other treatment dosages were examined to find a suitable amount that will result in a UCS value of 50 psi, and these results are presented in Table 5.1. The UCS and secant moduli are illustrated in Figure 5.3, and Figure 5.4, respectively. It can be noted from these figures that high plastic soil such as GF showed the low improvements with cement treatment while DC, BR, and NTF_LP, which are low plastic clays/silts, showed a strong increase in strength with increase in stabilizer content. For these soils, the addition of a minimal percentage of cement was sufficient to increase the UCS value to 50 psi.

Table 5.1: Summary UCS and secant moduli for cement treated samples cured at 100% humidity and 23°C (± 2°C)

Soil ID	USCS	AASTHO	Cement Content (%)	UCS (psi)	% Change (compared to untreated soil)	Secant Modulus (psi)	% Change (compared to untreated soil)
GF	CH	A-7	0	24.08	N/A	494.43	N/A
			3	27.45	14	899.43	82
			6	25.24	5	685.09	39
			7	104.13	332	4,513.03	813
			9	85.27	310	2,542.07	414
			11	132.68	451	2,759.61	458
			13	131.77	447	3,501.94	608
DC	CL	A-6	0	24.13	N/A	352.20	N/A
			2	91.56	279	3,279.22	831
			4	197.97	704	7,902.67	2144
			6	257.04	938	11,596.71	3193
			7	256.65	898	14,945.06	4143
			9	551.19	2184	31,630.99	8881
			11	484.04	2065	21,896.64	6117
BR	CL	A-7-6	0	53.69	N/A	1,521.90	N/A
			3	96.42	80	4,520.81	197
			7	216.33	303	10,159.78	568
			9	262.29	386	13,381.09	779
			11	251.38	415	10,369.86	581
CNK	SM	A-2-4	0	1.67	N/A	44.07	N/A
			5	59.60	3469	2,598.87	5797
			7	161.90	9595	6,635.69	14957
			9	203.73	12099	5,276.16	11872
NTF_LP	ML	A-5	0	47.93	N/A	1,536.15	N/A
			3	81.76	71	25,160.27	1538
			7	473.17	887	31,544.67	1953
			9	776.10	1519	32,285.01	2002
			11	874.14	1724	41,407.73	2596
NTF_HP	CH	A-7-6	0	35.03	N/A	621.50	N/A
			3	140.62	301	6,307.92	915
			9	374.01	968	16,874.81	2615
			11	483.03	1279	33,422.94	5278
			13	440.68	1158	24,178.50	3790

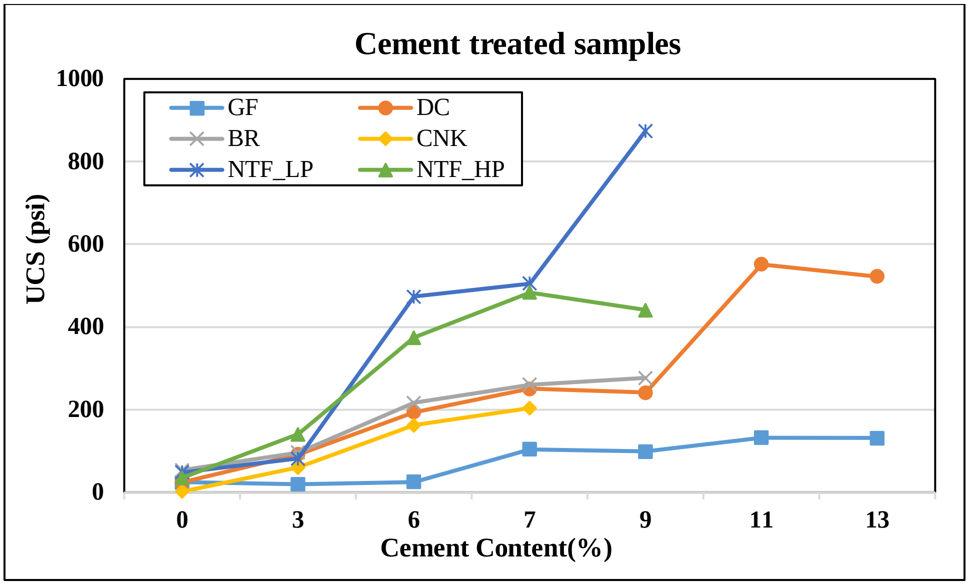


Figure 5.3: Unconfined compressive strength of 7- day cement treated samples at 100% humidity and 23°C (± 2°C).

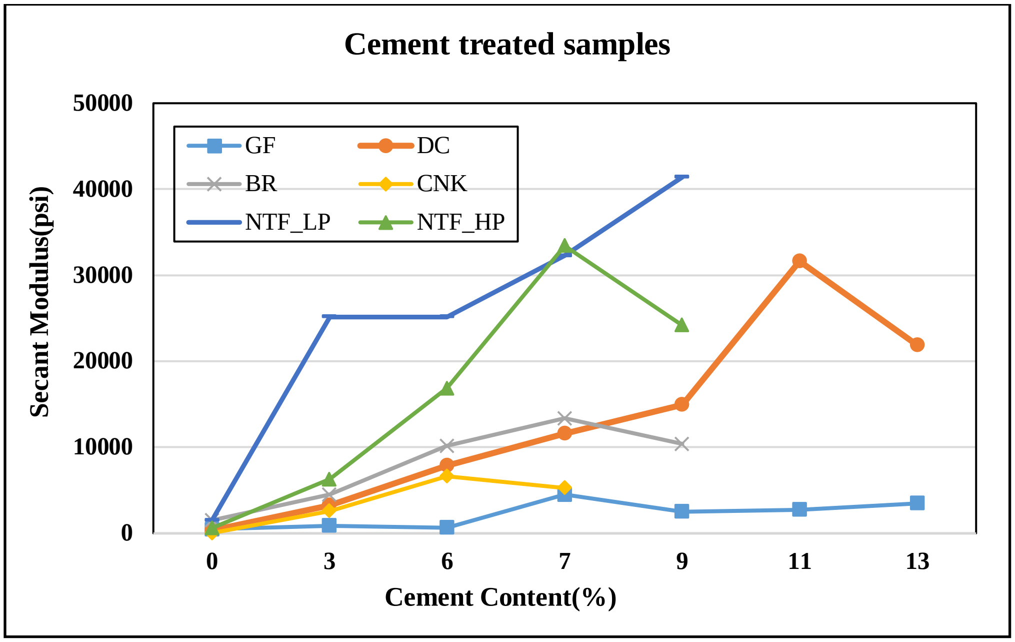


Figure 5.4: Secant modulus of 7- day cement treated samples at 100% humidity and 23°C (± 2°C).

The California Bearing Ratio (CBR) values for 0.1 in. and 0.2 in. penetration are given in Table 5.2. A treatment of 2% cement to the CNK soil increased the CBR value by three-fold, which suggests high suitability of cement stabilization for silty sands. CNK soil treated with 3% cement has a very high CBR value for 0.1 in. penetration. The same for 0.2 in. penetration couldn’t be determined, as the device capacity was reached before the 0.2 in. penetration.

Table 5.2: Summary of CBR results for CNK soil

Soil ID	USCS	AASTHO	Cement Content (%)	CBR (0.1 in) psi	CBR (0.2 in) psi
CNK	SM	A-2-4	0	28	34.9
			2	105.1	115.7
			3	186.5	> 200 (Exceeds device capacity)

5.2.1.2 Lime stabilized samples

The strength characteristics (UCS and secant modulus at peak strength) for lime treated samples are presented in Table 5.3. Graphical representation of UCS and secant modulus of lime treated soils are shown in Figure 5.5, and Figure 5.6, respectively. For GF soil, which is a highly plastic clay, 7% cement was required to increase the strength above 50 psi whereas 2% lime was sufficient for a similar amount of improvement. GF had the least increase in UCS value as well as secant modulus upon treatment with cement. This further confirms that lime is more suitable additive for treating GF soil. For lime stabilized soils, the maximum increase in the strength of GF and DC soils was observed with the addition of 4 percent lime. Further increase in the amount of lime led to a decrease in the strength of these soils. This threshold is called the lime fixation point, which represents an optimum point at which the amount of lime required for the pozzolanic reaction is equal to the available lime. Beyond this point, any excessive amount of lime reduces the strength because it does not have appreciable friction and cohesion (Bell, 1996a). Montgomery (2014) noted that at times this decline in strength exceeds 30% and is associated with a considerable decrease in dry unit weight.

The value of secant modulus also followed a similar trend to UCS. BR and NTF_HP soils have very high sulfate, and NTF_LP has very low plasticity which didn’t qualify them for lime treatment. Although, a small quantity of lime (2%) increased the strength of NTF_LP and NTF_HP above 50 psi. BR had the least increase in UCS after treatment compared to other soils as seen in Figure 5.5. The secant modulus of this soil seems to decrease after treatment, this could be due to the presence of lower amounts of reactive alumina and silica in BR soil compared to other soils. Lime reacts with alumina and silica to form pozzolanic compounds which contribute to an increase in strength, when the reactive alumina and silica of a soil are low, the excess lime cannot form pozzolanic compounds and the overall strength of the soil sample reduces. Therefore, the amount of reactive alumina and silica play an essential role in the performance of lime stabilization. It should be noted that none of the lime treated soils reached 150 psi which is the target for a base/subbase application per TxDOT (2005a).

Table 5.3: Summary of UCS and secant moduli for lime treated samples cured at 100% humidity and 23°C (± 2°C)

Soil ID	USCS	AASTHO	Lime Content (%)	UCS (psi)	% Change (compared to untreated soil)	Secant Modulus (psi)	% Change (compared to untreated soil)
GF	CH	A-7	0	24.08	-	494.43	-
			2	58.79	144	1,990.21	303
			4	79.74	231	2,520.75	410
			6	60.56	151	2,395.83	385
			8	58.10	141	1,691.37	242
DC	CL	A-6	0	24.13	-	352.20	-
			2	99.27	311	3,581.16	917
			4	231.38	327	5,069.16	1339
			6	92.48	283	5,143.11	1360
			8	73.75	206	3,062.34	769
BR	CL	A-7-6	0	53.69	-	1,521.90	-
			2	67.63	26	1,315.34	-14
NTF_LP	ML	A-5	0	47.93	-	1,536.15	-
			2	102.27	113	5,659.20	268
NTF_HP	CH	A-7-6	0	35.03	-	621.50	-
			2	58.75	68	2,223.72	258

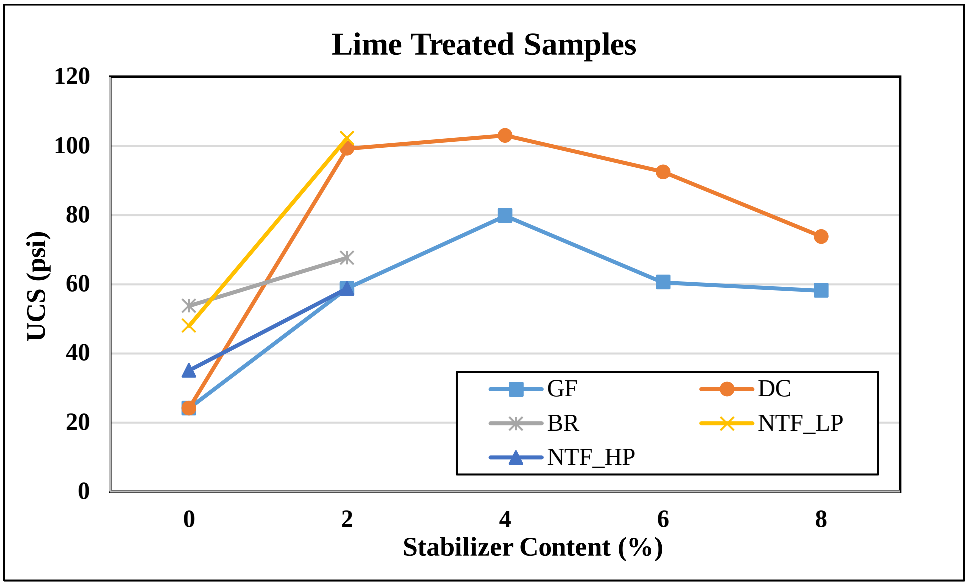


Figure 5.5: Unconfined compressive strength of 7- day lime treated samples cured at 100% humidity and 23°C (± 2°C).

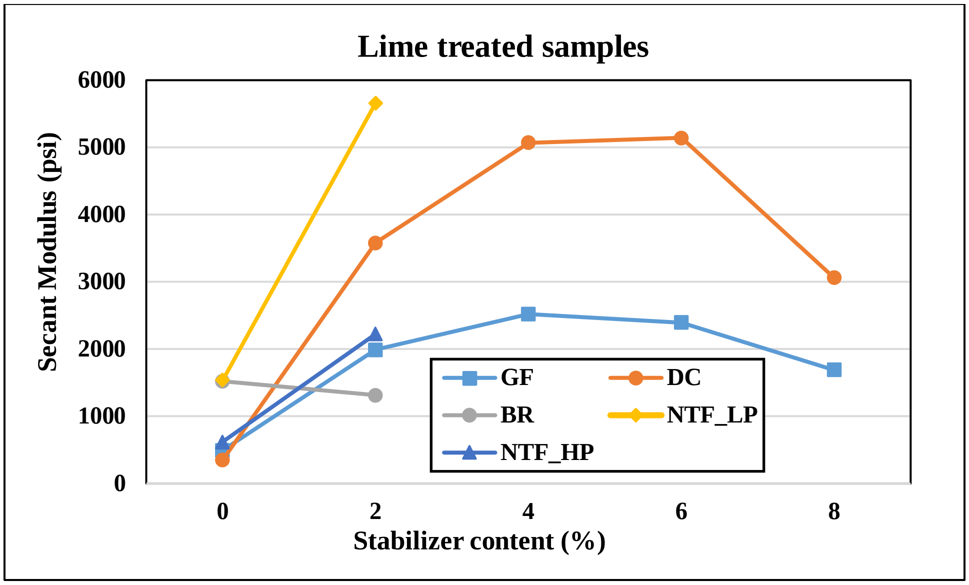


Figure 5.6: Secant modulus of 7- day lime treated samples cured at 100% humidity and 23°C (± 2°C).

5.3 Accelerated curing and moisture conditioning techniques evaluated in this study

Studies suggest a direct relationship between temperature and the rate of the pozzolanic reaction in stabilized soil. For example, a small increase in temperature can lead to improving strengths significantly. Conversely, temperature around 40˚F (4˚C) slows down the pozzolanic reactions and may stop it at lower temperatures. In fact, pozzolanic reactions may remain dormant during periods of low temperatures to regain reaction potential when temperatures increase (Bell 1996*a* and Celaya et al. 2011). Pozzolanic activity commences after only 1 day of curing at 72°F or 25°C whereas, it takes 7 days of curing at a lower temperature of 11.5°C. These suggest that strength development from the pozzolanic activity will occur more quickly in hot semi-arid climatic zones than in cool temperature zones (Rao and Shivananda, 2005). To assess the effect of temperature on curing for the soils selected in this study one additional curing temperature was studied for two different durations. Table 5.4 presents a summary of the accelerated curing protocols studied in this research.

Table 5.4: Accelerated curing protocols studied in this research

	Temperature	Duration	Humidity
Accelerated Curing Protocol-1 (ACP-1)	**140°F**	**1 day**	**Not controlled**
Accelerated Curing Protocol-2 (ACP-2)	**140°F**	**2 days**	**Not Controlled**

5.3.1 Role of moisture

The intrusion of moisture into the soil has a significant adverse effect on the performance of the pavement. The leaching of additives out of the host materials due to moisture movements may result in a variation of pH and Calcium/Magnesium ratio, and this will have severe implications on the sustainability of the chemical treatment (Veisi et al. 2010b). If the amount of additive does not ensure proper strength, stiffness, and durability, the stabilization would be ineffective and will result in costly rehabilitations. For this purpose, samples that were cured without moisture control were subjected to moisture conditioning via capillary saturation for several additional days and their unconfined compressive strength was measured. For the samples cured at higher temperatures (ACP-1 and ACP-2), the moisture conditioning was performed by fully submerging the samples under water for five hours. This submergence protocol was adapted from Veisi et al. (2010a).

As part of the protocol, the stabilized soil sample was prepared at their optimum moisture content (OMC) and maximum dry unit weight and (MDUW). The samples were dried for 48 hours (similar to the study of Veisi et al. (2010a) at 65.5^0C (150^0F) (Figure 5.7). Immediately following the oven-drying, the samples were moisture conditioned by submerging in the water as shown in Figure 5.8. For the moisture conditioning, the sample is kept inside a latex membrane with porous stone on both sides to avoid direct contact between soil and water and minimize surface erosion. An additional protocol with a period of 24 hours of oven-drying was also studied. After moisture conditioning, samples were tested for unconfined compressive strength (UCS) as shown in Figure 5.9. It can also be noted from this figure that the moisture percolation was not complete in five hours and the moisture was not uniform across the sample.

Figure 5.7: Oven-drying for a given amount of time.

Figure 5.8: Moisture conditioning by submerging the specimen.

Figure 5.9: UCS testing of an accelerated cured sample.

5.3.2 Novel curing protocol

The researchers at the SuRGE laboratory proposed a simplified version of accelerated curing by only increasing the temperature and keeping everything else constant (or close to static conditions). This protocol is called as humidity controlled accelerated curing (HCAC). In this protocol, the UCS sized samples were prepared at optimum moisture content and maximum dry unit weight and wrapped with impermeable material. The samples were placed inside a good quality zip-lock bag with water at the bottom of the bag. Heat resistant silicon plugs were placed inside the bag to separate the sample from the water. It was ensured that the samples rest on the silicone plug and do not touch the water. The primary intent of this setup was to avoid the submergence of the sample while maintaining 95% ± 5% humidity and the ability to put this whole setup inside a typical laboratory oven. The setup is shown in Figure 5.10. The whole setup is then kept inside the oven at 65.5°C (150°F) for 24 hours as shown in Figure 5.11. After 24 hours, the samples were tested for its strength characteristics.

Figure 5.10: HCAC Setup.

Figure 5.11: Placing the HCAC samples in the oven.

5.3.3 Results from accelerated curing protocols

A summary of all curing protocols conducted in this research is presented in Table 5.5. Cement treated GF, DC, BR and NTF_LP soil samples were subjected to all four protocols. The results were compared to evaluate which the three accelerated protocols predict UCS closest to the Conventional Curing Protocol (CCP). The protocol that best performed for cement treatments was then followed for lime treatments as well and compared with CCP data to ensure the protocol was yielding dependable results for lime treatments. The results from these analyses are presented in the following subsections.

Table 5.5 Summary of all curing protocols studied in this research

Protocol	**Temperature**	**Duration**	**Humidity**
Conventional Curing Protocol **(CCP)**	72°F	7 days	95%±5%
Accelerated Curing Protocol-1 **(ACP-1)**	150°F	1 day	Not Controlled
Accelerated Curing Protocol-2 **(ACP-2)**	150°F	2 days	Not Controlled
Humidity Controlled Accelerated Curing **(HCAC)**	150°F	1 day	95%±5%

Table 5.6: UCS test results for cement treated samples after different curing protocols

Soil ID	USCS	AASTHO	Cement Content (%)	CCP (psi)	ACP -1 (psi)	ACP -2 (psi)	HCAC (psi)
GF	CH	A-7	3	27.45	9.14	0*	37.15
DC	CL	A-6	9	551.19	190.12	299.16	477.34
BR	CL	A-7-6	9	260.78	215.50	320.03	384.93
NTF_LP	ML	A-5	9	776.10	160.12	84.02	475.00

Note: *Sample was too delicate and test could not be completed

5.3.3.1 Cement Treated Samples

UCS values of cement treated samples from different curing and moisture conditioning protocols are summarized in

Table 5.6 and visualized in Figure 5.12. The percentage of treatment was chosen as per the recommended guidelines discussed in sections 2.2.3 and 3.5. It can be noted in Figure 5.12 that HCAC curing method predicts the closest to CCP. At lower UCS values all three methods seem to predict well, however, at higher UCS values ACP-1 and ACP-2 seem to under predict. To better understand this data, the percentage differences in UCS values (compared to CCP strength) are tabulated in Table 5.7. The percentage difference between HCAC and CCP ranged from -48% to 39% while the same between ACP-1 and CCP ranged from 17% to 79%. In the case of ACP-2 the difference was between -23% and 100%. The average differences between the three new protocols and the CCP were 57%, 53%, and -8% for ACP-1, ACP-2 and HCAC, respectively. This shows that HCAC protocol is closest to predicting the CCP strengths on an average. Additionally, in contrast to ACP-1 and ACP-2 protocols, HCAC protocol has only one variable, i.e., temperature– enabling it to better represent the response of the stabilized soil and provide a better estimate of 7-day strength in most of the soil types.

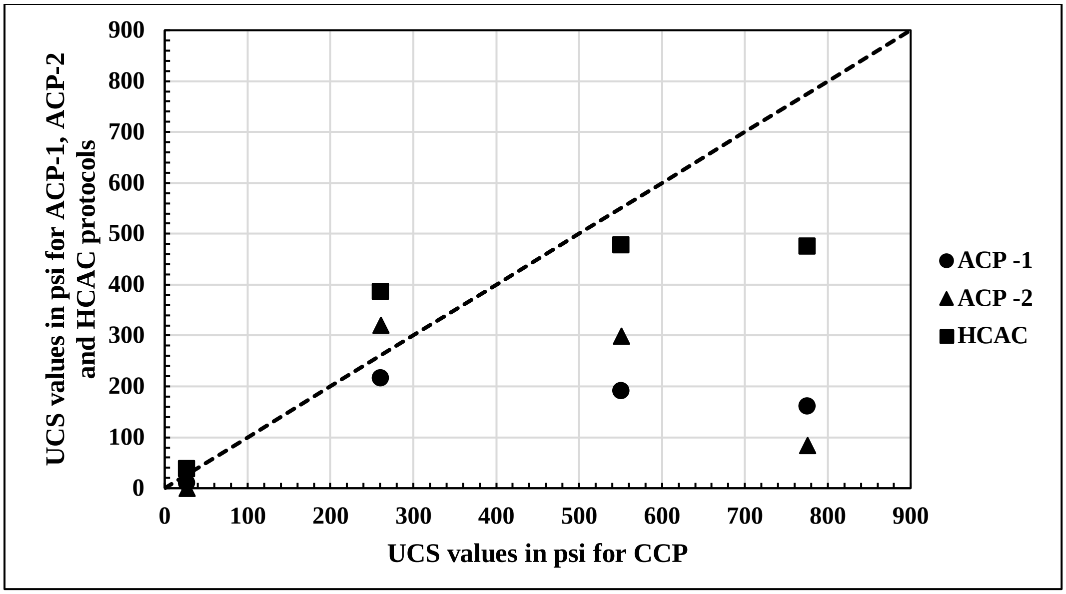


Figure 5.12: Comparing UCS data of cement treated soils for different curing methods

Table 5.7 Percentage difference in UCS values of the accelerated protocols compared to CCP

Soil ID	Cement Content (%)	CCP (psi)	Percentage Difference compared to CCP		
			ACP-1	ACP-2	HCAC
GF	3	27.45	67	100	-35
DC	9	551.19	66	46	13
BR	9	260.78	17	-23	-48
NTF_LP	9	776.1	79	89	39
Average Percentage Difference			*57*	*53*	*-8*

Further, the Student’s t-test was performed to determine if there were statistically significant differences between the protocols. Student’s t-test (Mongomery and Runger, 2014) is a commonly used statistical significance test when comparing two different protocols. The t-test is generally performed on samples of smaller sizes, and hence this test was best for this study as the sample size was five under each curing method (de Winter 2013). The t-test was conducted at different confidence levels (CL) indicating how likely are two protocols to be the same or different. The test results are presented in Table 5.8. These results show that the p-value for all three methods is above the significance level of 0.05 with the highest value of 0.832 for HCAC protocol. This indicates that HCAC is most likely to have UCS predictions similar to the conventional protocol (CCP).

Table 5.8: Statistical t-test results for the three accelerated curing protocols

Curing Protocol	**P-Value**	**Remarks**
ACP-1	0.077	The protocols **are significantly** different at 90% confidence level (CL) but **not significantly** different at 95% or 99% CLs
ACP-2	0.215	The protocols are **not significantly** different at 90%, 95% or 99% CLs
HCAC	0.832	The protocols are **not significantly** different at 90%, 95% or 99% CLs

5.3.3.2 Lime Treated Samples

UCS values for lime treated samples after different curing and moisture conditioning protocols are summarized in Table 5.9 and visualized in Figure 5.13. As seen in the case of cement treated soils, ACP-1 and ACP-2 almost always underestimated the 7- day UCS strength and caused samples to fail in some cases before strength measuring test could be performed. Therefore, HCAC was only used for the lime treated samples. The HCAC strength lies in between 50% and 171% of the 7-day strength. The lower percentage (underestimation) was seen in case of GF treated with lower lime content (i.e., 2%) whereas the high percentage (overestimation) was seen in the case of DC treated with 4% lime. Strength after HCAC on 4% GF and 2% DC was closer to 7-day strength. The p-value after the t-test on lime treated samples was 0.909 which is significantly higher than the accepted significance level of 0.05 (95%).

Table 5.9 Results of CCP and HCAC – Lime

Soil ID	**USCS**	**AASTHO**	**Lime Content (%)**	**CCP (psi)**	**HCAC (psi)**
GF	CH	A-7	2	58.70	31.38
			4	79.74	67.57
DC	CL	A-6	2	99.27	154.07
			4	231.38	287.38
			6	92.48	100.02
NTF_LP	ML	A-5	2	102.27	97.25

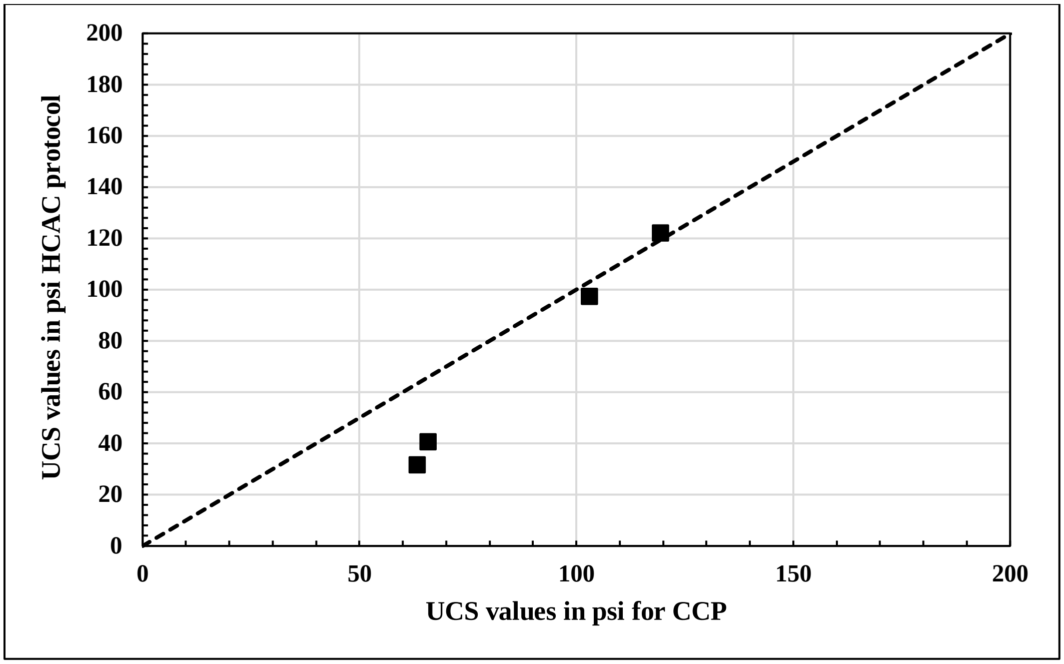


Figure 5.13: Comparing different curing methods based on the UCS data for lime treated soils

5.4 Summary and Findings

This chapter presented the results from testing accelerated moisture conditioning protocols. For this purpose, four different curing protocols were studied including, CCP, ACP-1, ACP-2, and HCAC.

The following observations were made:

1. All three accelerated curing techniques (ACP-1, ACP-2, HCAC) were determined to be producing similar results as CCP. However, HCAC is the most practical and highest reliable of the three.
2. For cement treated soils, the UCS value of the HCAC samples ranged from 61% to 209% of the CCP.
3. ACP-1 and ACP-2 procedures were problematic with GF soil due to surface cracking; this soil is a highly plastic soil.
4. HCAC cured lime treated soil samples had UCS values between 50% and 171% of the 7-day strength.

Curing and moisture conditioning methods for stabilized soil that accelerated the laboratory mix design while maintaining similar strength characteristics are preferred. Also, protocols that can be implemented comfortably in a typical geotechnical laboratory are preferred. Based on these requirements we recommend that HCAC protocol is used when time is of the essence. This method is simple, can estimate treated soil strength consistently, and is easier to conduct in the laboratory. CCP is recommended for all other cases.

6 DURABILITY STUDIES

6.1 Introduction

Durability studies were conducted primarily to simulate the seasonal moisture fluctuations that may transpire during summer and winter seasons. Past studies (Chittoori et al. 2018, 2009; Gabriele et al. 2016; Puppala et al. 2017; Uzer 2016) performed wet-dry and freeze-thaw related tests to address the durability issues and performance of stabilizers in different climatic conditions. These studies were reviewed, and test protocols that best simulate Montana environmental conditions were followed in this research. This chapter starts with a description of the freezing/thawing and wetting/drying protocols followed by an overview of the test results and significant findings. All of the tests were performed in the Sustainable and Resilient Geotechnical Engineering (SuRGE) laboratory at Boise State University.

6.2 Durability Studies

The main goal of this task was to study the longevity of the chemical stabilizers under different environmental conditions. Durability test protocols should replicate the seasonal changes experienced by the treated soil in the field. There are generally two types of such protocols, *Wetting/drying* (from here on referred to as W/D) and *Freezing/thawing* (from here on referred to as F/T). W/D protocol is generally preferred in dryer climates where prolonged summer at high temperatures (~100°F) can cause considerable drying. This, when followed by a rainfall event, may cause the treated soil to lose strength and/or have volumetric changes. F/T protocol is preferred in colder climates where the soil freezes during winter. This, when followed by thawing in summer, can have a similar effect and may lose the positive effects of stabilization. For the type of climate Montana experiences, the research team decided to proceed with F/T durability studies. However, W/D studies were conducted on two of the soils to understand its impact on the stabilization.

6.2.1 Freeze/Thaw Protocol

To study the performance of chemically treated soils in cold climates, treated soil specimens are exposed to repeated freezing and thawing to determine soil losses, moisture changes, and volume change. ASTM D 560 method is the standard method used for conducting F/T investigations and helps determine the minimum additive required to achieve and sustain adequate strength under field weathering. The following section details the test procedure followed in this research and the modifications made to the ASTM D 560 method. Figure 6.1 shows an example of cement-treated soil during the process of freezing, thawing, and UCS testing.

a. b. c.

Figure 6.1: Photographs showing the F/T Testing – (a) Samples inside freezer (b) Samples being thawed under humidity-controlled conditions (c) UCS sample being tested after several F/T cycles

Test method A of ASTM D 560 was followed using two Proctor-size specimens of dimensions, 4.58 in. (11.6 cm) in height and 4.0 in. (10.2 cm) in diameter. In addition, two UCS-size specimens {2.8 in. (7.1 cm) in diameter and 5.6 in. (14.2 cm) in height} were prepared. One of the two specimens (specimen #1) in both sizes (Proctor-size and UCS-size) was used to collect data on water content and volume change while the second specimen (specimen #2) for each size were used to gather data on soil loss during the durability cycles. The UCS-size specimens made it possible to determine UCS after the specimens have undergone F/T cycles. This was a modification from the standard ASTM D560 method, along with the measurement of volume change.

After samples were compacted to the required dimensions at OMC and MDUW, they were cured for 7 days. At the end of the 7th day, the samples were allowed to freeze inside a freezing cabinet at a temperature lower than -10°F (-23°C) for 24 hours. The weight and dimensions of all samples were measured to detect moisture and volume changes. All the specimens were then placed in a covered container at 70°F (21°C) and 100% relative humidity for 24 hours and allowed to thaw. The specimen #2 of both sizes were scratched with two firm strokes all around the sample with a wire brush to check for soil loss. Weight and dimensions of all four samples were measured. This constitutes one cycle. A total of 12 cycles or until specimen fails were performed. Volume and water content change in specimen #1 and soil loss in specimen #2 were reported. At the end of the 12th cycle, the UCS-size samples were tested for strength.

6.2.2 Wet/Dry Protocol

A correct approach to address the durability of chemically treated soils in arid environments is by exposing the treated soil specimens to various cycles of wetting and drying processes. During these processes, both volume and moisture changes along with soil loss can be determined. These properties will provide insights into the effects of seasonal moisture fluctuations on the soil property variations. ASTM D 559 method is the standard method often used for these W/D durability investigations. The following section details the test procedure followed in this research and the modifications made to the ASTM D 559 method. Figure 6.2 shows photographs of the wetting and drying setups used in this study.

Figure 6.2: W/D Testing Cycles – a) Wetting Cycle b) Drying Cycle

The procedure outlined by the ASTM D 559 method was closely followed in this research for the two soil samples tested for W/D durability. This method simulates both wet and dry cycle conditions close to field conditions in a reasonably short time period. The soil specimens were allowed to swell and shrink in both lateral and vertical directions. Test method A of ASTM D 559 was followed using two specimens of dimensions, 4.6 in. (11.7 cm) in height and 4.0 in. (10.2 cm) in diameter. The first specimen (specimen #1) was used to collect data on water content and volume change while the second specimen (specimen #2) was used to gather data on soil loss during W/D cycles. UCS tests were not performed on these samples.

After samples were compacted to the required dimensions at OMC and MDUW, they were cured for 7 days. At the end of the 7th day, the samples were submerged in potable water at room temperature for 5 hours. The weight and dimensions of specimen #1 were measured. Both specimens were then placed in an oven at 160˚F (71˚C) for 42 hours. After this time, specimen #2 was then scratched with two firm strokes all around the sample with a wire brush. Weight and dimensions of specimen #2 were measured. This constitutes one cycle. A total of 12 cycles or until specimens failed were performed. Volume and water content change in specimen #1 and loss of weight in specimen #2 were reported.

6.2.3 Testing Approach

A summary of all stabilizers content that satisfied the target UCS strength of 50 psi in this research is presented in

Table 6.1. The highest additive dosages (for both lime and cement) that passed the targeted strength of 50 psi were tested. If samples failed the durability tests at the highest dosage level samples were not tested with any further increase in dosage as that would turn out to be uneconomical beyond the highest dosage levels.

Table 6.1: Stabilizer contents that satisfied the target UCS

Soil	USCS Classification	Stabilizer Content	UCS Strength
LIME as Additive			
GF	CH	2%	58.8
		4%	65.0
DC	CL	2%	118.6
		4%	166.8
CEMENT as Additive			
GF	CH	7%	104.1
		9%	85.3
		11%	131.9
DC	CL	2%	91.6
		4%	197.9
		6%	276.0
		7%	256.6
		9%	551.2
		11%	484.1
BR	CL	3%	96.4
		7%	216.3
		9%	261.0
CNK	SM	3%	103.9
		7%	162.0
NTF_LP	ML	3%	81.7
		7%	473.4
		9%	775.7
NTF_HP	CH	3%	140.6
		9%	374.0
		11%	483.0

An additive dosage is considered to have passed the durability criterion if the percentage of weight loss observed after 12 cycles of durability (F/T or W/D) is within the limit shown in Table 6.2 for different soil types. These limits are established by the Army Corps of Engineers for stabilized pavement subgrades (U.S. Army TM 5-882-14/AFM 32-1019 1994). In addition to this criterion, the research team established a volumetric strain threshold of 9%. This percentage was established using the following approach. It was established by TxDOT (2005a) that a potential vertical rise (PVR) of 1 in. (2.5 cm) was acceptable for pavements on expansive soils. This PVR can be converted into an axial strain using a typical pavement section of 3 ft. (91.4 cm), which gives an axial strain of 2.7%. Since the deformation measured in both W/D and F/T protocols was volumetric, the axial strain was converted to volumetric strain using the Poisson's ratio of 0.3. This gave an acceptable volumetric strain of 4.3%. It should be noted here that PVR of 1 in. (2.54 cm) was established under the surcharge of the pavement section. In the current protocols (F/T and W/D), since there was no surcharge, and the samples were exposed directly to extreme conditions, the allowable volumetric strain could be higher. Hence, an allowable volumetric strain of 5% was targeted after 12 cycles of treatment for all soil types. It was targeted that the UCS cannot drop below 50% of the initial targeted 50 psi of strength after the durability treatments.

Table 6.2: Durability requirements

Soil	**USCS Classification**	**Maximum allowable weight loss**	**Maximum allowable volume change**	**Minimum allowable UCS (psi)**
GF	CH	6%	5%	25
DC	CL	6%	5%	25
BR	CL	6%	5%	25
CNK	SM	11%	5%	25
NTF_LP	ML	8%	5%	25
NTF_HP	CH	6%	5%	25

6.2.3.1 Volume and weight change measurements

The volume change was calculated as the difference between the volume of the soil specimen at the time of molding and subsequent volumes as a percentage of the original molding volume. Similarly, the soil loss percentage was calculated as a difference between the dry soil weight at the time of molding and subsequent dry weights as a percentage of initial dry weight (at molding). However, there is a portion of water that reacts with cement and stays in the soil after oven drying at 230°F (110°C), hence a correction was made to the measured dry mass as per ASTM D560. The corrected oven-dry weight of the soil sample after an F/T cycle was calculated as per Eq. (1).

$Corrected\ dry\ weight = \frac{A}{B} \times 100$ ------------(1)

Where,

A = oven-dry weight after drying at 230°F (110°C)

B = percentage of water retained in specimen plus 100

ASTM D560 recommends the use of prescribed values for B based on soil type. These values are from ASTM D560 are provided in Table 6.3 for reference. After calculating the corrected oven-dry weight, loss in the specimen was calculated as per Eq. (2)

$$Soil\ loss, \% = \frac{X}{Y} \times 100 \text{ ----------(2)}$$

Where,

X = original calculated oven-dry weight minus corrected oven-dry weight after F/T cycle

Y = original calculated oven-dry weight

Table 6.3: Average values of retained water after cement reactions (as per ASTM D560)

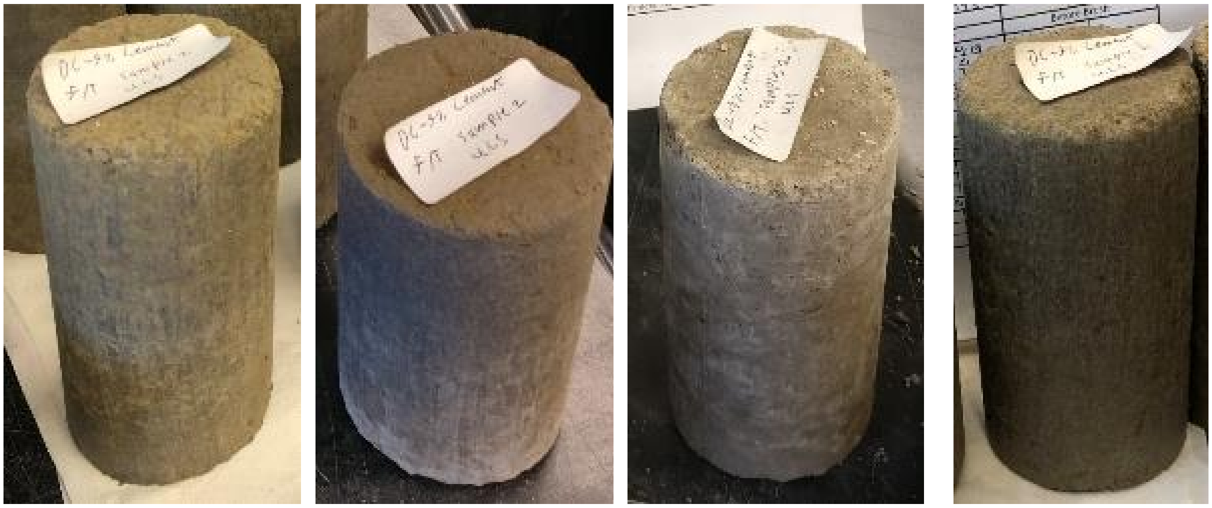
Soil Classification (ASTM D3282 or AASHTO M 145)	**Average water retained after drying at 230°F (110°C) %**
A-1, A-3	1.5
A-2	2.5
A-4, A-5	2.0
A-6, A-7	3.5

6.3 Results and Discussion

The relative merits of the stabilizers from volume change, weight loss, and UCS perspectives are discussed in the following section. Figure 6.3 and Figure 6.4 present example photographs showing how samples looked at different cycles of durability for DC soil. Figure 6.3 presents sample photos for DC soil treated with 4% lime, while Figure 6.4 presents DC soil treated with 9% cement. It can be observed from Figure 6.3 that DC soil treated with 4% lime experienced substantial distress during the durability cycles while Figure 6.4 clearly shows that DC soil treated with 9% cement did not show weakness with durability cycles. This could be due to the low plastic nature of this soil and lack of soluble sulfates in this soil.

After 4 F/T cycles **After 5 F/T cycles** **After 6 F/T cycles**

Figure 6.3: DC soil samples treated with 4% lime at different F/T cycles



After 5 F/T cycles **After 7 F/T cycles** **After 9 F/T cycles** **After 10 F/T cycles**

Figure 6.4: DC soil samples treated with 9% cement at different F/T cycles

6.3.1 Freezing/Thawing

Soil samples treated with both cement and lime additives were tested for F/T durability.

Table 6.4 presents the dosage at which durability testing was conducted in this research. Two out of six soils tested in this research qualified to be treated using lime based on current stabilization guidelines. All six soils were treated with cement as an additive and tested for durability at the dosages that satisfied the UCS requirement. The following sections present this data.

Table 6.4: Initial Stabilizer Content used in freeze-thaw durability studies

Soil	USCS Classification	Stabilizer Content
Lime		
GF	CH	4%
DC	CL	4%
Cement		
GF	CH	11%
DC	CL	9%

BR	CL	9%
CNK	SM	7%
NTF_LP	ML	9%
NTF_HP	CH	11%

6.3.1.1 Lime Treatment

DC and GF soils met the criterion to be stabilized with lime, based on PI, gradation, and soluble sulfate content as per the current stabilization guidelines. For both soils, dosages of 2% and 4% satisfied the target strength requirement of 50 psi; however, the higher dosage was tested. Figure 6.5 presents the volume change data obtained from specimen #1 for both soils tested using lime. It can be observed from Figure 6.5 that both soils experienced significant volume changes after three F/T cycles which indicates that the positive effects of stabilization were lost after three F/T cycles, clearly indicating that this dosage was not durable in the long-term.

Figure 6.5 shows the loss in soil over the different F/T cycles for DC and GF soils. It can be observed the weight loss threshold of 6% (Table 6.2) was exceed by GF soil after three F/T cycles and by DC soil after four F/T cycles. The samples were also not intact after the 10th cycle to test for UCS. This indicates that the stabilizer contents are not feasible in the long-term for these soils. Higher dosages may be needed for effectively stabilizing these two soils with lime.

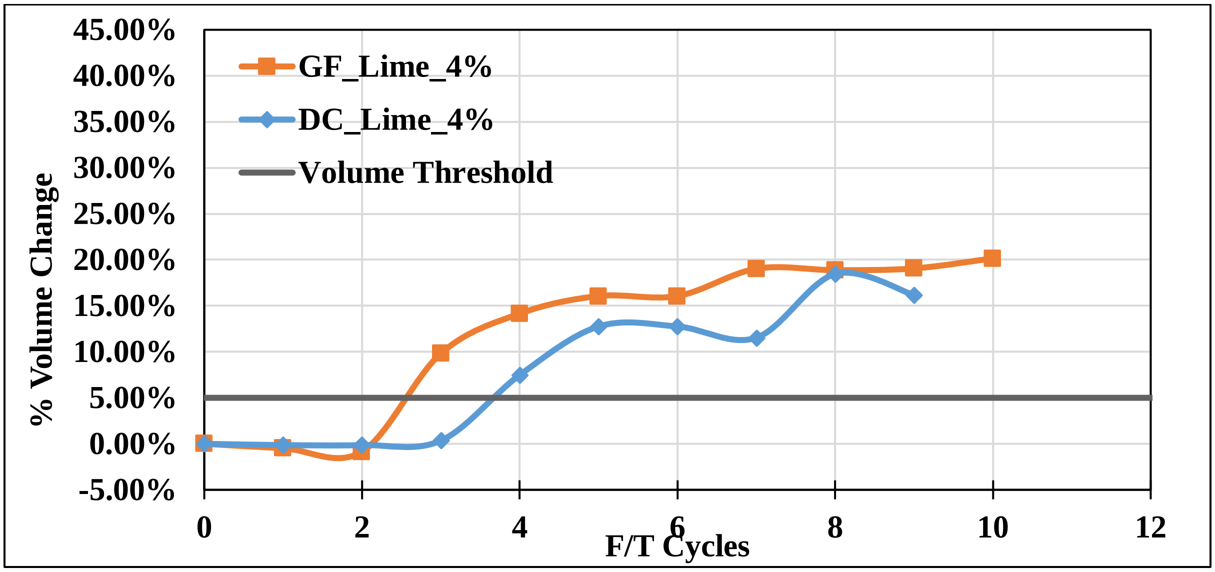


Figure 6.5: Plot showing the change in volume with F/T cycles for the two lime treated soils

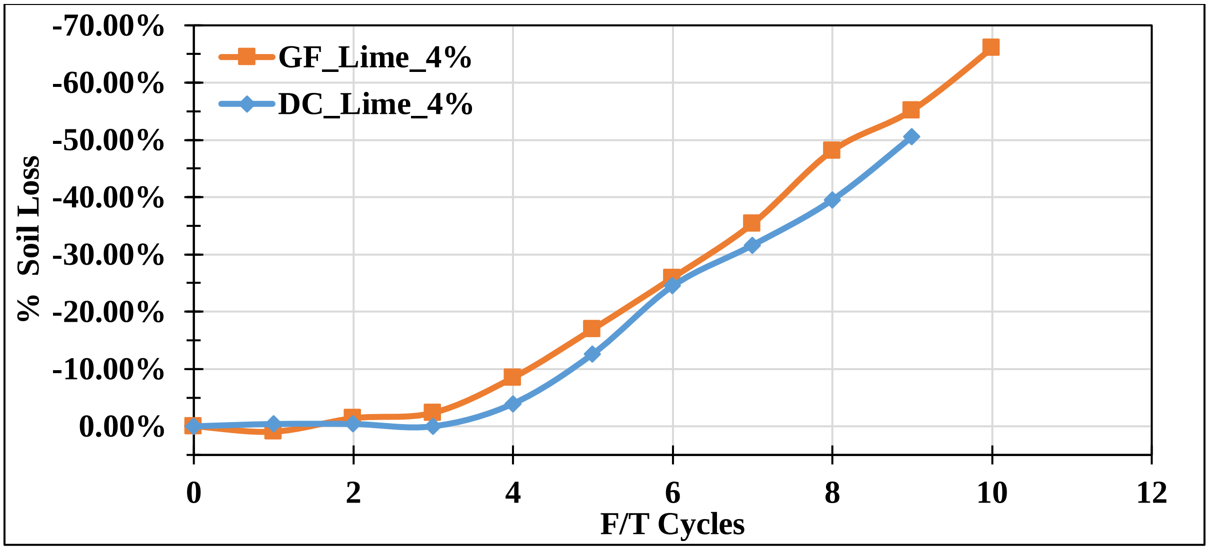


Figure 6.6: Plot showing the gross weight loss in each F/T cycle for two lime treated soils

6.3.1.2 Cement Treatment

F/T durability tests for cement-treated soils were conducted on all six soils – Dry Creek (9%), Bad Route (9%), Great Falls (11%), NTF_HP (11%), NTF_LP (9%), CNK (7%). Figure 6.7 shows the variation of percentage volume change in proctor sized samples with different F/T cycles. It can be observed from the figure that CNK and DC samples consistently showed less than 5% volume change throughout the 12 F/T cycles indicating that these soils at these dosage levels have long-term durability. BR and NTF_LP soils showed larger than 9% volume change after 8 F/T cycles indicating that these soils may not have high durability in the long-term but certainly have medium to low durability at this dosage level. Both GF and NTF-HP soils showed larger than 9% volume change after 5 F/T cycles meaning these two soils would be least effective in the long-term at this dosage level.

Figure 6.8 shows soil loss percentage for proctor sized samples for all six soils tested in this research. In the case of weight loss, the threshold soil loss percentage depends on the soil type, as indicated in Table 6.2. CNK (7% cement), DC (9% cement) and NTF_LP (9% cement) performed well with less than 3% weight loss over the entire 12 cycles. NTF_HP (11% cement) failed the durability requirement on the 10th cycle and GF (11% cement) on the 5th cycle. BR (9% cement) failed the durability threshold on the 6th cycle.

According to the results from volume change and weight loss, it can be inferred that the stabilizer content loses its effectiveness in the long-term for the BR, NTF_HP, and GF soils. Long-term performance is better in the case of CNK, DC, and NTF_LP soil samples. This performance difference can be attributed to the low plastic nature of the DC and NTF_LP and CNK soils and the lack of soluble sulfates in these soils. About 50% of the soils tested in this study did not show good durable performance after chemical treatments.

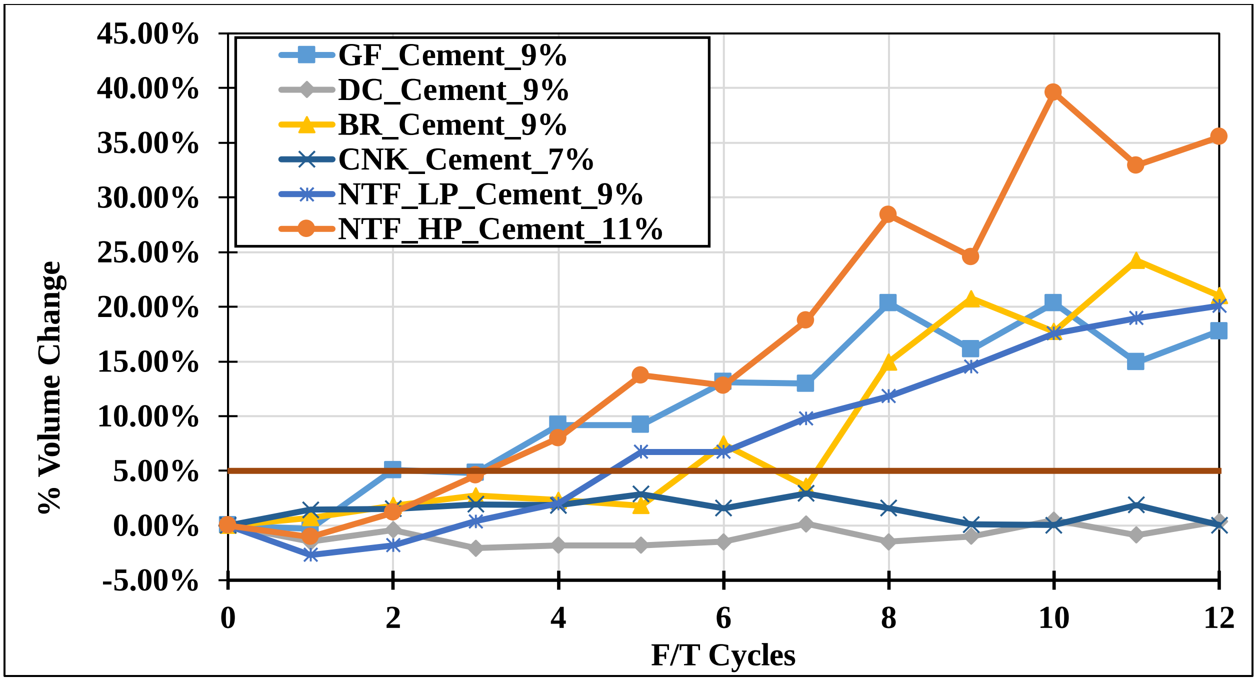


Figure 6.7: Plot showing the volume change in each F/T cycle cement-treated soils – proctor size samples

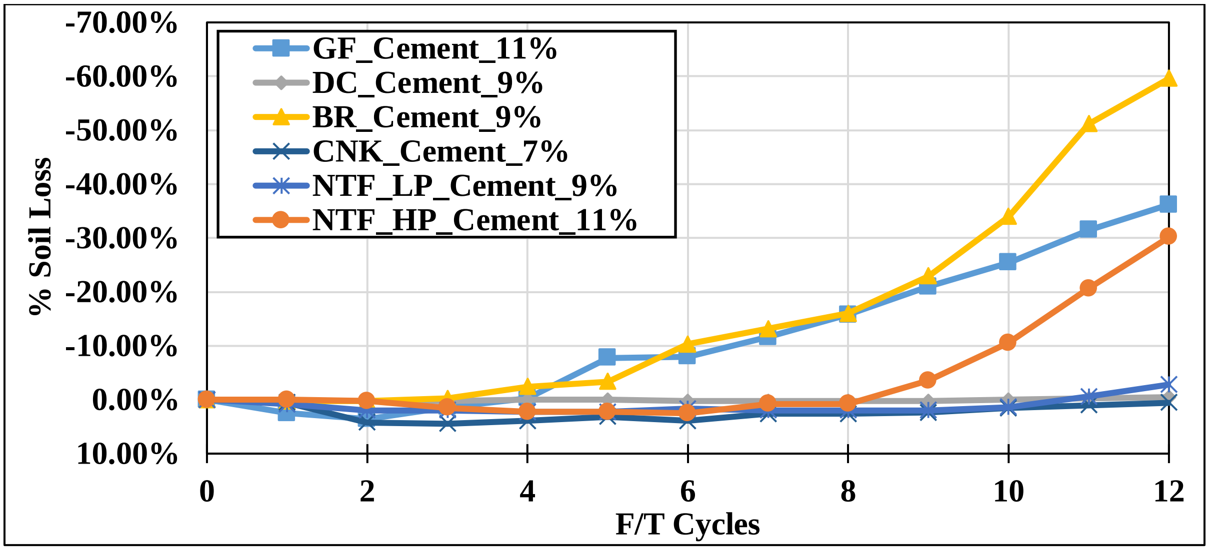


Figure 6.8: Plot showing gross weight loss in each F/T cycle cement-treated soils – Proctor size samples

In addition to the volume change and weight loss measurements, UCS tests were performed on the UCS sized soil samples at the end of the 12 F/T cycles (see Table 6.5). Between the two UCS sized samples, sample #1 was subject to freezing and thawing alone, while sample #2 was subject to brushing, in addition to the freezing and thawing. The CNK (7% cement), DC (9% cement) and NTF_LP (9% cement) samples retain high UCS strengths, even after undergoing the freeze-thaw and brushing for the entire 12 cycles.

Table 6.5: Average water content (W/C) and UCS values at the beginning and end of F/T durability studies (cement-treated only)

Soil	W/C after 0 F/T Cycles	UCS after 0 F/T Cycles (psi)	W/C after 12 F/T Cycles	UCS after 12 F/T Cycles (psi)
GF	35.0%	131.9	41.6%	14
DC	17.4%	484.1	17.8%	325
BR	21.0%	261.0	29.4 %	14
CNK	10.7%	162.0	12.5%	53.2
NTF_LP	27%	775.7	30.8%	166
NTF_HP	24.3%	483.0	39.2%	13

6.3.2 Wetting/Drying

Soil samples with stabilizer content that passed the freeze-thaw durability criteria were chosen for wetting-drying durability tests (see section 6.2 for test protocols). Table 6.6 shows the stabilizer content and soil types used in the wetting-drying durability study. DC (9% cement) and NTF_LP (9% cement) passed with less than 3% weight loss over the entire 12 cycles during the freeze-thaw tests.

Table 6.6: Initial Cement Content used in wetting-drying durability studies.

Soil	USCS Classification	Stabilizer Content
DC	CL	9%
NTF_LP	ML	9%

Note: Only samples that passed freeze-thaw durability criteria were chosen for wetting -drying durability.

Figure 6.9 and Figure 6.10, respectively show the percentage change in volume and percentage weight loss at the end of each wetting and drying cycles. The plots for volume change represent the percentage shrinkage of soil samples compared to initial sample volume at the end of each cycle (Figure 6.9), and the plots for percentage weight loss represent the loss of weight after brushing of dry samples at the end of each cycle (Figure 6.10). The volume shrinkage at the end of wetting and drying cycles are below 5% for the DC (9% cement) as well as NTF_LP (9% cement). The DC (9% cement) sample performed well below the threshold for weight loss of 6%

(Table 6.2) during the entire 12 cycles. NTF_LP (9% cement) failed in weight loss durability criteria of 8% (Table 6.2) after cycle 4.

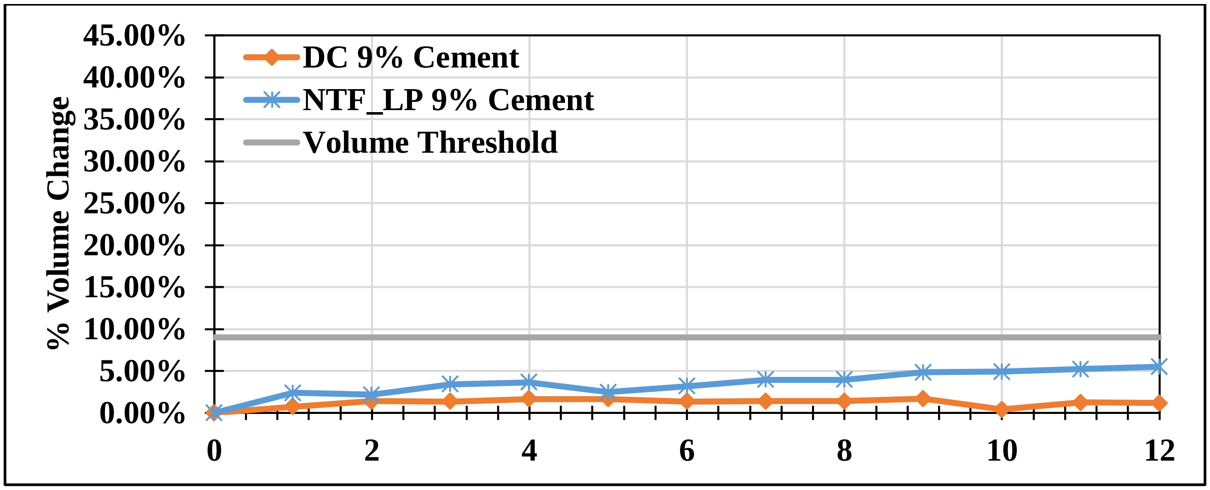


Figure 6.9: Plot showing percentage volume change in each W/D cycle for treated soils – proctor size samples

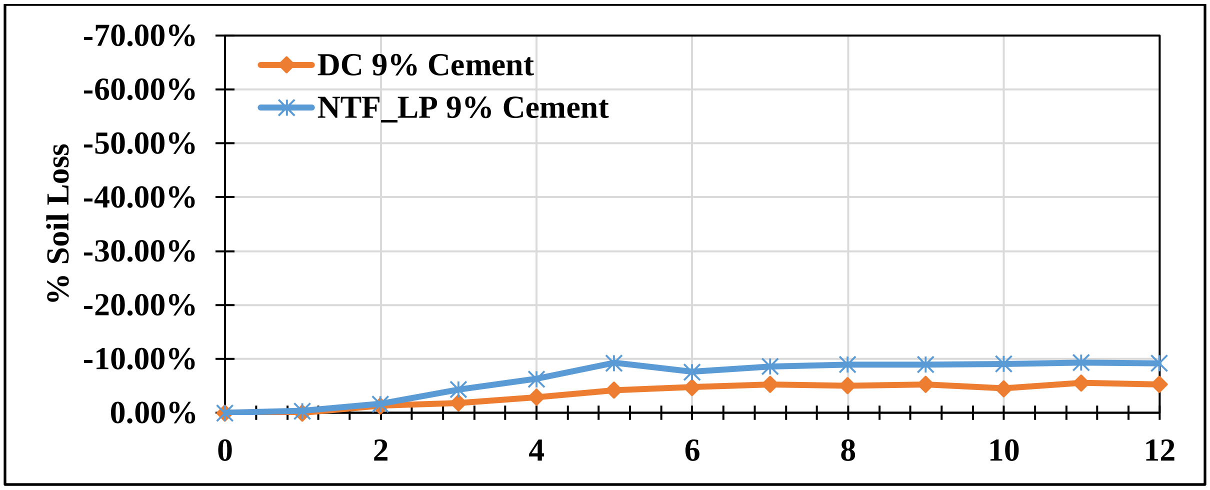


Figure 6.10: Plot showing gross weight loss in each W/D cycle for treated soils – proctor size samples

6.4 Summary and Findings

In this task, the long-term performance of stabilization was tested for all six soils studied in this research. Two types of durability studies were conducted, Freeze/Thaw, and Wetting/drying. More focus was given to the Freeze/thaw durability as Montana soils are more likely to undergo freezing and thawing in the field. However, wetting/drying durability was also performed on select soils to observe their performance under these conditions. A summary of these findings is presented in

Table 6.7.

Table 6.7: The long performance of stabilized soils under Freeze/thaw and Wetting/ drying durability studies

Soil	USCS Classification	Stabilizer Content	The Cycle at which they failed	Max Weight Loss
Freeze/Thaw (Lime as Additive)				
GF	CH	4%	3	66%
DC	CL	4%	4	59%
Freeze/Thaw (Cement as Additive)				
CNK	SM	7%	Did not fail @12 cycles	< 3%
DC	CL	9%	Did not fail @12 cycles	< 3%
NTP_LP	ML	9%	Did not fail @12 cycles	< 3%
NTP_HP	CH	11%	10	30%
GF	CH	11%	5	36%
BR	CL	9%	5	60%
Wetting/Drying (Cement as Additive)				
DC	CL	9%	Did not fail @12 cycles	5.5%
NTP_LP	ML	9%	4	9%

Freeze/Thaw Results

- The limit for durability requirement in terms of weight loss of 6% was surpassed after cycle 3 for GF (4% lime treated) soil and cycle 4 for DC (4% lime treated) soil.
- In case of durability with cement treatment of Montana soils: CNK (7% cement), DC (9% cement) and NTF_LP (9% cement) performed well with less than 3% weight loss over the entire 12 cycles. NTF_HP (11% cement) failed the durability requirement on the 10th cycle, and GF (11% cement) along with BR (9% cement) failed around the 5th cycle.

Wetting/Drying Results

- The DC (9% cement) sample performed well below the threshold for weight loss of 6% (Table 4.2) during the entire 12 cycles. NTF_LP (9% cement) failed in weight loss durability criteria of 8% after cycle 4.

Chemical treatments performed poorly under durability tests for both GF and DC soils when treated with lime. Other types were excluded from treating with lime due to high plasticity or chemical compositions. In general, the combined results show that cement treatment is most compatible in terms of durability with the DC soil (at 9% cement), CNK (at 7% cement), and NTF_LP (at 9% cement). It should be noted here that CNK and NTF_LP soils would be suitable to be treated with cement but did not fare as well as DC soil. The prospect of chemical treatment to be durable on the NTF_HP and BR soils are poor compared to other Montana soils. This could be due to the high amounts of sulfates present in these soils.

7 LIFE CYCLE COST ANALYSIS

7.1 Introduction

An important goal of this project was to help engineering managers make informed decisions on adopting appropriate methods in handling problematic soils. Several life-cycle cost analyses were performed on various pavement alternatives to show how the overall lifetime cost changes when different techniques are used for tackling problematic soils. Strength and durability results, discussed in chapters 5 & 6, were used to design alternative pavement sections and their life cycle costs were compared. Alternatives included in the cost analysis include pavement sections on top of untreated subgrade, chemically stabilized subgrade and soil replaced with special borrow (which is the preferred MDT practice to mitigate problematic soils). Comparison of pavement sections has been done based on changes in costs due to a reduction in long term repair and maintenance activities when using a treated subgrade soil.

7.2 Introduction to Life Cycle Cost Analysis (LCCA)

LCCA is an economics based method to compare design alternatives that satisfy a need in order to determine the lowest cost alternative over the life of the alternative. According to Chapter 3 of the AASHTO Guide for Design of Pavement Structures, life cycle costs "*refer to all costs which are involved in the provision of pavement during its complete life cycle.*" Part of these costs are borne by an agency and include the costs associated with initial construction and future maintenance and rehabilitation. Additionally, costs are borne by the traveling public and overall economy in terms of user delay. The life cycle starts when the project is constructed and opened to traffic and ends when the initial pavement structure is no longer serviceable, and reconstruction is necessary.

The major procedures for LCCA consist of calculating initial construction costs, future maintenance costs over the analysis period and the salvage value based on the remaining life of the structure at the end of the analysis period. These values are compared for various alternatives based on their net present value (NPV), and the pavement option with the lowest NPV is considered the best value. The following section discusses the significant components of LCCA.

7.2.1 Determination of Analysis Period and Costs

LCCA periods should be sufficiently long to reflect long-term cost differences associated with maintenance and repair activities. The analysis period is generally longer than the pavement design period and should be long enough to incorporate at least one complete cycle of rehabilitation activity. FHWA's LCCA Policy Statement recommends an analysis period of at least 35 years for all pavement projects (FHWA, 1998). The MDT recommends an analysis period of 40 years or more. Regardless of the analysis period selected, the same period should be used for all alternatives. Figure 7.1 shows a typical analysis period for a pavement design alternative.

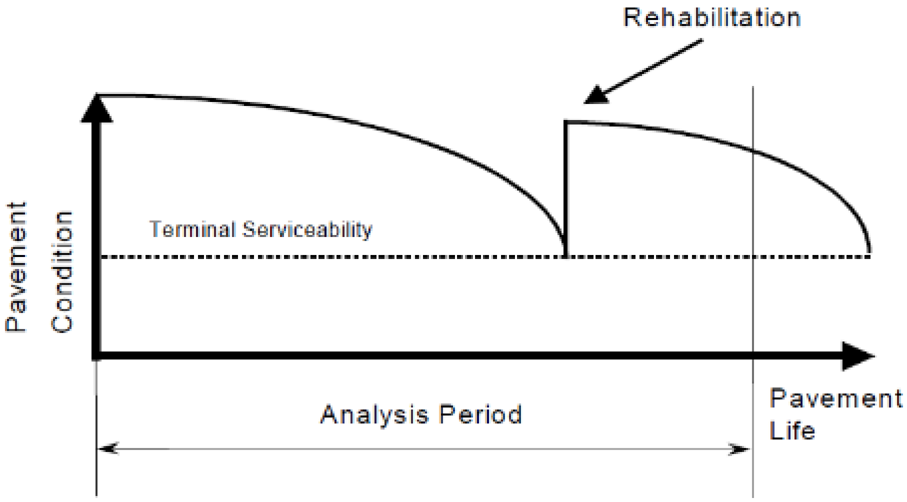


Figure 7.1 Analysis period for a pavement design alternative (FHWA, 1998)

The cost outputs during the analysis period include:

- *Initial Pavement Construction Cost*; which is the cost of the initial construction of a pavement option.
- *Maintenance Cycle and Costs*; which include the cost of future maintenance activities in a predicted year of maintenance. The maintenance activities may include activities like seal and cover, thin overlay, major or minor rehabilitation, mill, and overlay or micro surfacing at various stages during the analysis period. The cost/yd^2 prices in annual Pavement condition and treatment report published by the MDT were used to calculate the costs associated with these activities.
- *User Cost;* which includes the costs borne by the pavement users over the life of the project. These typically include costs due to traffic delays, vehicle operating costs and accident and safety costs. User costs depend highly on specific locations and conditions for a highway. For the purpose of this report, *user costs were not included* in the results of LCCA. It was assumed that these costs would remain the same across the alternatives.

7.2.2 Determination of Net Present Value

The costs related to future activities in a project are accounted for by using the time value of money. A discount rate is used to convert the cost of future maintenance activities and a salvage value of a pavement option into present value. The basic NPV formula for discounting discrete future amounts at various points in time back to a select base year is:

$$NPV = Initial\ Cost + Rehab\ Cost \times \frac{1}{(1+i)^n}$$

Where i = discount rate and

n = year of expenditure

The FHWA's Interim Technical Bulletin (1998) states that a discount rate of 3 to 5 percent is within an acceptable range and is consistent with values historically reported in Appendix A of

OMB Circular A-94 (Office of Management and Budget, 2016). The Net Present Value (NPV) is calculated after discounting all costs to the period of initial construction.

7.3 Determining Alternative Pavement Sections

The life cycle cost comparisons were made based on typical flexible pavement sections designed using the 1993 AASHTO Guide. The following empirical equation from the 1993 Guide was used for flexible pavement design (AASHTO, 1993).

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\frac{\Delta PSI}{4.2-1.5}}{0.40+\frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07 \quad (1)$$

Where:

W_{18} = predicted number of 80 kN (18,000 lb.) equivalent single axle loads (ESAL)

Z_R = standard normal deviate

S_o = combined standard error of the traffic prediction and performance prediction

SN = Structural Number (an index that is indicative of the total pavement thickness required)

ΔPSI = difference between the initial design serviceability index, p_o, and the design terminal serviceability index, p_t

M_R = subgrade resilient modulus (psi)

The structural number is calculated using the following equation (AASHTO, 1993):

$$SN = a1D1 + a2D2m2 + a3D3m3 + \ldots aiDimi \quad (2)$$

(Where: a_i = i_{th} layer coefficient, D_i = i_{th} layer thickness (inches), m_i = i_{th} layer drainage coefficient)

The pavement sections were designed as a two-layer structure consisting of asphalt layer surface and crushed aggregate course (CAC). In the case of chemically stabilized subgrade, a 12-inch-thick stabilized soil subbase was assumed to be part of the pavement structure. Similarly, in the case of current MDT practice of using a special borrow a 12-inch-thick layer was used and required pavement thickness was calculated. The MDT guidelines for the design of flexible pavement were closely followed for the determination of alternative pavement sections.

7.3.1 Incorporation of Chemically Treated Subgrade

The subgrade resilient modulus (M_R), is an important parameter that is used in the design equation. In order to determine the subgrade resilient modulus, a correlation with unconfined compressive strength (UCS) of 50 psi was used. The following empirical correlation was proposed by Hossain and Kim to estimate the resilient modulus for fine grained soils based on the unconfined compression test (Hossain & Kim, 2013).

$$M_R = 142.32 \text{ x UCS} + 4283.3 \text{ (psi)} \quad (3)$$

Average values of UCS results obtained for the untreated soils were used to estimate a resilient modulus for the subgrade. In a pavement section that consists of chemically treated soil subgrade, the stabilized soil layer was considered as a soil sub-base layer in the pavement structure. A structural number was assigned to the treated subgrade soil using the following relation provided in the AASHTO 1993 design guide (AASHTO, 1993).

$$a_2 = 0.249 log_{10} M_R - 0.977 \quad (4)$$

Where, M_R, is the resilient modulus of the treated subbase soil and a_2 is the structural coefficient of that layer. Table 7.1 summarizes the modulus values assigned to the subgrades for various treatments and their corresponding structural coefficients.

Table 7.1 Resilient Modulus of untreated soil samples based on Unconfined Compressive Strengths

Soil	GF	DC	BR	NTF_LP	NTF_HP
UCS (psi)	24.08	24.13	53.69	47.93	35.03
M_R (psi)	7710	7717	11924	11105	9269

In the case of Chinook soils (CNK), the strength of untreated soil was measured using the California Bearing Ratio (CBR). The resilient modulus for CNK soils was obtained using the relationship provided in AASTHO 1993 to relate CBR percentage values to M_R, i.e.,

$$\mathrm{M}_R = 1500 * \mathrm{CBR} \text{ psi} \quad (5)$$

The following CBR percentage values and corresponding resilient moduli were used for the CNK soil.

- Untreated CNK soil: CBR = 2.8% , M_R = 4200 psi

When accounting for the chemically treated subgrade soil, the target UCS value of 50 psi was used to calculate resilient modulus and a structural layer coefficient of 0.033 was obtained. All chemically stabilized layers were assigned with a layer coefficient based on the minimum target strength. It should be noted that, although the actual UCS values obtained from chemical stabilization were higher than 50 psi for specific soil types, a conservative minimum target strength has been used for the purpose of pavement design.

7.3.1.1 Preferred MDT Practice:

The MDT currently prefers the practice of excavating problematic soils up to a depth of 2 feet and replacing with geotextile underlayment and imported special borrow. For alternative pavement sections of special borrow, a layer coefficient of 0.07 was applied to the special borrow layer as recommended in the flexible pavement design guidelines published by the MDT.

An example has been provided to show the design process used for one of the alternative pavement sections on NTF_HP soil using the AASTHO 1993 equation.

7.3.1.2 *Sample Layer Thickness Calculation:*

The following values were assumed as design inputs across all pavement alternatives.

- Daily ESAL = 200.00
- Design Life = 20 years
- Design ESAL (W_{18} required) = 1,460,000

The following parameters were used as per the pavement design guidelines from the Montana Department of Transportation (MDT) for flexible pavements.

- Initial Serviceability, p_o = 4.2
- Terminal Serviceability, p_t = 2.5
- Reliability Level = 95% (correspondingly, $Z_R = 1.645$)
- Standard Deviation, S_o = 0.45
- Drainage Coefficient, m_i = 1

The thickness of the asphalt layer is chosen as recommended for daily ESAL in the MDT guidelines. The thicknesses of the remaining layers are then chosen to satisfy the required design ESAL. Table 7.2 shows the thicknesses of various layers and their equivalent structural numbers calculated as per equation 2 for pavement on NTF_HP soil (**untreated**).

Table 7.2 Sample pavement thickness design for NTF_HP (untreated subgrade)

Layer	Layer Coefficient (a_i)	Drainage Coefficient (m_i)	Thickness (D_i)	SN (Eq. 2)
Surface Course (asphalt)	0.41	1	4	1.64
Base Course (crushed aggregate)	0.14	1	15	2.1
			Total SN =	**3.74**

From equation 1, the corresponding value of ESAL (W_{18}) for this structural number is 1,766,409 which is greater than the required ESAL.

Similarly, for a pavement section on the **treated** soil layer, the structural number and corresponding ESAL are calculated as shown in Table 7.3. The stabilized soil subbase is incorporated in the pavement structure for structural number calculation.

Table 7.3 Sample pavement thickness design for NTF_HP (3% cement treated subgrade)

Layer	Layer Coefficient (a_i)	Drainage Coefficient (m_i)	Thickness (D_i)	SN (Eq. 2)
Surface Course (asphalt)	0.41	1	4	1.64
Base Course (crushed aggregate)	0.14	1	12	1.68
Stabilized Soil Sub Base	0.033	1	12	0.396
			Total SN =	**3.72**

From equation 1, the corresponding value of ESAL (W_{18}) for this structural number is 1,697,254 which is greater than the required ESAL.

Another alternative section based on the **current MDT practice** of using a 12-inch-thick special borrow is calculated as shown in Table 7.4.

Table 7.4 Sample pavement thickness design for NTF_HP (special borrow)

Layer	Layer Coefficient (a_i)	Drainage Coefficient (m_i)	Thickness (D_i)	SN (Eq. 2)
Surface Course (asphalt)	0.41	1	4	1.64
Base Course (crushed aggregate)	0.14	1	9	1.26
Special Burrow	0.07	1	12	0.84
			Total SN =	**3.74**

From equation 1, the corresponding value of ESAL (W_{18}) for this structural number is 1,766,409 which is greater than the required ESAL.

Table 7.5 Designed pavement layer thicknesses for all alternatives based on MDT guidelines

Soil	Treatment Type	Asphalt Surface Course	CAC Base Course	Stabilized Soil Subbase	Design Structural Number
GF	Untreated	4	17		4.0
	Cement Treated	4	14	12	4.0
	Lime Treated	4	14	12	4.0
	Special Borrow	4	11	12	4.0
DC	Untreated	4	17		4.0
	Cement Treated	4	14	12	4.0
	Lime Treated	4	14	12	4.0
	Special Borrow	4	11	12	4.0
BR	Untreated	4	14		3.6
	Cement Treated	4	11	12	3.6
	Special Borrow	4	8	12	3.6
NTF_LP	Untreated	4	14		3.6
	Cement Treated	4	11	12	3.6
	Special Borrow	4	8	12	3.6
NTF_HP	Untreated	4	15		3.7
	Cement Treated	4	12	12	3.7
	Special Borrow	4	9	12	3.7
CNK	Untreated	4	23		4.9
	Cement Treated	4	20	12	4.8
	Special Borrow	4	17	12	4.9

7.4 Life Cycle Cost Comparisons

The life cycle costs of the alternatives were compared based on changes in the long-term treatment and repair activities on the pavements over an analysis period of 40 years.

7.4.1 Alternative Pavement Sections

Table 7.5 lists the layer thicknesses and the design structural numbers of various pavement sections used for the life cycle cost comparisons. The thickness of the asphalt surface course was chosen as 4 inches for all alternative sections based on the assumed ESALs, as recommended in the MDT pavement design guidelines. The crushed aggregate base course thickness was determined based on the required structural number as per AASTHO 1993. The minimum thickness of the base course layer was limited to 8 inches.

7.4.2 Prediction of Life Cycle Activities

The Federal Highway Administration (FHWA) Long-Term Pavement Performance (LTPP) program collects research quality pavement performance data from in-service test sections across the U.S. and Canada. A historical database of repair and construction activities performed on 38 different pavement sections from Montana were obtained from the LTPP website. The data was analyzed to group similar activities within a range of 5 years on different pavements. The average of age at which the construction or repair was performed was taken to predict future activities. The predictions for future activities on pavements constructed over untreated subgrade and chemically treated subgrades and special borrow are shown in Table 7.6.

Table 7.6 Life cycle activities on pavements over the alternative subgrades

Age of Pavement	**Life Cycle Activities**		
	Untreated Soil Subgrade	**Chemically Stabilized Subgrade (Durability Failed)**	**Special Borrow and Chemically Stabilized (Durability Passed)**
0	Construction	Construction	Construction
6	Crack Seal and Cover on 50% area	Crack Seal and Cover on 50% area	Crack Seal and Cover on 50% area
9	Thin Overlay	Crack Seal and Cover on 50% area	Crack Seal and Cover on 50% area
14	Crack Seal and Cover on 50% area	Crack Seal and Cover on 50% area	Crack Seal and Cover on 50% area
18	Minor Rehab	Minor Rehab	Minor Rehab
23	Crack Seal and Cover on 50% area	Crack Seal and Cover on 50% area	Crack Seal and Cover on 50% area
30	Thin Overlay	Thin Overlay	Crack Seal and Cover on 50% area
35	Crack Seal and Cover on 50% area	Crack Seal and Cover on 50% area	Crack Seal and Cover on 50% area
40	End of Analysis (no residual value)	End of Analysis (no residual value)	End of Analysis (no residual value)

Durability studies for chemical treatment on GF, BR and NTF_HP soils did not show long term viability. However, the strength improvement on these soils during the short term is high and that should impact the life cycle activities. To account for this, one round of thin overlay activity in the duration of 9 years was replaced with crack seal and cover for pavements on chemical stabilized subgrades for GF, BR and NTF_HP soils. The chemically treated CNK, DC and NTF_LP soils performed well during the durability tests. To account for the long-term advantage these soils can provide to the pavement life, an additional thin overlay activity was replaced with crack seal and cover at the age of 30 years. The same condition was also assumed for the pavements on special borrow.

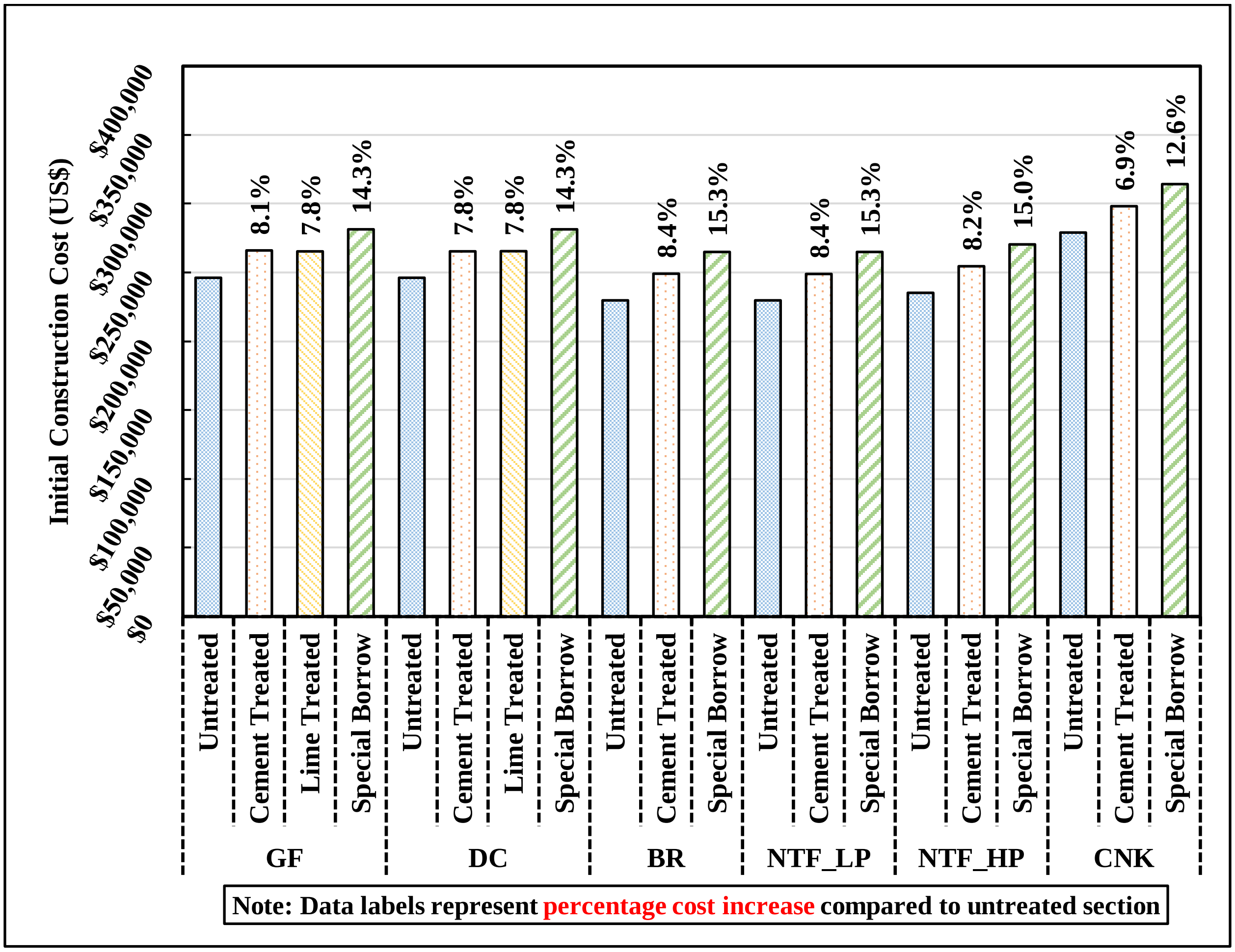
A database on the historical performance of pavements on treated subgrade soil cannot be found in the LTPP yet. Given that the durability and strength of the subgrade are substantially improved after treatment (on CNK, DC and NTF_LP soils), an assumption was made for predicting the change of life cycle activities for pavements constructed on a treated subgrade as they should eventually need a smaller number of repair activities. The repair and maintenance activities and their periods are only a guess at best, however the durability studies on the chemically treated soils show sufficient strength improvements to reinforce these assumptions.

7.4.3 Cost Comparison

A 12 ft wide and 1-mile long pavement section was considered for calculating quantities of items involved in construction as well as repair and maintenance. The unit costs of items for asphalt, crushed aggregate base course, and special borrow are the general cost averages used by the MDT. The cost of subgrade modification and that of cement and lime were obtained from a report published by the Strategic Highway Research Program (SHRP2, 2012). The cost of intermediate life cycle activities was calculated based on the cost/yd^2 rates provided in the annual Pavement condition and treatment report published by the MDT. The following unit rates were used in the cost analysis:

- Asphalt: US$100/ton of the mix
- CAC: US$28/cuyd of the mix
- Subgrade work for stabilization: US$15/cuyd of the mix
- Lime: US$215/ton
- Cement: US$165/ton
- Special borrow: US$19/cuyd
- Excavation and haul: US$10/cuyd
- Seal and Cover: US$2.24/sqyd
- Thin Overlay: US$12.43/sqyd
- Minor Rehab: US$14.95/sqyd

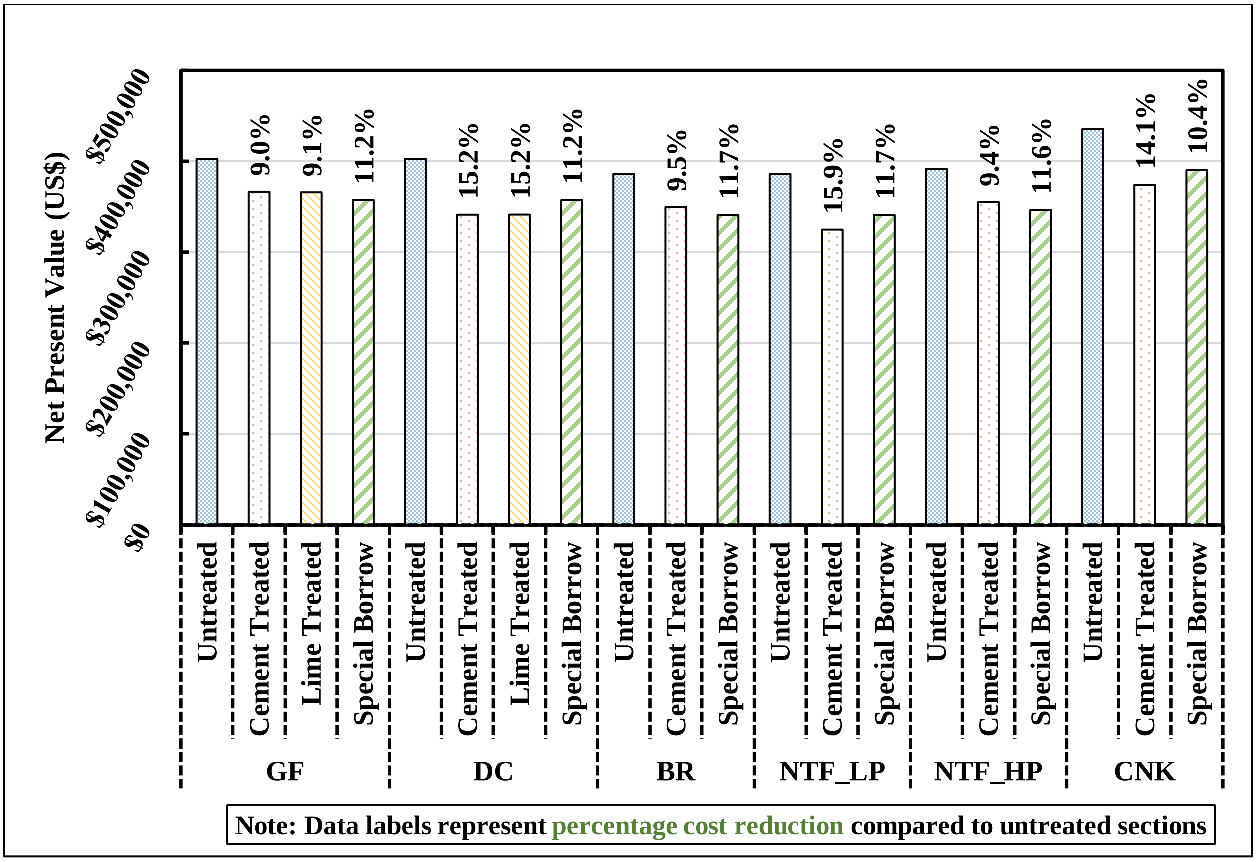
Figure 7.2 show the initial construction costs for pavements on various soil types and treatment alternatives. The general cost increase in construction is higher for special borrow than chemical stabilization. The percentage increase in initial construction cost due to the use of a chemically treated subgrade soil varied from 6.9% to 8.4%. The increase in construction cost for pavements on special borrow varied from 12.6% to 15.3%. The practice of special borrow would involve excavation and transportation of in-situ soil and bringing in new material for the subgrade. In case of chemical stabilization, the in-situ soil is mixed and compacted in place. This results in a lower cost of construction for chemical stabilization when compared with special borrow.



Note: Data labels represent percentage cost increase compared to untreated section

Figure 7.2 Initial construction cost comparisons for pavements on the treated subgrade

Figure 7.3 shows the NPV of the life cycle costs for various alternatives calculated at a discount rate of 4%. Percentage life cycle cost savings from chemical stabilization vary from 9.0% to 15.9% whereas, cost savings from special borrow vary from 10.4% to 11.7%. Special borrow is more favorable than chemical stabilization in the long term for soils that failed in the durability tests (BR, GF and NTF_HP). When considering soils that performed well in the durability test (CNK, DC, NTF_LP), chemical stabilization is the more favorable alternative than special borrow.



Note: Data labels represent percentage cost reduction compared to untreated sections

Figure 7.3 Life cycle costs at the discount rate of 4% (NPV)

Discount rates in the range of 2% to 6% were used to observe the sensitivity of the cost comparison. The discount rate is used to convert the future costs on the pavement to present dollar values. However, discount rates are variable and can change each year depending on the economy. Comparing the net present values at varying discount rates can help analyze the sensitivity of costs/benefits against the uncertainty in discount rates. The sensitivity of the percentage change in life-cycle cost against the change in discount rates is shown in Figure 7.4 and Figure 7.5. The life cycle cost advantage varies by almost up to 9% on special borrow and durable chemical treatments when the discount rate change from 2 to 6%. This change reflects the sensitivity of assumptions made during selection of long-term pavement activities to the net present value. The slope of the line on the sensitivity chart shows the magnitude of effect that change in discount rates can have on the cost advantage. The choice of long-term activities, costs and the age at which these activities are performed affect how net present values can change against discount rates. The most favorable alternative is the one that shows good cost advantage in all scenarios.

The type of soil and the durability of chemical treatment can have different magnitudes of impact on the life-cycle cost advantage. The cost analysis done here was based on the predictions that only one or two cycles of treatment or repair activity would change in the use of treated subgrades or special burrow sections. The actual benefits of the treatment on pavement structure over the long term could be greater if the durability of treatments is high.

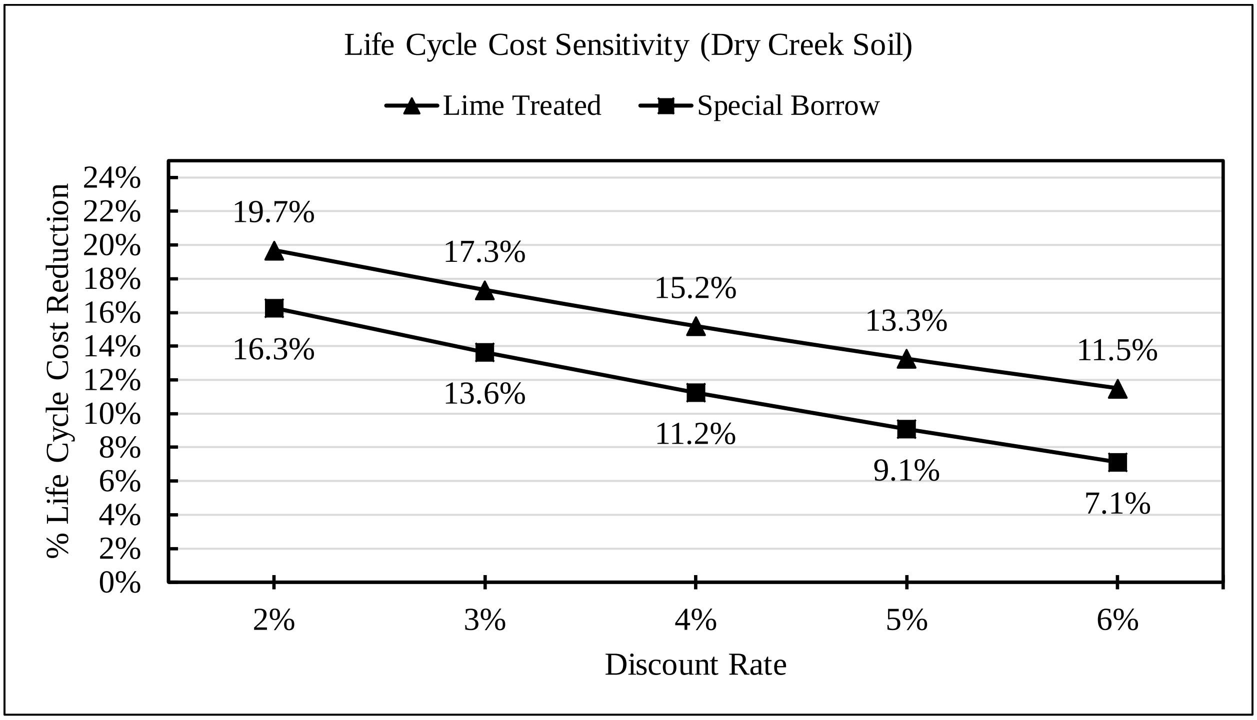


Figure 7.4 Sensitivity of percentage life cycle cost reduction to discount rates (DC Soil)

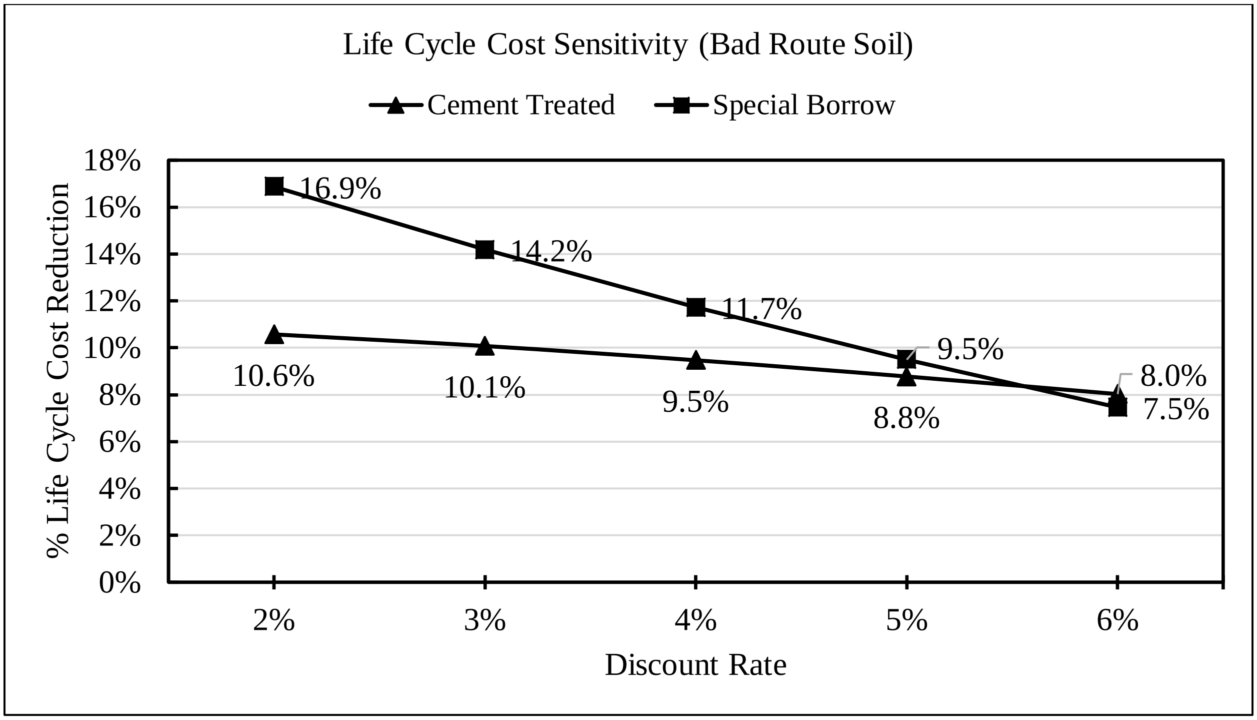


Figure 7.5 Sensitivity of percentage life cycle cost reduction to discount rates (BR soil)

7.5 Summary and Findings

A life cycle cost analysis was performed to understand the benefits and feasibility of using chemical treatments to address various problematic soils in Montana. A comparison was made based on changes in costs due to a reduction in long term repair and maintenance activities on pavements constructed on chemically treated soil or special borrow vs. untreated subgrades. The following observations were made:

1. The general cost increase in construction is higher for special borrow than chemical stabilization. The percentage increase in initial construction cost due to the use of a chemically treated subgrade soil varied from 6.9% to 8.4%. The increase in construction cost for pavements on special borrow varied from 12.6% to 15.3%.
2. Percentage life cycle cost savings from chemical stabilization vary from 9.0% to 15.9% whereas, cost savings from special borrow vary from 10.4% to 11.7%.
3. Special borrow is more favorable than chemical stabilization in the long term for soils that failed in the durability tests (BR, GF and NTF_HP).
4. When considering soils that performed well in the durability test (CNK, DC, NTF_LP), chemical stabilization is the more favorable alternative than special borrow.

Based on these observations, it can be concluded that using chemical stabilization on problematic soils is more advantageous than special borrow, if the durability of the treatment is good. When durability results are poor for chemical treatments, special borrow is more cost advantageous in the long term. It should be noted that the life cycle costs and analyses for this research project are based purely upon construction costs. Additional incurred costs and/or time that would be included during project design and development (i.e. subsurface investigation, laboratory testing, engineering analyses) to implement chemical stabilization have not been included in this research project.

8 STABILIZATION GUIDELINE

This chapter presents a procedure to be followed to determine the optimum amount and type of additive to chemically stabilize a given soil.

8.1 Determine the type of additive

Figure 8.1 provides a flowchart to select an additive(s) based on the plasticity characteristics of the soil. To determine an appropriate additive type, soil classification information from soil exploration results are required. The information from this chart applies to most, but not all cases. This serves as a good rule of thumb in selecting an initial additive(s). Validation testing must be performed to verify whether the selected additive(s) accomplishes the goals and requirements of the treated soil. Also, life cycle cost analysis should be considered by accounting for material availability, construction costs, construction time, and the overall benefit of the improved performance during the selection of an additive. FHWA (1998) published a guideline for Life-Cycle Cost Analysis in Pavement Design in their Interim Technical Bulletin which can help designers in make an informed decision regarding the choice of a stabilization method.

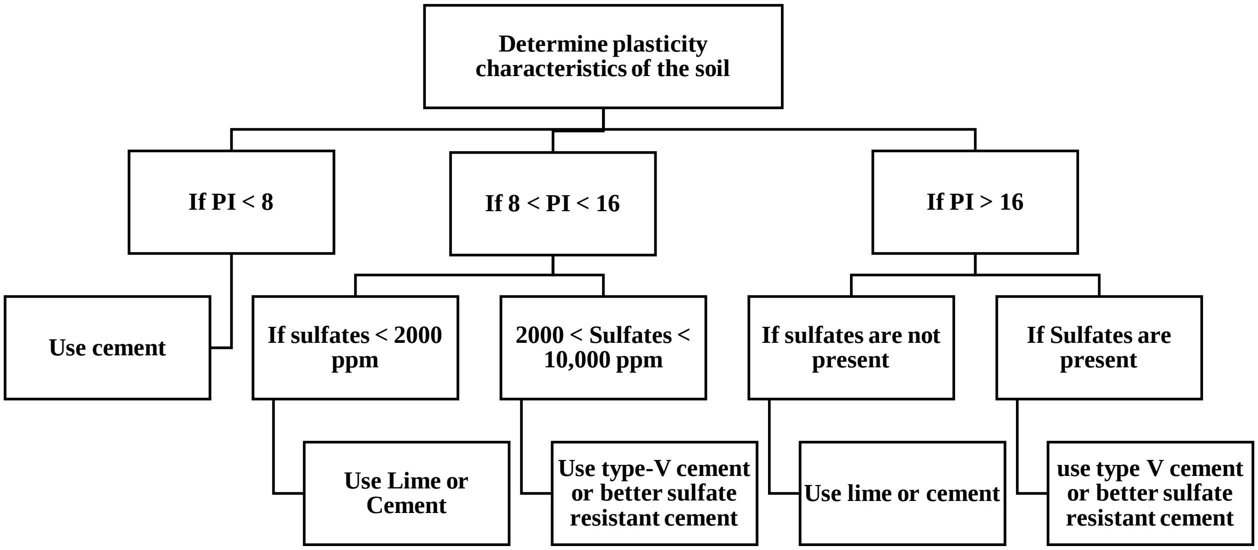


Figure 8.1 Flowchart showing the procedure to select the optimal additive type

8.2 Determine the optimal additive content (Mix Design)

The determination of optimal additive content depends on the purpose of stabilization. For example, soil stabilization is the treatment of subgrade soil with an additive to provide a stable working platform under construction traffic. The optimum additive content can be defined as the percent additive (by dry weight of the soil) that provides a minimum UCS of 50 psi (345 kPa) at OMC and MDUW. This process differs for different additive types. Here we will highlight the procedures for lime and cement.

8.2.1 Lime Stabilization

Success with lime stabilization depends on the soil-lime reactivity which largely depends on the clay fraction in the soil. The soil shall contain a minimum of 15% clay in order to be reactive with lime. The following steps should be followed to arrive at design lime content.

Step 1: Verify that the sulfate and organic contents are within acceptable limits. Measure the sulfate and organics content prior to the addition of the additive. Sulfates should be less than 2000 ppm (if the PI of the soil is less than 16) and organics should be less than 1%. If the sulfates are more than 2000 ppm, do not use lime as additive. If organics are more than 1%, be cognizant that the soil may require higher dosages to counter the effect of cation exchange capacity.

Step 2: pH test. The initial optimum lime content is established using a procedure developed by Eades and Grim (1966) which targets a pH of 12.4 or higher.

Step 3: Moisture Density curve. Establish the moisture density curve using the lime content established in Step 2. This will be used to prepare soil samples for UCS testing.

Step 4: Plasticity Index. Conduct a plasticity index test to evaluate shrink/swell characteristics of treated soil. Most soils turn non-plastic at optimum lime content.

Step 5: Strength Testing. Conduct a UCS test at OMC and MDUW established in Step 3. Verify if the strength meets the governing specification.

Step 6: Durability Testing. Perform durability tests if stabilization is targeting long-term performance.

Step 7: Select the optimal content. Select the lowest modifier content necessary to satisfy the project requirements.

8.2.2 Cement Stabilization

Cement stabilization is generally preferred when the strength improvements required are high. The following steps should be followed to arrive at the design cement content.

Step 1: Verify that the sulfate and organic contents are within acceptable limits. Measure the sulfate and organics content prior to the addition of the additive. If the sulfates are less than 2000 ppm use type I/II Ordinary Portland Cement. If sulfate content exceeds 2000 ppm, use type V or better sulfate resistant cement. If organics are more than 1%, be cognizant that the soil may require higher dosages to counter the effect of cation exchange capacity.

Step 2: Moisture Density curve. Establish the moisture density curve using an initial cement content. This will depend on the soil type and the target performance requirements. Start with minimal percent typically 2%.

Step 3: Strength Testing. Conduct a UCS test at OMC and MDUW established in Step 3. Verify if the strength meets the governing specification.

Step 4: Durability Testing. Perform durability tests if stabilization is targeting long-term performance.

Step 5: Select the optimal content. Select the lowest modifier content necessary to satisfy the project requirements.

9 SUMMARY, FINDINGS, AND RECOMMENDATIONS

9.1 Summary and Findings

The Montana Department of Transportation (MDT) initiated the project, in 2017, to develop guidelines for chemical stabilization of problematic subgrade soils and a research was conducted through the Sustainable and Resilient Geotechnical Engineering (SuRGE) lab at Boise State University (BSU). It was found through survey and literature review that states neighboring Montana, except Canada, do not have much experience with chemical stabilization practices or established protocols for doing so. To develop a stabilization guideline that could incorporate the diverse range of soils found in Montana, the research team studied the effects of lime and cement treatments on soils from six different locations. The six soils were labelled after their location of origin or plasticity characteristics: Great Falls (GF), Dry Creek (DC), Bad Route (BR), Chinook (CNK) and North Three Fork-High Plastic (NTF_HP), North Three Fork-Low Plastic (NTF_LP).

The selected soils were stabilized referring to various existing guidelines on cement and lime treatment, and changes in physical, chemical and mineralogical properties were studied. The response of the soils to different additive types and contents were initially tested by targeting an unconfined compressive strength of chemically treated soils to a value of 50 psi. The treated samples were cured using various protocols and a new curing protocol was developed that reduced the standard curing time from 7 days to 1 day. The treated soil samples that passed the strength requirements were then tested for durability against freezing/thawing and wetting/drying cycles. A life cycle cost analysis was performed following the durability studies to compare various hypothetical pavement sections designed on untreated soil, chemically treated soil and special borrow. A long-term cost comparison of chemical treatments against the preferred MDT practice of using special borrow was made as a final task of the study. Some of the important observations and findings from this study are as follows:

1. Out of the six soils collected, there were two high plastic clays, two low plastic clays, one low plastic silt, and, one silty sand. Two out of six soils contained soluble sulfates in excess of 10,000 ppm and, all soils except CNK contained organic content greater than 1%. Such soils require special attention in selecting stabilization method and durability.
2. It was noted that only 2% of lime was sufficient to increase strength above 50 psi for all soils tested in this research. GF required 7% cement to increase the strength above 50 psi whereas 2% lime was enough. However, some of these samples have high sulfate contents which can cause issues with durability.
3. Of the three different accelerated curing protocols studied in this research, Humidity Controlled Accelerated Curing (HCAC) is the most practical and reliable. This protocol is recommended when time is of essence, otherwise, the ASTM standard 7-day curing protocol should be used.
4. Based on the Freeze -Thaw and Wetting – Drying durability studies, the results generally show that cement treatment is most compatible in terms of durability with the DC soil (at 9% cement), CNK (at 7% cement), and NTF_LP (at 9% cement). It should be noted here that CNK and NTF_LP soils would be suitable to be treated with cement but did not fare as well as DC soil.

5. The durability of chemical treatment on the NTF_HP and BR soils is poor compared to other Montana soils. This could be due to the high amounts of sulfates present in these soils.
6. The general cost increase in construction is higher for special borrow than chemical stabilization. The percentage increase in initial construction cost due to the use of a chemically treated subgrade soil varied from 6.9% to 8.4%. The increase in construction cost for pavements on special borrow varied from 12.6% to 15.3%.
7. Special borrow is more favorable than chemical stabilization in the long term when compared on soils that failed in the durability tests (BR, GF and NTF_HP). When considering soils that performed well in the durability test (CNK, DC, NTF_LP), chemical stabilization is the more favorable alternative than special borrow.
8. Based on the results of LCCA, it can be concluded that using chemical stabilization on problematic soils is more advantageous than special borrow, if the durability of the treatment is high. When durability results are poor for chemical treatments, special borrow is more cost advantageous in the long term.
9. It should be noted that the life cycle costs and analyses for this research project are based purely upon construction costs. Additional incurred costs and/or time that would be included during project design and development (i.e. subsurface investigation, laboratory testing, engineering analyses) to implement chemical stabilization have not been included in this research project.

9.2 Recommendations for further research

It is recommended that a field implementation project be undertaken with the goal of:

1. The first objective of this field implementation would be the development of field protocols to use chemical stabilization in the field.
2. The second objective will target the understanding of field performance of the stabilization with respect to laboratory durability studies. This type of study will help understand the suitability of durability studies in evaluating long term performance. To date, it is not clear how many years of field life is represented in 12 cycles of durability. This objective will help evaluate this aspect.

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11 APPENDIX A

Description of tests and procedures used for establishing baseline properties of Montana soils.

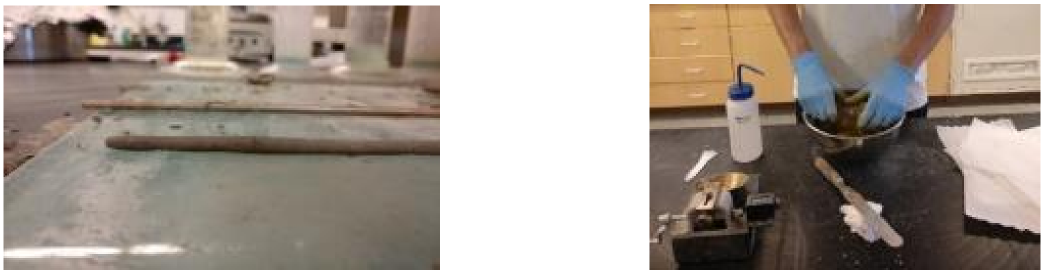
1. Specific gravity

Specific gravity (G_s) is an essential parameter for calculating the weight-volume relationships which are very important in determining various physical parameters. G_s ranges from 2.63-2.67 for sands, 2.65-2.70 for silts, <2 for organic soils and 2.67-2.90 for clays and silty clays. A 500 ml flask was filled with distilled water and de-aired using a vacuum and weighted. The weight of 50 to 100 gm of soil with distilled water in 500 ml flask was determined. Mass of the dry soil and displaced water were calculated, whose ratio gives the value of G_s. Temperature correction was applied to report G_s at 20^0C.

2. Atterberg limits

Atterberg limits are related to the consistency of the soil which proceeds from dry, semisolid, plastic and finally to liquid states. Water contents at the boundary of these states are termed, shrinkage limit (SL), plastic limit (PL) and liquid limit (LL), respectively. The water content at which the soil starts to behave like a liquid corresponds to LL and water content at which soil starts to crumble when rolled into a 1/8” (3mm) diameter thread corresponds to PL. The numerical difference between LL and PL is known as the plasticity index (PI) which characterizes the plasticity nature of the soil.

The sample for LL and PL was oven dried at 105 ºC and broken into smaller pieces. The ground sample passing #40 sieve was then mixed with distilled water. Casagrande’s device was used to determine LL of the soil, which is defined as the moisture content at which a groove closure of 0.5 in. occurs at 25 blows. Similarly, PL was determined by hand rolling a soil sample on a glass plate to a diameter of 1/8” until cracks appear on the roll. Figure 11.1 shows these tests being conducted on soil collected. The LL, PL and PI data for the six soils are summarized in Table 3.4.



(a) (b)

Figure 11.1: Plastic limit (a) and liquid limit (b) tests

3. Sieve analysis and hydrometer

The grain size distribution of the selected soils was obtained using *sieve analysis* and *hydrometer* tests. A representative soil sample was prepared for each site by mixing with water and washing through #200 sieve. The soil sample washed through #200 sieve was collected for hydrometer tests while the soil retained on the #200 sieve was oven dried to perform the sieve analysis tests. Sieve analysis establishes the percentage of the coarse fraction of the soil (Gravel and Sand) while hydrometer analysis establishes the percentage of fine fraction in the soil specimens (Silt and Clay). The percentages of sand, silt, and clay are summarized in Table 3.4.

4. Standard Proctor test

Standard Proctor test was used to determine the moisture-density relationship to establish the maximum dry unit weight and optimum moisture content for a given soil. For this test, a 6 lb. soil sample passing #4 sieve was mixed with water to bring moisture content 4%-5% below the estimated optimum moisture content. The soil was compacted in 3 layers with 25 blows per layer using a 5.5 lb. hammer falling from a height of 12 in. The moisture content of the compacted sample and weight of the compacted samples with mold were measured. This procedure was repeated with increments of 2% moisture content until a decrease in mold weight was observed. Calculation of dry density for each moisture content was done using mass-volume relations. Maximum dry unit weight (MDUW) and corresponding optimum moisture content (OMC) were determined by plotting the dry unit weight vs. moisture content. Figure 11.2 shows this test being conducted on BR soil. The maximum dry density and optimum moisture content for the six soils are presented in Table 3.5.

Figure 11.2: Proctor Test on BR soil

5. Unconfined Compression Strength (UCS)

UCS strength was determined by applying axial stress to a cylindrical soil specimen without confining pressure. A sample of 2.8 in. (diameter) x 5.6 in. (height), was prepared at OMC and MDD and loaded at a rate of 0.078 in/min. Axial strain corresponding to each axial stress was measured until the failure of the sample was observed. Maximum stress at failure was recorded as the UCS strength of the soil. Secant modulus was determined as the slope of the line connecting the origin and maximum UCS strength on the stress-strain plot. Figure 11.3 shows the sample setup and failed sample specimen after the test. UCS strength and modulus for the six soils are presented in Table 3.5.

(a)

(b)

Figure 11.3: UCS sample (a) prior to test inception (b) after completion of tests

6. 1-D swell tests

It is a general method for evaluation of swelling behavior of expansive soil under realistic field conditions, which involve 1-D loading and moisture inundation inside an oedometer. Method-A in ASTM D4546 was followed which allowed the test specimen to expand after being inundated with water under a 6.5 kPa vertical stress. The swell in the specimen was calculated as a percentage of their original height which is referred to as the *free swell*. After the completion of free swell, the specimen was loaded incrementally – as done in the conventional 1-D consolidation test. This was done until the specimen height is equal to or smaller than the original height. The pressure at which the specimen had to be loaded to bring the height back to its original thickness is the swell pressure. Figure 11.4 shows the sample setup for the 1-D swell test. Swell pressures and swell strains for the six soils are presented in Table 3.5.

Figure 11.4: Sample setup for the 1-D swell test

7. Permeability tests

ASTM D5084 was used to determine the hydraulic conductivity of the soils. Method B was used which involved a falling head constant tailwater elevation hydraulic system. The head loss across the specimen at the beginning and end of the test was calculated. This along with the time-elapsed between the start and end of the test, and volume of the water collected was used to calculate the hydraulic conductivity. Permeability for the six soils is presented in Table 3.5.

8. Soluble Sulfate

Calcium-based chemical treatment of sulfate-rich soil induces heave distresses due to the formation of high swelling minerals like Ettringite and Thaumasite. Therefore, quantification of soluble sulfates present in the subgrade soil before chemical treatment is necessary. The modified UTA method outlined by Puppala et al. (2002) was used in this study. As per this procedure, 10 grams of dried soil was taken and diluted with 100 mL of distilled water. This solution was shaken on an Eberbach shaker for 30 minutes. After which the solution was separated from the soil by centrifuging at a speed of approximately 14,000 rpm. The pH of the solution was maintained between 5 and 7 with the help of Hydrochloric acid. The extracted solution was put on a hot plate to boil. Barium Chloride ($BaCl_2$) was then added to the boiling solution to bring out sulfate in the form of Barite ($BaSO_4$). The solution was then placed in an 85°C oven for 12 hours to continue the digestion process in which precipitation takes place to obtain Barite by the gravimetric process. The barite precipitated from this process, was used in the calculations to obtain the soluble sulfate content in the soil samples. The photographic representation of the process is shown in Figure 11.5. Soluble sulfate results for six soils are listed in Table 3.6.

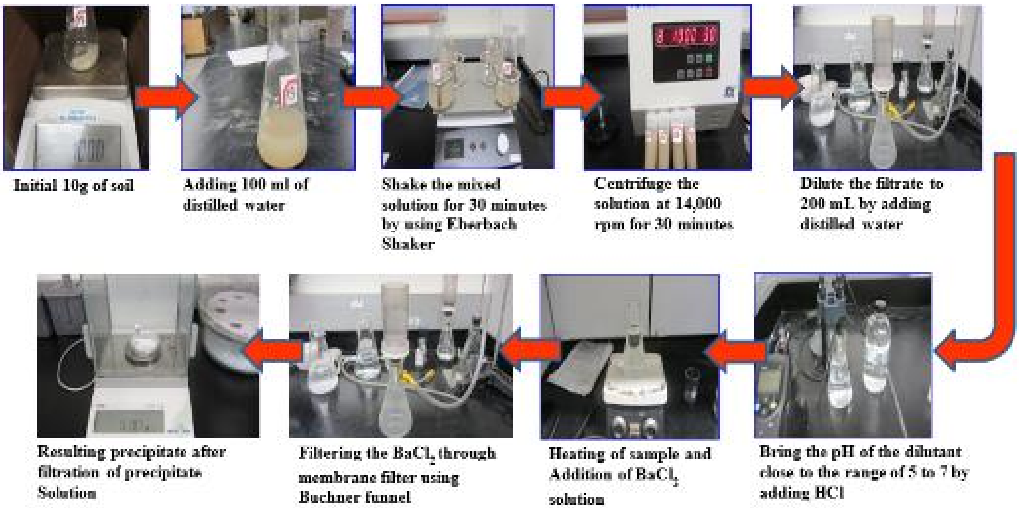


Figure 11.5: Photographic representation of sulfate test procedure (taken from Chittoori et al. 2015)

9. Organic Content

Chemical stabilization using lime or cement may not be effective in soils containing high organic content. Therefore, the determination of organic content is an important screening procedure before conducting chemical stabilization. Hence, the six soils were screened for the presence of organic material using the loss-on-ignition method. In this process, the oven-dried soil was ignited in a furnace at 440 ± 40^0C. This was done until there was no change in the weight of the soil after 1 hour in the furnace. The final mass of the ash was determined as the percentage of the oven-dried soil's weight and subtracted from 100% to get the organic content of the soil. The data obtained from this test are presented in Table 3.6, and the photographic representation of the process is shown in Figure 11.6.

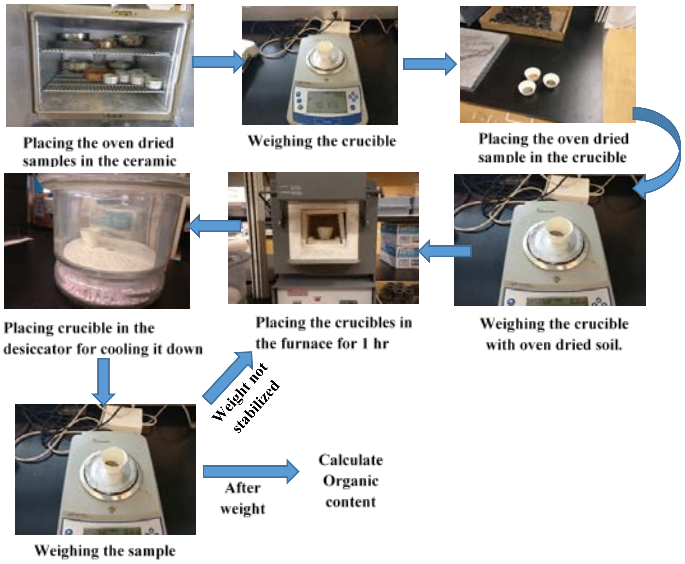


Figure 11.6: Photographic representation of the organic content test

10. Reactive Alumina and Silica

Pozzolanic reactions are dependent on the availability of reactive alumina and silica in the soil. Determining their concentrations will help in assessing the amount of improvement possible after chemical treatments. Reactive alumina and silica were determined using the procedure outlined in Veisi et al. (2010a). As per this procedure, 15 gm of soil was mixed with 150 ml of 0.5N NaOH and boiled. The solution was then centrifuged and passed through a membrane filter using Buchner funnel. The filtrate was analyzed for the amount of alumina and silica using inductively coupled plasma mass spectrometry (ICP-MS). The data obtained from this test are presented in Table 3.6, and photographic representation of the process is shown Figure 11.7.

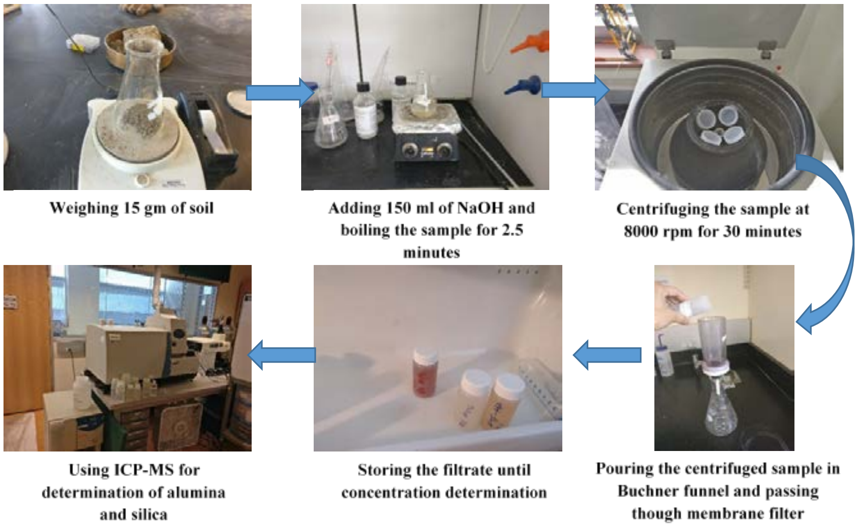


Figure 11.7: Photographic representation of reactive alumina and silica test

11. Cation Exchange Capacity

Cation Exchange Capacity (CEC) is the ability of the soil to exchange free cations in the exchangeable locations. High CEC corresponds to high amounts of expansive clay minerals such as Montmorillonite. CEC can be used to determine the mineral composition of a given soil. A soil with a high CEC value indicates a high amount of expansiveness due to the presence of the clay mineral Montmorillonite whereas a low CEC indicates the presence of non-expansive clay minerals such as Kaolinite and Illite. One of the earliest methods proposed by Chapman (1965) is the most commonly used method in the field, and this method had been selected for the current study. The method involves the addition of a saturating solution and the removal of the adsorbed cations using an extracting solution.

The saturating solution used here is ammonium acetate (NH_4OAc) at pH 7. This solution is added to prepared soil specimens and set aside for 16 hours after shaking for half an hour, to ensure that all the exchange locations are occupied by the ammonia ion (NH_3-N). Then the solution is filtered through a Buchner funnel and washed with 4 different 25 mL additions of NH_4OAc. This step is to bring out all the cations from the soil sample solution that has been replaced by ammonium ion. Excess NH_4OAc was removed by the addition of 8 different 10 mL additions of 2-propanol. Now, all the cation places are replaced by the ammonium ion, and excess ammonium was also removed. The CEC of the soil sample can be obtained by extracting the ammonia ions in the exchange locations. This was done by washing the sample with 8 different 25 mL additions of 1M potassium chloride (KCl) solution. Though potassium ion (K+) has similar electronegativity as ammonia, it has higher molecular weight and has the ability to substitute the NH_3-N ion. After the exchangeable cations are extracted in the form of NH_3-N) in KCl extract, the salicylate method using TNTplus 832 reagent was used to determine the concentration of NH_3-N with the help of HACH D2800 spectrophotometer. The concentration of NH_3-N was used to calculate the CEC of the soil. The data obtained from this test are presented in Table 3.6, and a photographic representation of the steps involved in this procedure are presented in Figure 11.8.

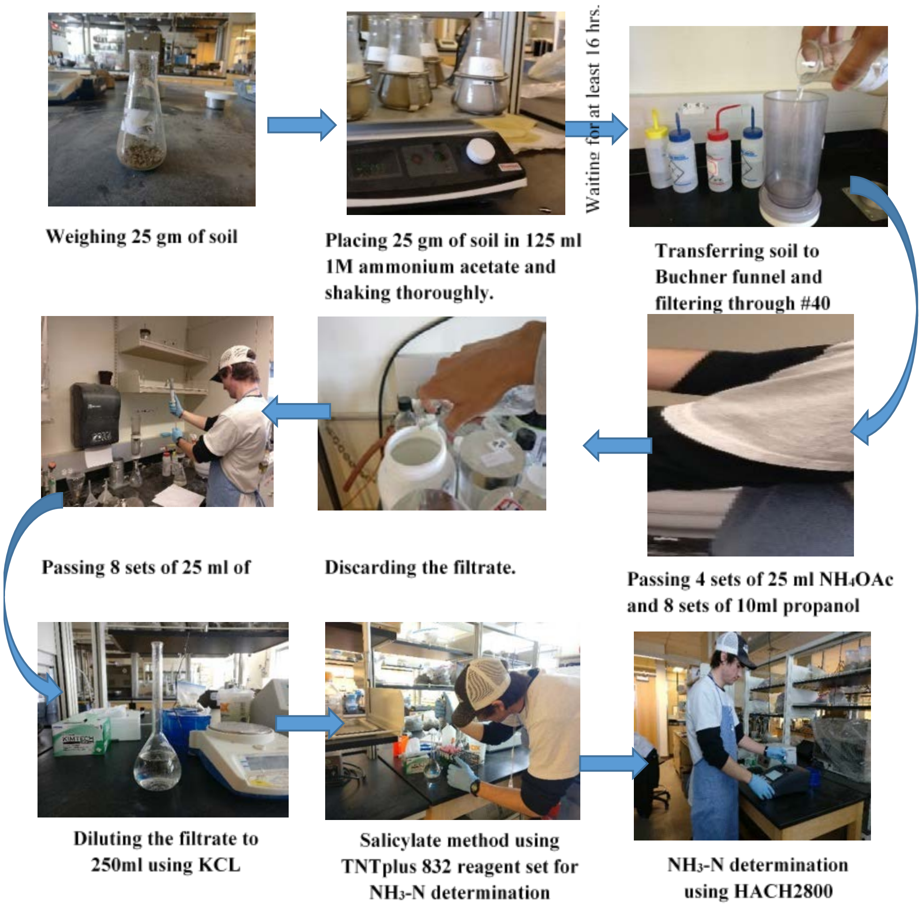


Figure 11.8: Photographic representation of CEC test

12. Specific Surface Area

Specific Surface Area (SSA) of a soil sample is defined as the total surface area of the particles contained in a unit mass of soil which is directly proportional to the particle size of the soil. High SSA corresponds to the presence of large portions of smaller size particles which typically are clays with higher water holding capacity and greater swell potential. SSA is a good indicator of the mineral content in the soil. Higher SSA indicates the presence of swelling minerals while lower SSA indicates their absence. The method developed by Cerato and Lutenegger (2002) was closely followed to determine the SSA of the six soils. This procedure involves saturating prepared soil samples in ethylene glycol monoethyl ether (EGME). The saturated soil is then placed in vacuum over calcium chloride-EGME solvate to find a point of equilibrium. The soil sample is weighed every two hours to verify equilibrium. Equilibrium is established when there is no weight change in successive two-hour measurements.

The specific surface is then determined from the mass of retained EGME in comparison to the amount retained by pure Montmorillonite clay, which is assumed to have a surface area of 810 m2/g (Carter et al. 1986).The test procedure typically takes two days to complete. This method was fully evaluated for geotechnical usage by Cerato and Lutenegger (2002) and concluded that the method is applicable to a wide range of mineralogies and is capable of determining specific surface area ranging from 15 to 800 m^2/gm. They also indicated that the procedure is repeatable and gives reliable results. A detailed procedure for the determination of SSA by EGME method that has been followed in the current study is given in Figure 11.9.

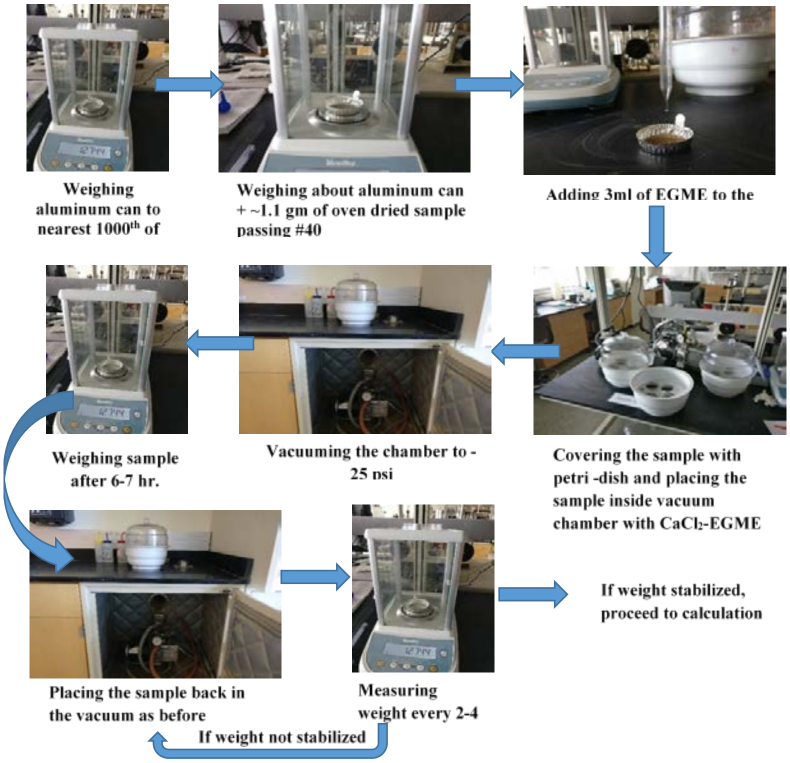


Figure 11.9: Photographic representation of SSA Test

12 APPENDIX B

Secant Modulus:

Secant modulus is the slope of a line drawn from the origin of the stress-strain diagram and intersecting the curve at the point of interest. Therefore, the secant modulus can take different values depending on the location of intersect. The figure below shows how the secant modulus is obtained at point A on the curve. The secant modulus can be expressed as a percentage of the Young's Modulus (e.g., 0.7E or 0.85E), and it is used to describe the stiffness of a material in the inelastic region of the stress-strain diagram. Secant modulus is commonly denoted by E_s.

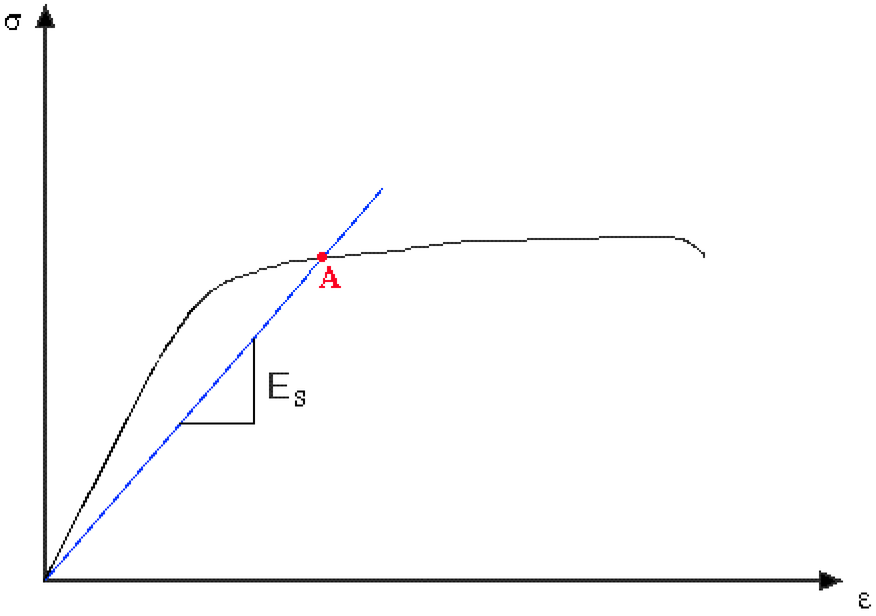


Figure 12.1 Definition of Secant Modulus