FREEZE-THAW PERFORMANCE OF LOW-CEMENT CONTENT STABILIZED SOILS FOR CONTAINMENT APPLICATIONS

by

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DEDICATION

This work is dedicated to:

Family and friends

Thank you for all of the love and support.

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Abstract

A laboratory-scale experimental study was performed to extend the available knowledge on the hydraulic and mechanical performance of cement-stabilized soils in cold regions. Experiments were performed on soil-cement (3 percent or 6 percent cement content by dry weight of soil) of different mix proportions before and after exposure to freeze/thaw cycles. For control specimens (i.e. no freeze-thaw), experiments showed more improvement in hydraulic conductivity when compacted at optimum and wet of optimum water content standard proctor conditions. Experiments showed increases in hydraulic conductivity after freeze/thaw cycling. Freeze/thaw damage related to hydraulic performance was observed to be more at optimum moisture content (i.e. maximum density) compared to dry and wet of optimum compaction water content conditions. Unconfined compressive strength also showed a decrease after exposure to three freeze/thaw cycles.

To further study the mechanisms of freeze/thaw damage, thin sections were obtained from control and exposed soil-cement specimens. Thin sections were examined using an optical microscope to study the structural changes of soil cement specimens due to exposure to freeze/thaw cycles. Mercury Intrusion Porosimetry (MIP) test was also used to examine the changes in porous structure of the soil-cement due to changes in moisture and cement contents, as well as due to exposure to three freeze/thaw cycles. Thin section results showed a lack of ice lenses in the samples after freeze thaw and the formation of both cracks and matrix disruption after freeze thaw. MIP results comparing both before and after freeze thaw failed to show major changes in pore size distribution of the soil-cement samples which tends to agree with the hypothesis that most of the damage observed from hydraulic conductivity testing is likely due to cracking and macroscale pore changes.

List of Abbreviations and Symbols Used

ACI: American Concrete Institute

ASTM: American Society for Testing and Materials

CSBC: Cement-Stabilized Base Courses

ITRC: Interstate Technology and Regulatory Council

MIP: Mercury Intrusion Porosimetry

PCA: Portland Cement Association

RF: Resonant Frequency

S: Degree of saturation of soil-cement specimen

UCS: Unconfined Compressive Strength

W: Moisture content of specimen after curing

Gs: Specific gravity

ρd: Dry density

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Chapter 1: Introduction

1.1 General

Some soils don't have the required engineering properties for construction projects. The common practice for problematic soils is to replace a soil layer by another layer of better engineering properties. This is not always a cost effective or a practical solution. Moreover, there is an environmental impact of disposing large amounts of soil on another site. Geotechnical engineers introduced soil-cement as a feasible solution to improve engineering properties of problematic soils and make them suitable for the project requirements.

Soil-cement is the process of stabilizing soil by the addition of cement. The soil is strengthened and made resistant to softening by the addition of cement, though some admixtures may be added with the cement for various conditions (Andrews, 1960). The soil layer can be made less permeable through the addition of cement which fill the voids and improves bonding between soil particles. Soil-cement can be further defined as a material produced by blending, compacting, and curing a mixture of soil/aggregate, Portland cement, possibly admixtures including pozzolans, and water to form a hardened material with specific engineering properties. The soil/aggregate particles are bonded by cement paste, but unlike concrete, individual particles are not completely coated with cement paste (AC Institute 1997). Soil-cement was first being used almost hundred years ago. The focus of early studies was on the mechanical properties when most of the applications were roadway construction. Soil-cement hydraulic properties were studied when applications of hydraulic barriers and contaminant containment start being used almost sixty years ago. As discussed by the AC Institute (1997), there are several reasons to use cement as soil stabilizer:

1) Cement stabilization improves soil strength and stiffness which leads to less deflections caused by traffic loads. This delays fatigue cracking and extends pavement structure life.
2) A thinner cement-stabilized layer can reduce stresses more effectively than a thicker un-stabilized layer of soil by providing uniform strong support which results in less stresses on the subgrade. This reduces sub-grade failure.
3) Cement-treated bases have less risk of pumping subgrade fines due to lower hydraulic conductivity.
4) Cement stabilization improves moisture resistance to keep water out of the base material; resulting in higher structural strength and semi impermeable soil cement-layer (PCA 2013a).

Design of soil-cement relies on the construction applications and the final intended engineering properties. This research will focus on the hydraulic and mechanical performances of soil cement and its durability. This research will examine the use of low cement content (3 percent cement or 6 percent cement) for which not many studies have discussed freeze-thaw durability with respect to hydraulic conductivity, despite the wide use of soil-cement stabilization.

Exposure to multiple freeze thaw cycles can cause soil cement to undergo hydraulic and/or mechanical damage due to expansive forces created by water freezing (Dempsey & Thompson 1973). The addition of cement can prevent soil swelling and softening from absorption of the moisture and from freeze and thaw effects as the cement increases the shear strength and minimizes the water holding capacity of soils (Guyer 2011). This research will study the hydraulic and mechanical damages occurred due to exposure to multiple freeze/thaw cycles, as well as studying the mechanisms of the damage by examining the structural changes of soil cement specimens after exposure to freeze/thaw cycles.

1.2 Research Objectives

The goal of this research is to enhance the knowledge of hydraulic and mechanical performance of soil-cement and assess its durability under freeze-thaw conditions. The main objectives of this research include:

- Assessing the impact of using low cement content (3 percent or 6 percent cement by dry weight of soil) on the hydraulic and mechanical performance of soil cement by performing laboratory testing on soil-cement mixes at different cement contents and different moisture contents (dry of optimum water content, optimum water content, and wet of optimum water content).
- Evaluating the freeze/thaw damage on soil-cement of different moisture and cement contents by performing laboratory testing on exposed soil cement mixes.
- Examining the mechanisms of freeze/thaw damage by studying the structural changes and pore size distribution of control (not exposed to freeze/thaw) and exposed soil cement mixes. Jamshidi (2014) showed that for high cement contents (>10%), damage to soil cement was caused primarily by cracking of the material via matrix and aggregate/cement contact while Othman and Benson (1992) showed that for compacted clays, damage under freeze thaw was caused by pore-size increases, redistribution and frost lenses. This thesis will explain what happens to soil-cement at low cement contents and if it is going to behave similar to a high cement content stabilized soil or similar to a raw (non-stabilized) soil.

1.3 Thesis Organization

This thesis contains four chapters. Chapter 1 has provided an introduction to the research subject and specified the main objectives of the research. Chapter 2 of this thesis provides a literature review on soil-cement stabilization technology and mechanisms of freeze thaw damage. Chapter 3 describes the materials and testing procedures used to carry out the research objectives. Chapter 4 explains testing results and soil cement performance under the different conditions examined. Chapter 5 provides conclusions and recommendations for future work.

Chapter 2: Literature Review

2.1 Soil Cement Mixing Methods

The addition of cement to soil can be performed either by wet mixing or dry mixing. Wet mixing is performed on dry soils without sufficient water content for the cement to react with the soil. Cement is therefore added in grout form during mixing (Kempfert and Gebreselassi 2006). Dry mixing is performed on high moisture content soils where the cement has sufficient water react with the soil. Hence, cement is added in dry form during mixing (Kempfert and Gebreselassi 2006). Soil-cement can be mixed in place or in a central mixing plant. There are different types of in-place mixers such as transverse single-shaft mixer and windrow-type pugmill. Common types of central mixing plant equipment are continuous flow-type pugmill, batch-type pugmill and rotary drum mixer (AC Institue 1997).

2.2 Basics of Soil Cement Stabilization

As with concrete, soil-cement hardening is a chemical process which builds strong linkage between the soil minerals and soil aggregates resulting in a stiffer structure (Andromalos et al. 2000). Equations 2.1 and 2.2 show the hydration reactions of tricalcium silicate and dicalcium silicate of Portland cement of water (Mindess et al. 2003).

$$2(3CaO\text{-}SiO2) + 11H2O \rightarrow 3CaO\text{-}2SiO2\text{-}8H2O + 3Ca(OH)_2 \tag{2.1}$$

alite + water → calcium-silicate-hydrates (C-S-H) + calcium hydroxide

$$2(2CaO\text{-}SiO2) + 9H2O \rightarrow 3CaO\text{-}2SiO2\text{-}8H2O + Ca(OH)_2 \quad (2.2)$$

belite + water → calcium-silicate-hydrates (C-S-H) + calcium hydroxide

A series of strong bonds form between the particles, making a network in which the soil particles are trapped. In order for the soil-cement to gain strength, sufficient water must be available for this chemical reaction to happen (Bofinger 1978). Sufficient curing will result in the best engineering performance for soil stabilization. Weather and moisture are two critical factors affecting curing that can have a direct influence on the hydraulic and mechanical performance of the stabilized base (Caterpillar 2006).

Based on mixing proportions and required final properties, soil-cement is designed to be classified as one of the following categories:

Cement-modified soils is a term used when adding low cement content. However the soil-cement material will have slightly improved strength and reduced hydraulic conductivity when compared to initial soil being (Portland Cement Association 2011).

Compacted soil-cement is a term used when using a water content close to the optimum water content in the mix design of the soil-cement which provides the maximum density when compacted on site. The soil-cement product is hardened due to hydration of cement. The final soil-cement product will have significantly higher strength than the raw soil (Portland Cement Association 2011).

Plastic soil-cement (flowable mortar) is a term used when using very high moisture content. It is a self-compacting soil-cement. Plastic soil-cement requires higher cement content in the mix than compacted soil-cement and cement-modified soils due to the high water content (ACI 1999). In this thesis, the term “soil-cement” is used throughout to avoid confusion.

2.3 History of Soil Cement Stabilization

According to the AC Institute (1997), the earliest documented application of stabilising soil with cement was 1915 when a street in Sarasota, Florida was constructed using a mixture of shells, sand, and Portland cement mixed with a plow and was subsequently compacted. Since then, soil-cement has become one of the most widely used forms of soil stabilization for highway construction. More than 170,000 kilometers of equivalent 7.3 meter wide pavement using soil-cement have been constructed between 1915 and 1997 (AC Institute 1997). In severe weather conditions and different climates, this technology has very few failures (less than 1 percent of the pavement area). These failure were mainly due to the inexperience of the construction methods and inadequate appreciation of the soils and soil-cement properties (Andrews 1960).

In Europe, the leader in soil-cement construction was Germany. During the war, the scale of soil-cement construction work was comparable to that of the United States where machinery for the soil-cement construction work was specially manufactured. In Holland, the scale of soil-cement construction has been high since 1956 due to construction materials shortage which led engineers to seek alternatives to earthen materials (Andrews 1960).

Following World War II, there was a rapid expansion of water resource projects in the Great Plains and South Central regions of the United States. Rock riprap of satisfactory quality for upstream slope protection was not locally available for many of these projects. The cost of transporting riprap from distant quarries to the water resource projects was high and that high cost threatened the economic feasibility of some projects. The U.S. Bureau of Reclamation (USBR) launched a major research effort to study the soil-cement as a proper alternative to conventional riprap.

Laboratory studies were performed and mixing cement made with sandy soils, the application being as an efficient durable erosion-resistant facing. The USBR constructed a full-scale test section in 1951 (AC Institute 1997). Tests were performed on a test-section along the shores of Bonny Reservoir in eastern Colorado. Conditions such as waves, ice and more than 100 freeze-thaw cycles per year might have impacted the performance of soil-cement. Soil-cement proved to be efficient throughout ten years of testing in this site before USRB specify the soil-cement as an efficient alternative to riprap for slope protection in 1961 (AC Institute 1997).

2.4 Various Soil Cement Stabilization Applications

One important aspect of soil-cement is that its cost compares favorably with that of granular-base pavement. When built for equal load carrying capacity, soil-cement is almost always less expensive than other low-cost site treatment or pavement methods. The use or reuse of in-place or nearby borrow materials eliminates the need for hauling of potentially expensive, granular-base materials; thus both energy and materials are conserved (PCA, 2013a). Soil-cement has also been used as conventional riprap alternative in projects requiring slope protection including dam facing, channels, spillways, railroad embankments and embankments for inland reservoirs as shown in Figure 2.1. Soil-cement facing has been shown to how high durability resistance to moderate and severe wave action (AC Institute 1997).

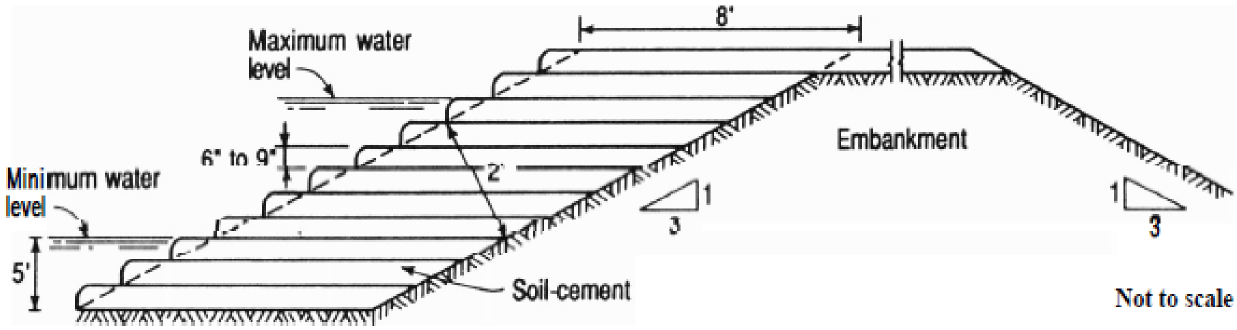


Figure 2.1 Soil-Cement Slope Protection Showing Layered Design (AC Institute 1997)

Soil-cement has been used to form liners around contaminated sites such as refineries and landfills as it can prevent contaminant migration through the soil. Soil-cement liners proved to be efficient for a long term against exposure to various hazardous and toxic materials (AC Institute 1997). Mixing weak soils with cement can also provide an efficient foundation stabilization alternative to installing piles as soil-cement can significantly improve weak soil strength under the structure instead of utilizing piles which may add to the high costs of the project (AC Institute 1997).

The use of soil-cement has been expanded over the years to include:

- Airports
- Parking areas
- Storage areas
- Reconstruction and recycling of failed flexible pavements
- Channel and ditch linings (PCA 1995).

2.5 Typical Mix Design Proportions

The purpose of a soil-cement mix design is to establish the proportions of cement and water required to stabilize a soil to its designed properties. These properties are inherently related to the level of compaction to achieve the maximum dry density and optimum moisture content that governs field control (Andromalos et al. 2000).

The soil-cement mix design proportions will vary based on the properties of a soil and the particular application of soil-cement. Strength properties are usually determined by performing unconfined compressive strength tests of the soil-cement specimens under study and they are a function of water to cement ratio which typically ranges from 0.9 to 1.3 by weight. Higher cement ratio and/or cohesion-less soils lead to upper limits of shear strength properties while lower limits are usually obtained for lower cement ratio and/or cohesive soils. Settlement properties are usually determined by performing consolidation and standard penetration tests on the soil-cement specimens (Andromalos et al. 2000).

Portland cement type I and II are the most common cement types of Portland cement being used in soil-cement mix designs. Cement content varies based on local soil properties, soil-cement application and additives being added to the mix. The typical cement content proportion ranges from 3 to 8% for most transportation applications (Halsted 2011). The US Army Corps of Engineers frequently increases cement content by 1 or 2 percent to account for the variation of soil properties over the site (AC Institute 1997). For containment applications, this amount of cement can be higher to achieve target hydraulic conductivity.

2.6 Common Additives in Soil Cement Mix Designs

There are multiple soil stabilization additives that can be utilized in a mix design to meet the required engineering soil properties. The selection of these additives is based on soil granularity, plasticity, texture and the engineering properties to be improved in the mix design.

Portland cement is the most common additive in a mix design and it has been proven to be efficient with most type of soils except plastic soils (Guyer 2011). The more plastic the soil is, the less desirable to use cement. High plasticity soils are difficult to be pulverized and uniformly mixed with cement. For highly plastic clay soils, hydrated lime or quicklime may sometimes be used as a pretreatment to reduce plasticity and make the soil more friable and susceptible to pulverization prior to mixing with cement (AC Institute 1997). Well-graded granular soils that possess sufficient fines result in better performance of Portland cement in stabilizing the soil. Lime is another type of stabilizer which can perform well with soils of medium to high plasticity and produce a mix of decreased plasticity, higher workability, reduced swell and increased strength. Lime can also be used with different types of weak soils and clay-gravels; stabilizing them to perform well as a base course (Guyer 2011).

Fly ash is a pozzolonic material that is almost always mixed with lime stabilizers in soils that have little or no plastic fines. Other types of traditional additives are bitumen and cement kiln dust (Guyer 2011). There are some non-traditional additives such as:

1) Polymers Based Products: can be used for stabilization and erosion control of poorly graded sands
2) Copolymer based products: can be used for soil stabilization and dust control

3) Fiber Reinforcement: have a high resistance towards chemical and biological degradation and do not cause leaching in the soil
4) Calcium Chloride: mainly used in highway constructions, dust control and maintenance (Al-khanbashi and Abdalla 2006).

2.7 Testing and Performance Criteria of Soil Cement

Performance tests are carried out to investigate the suitability of soil-cement application. It is essential to prove that the contractor has produced a constructed material in accordance with the design and specifications. Performance tests have the ability to evaluate the damage of weathering conditions including wetting-drying and freezing-thawing on soil-cement. The following sections provide a brief overview of these performance tests.

2.7.1 Leaching Tests

The main purpose of leaching tests for soil-cement materials is to obtain aqueous phase concentrations of constituents which are released from solids when placed in a land disposal unit. Leaching potential for the same chemical can be quite different depending on a number of factors such as characteristics of the leaching fluid, form of the chemical in the solids, and the disposal conditions. ASTM has developed standard leaching tests which use alternate leaching fluid with very little additional difference in the test methodology (Centioli et al. 2008).

There are different types of leaching tests such as Laboratory Batch Equilibrium (ASTM D5233 - 92(2009)), Laboratory Column (ASTM D4874 - 95(2014)) and Solubility Based (ASTM D7190 - 10(2011)). Leaching tests can be used to estimate the migration of the contaminants through the soil and assess the subsequent risk of contamination. This can provide important parameters in the design of environmental protection such as soil-cement liners around landfills and contaminated sites (Centioli et al. 2008).

2.7.2 Hydraulic Conductivity

The hydraulic conductivity test is a common soil and soil-cement test. It reflects the rate of water flow through the soil or soil-cement materials. Hydraulic conductivity testing is very important in all soil-cement applications especially as liner around landfills and contaminated sites. In roadway construction, hydraulic conductivity test can be used to estimate the amount of water penetrates the soil-cement base under the pavement.

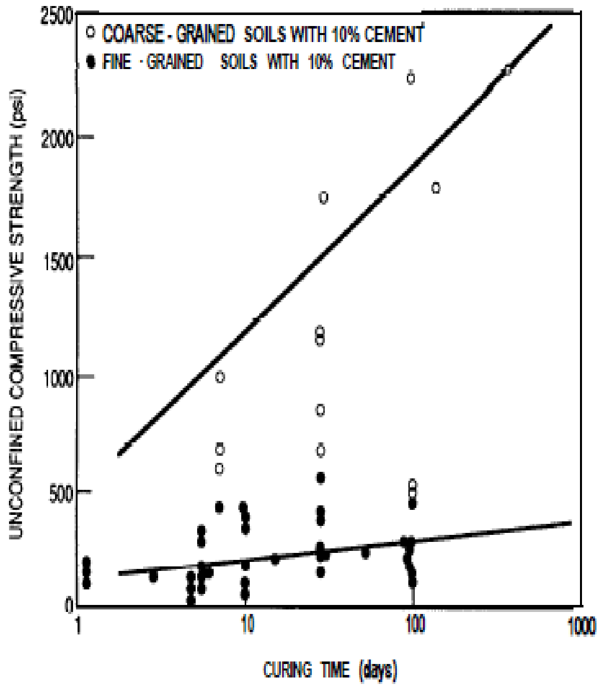
For water-saturated porous media, there are two general hydraulic conductivity tests, Constant head and falling head tests. The constant head test is used for permeable soils of hydraulic conductivity greater than 10^{-4}cm/s and falling head test for less permeable soils of hydraulic conductivity less than 10^{-4} cm/s (ASTM D2434-68 (2006)). Sample preparation for standard soil-cement specimen is provided in ASTM D2434-68 (2006).

In this research, constant head flexible wall permeameter (ASTM D5084-10 (2010)) was used to determine the hydraulic conductivity of the soil-cement specimens. The advantages of a flexible-wall cell include complete control over the state of stress existing within the test specimen and the ability to back-pressure saturate and consolidate the specimen prior to membrane testing. The disadvantage of flexible-wall cells included higher cost, leakage problems between the flexible confining membrane, and the need to apply significant confining pressures when testing soil specimens under high hydraulic gradients (Daniel & Trautwein 1994).

2.7.3 Unconfined Compressive Strength

Unconfined compressive strength is the most widely referenced property of soil-cement and it is usually measured according to standard test methods for compressive strength of molded soil-cement cylinders (ASTM D1633 (2007)). The test is used to assess soil-cement compressive strength and hence the degree of reaction of soil-cement water mixture. Soil-cement compressive strength is one of the more common parameters to determine the minimum cement requirements for soil-cement mix design proportions. This is often related to the application (i.e. pavement, etc.) however, unconfined compressive strength can be affected by the degree of compaction and water content.

Cement content is one of the most important factors the unconfined compressive strength of a soil-cement material. Laboratory test results show that increasing the cement content generally gives a higher compressive strength soil-cement mixture. This increase in compressive strength can vary according to the soil grain size. Another factor that can have a significant impact on the soil-cement compressive strength is the curing time. Longer curing times mean longer time allowed for the cement and water to react and better bonding between the soil and cement particles which result in a stronger and less permeable soil-cement mix. The typical curing times for the ASTM standard unconfined compressive strength test are 7 and 28 days. Figures 2.2 and 2.3 show the curing time and cement content relations versus the unconfined compressive strength of soil-cement (AC Institute 1997).



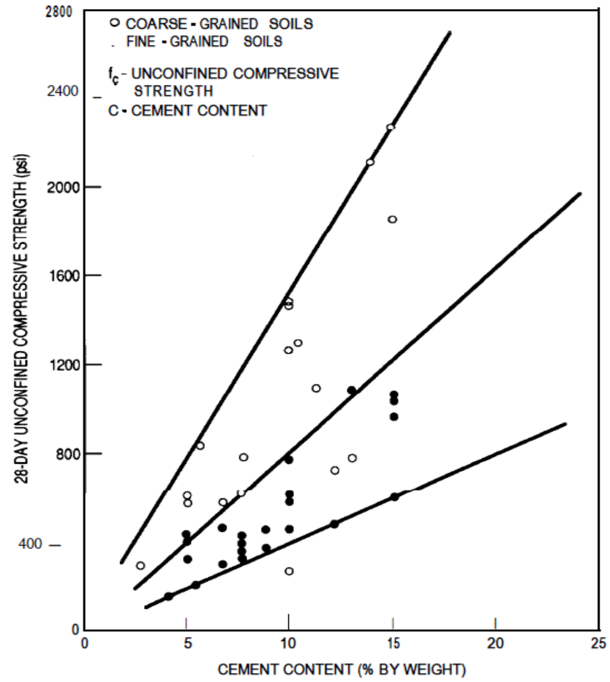


Figure 2.2 Curing time vs 28-days UCS (AC Institute 1997)

Figure 2.3 Cement content vs UCS (AC Institute 1997)

Also, freezing and thawing is one of the factors that have significant impact on the long term compressive strength performance of soil-cement. To evaluate this effect, soil-cement specimens in this research will undergo unconfined compressive after exposure to three freeze-thaw cycles. Jamshidi (2014) performed testing on soil-cement specimens after exposure to multiple freeze/thaw cycles and noticed a significant impact after only three freeze/thaw cycles. However, the Alberta Department of Transportation (2004) recommended testing after exposure to 12 freeze/thaw cycles when most of the damage has occurred.

2.8 Freeze Thaw Performance of Soil, Concrete and Soil Cement

2.8.1 Soil Freezing and Ice Lens Formation

Given that soils under structures are susceptible to freeze/thaw cycles, it is very important to understand their thermal and subsequent mechanical behavior. Thermal changes to frost susceptible soil will often lead to irreversible restructure of the system causing thermal stress relaxation and creep deformation (Watanabe 1999). When soil-temperature is cooled below 1 C°, water begins to solidify or freeze. Two conditions should exist for soil freezing to happen; sufficient cold weather and a soil that has sufficient moisture content (Nixon et. al 1998).

When soil is exposed to sub-zero temperature conditioning, pore water freezes and subsequent heave occurs due to ice lens formation. Nixon et. al (1998) state that a suction gradient will develop due to water freezing resulting in water migration from the unfrozen soil through the frozen fringe where it accumulates and freezes. The frozen fringe is a zone of soil located between the active ice lens and the unfrozen soil. The frozen fringe acts as a zone of flow obstruction caused by partial freezing of pore water. Water in the soil flows through the frozen fringe in a thin layer of unfrozen water that remains close to the soil particles. When soil freezes, water expands up to nine percent causing heave in the frozen soil which could lead to cracking of pavement on the surface as shown in Figure 2.4 (Nixon et. al 1998).

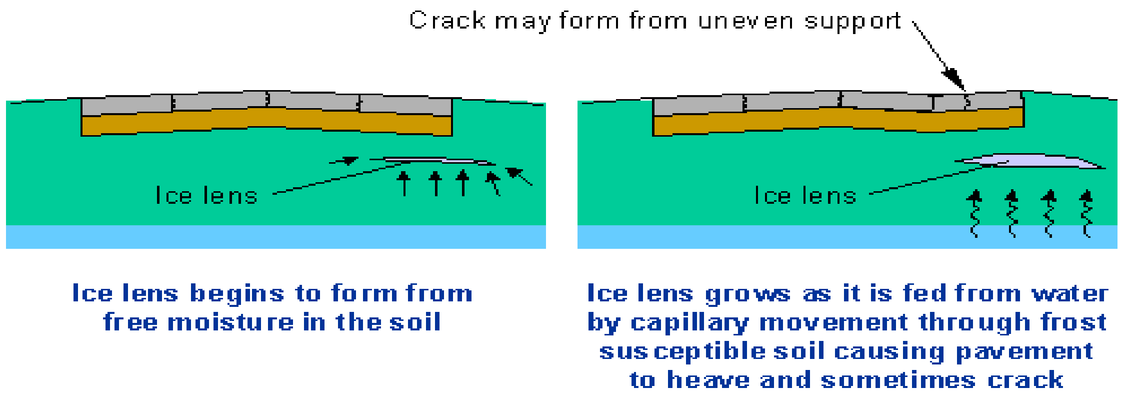


Figure 2.4 Formations of Ice Lens and Frost Heave (American Concrete Pavement Association 2013).

Konrad and Morgenstern (1980) mentioned that for fine-grained soils, if rapid change in temperature occurred across the soil depth, it will create invisible ice lenses. Thin ice lenses will appear when the freezing front advances and the temperature gradient decreases. When the temperature becomes constant, the frost front will stop advancing and a final lens will form (see Figure 2.5). In Figure 2.5, Tc and Tw represent the cold and warm temperatures. Ts and Tp represent segregation freezing temperature and pore-freezing temperature respectively (Konrad and Morgenstern 1980).

When frost penetration happens at slow rates, a linear temperature profile could be maintained throughout the soil while in very quick frost penetration rates, the thickening frozen fringe will prevent or delay heat flow through the fringe developing a bilinear temperature gradient (see Figure 2.5) (Konrad and Morgenstern 1980).

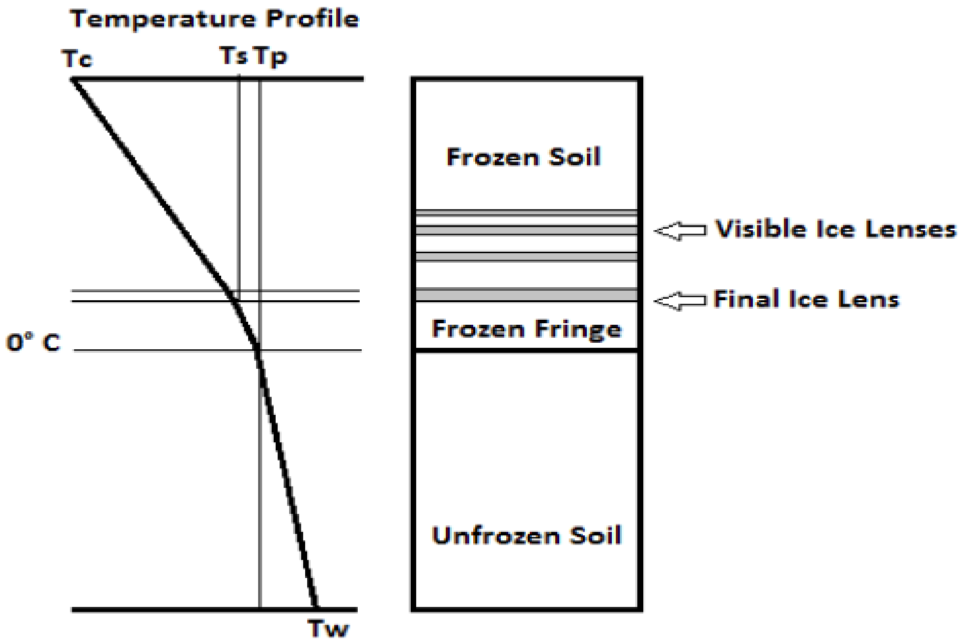


Figure 2.5 Schematic of Ice Lens Formation (Konrad and Morgenstern 1980).

2.8.2 Physical Processes Responsible for Moisture Movement in Frozen Soils

The existence of an ice phase significantly enhances the transfer of moisture under a temperature gradient. Free water migrates through the soil to a forming ice lens because the forming ice lens is a higher suction area. This migration of water can be as far as 6 m for certain frost susceptible soils (Andersland and Ladanyi 2004). When soil water freezes, a suction gradient will develop which can lead to water movement upward from a deeper unfrozen zone to the lower-temperature freezing zone, which will cool down and freeze (Nixon et. al 1998). The rate of water flow in the soil goes down as the soil temperature goes down (see Figure 2.6).

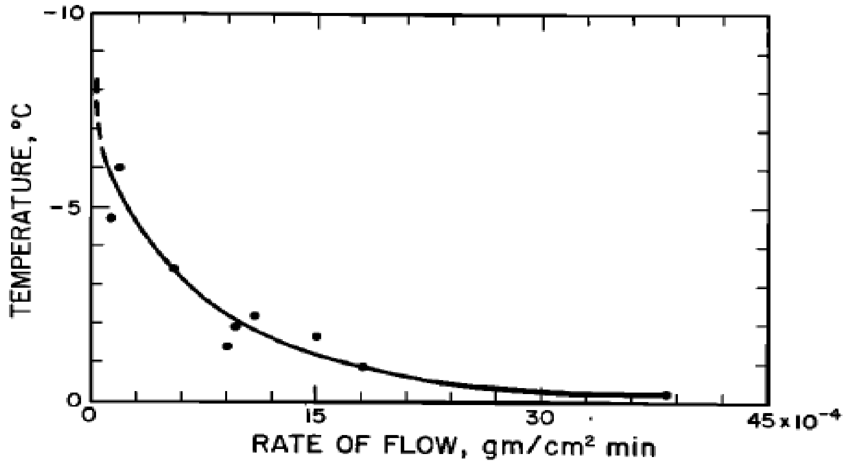


Figure 2.6 Rate of Moisture Content Flow in Frozen Soil as a Function of Temperature below 0C˚(Hoekstra 2010).

In porous media and any flow system, water transfer from high energy area to low energy areas to achieve equilibrium. The energy varies due to different temperature, elevations, pressures and ion concentrations throughout the soil layers. The vapor from water content of warmer underlying soil layers has greater vapor pressure and moves upward to a cooler, lower vapor pressure layer, condensed into water and ultimately crystallizes into ice. Salts in pore cavities dissolved in freezing water expels outside the pore cavity into adjacent unfrozen water creating a region of high ion concentration in underlying soil layer. Therefore, water will flow from higher ion concentration region upward to the frost heave where there is lower ion concentration region. This mechanism called osmosis. The last mechanism is capillary rise, before soil freezing, water on the surface of soil particles and in the pore space forms a network of channels for the water to flow. As freezing front passes through the soil, ice crystals form in the water between soil particles as shown in Figure 2.7 (Guthrie et al. 2007).

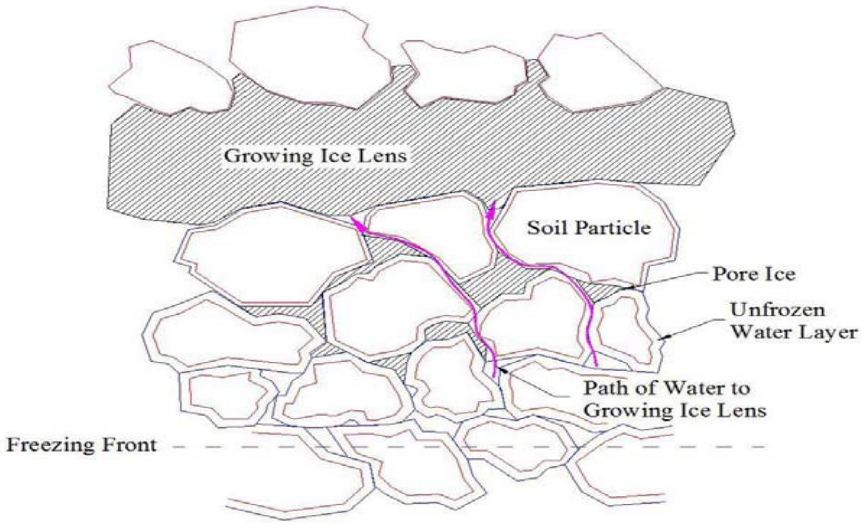


Figure 2.7 Schematic of Growing Ice Lens (Guthrie et al. 2007).

There are two theories of moisture movement in frozen soils. The first theory states that water moves through unfrozen films existing on the surface of the soil particles. The pore pressure and the gravitational potential are the dominant driving forces. The second theory of moisture movement states that water moves through frozen soils is referred to as regelation. Regelation is the phenomenon of melting under pressure and freezing again when the pressure is reduced. In Regelation, the mechanism responsible for moisture movement is the heat transfer with phase change (Kane & Stein 1983).

2.8.3 Physical Changes and Damages of Frozen Soils

The volume of water in the soil will increase after freezing which leads to increase in soil volume that can cause cracks and deformations in foundations, pipelines and roadways. The volume change can lead to differential settlement and change in soil density. To solve those engineering problems it is important to study the ice lens mechanism in freeze and thaw conditions. Other than the soil, other porous media involving soil-cement and concrete are also susceptible to ice lensing problem (Watanabe 1999). It is more usual that pavements undergo cracking and straining during the thawing period of the underneath soils. Insufficient internal drainage or no drainage can lead to significant reduction in the strength of the underneath active soil layers. The usual difference in soil characteristics over the pavement results in non-uniform heave which leads to unacceptable deformations and uncomfortable driving (Andersland and Ladanyi 2004).

Andersland and Ladanyi (2004) suggest that repeated freeze/thaw cycles produce an increase in effective void ratio which result in increase of the soil hydraulic conductivity. Konrad (1989) detected primary change in hydraulic conductivity during the first three freeze/thaw cycles.

2.8.4 Physical Changes and Damages of Frozen Concrete

Similar to soil, concrete moisture content is also susceptible to physical changes from freezing. When water freezes, it expands 9% by volume which produces pressure in the pores of the concrete. The cavity will dilate and rupture when the pressure due to volume change exceeds the concrete tensile strength. The accumulative effect of multiple freeze/thaw cycles and the disturbance of concrete constituents can eventually lead to expansion and cracking, scaling, and crumbling of the concrete (PCA, 2013b).

D-Cracking of concrete pavements is one of the common cracks caused by freeze/thaw effect on concrete (see Figure 2.8). D-cracks are closely spaced crack formations parallel to transverse and longitudinal joints. Naturally, water accumulates under the pavements in the base and subbase layers and saturates the aggregates. After multiple freeze/thaw cycles, cracking of the concrete starts in the saturated aggregate layer at the bottom of the slab and progress upward until it reaches the concrete pavement. This problem can be mitigated by using aggregates more resistant to freeze/thaw effects or by designing a more effective drainage system to avoid water accumulation under pavements. Another concrete damage due to freeze/thaw effects is the concrete scaling (see

Figure 2.9). Concrete scaling is local flaking or peeling of a finished surface of hardened concrete due to the exposure to freeze/thaw cycles (PCA 2013b).

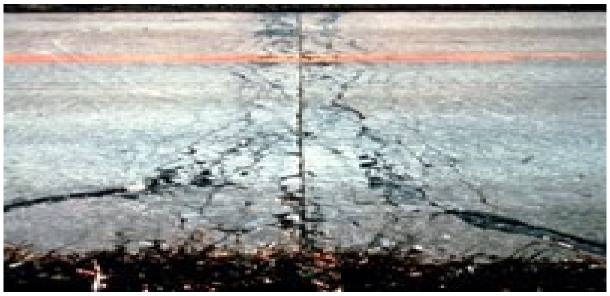


Figure 2.8 D-Cracking (PCA 2013b)

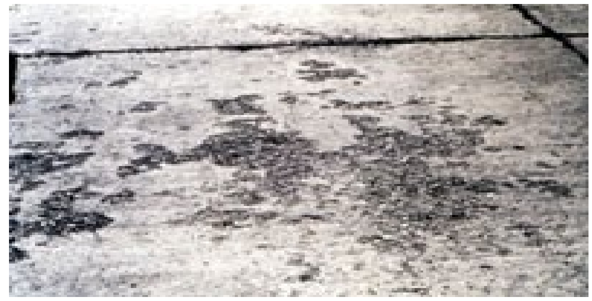


Figure 2.9 Concrete Scaling (PCA 2013b)

Concrete air entrainment technology improves the concrete resistance to freeze/thaw effects. The tiny entrained air voids act as empty chambers in the freezing and migrating water, thus relieves the pressure in the pores and prevents concrete damage (PCA 2013b). Mixing cement with soil can prevent those damages, as soil-cement has a very low permeability that can prevent water accumulation and subsequent freezing (PCA 2013b).

2.8.5 Physical Changes and Damages of Frozen Soil Cement

Soil-cement characteristics represent an intermediate between concrete and soil and its mix proportions can be set according to its purpose. In cold regions, it is likely that the quality of the soil-cement may be reduced due to cycles of freeze/thaw of water in soil-cement. Soil-cement bases are designed to have very low permeability that can keep the water away from the soil-cement base so under freezing conditions no ice lens can be formed (PCA 2005). Guthrie et al. (2007) provided testing results to show that stabilizing the soil with sufficient amount of cement proved to be effective in improving the resistance to frost heave. However, adding insufficient amount of cement can lead to a frost heave of greater than that occurring in untreated soil samples. This behavior was associated with the ingress of a substantial amount of water which proved in further testing to be the reason behind the changes in suction and permeability properties. Soil-cement testing results show that freeze/thaw testing have to be one of the controlling factors in determining the amount of cement needed in soil-cement mixture as too much cement content may cause shrinkage cracking, water ingress and structural deterioration while too little cement content may cause ice lens formation and worse frost heave behavior than in untreated samples (see Figure 2.10). More ice lens formation means greater thaw straining and deformation in the exposed pavement layer during spring time (Guthrie et al. 2007).

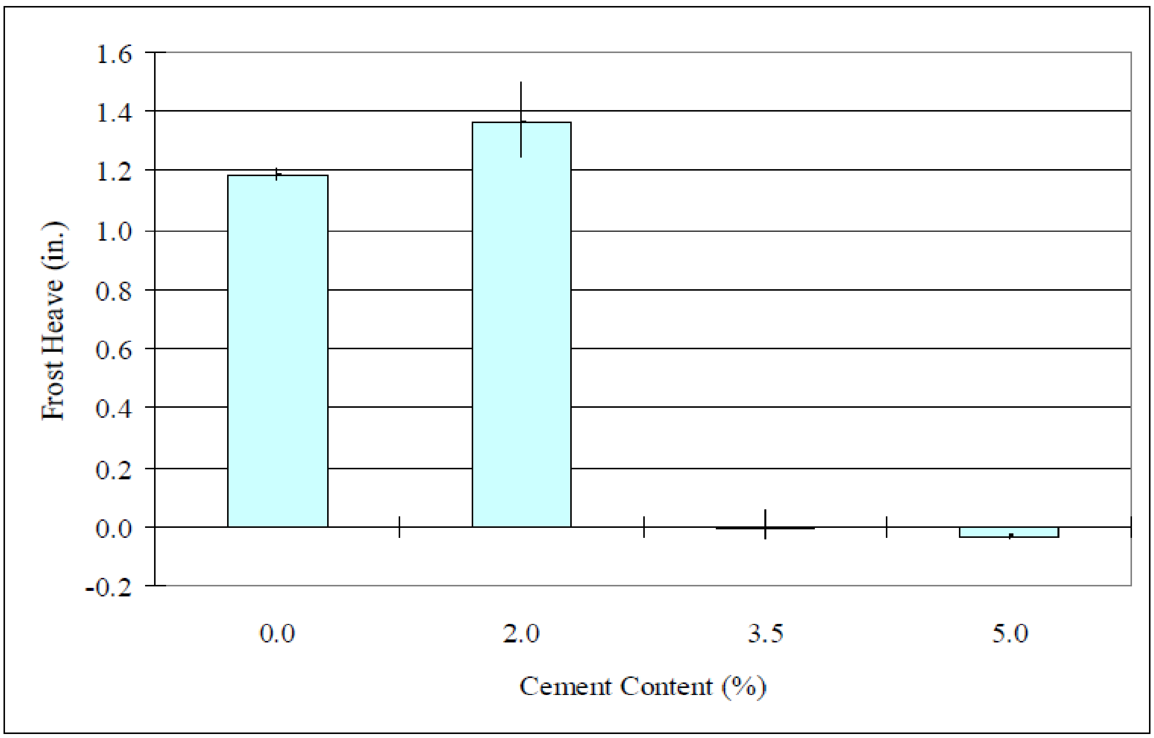


Figure 2.10 Cement Content vs. Frost Heave (Guthrie et al. 2007)

(Jamshidi et. al 2011) examined hydraulic conductivity and UCS performance for soil-cement samples of silty sand soil, 10% of cement and 2.28 water to cement ratio. After 7 freeze/thaw cycles, testing results showed an increase in hydraulic conductivity of up to two orders of magnitude as well as decrease in UCS values. (USEPA 1997) states a maximum hydraulic conductivity value of 10^{-8} m/s in the design of soil-cement liners. The initial values of hydraulic conductivity in the Jamshidi et. al (2011) research averaged 1.2×10^{-10} m/s. After 4 cycles of freeze/thaw the values approached 10^{-8}m/s as shown in Figure 2.11.

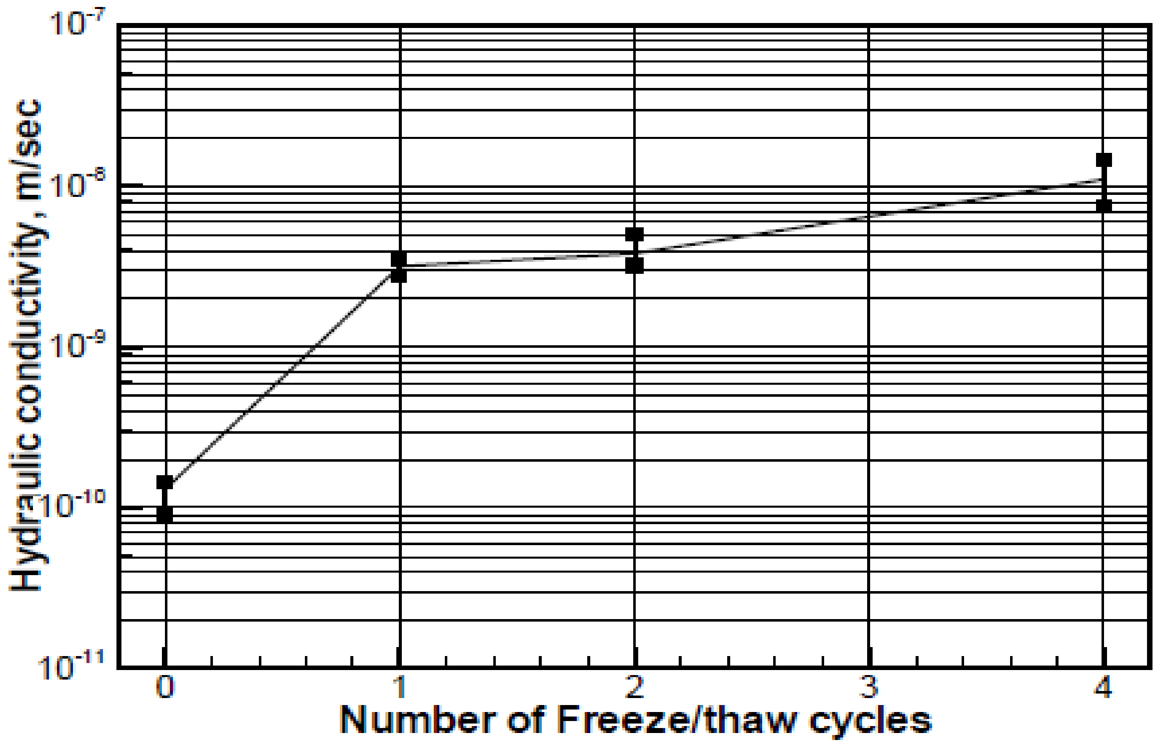


Figure 2.11 Number of Freeze/Thaw Cycles vs. Hydraulic Conductivity (Jamshidi et. al 2011)

Guthrie et al. (2007) carried out UCS, hydraulic conductivity and frost heave testing procedures on a silty subgrade soil mixed with 2, 3.5 and 5% cement contents. Mixing soil with 2% of cement showed higher permeability results than with untreated samples. However, at higher cement contents, results showed a significant decrease in hydraulic conductivity values. The higher hydraulic conductivity results detected at 2% of cement reflected more frost heave impact while less values of hydraulic conductivity at 3.5 and 5% of cement showed a significant decrease in or almost eliminated the frost heave phenomena in the soil as shown on Figure 2.10 (Guthrie et al. 2007).

The Interstate Technology and Regulatory Council ITRC (2011) explained the mechanism of inelastic deformation of soil-cement due to cycles of freezing and thawing. The constrained expansion of pore-water upon freezing can lead to significant stress in brittle small-pore material such as soil-cement. The stress caused by the increase in molar volume of ice over water causes a strain within the pore structure. If this strain was greater than the strength of the material, the material will deform inelastically by cracking. The internal stresses and resultant cracking due to freeze/thaw cycles lead to increase in hydraulic conductivity and decrease in compressive strength of soil-cement (ITRC 2011). At soil-cement of low cement content, less voids will be filled by the cement. Pressure developed due to water freezing and expansion will lead soil-cement particles to move, rearrange and consolidate similar to soils with no cement content. When ice lenses melt, micro cracks and macro cracks remain in the soil-cement structure leading to an increase in the hydraulic conductivity of the exposed material (Daniel & Kim 2001).

2.9 Freeze Thaw Damage Evaluation

As discussed earlier, mechanical and hydraulic performance of soil-cement can be damaged under freeze/thaw conditioning. So far, there is no standard method to detect changes in soil-cement performance in cold regions. Durability studies of Stegemann & Côté (1996) and Paria & Yuet (2006) suggested percent mass loss as an indicator of acceptability for performance of soil-cement in cold regions. Percent mass loss may sufficiently indicate changes in strength parameters (Shihata & Baghdadi (2001)) but Jamshidi (2014) showed that it may not be a reliable method for predicting changes in hydraulic performance of soil-cement.

2.9.1 Resonant Frequency Measurements

Resonant frequency was used in this research as a non-destructive test to evaluate the hydraulic damage of freeze/thaw cycles. Resonant frequency is a standard technique for concrete to estimate the changes in dynamic modulus of elasticity due to weathering and other potential deteriorations. It has also been used to monitor the development of dynamic elastic modulus of concrete with increasing maturity of test specimens (ASTM-C215 (2008)). There are factors such as manufacturing conditions and moisture content that can affect the results. Jamshidi (2014) performed longitudinal resonant frequency testing on control and exposed soil-cement specimens. Testing results by Jamshidi (2014) showed conformance with structural deterioration and subsequent increase in hydraulic conductivity due to freeze/thaw exposure.

2.9.2 Optical Microscope

Optical microscope is a useful method to observe fracture surfaces of thin sections that can be obtained from soil-cement specimens. Through optical observation, several changes due to freeze/exposure can be noticed such as cracking and matrix disruption. Different types of microscopes can be used to look at different characteristics of soil-cement specimens. Koskiahde (2004) examined the structural changes of concrete due to frost damage using micrographs taken by optical microscope. Horpibulsuk (2012) used the electron microscope to study the microfabrics of cement-stabilized clay. Jamshidi (2014) used optical microscope to examine the structural changes of soil-cement under various curing and exposure scenarios.

2.9.3 Mercury Intrusion Porosimetry (MIP)

Mercury Intrusion Porosimetry is a standard method for determination of pore volume distribution of soil and rock (ASTM D4404 (2010)). Previous study by Winslow & Lovell (1981) examined pores size distributions in cements, aggregates and soils while Lawrence (1977) studied pore sizes in fine-texture soils. The range of pore diameter for which this test method is applicable is determined by the operating pressure range of the testing instrument. Mercury can be intruded into the pores by the application of external pressure. The size of the pores that are intruded is inversely proportional to the applied pressure. MIP method only determines the volume of intrudable pores that are open to the outside (ASTM D4404 (2010)).

Summary

Cement stabilized bases are improved soil materials that have been treated with different proportions of Portland cement based on local soil properties and weather conditions. The objective of the treatment is to improve specific soil properties (i.e. hydraulic conductivity and UCS) based on intended soil properties for a specific project.

The improvement of engineering soil properties due to cement stabilization can be measured by different methods of testing including hydraulic conductivity and unconfined compressive strength (Halsted 2011).

The addition of cement content at specific design proportions can minimize the effects of freeze/thaw conditioning. The addition of cement at specific proportions can decrease the hydraulic conductivity of the soil significantly that can prevent ice lens and frost heave formation by leaving no space for the water to accumulate and freeze in the soil pores (Guthrie et al. 2007). Another soil property that can be improved significantly by the addition of cement and water is compressive strength. The reaction between soil-cement components provides stronger bonding between the particles that lead to increase in compressive strength of the soil layer (AC Institute 1997).

To study mechanisms of freeze/thaw damage on soil cement, macro and micro-structural changes can be studied using a thin section of soil-cement specimen under the microscope. Pore-size distribution can also be used to evaluate the damage of freeze/thaw as it can directly affect the hydraulic and mechanical behavior of soil cement (Lawrence 1977). Techniques of other porous media such as resonant frequency for concrete could be a good method for soil-cement to evaluate freeze-thaw damage on hydraulic performance (Jamshidi 2014).

Chapter 3: Materials and Methods

3.1 Materials Description

All soil cement mixes in this thesis utilized soil, cement and tap water. The soil was manufactured in the lab by blending two different soils, referred to as soils A and B in this thesis, at the proportions shown in Table 3.1. This soil blend shown in Table 3.1 allowed comparison of results to that obtained by Jamshidi (2014). Soil A is glacially-derived silty sand from the Timberlea area in Halifax, Nova Scotia that was initially air dried and sieved through a 9.5 mm (ASTM-D6913 (2004)) mesh to remove oversize material. Soil A was then separated into the proportions shown in Table 3.1 by passing the soil through 4.75, 1.2, 0.3 and 0.08 mm sieves. Any soil remaining on the 0.08 mm sieve was washed through this sieve to minimize the fine content in the soil and then dried at 110 ºC. Soil B was used to provide sufficient amount of fines in the soil blend. Soil B is the dust waste by-product of a Nova Scotia quarry operation that was prepared by air drying and sieving through a 0.08 mm mesh with any oversize material being discarded. The various soil proportions described above were then stored in separate plastic bags to be used for future blending according to each mix. The blended soil is classified as silty sand (SM) by the Unified Soil Classification System (USCS).

Table 3.1: Proportions of soils A and B used in the research (referred hereafter as "the soil").

<table>
<tr><th colspan="4">Soil A (weight as percentage of total)</th><th>Soil B (weight as percentage of total)</th><th rowspan="2">USCS classification of blended soil</th></tr>
<tr><td>9.5-4.75 mm</td><td>4.75-1.20 mm</td><td>1.20-0.30 mm</td><td>0.30-0.08 mm</td><td><0.08 mm</td></tr>
<tr><td>11%</td><td>36%</td><td>25%</td><td>13%</td><td>15%</td><td>Silty Sand</td></tr>
</table>

To provide information on the compaction characteristics of the soil when blended with mixtures of cement and water, standard proctor moisture density tests (ASTM-D558 (2011)) were performed at each of the two cement contents (ie. 3% and 6%) which are low cement contents compared to (>10%) cement contents used by Jamshidi (2014). General use Portland-limestone cement (CSA type GUL) was used as the binding agent for all the soil cement specimens in this research. The first compaction test involved blending the soil with 3% cement at water contents ranging from 6 to 14 percent. A second compaction test with 6% cement was repeated with water contents ranging from 6 to 14 percent. The compaction curves of both tests are shown in Figures 3.1 and 3.2. As shown in the figures, the optimum water content was approximately 10% for both tests with maximum dry densities of 2016 kg/m^3 and 2010 kg/m^3 for the 3% and 6% cement mixtures respectively.

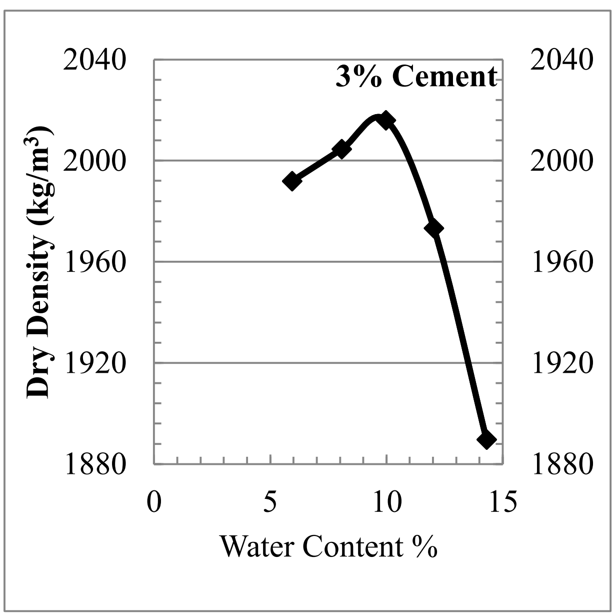


Figure 3.1: Compaction curve of soil with 3% cement content

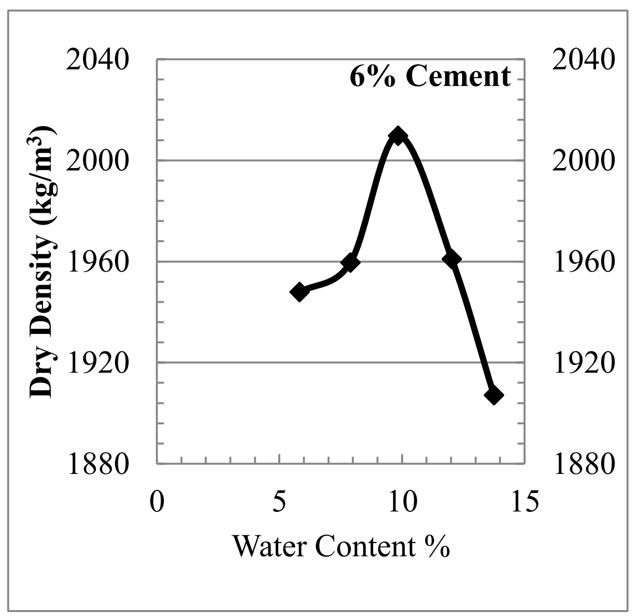


Figure 3.2: Compaction curve of soil with 6% cement content

Previous work by Jamshidi (2014) showed that silica and aluminum are the major oxides present in both soils A and B, which represent more than 80% of the entire composition, as shown in Table

3.2. X-ray diffraction tests performed by Jamshidi (2014) on soils A and B showed quartz and feldspar as the main mineralogical components of those soils.

Table 3.2: Mineral oxides analysis of soils used in this research (Jamshidi 2014)

Mineral Oxide	Soil A Wt. %	Soil B Wt. %
Al_2O_3	14.57	15.31
CaO	0.49	1.85
Fe_3O_3	2.66	5.66
K_2O	3.27	4.02
MgO	0.61	1.6
MnO	0.09	0.12
Na_2O	3.03	3.08
P_2O_5	0.23	0.39
SiO_2	71.82	65.65
LOI(1000°C)*	2.41	1.17
Total	99.18	98.85

*LOI: Loss on ignition

3.2 Specimens Preparation for Soil Cement Mix Designs

After a review of the compaction test results presented in section 3.1, a total of six mix designs were used in this research as shown in Table 3.3. At the 3% and 6% cement contents, three different moisture contents (i.e. 6%, 10% (OWC) and 14%) were selected to examine “dry”, “optimum” and “wet” compaction conditions (see Table 3.3). A full summary of the dry preparations of soil and cement as well as tap water are shown in Table 3.3. Also shown in Table 3.3 are the calculated water to cement ratio of the six mixtures used.

Table 3.3: Soil cement mixes proportions used in this research

Mix ID	Soil A (g)				Soil B (g)	Cement * %	Cement (g)	Water * %	Water (ml)	W/C Ratio
	9.5-4.75 mm	4.75-1.2 mm	1.2-0.3 mm	0.3-0.08 mm	<0.08 mm					
A	1760	5760	4000	2080	2400	3%	480	6%	960	2
B	1760	5760	4000	2080	2400	3%	480	10%	1600	3.3
C	1760	5760	4000	2080	2400	3%	480	14%	2240	4.7
D	1760	5760	4000	2080	2400	6%	960	6%	960	1.0
E	1760	5760	4000	2080	2400	6%	960	10%	1600	1.7
F	1760	5760	4000	2080	2400	6%	960	14%	2240	2.3

Note:*expressed as a percentage of the dry soil.

3.3 Testing Procedure

After compaction, all specimens were cured for at least 28 days at room temperature. The first five days of curing were spent inside the mold covered with a damp cloth inside a plastic bag to minimize water evaporation. After these first five days, specimens were extruded from the mold and stored in a moist room for the remaining 23 days. Specimens were then subjected to the testing plan shown in Figure 3.3. A total of 60 specimens have been used in this research (6mixes × 10specimens). The following sections will describe in detail the various tests that were performed, as described in Figure 3.3.

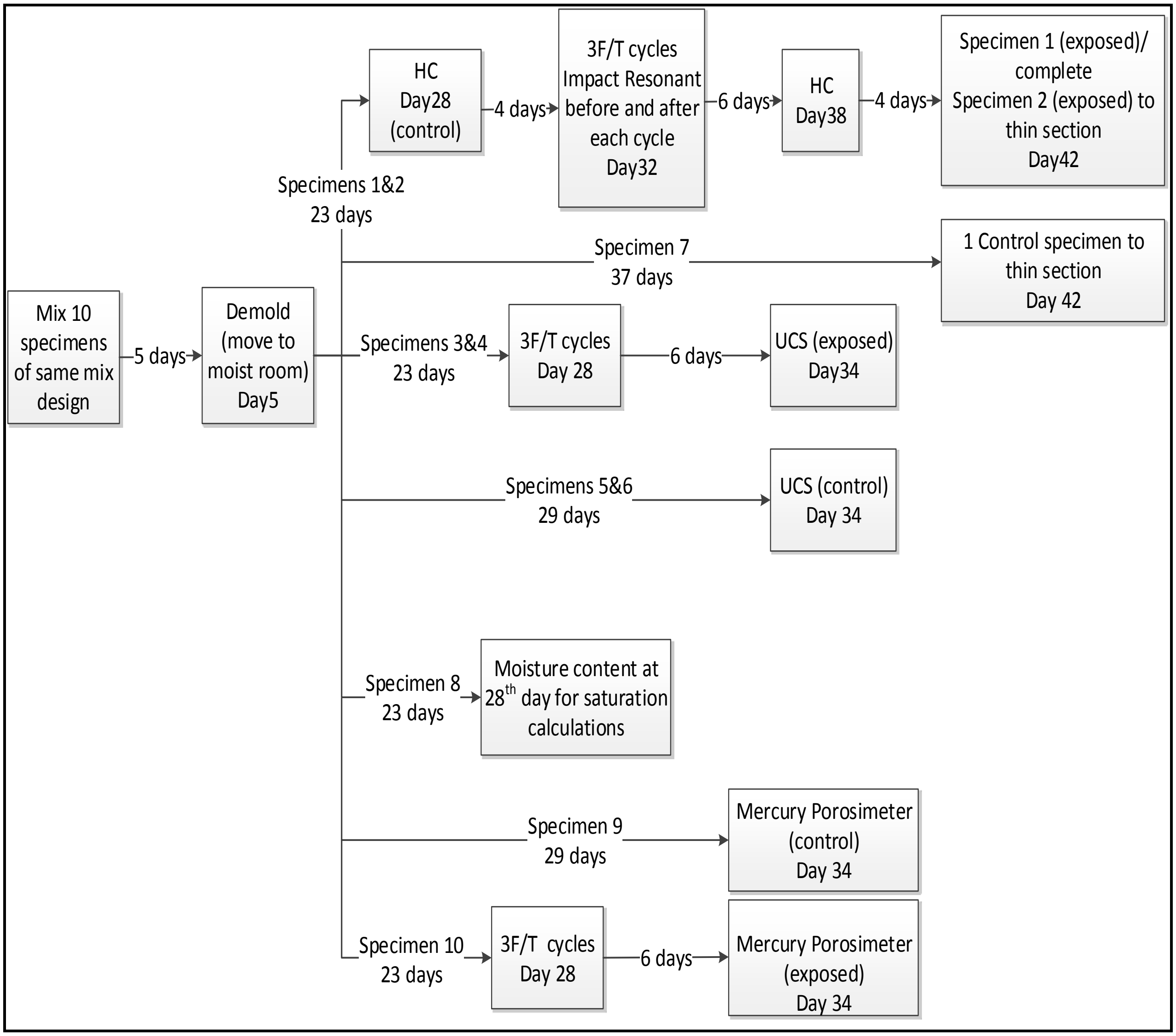


Figure 3.3: Testing procedure performed on 10 specimens of a given mix design

3.3.1 Hydraulic Conductivity Testing Procedure

Duplicate specimens (Specimens 1 and 2 in Figure 3.3) were used for hydraulic conductivity testing. After 28 days of curing, the duplicate specimens underwent flexible wall hydraulic conductivity testing according to ASTM-D5084 (2000) for 4 days including saturation, consolidation and permeation stages (i.e. prior to freeze/thaw). Back-pressure saturation of 524kPa was performed under an effective confining pressure of 35kPa, followed by consolidation under an effective confining pressure of 138kPa. The 138kPa was used for an attempt to keep sidewall leakage to a minimum. Permeation was then performed with de-aired water under a hydraulic gradient of approximately 30. Specimens 1 and 2 were then exposed to three freeze/thaw cycles before hydraulic conductivity test was again performed on both specimens. Each cycle of freeze/thaw consisted of three-dimensional freezing of the specimens for 24 hours at -10C° ($\pm$1) followed by thawing for another 24 hours at 22C° ($\pm$1) similar to freeze/thaw conditions used by Jamshidi (2014).

3.3.2 Resonant Frequency Testing Procedure

Resonant frequency testing covers measurement of the fundamental longitudinal resonant frequencies of soil cement cylinders. Resonant frequency is a non-destructive test widely used to predict the dynamic properties of cementitious materials. It has been used in this research to detect the damage of freeze/thaw conditioning as discussed by Jamshidi (2014). It was performed according to ASTM C215 (2008) on duplicate specimens (specimens 1 and 2 in Figure 3.3) before and after each freeze/thaw cycle.

To prepare specimens for resonant frequency testing, a thin square steel sheet of 1cm by 1cm dimensions and a thickness of 1mm was glued on the face of each specimen end. The reason to steel sheet allows a magnetic link of an accelerometer at one end and act as a stiff material at the other end of the specimen to receive the applied impact. Impact was applied using a steel ball of 9.5mm diameter attached to a plastic rod. The combined weight of the plastic rod and the steel ball is 5.3g. During the test, specimens 1 and 2 were placed on a sponge box of 22cm × 9cm × 7cm.

The response signals were obtained using a PCB model 353B02 accelerometer attached magnetically to the square steel tab on the other end of the specimen which transfers the signal to an amplifier and computer (Freedom NDT Data PC Platform, Olson Instrument Inc.) similar to the way used by Jamshidi (2014). The signal was processed by the computer's software using a Fast Fourier Transform to calculate the longitudinal resonant frequency. Five replicates of resonant frequency readings were taken and averaged for each specimen before and after each freeze/thaw cycle.

3.3.3 Unconfined Compressive Strength (UCS) Testing Procedure

Unconfined compressive strength testing was used to compare the strength of soil-cement before and after three freeze/thaw cycles. After 28 days of curing, duplicate specimens (3 and 4 in Figure 3.3) were exposed to 3 freeze/thaw cycles which took 6 days while duplicate specimens (control) (5 and 6 in Figure 3.3) were stored in the moist room for the same period to ensure testing was performed on the same day. At an age of 34 days, unconfined compressive strength testing was performed on the four specimens according to ASTM D1633 (2007). The displacement rate used to acquire UCS results was 0.5mm/min for all specimens. Specimens used have an average length of 116.5mm which is not a standard length for UCS test but are similar to all other specimens used in this thesis for the results to be comparable.

3.3.4 Thin Sections of Control and Freeze-Thaw Exposed Specimens

Thin sections were taken from exposed specimen 1 and control specimen 7 (see Figure 3.3) to be used under the optical microscope to examine the mechanisms of freeze/thaw damage as well as studying macro- and micro-structural changes of exposed soil cement specimens. Thin section samples were prepared by a specialized laboratory at the Geology and Earth Sciences Department of Dalhousie University. Specimens 1 and 7 were initially resin impregnated before thin sections were obtained from horizontal and vertical planes of those two specimens.

3.3.5 Saturation Measurement of Control Specimens

To obtain information on the degree of saturation of specimens not subjected to saturation in freeze-thaw testing, specimen 8 in Figure 3.3 was used to estimate the saturation value of each mix. Equation 3.1 was used to calculate saturation as follows:

$$S = \frac{(w \times G_s)}{\frac{\rho_w (G_s - \rho_d)}{\rho_d}} \qquad \text{Equation 3.1}$$

Where:

S: Degree of saturation of soil-cement specimen.

w: Moisture content of specimen after curing. Specimen weight was recorded after curing for 28 days, then over dried for 72 hours before being recorded again to calculate moisture content.

G_S: Specific gravity of the specimen measured by specialized laboratory at the Civil and Mineral Resources Engineering Department of Dalhousie University.

ρ_d: Dry density of soil-cement specimen after 28 days of curing.

3.3.6 Optical Microscope

Transmitted light optical microscopy was used on thin sections obtained from control and exposed soil-cement specimens to examine the mechanisms of freeze/thaw damage. To achieve this goal, microscopic pictures were taken at different magnifications (i.e. 0.25mm, 1.25mm, 2.5mm and 6.25mm) to study the micro- and macro-structural change due to exposure to three/freeze thaw cycles. Nikon Optiphot-Pol polarized light microscope was used in this thesis. Microscopic pictures were taken using a 12 megapixel digital scanning camera (Kontron ProgRes 3012) equipped with the microscope.

3.3.7 Mercury Intrusion Porosimetry (MIP)

Mercury Intrusion Porosimeter tests was performed on control specimen 9 and exposed specimen 10 from each soil-cement mix as shown in Figure 3.3. MIP test was used to examine the changes in porous structure of the soil-cement due to changes in moisture and cement contents as well as due to exposure to three freeze/thaw cycles. To achieve this goal, MIP test was performed on soil-cement specimens under different cement and water contents, before and after exposure to freeze/thaw cycles. MIP test was performed according to ASTM D4404 (2010). Pore Master GT (model number: PM33-7) was used to perform this testing method. After curing and exposure, compacted soil-cement specimens were oven dried for 48 hours before MIP testing was performed. MIP testing method consist of generally two stages, low pressure and high pressure. Each specimen went through those two stages and followed the same exact procedure. Changes in pore size distribution parameter due to changes in cement and water contents, and exposure to freeze/thaw cycles were examined by this testing method.

Chapter 4: Results

This chapter presents the results of soil-cement testing at cement contents of 3% and 6%; at optimum water content, 4% below optimum and 4% above optimum water content. In addition, results also include testing performed on soil-cement specimens exposed to 3 f/t cycles to examine the physical damage due to f/t conditioning. As discussed in Chapter 3, observations of physical damages was observed by testing hydraulic conductivity, unconfined compressive strength, resonant frequency and mercury intrusion porosimetry on the samples. Electron microscopic photographs were also taken to examine macro- and micro-structural states of soil-cement specimens at these different water and cement contents and under f/t conditioning.

4.1 Hydraulic Conductivity Results

Hydraulic conductivity results of the control soil-cement specimens (i.e. without freezing) for each mix design are presented in Table 4.1. As discussed in Chapter 3, flexible-wall hydraulic conductivity testing was performed on two replicate specimens to show specimen variation. Table 4.1 presents the results for each specimen as well as the average of both tests. Figure 4.1 plots the average hydraulic conductivity as well as the range (denoted by error bar) for control specimens at 3% cement content and 6% cement content for each water content of the mixes. As discussed in Chapter 3, the moisture contents of the tested specimens represent optimum water conditions (10 %), dry of optimum water conditions (6%) and wet of optimum water conditions (14%). Data points at 10% and 14% water content in Figure 4.1 has no visible error bars which means the error in the two samples is relatively small.

Table 4.1 Hydraulic conductivity results under control conditions

Mixture ID	Cement (%)	Water (%)	Water/Cement Ratio	Hydraulic Conductivity, K (m/sec)		
				Specimen 1	Specimen 2	Average
A	3	6	2.00	9.6E-08	4.3E-07	2.6E-07
B	3	10	3.33	2.1E-08	1.7E-08	1.9E-08
C	3	14	4.67	4.5E-08	4.4E-08	4.4E-08
D	6	6	1.00	1.1E-07	5.4E-08	8.2E-08
E	6	10	1.67	3.1E-10	2.4E-10	2.7E-10
F	6	14	2.33	3.7E-10	4.1E-10	3.9E-10

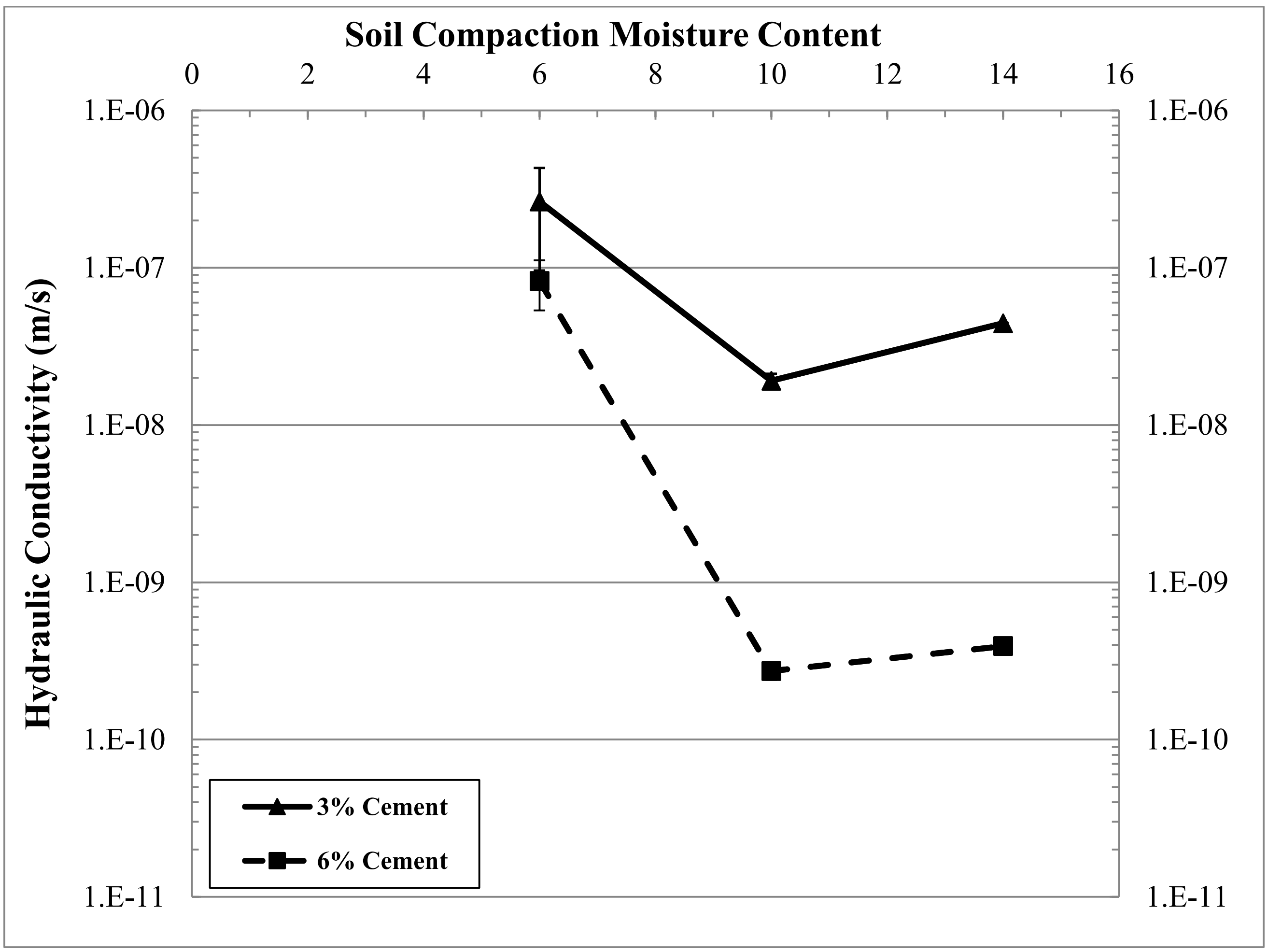


Figure 4.1 Variation of hydraulic conductivity with soil compaction moisture content

4.1.1 Effect of Soil Compaction Water Content and Cement Content on Hydraulic Conductivity

Testing results in Table 4.1 and Figure 4.1 show that compaction water content plays an important role in the hydraulic conductivity of the soil cement specimens. Examining either the 3% cement or 6% cement content specimens in Figure 4.1 shows that optimum water content provided the minimum hydraulic conductivity of the three water contents examined. Under dry of optimum conditions, the highest hydraulic conductivity was observed which is likely due to a combination of inadequate water necessary for cement hydration as well as inadequate water for lubrication and subsequent densification during compaction leading to creation of more voids in the specimen (i.e. more space for water to go through). At wet of optimum water condition conditions and 3% cement, the average hydraulic conductivity is 132% higher than that at optimum water condition which is likely due to the existence of additional voids due to excessive water. However, this hydraulic conductivity at 3% cement content is 83.2% lower than that at dry of optimum condition which could be due to sufficient water for cement hydration. Similar trends can be seen for wet of optimum condition and 6% cement, where hydraulic conductivity is 44% higher than that at optimum water content. Jamshidi (2014) saw that wet of optimum water content gave the lowest hydraulic conductivity which is likely due to the use of high cement content that requires more water for hydration process.

Figure 4.1 shows that at a given water content, increasing the cement content leads to a decrease in hydraulic conductivity which agrees with soil-cement studies done by Hammad (2013), Felt & Abrams (1957) and ACI (1990). The addition of cement fills more voids in the soil-cement mix leaving less space for water flow. At dry of optimum conditions, adding 3% cement resulted in minimal reduction of hydraulic conductivity when compared to optimum and wet conditions. The reason may be that the water at dry conditions is not sufficient for all the extra cement to hydrate, resulting in less improvement in hydraulic conductivity.

4.1.2 Changes in Hydraulic Conductivity Due to Three Freeze/Thaw Cycles

After performing hydraulic conductivity tests on control condition specimens (i.e. without freeze thaw), specimens were exposed to three f/t cycles before repeating hydraulic conductivity tests, as discussed in Chapter 3. Table 4.2 and Figures 4.2 and 4.3 compare hydraulic conductivity results of control and f/t exposed conditions. As previously discussed for control specimens, Figures 4.2 and 4.3 display the average hydraulic conductivity as well as the range (i.e. error bars).

Table 4.2 Hydraulic conductivity results comparison between control and f/t exposed conditions

Mixture ID	Control K (m/sec)			Exposed K (m/sec)			$\frac{\text{Exposed K}}{\text{Control K}}$
	Specimen 1	Specimen 2	Average	Specimen 1	Specimen 2	Average	
A	9.6E-08	4.3E-07	2.6E-07	6.5E-07	6.9E-07	6.7E-07	2.6
B	2.1E-08	1.7E-08	1.9E-08	8.5E-08	8.5E-08	8.5E-08	4.5
C	4.4E-08	4.4E-08	4.4E-08	1.2E-07	1.4E-07	1.3E-07	3.0
D	1.1E-07	5.4E-08	8.2E-08	1.3E-07	1.2E-07	1.3E-07	1.6
E	3.1E-10	2.4E-10	2.7E-10	7.7E-10	6.5E-10	7.1E-10	2.6
F	3.7E-10	4.1E-10	3.9E-10	8.7E-10	1.1E-09	9.8E-10	2.5

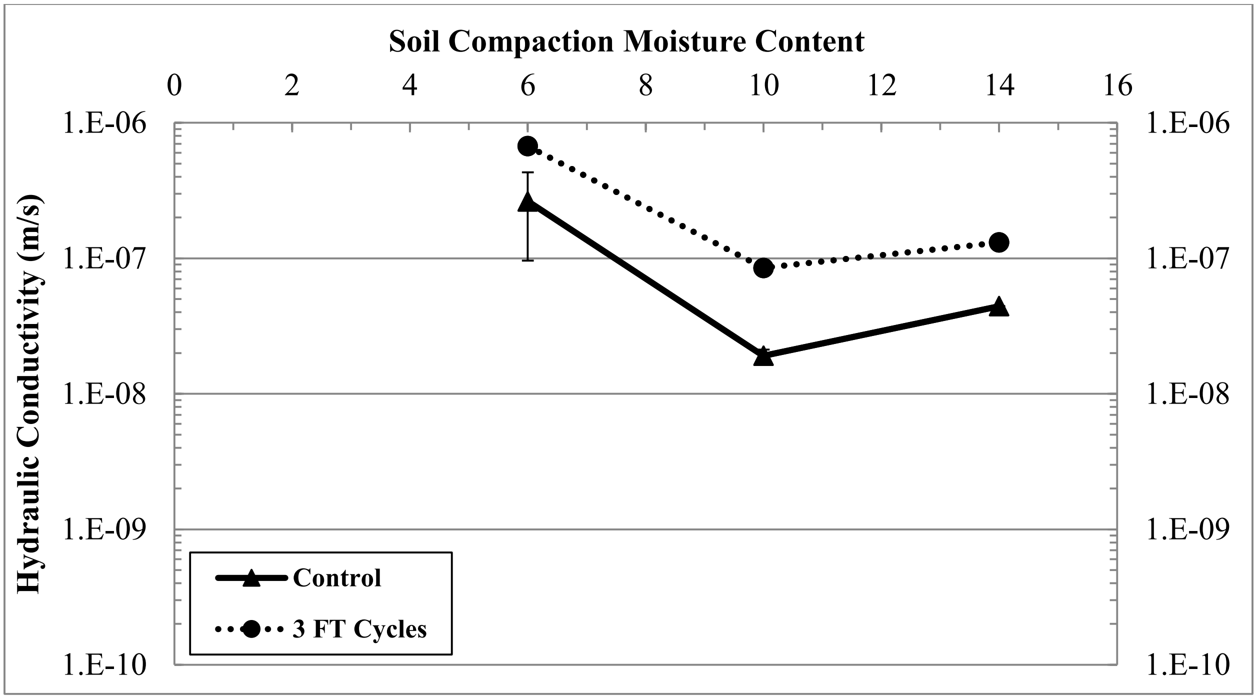


Figure 4.2 Variation of hydraulic conductivity with soil compaction moisture content before and after freeze/thaw exposure at 3% cement

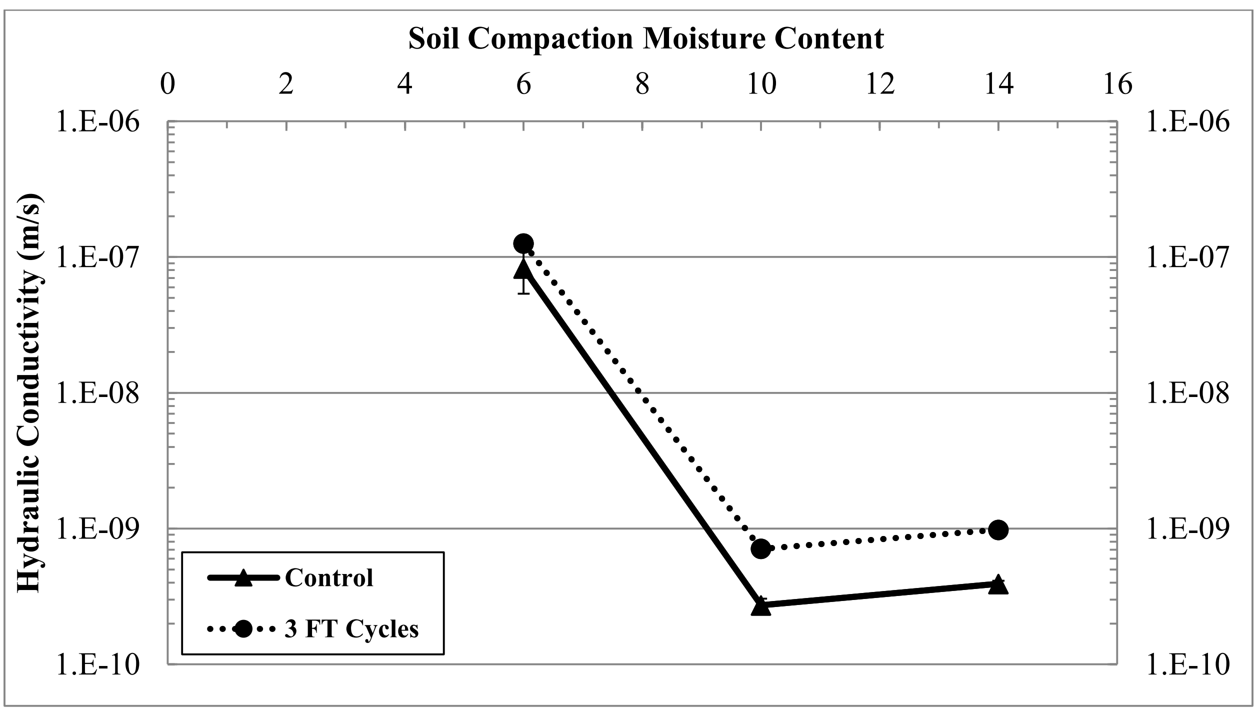


Figure 4.3 Variation of hydraulic conductivity with soil compaction moisture content before and after freeze thaw exposure at 6% cement

Figures 4.2 and 4.3 show an increase in hydraulic conductivity for both the 3% and 6% cement contents when soil-cement specimens are subjected to three cycles of freeze and thaw. Figures 4.2 and 4.3 also show similar hydraulic conductivity trends with compaction water content for 3% and 6% cement specimens as those developed before f/t exposure. Figures 4.2 and 4.3 are re-plotted in Figure 4.4 to show hydraulic conductivity ratio (Exposed K/Control K) with respect to moisture content. Hydraulic damage due to f/t exposure appeared to be consistently less for 6% cement specimens than 3% cement specimens which is likely due to the existence of more cement that provides better resistance to internal stresses created by f/t exposure. Hydraulic damage was less at dry of optimum conditions (highest hydraulic conductivity) than at optimum and wet conditions. The reason for this could be that at dry of optimum conditions, more pores space exists for water to expand when frozen. And hence, less internal stresses created in the specimens. It should be noted that less than an order of magnitude damage resulted for all compaction water content conditions for the soil-cement examined.

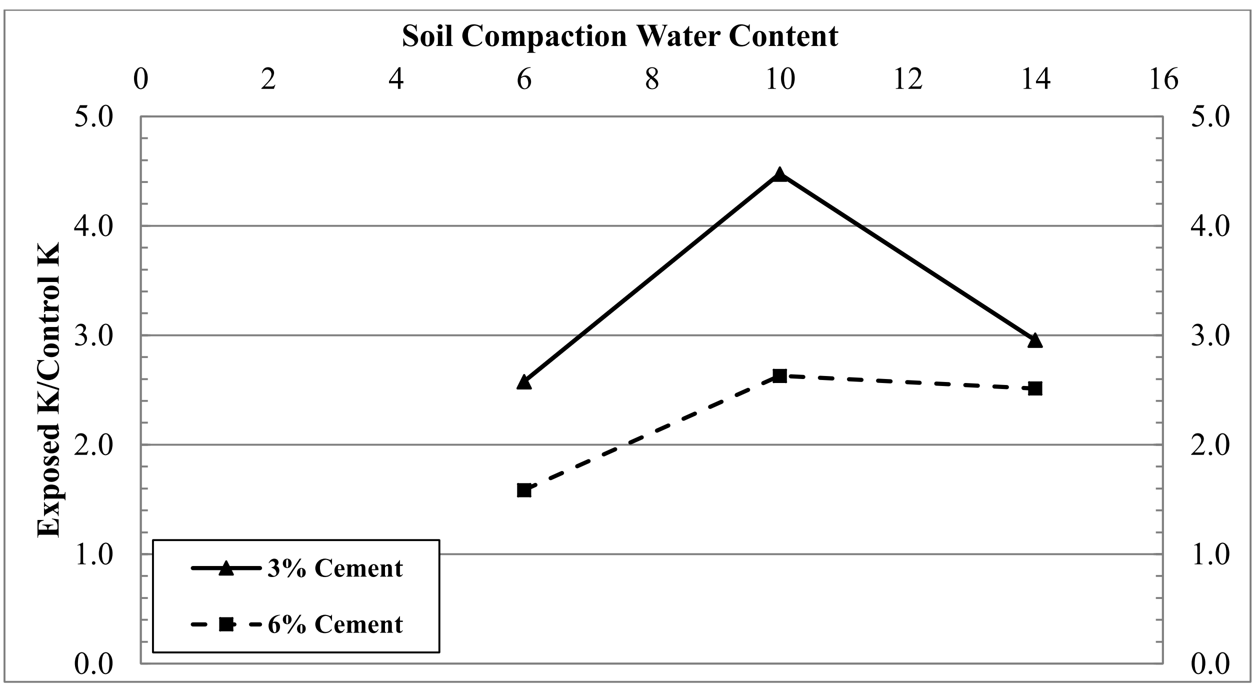


Figure 4.4 Variation of hydraulic conductivity ratio with soil compaction moisture content

Jamshidi (2014) also used a similar silty sand soil but with higher cement contents and 12 f/t cycles instead of three. Jamshidi (2014) testing results showed more hydraulic damage due to f/t exposure. The first reason could be that at higher cement contents, less space is provided for the water to expand when freezes. Therefore, more stresses could be created. Secondly, exposure to more f/t cycles leads to creation of more stresses. However, Jamshidi (2014) also noticed most f/t damage occurred after 1-3 cycles of f/t. Guney et al. (2006) examined soil-cement compacted at optimum water content with 5% cement. After exposure to eight f/t cycles, results showed an increase of 20 fold in hydraulic conductivity.

4.2 Unconfined Compressive Strength Results

Unconfined compressive strength (UCS) results of the control soil-cement specimens are presented in Table 4.3 and Figures 4.5 and 4.6. Similar to hydraulic conductivity testing, UCS testing was performed on control specimens at dry of optimum water content, optimum water and wet of optimum water content compaction conditions after a curing time of 28 days. UCS testing was performed at cement contents of 3% and 6%, as discussed previously. Figure 4.5 plots the average and range of unconfined compressive strength test for control specimens at 3% and 6% cement contents relative to soil compaction moisture content.

Table 4.3 UCS results at control condition

Mixture ID	Cement (%)	Water (%)	Water/Cement Ratio	Degree of Saturation	Control UCS (KPa)		
					Specimen 1	Specimen 2	Average
A	3	6	2.00	0.46	512	622	567
B	3	10	3.33	0.81	740	894	817
C	3	14	4.67	0.89	102	92	97
D	6	6	1.00	0.42	2320	1124	2222
E	6	10	1.67	0.79	3178	3131	3155
F	6	14	2.33	0.91	1479	1489	1484

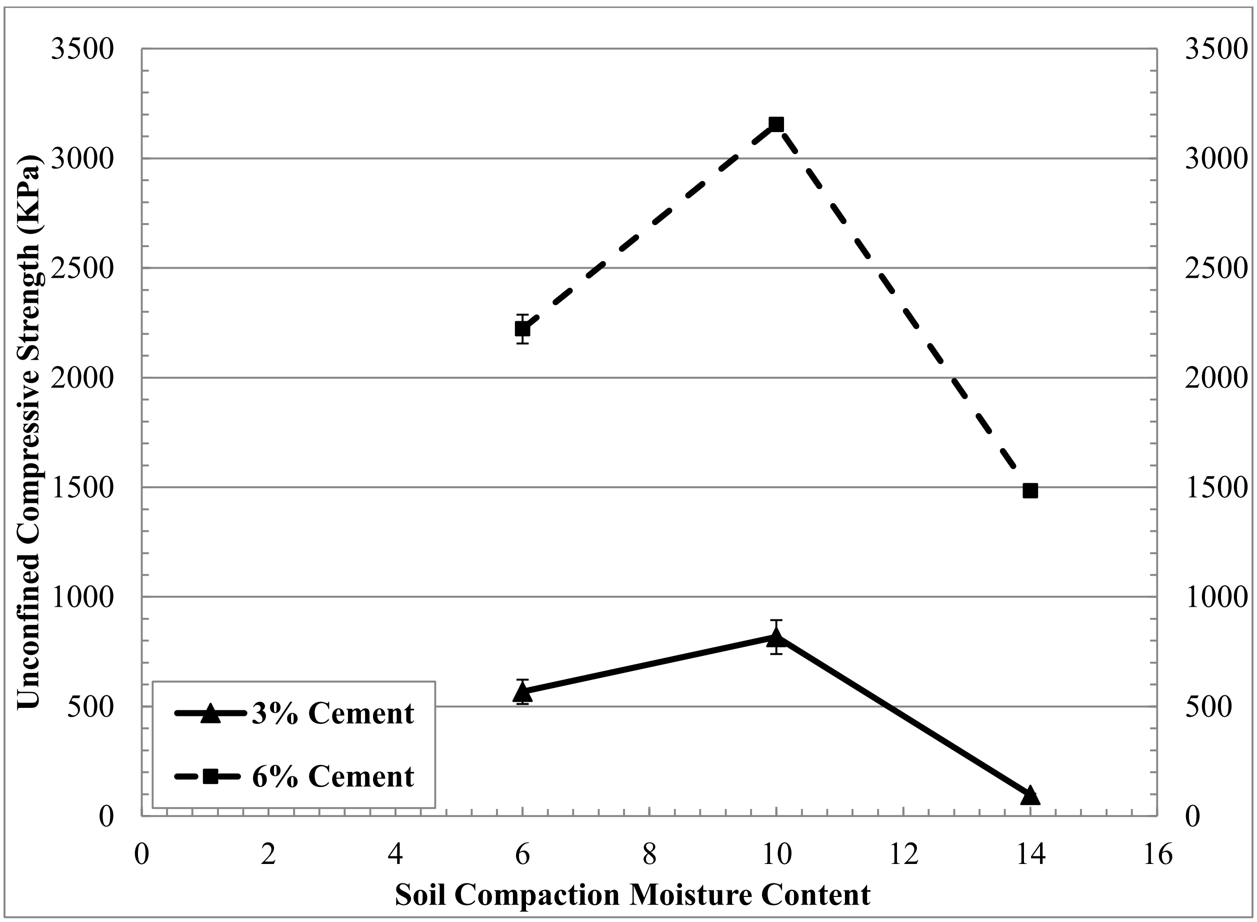


Figure 4.5 Variation of unconfined compressive strength with soil compaction moisture content

4.2.1 Effect of Soil Compaction Water Content and Cement Content on Unconfined Compressive Strength (UCS)

Figure 4.5 shows how UCS results follow the same trends as their moisture-density curves for each cement content. For a given cement content, reduction of UCS values at dry of optimum water content conditions are likely due to limited water for cement hydration as well as inadequate lubrication of soil particles during compaction. Reduction of UCS values at wet conditions is likely due to excessive water content in the soil-cement mix that could lead to bleeding in the paste and consequently create high porous areas and lowers the density. Jamshidi (2014) performed UCS tests on similar type of soil (silty sand) at cement contents of 10%, different water contents and different curing ages. Jamshidi (2014) showed a similar trend in UCS with the maximum UCS values occurring at compaction moisture contents close to optimum, as well as showing that the UCS values tend to decrease as additional water is added during compaction.

Figure 4.5 shows a significant increase in UCS values by increasing cement contents from 3% to 6%. Adding extra cement resulted in improved bonding and link between soil particles due to the chemical reaction between cement and water which increases the resistance of the soil-cement specimen under an applied compressive load. The percent increase in UCS values due to increased cement content presented in Figure 4.4 was more at wet conditions (1424%) than at dry conditions (290%) which is likely due to the existence of more water for cement hydration at wet conditions. At dry conditions, the water content was probably not sufficient for all the cement to react which resulted in less improvement in UCS values.

4.2.2 Changes in Unconfined Compressive Strength (UCS) Due to Three Freeze/Thaw Cycles

To further examine the physical damage of soil-cement mixes after three f/t cycles, UCS testing was repeated on specimens exposed to three f/t cycles. Table 4.4 shows comparison of UCS results between control and exposed condition.

Table 4.4 UCS results comparison between control and exposed conditions

Mixture ID	Control UCS (KPa)			Exposed UCS (KPa)			$\frac{\text{Exposed UCS}}{\text{Control UCS}}$
	Specimen 1	Specimen 2	Average	Specimen 1	Specimen 2	Average	
A	512	622	567	422	376	399	0.7
B	740	894	817	476	549	513	0.6
C	102	92	97	74	74	74	0.8
D	2320	2124	2222	1779	1650	1714	0.8
E	3178	3131	3155	2860	2743	2801	0.9
F	1479	1489	1484	1415	1396	1406	0.9

Figures 4.6 and 4.7 show the difference in UCS values between control and exposed specimens at 3% and 6% cement content respectively. It also showed that at all cement and water contents, UCS values decreased after exposure to three f/t cycles. Although, residual strengths of 6% cement specimens are still high. Degree of saturation (S) was calculated to examine any possible relation to strength performance and f/t damage. There was no relation observed.

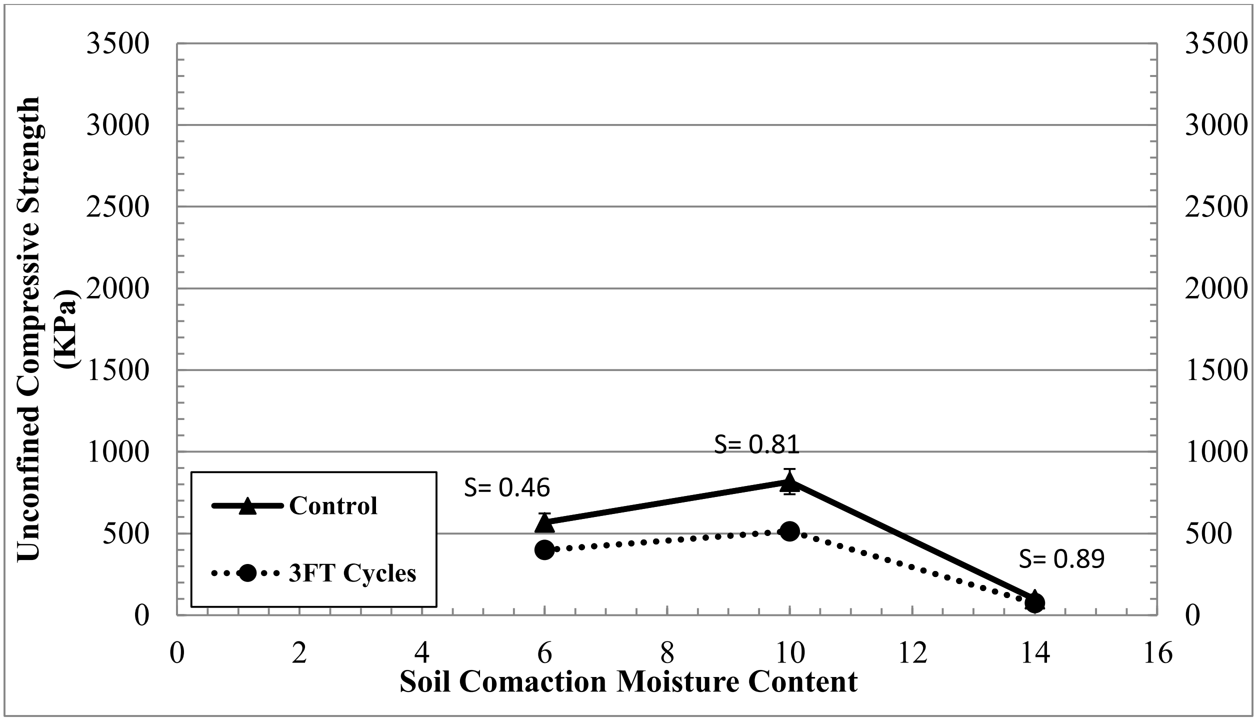


Figure 4.6 Variation of unconfined compressive strength with degree of saturation before and after freeze thaw at 3% cement

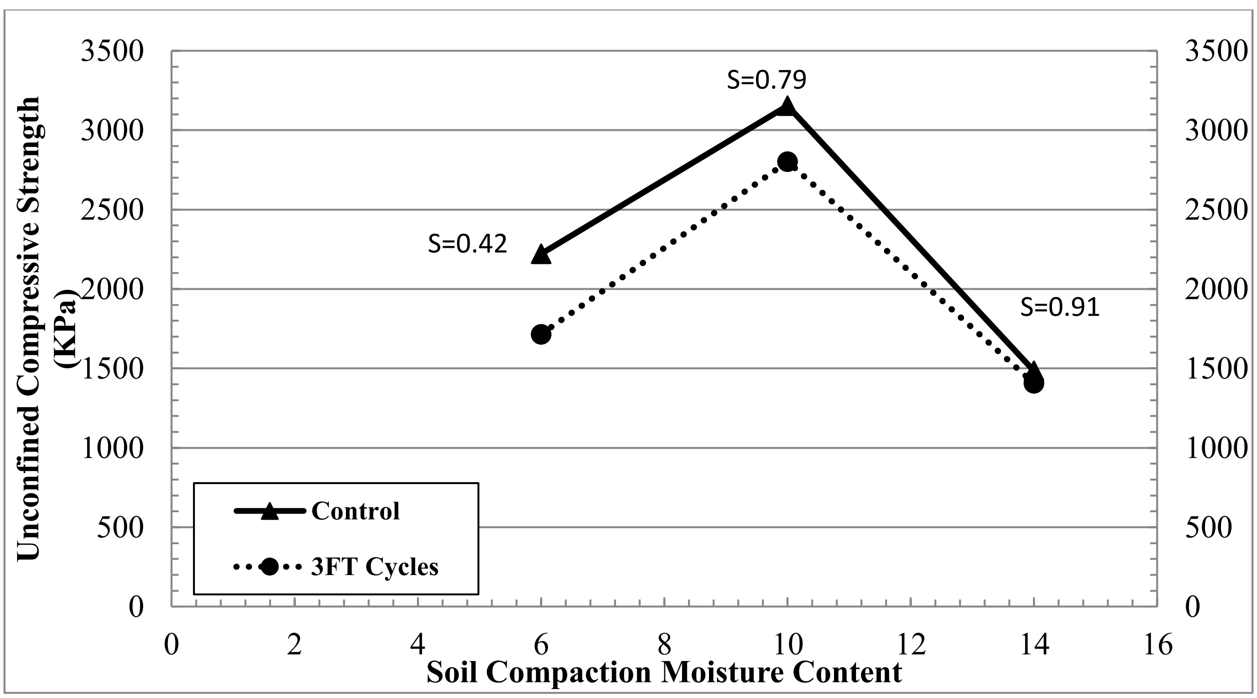


Figure 4.7 Variation of unconfined compressive strength with soil compaction moisture content before and after three f/t cycles at 6% cement

Figures 4.6 and 4.7 are re-plotted in Figure 4.8 to show the unconfined compressive strength ratio (Exposed UCS/Control UCS) with respect to soil compaction moisture content. Figure 4.8 shows more damage at dry and optimum conditions than at wet conditions which is likely due to the ductile behavior of wet specimens.

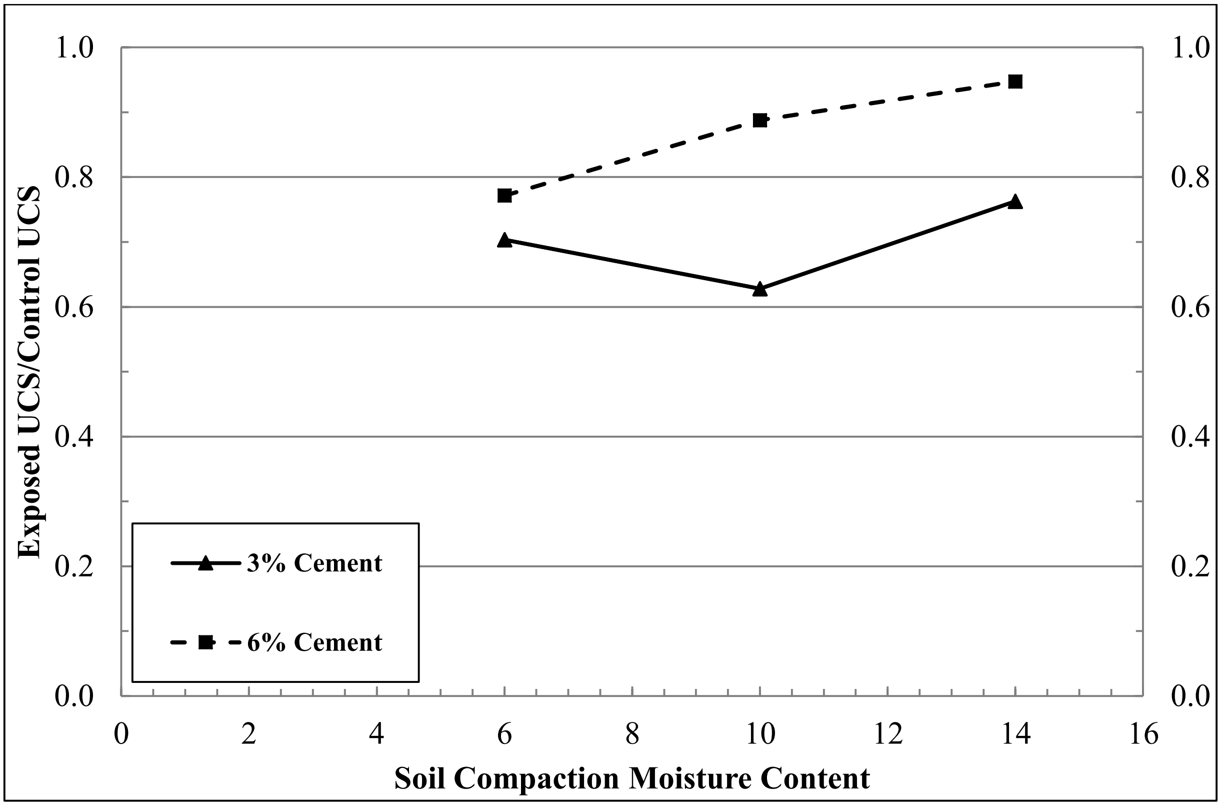


Figure 4.8 Variation of unconfined compressive strength ratio with soil compaction moisture content after three f/t cycles

4.3 Resonant Frequency Results

Resonant frequency (RF) testing was used as a technique to monitor the macro-porosity changes at each f/t cycle. As shown by Jamshidi (2014), it is also a useful technique to track damage of soil-cement specimens during freeze f/t exposure. Figures 4.9 and 4.10 show the variance of resonant frequency over three f/t cycles at 3% and 6% cement contents respectively.

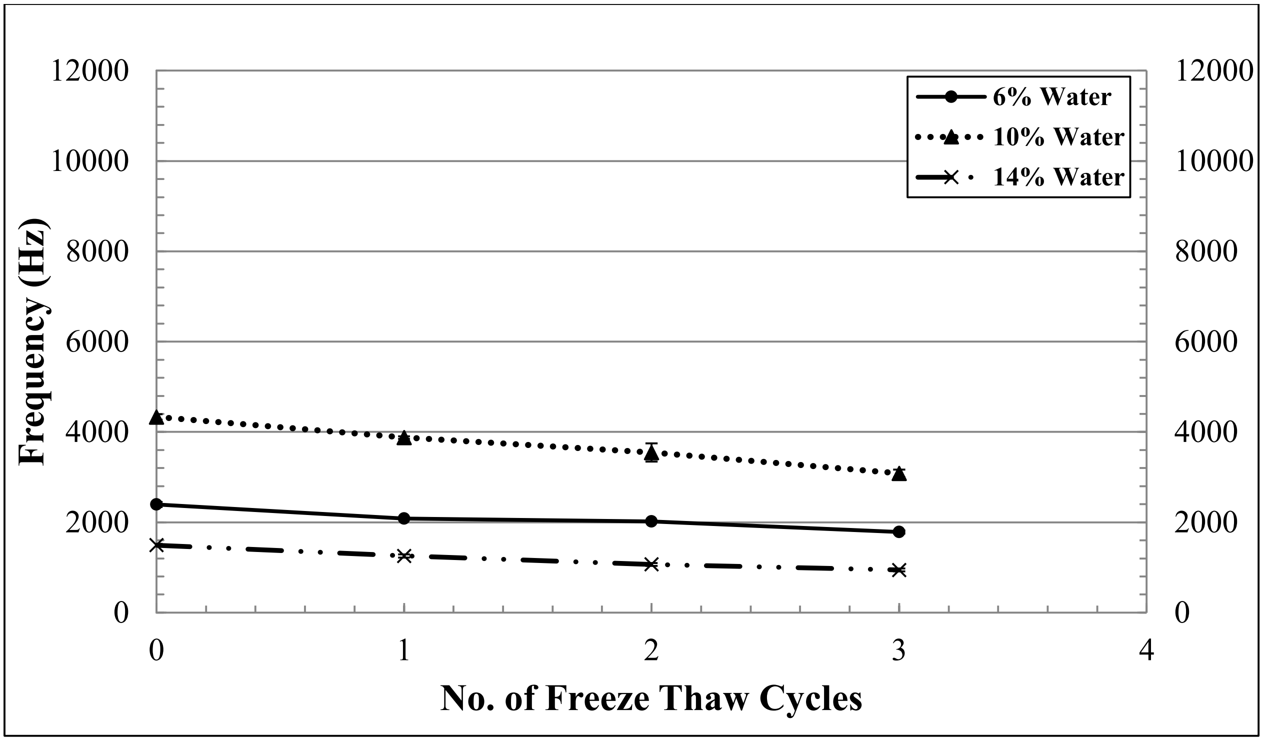


Figure 4.9 Resonant frequency variance over three f/t cycles at 3% cement content

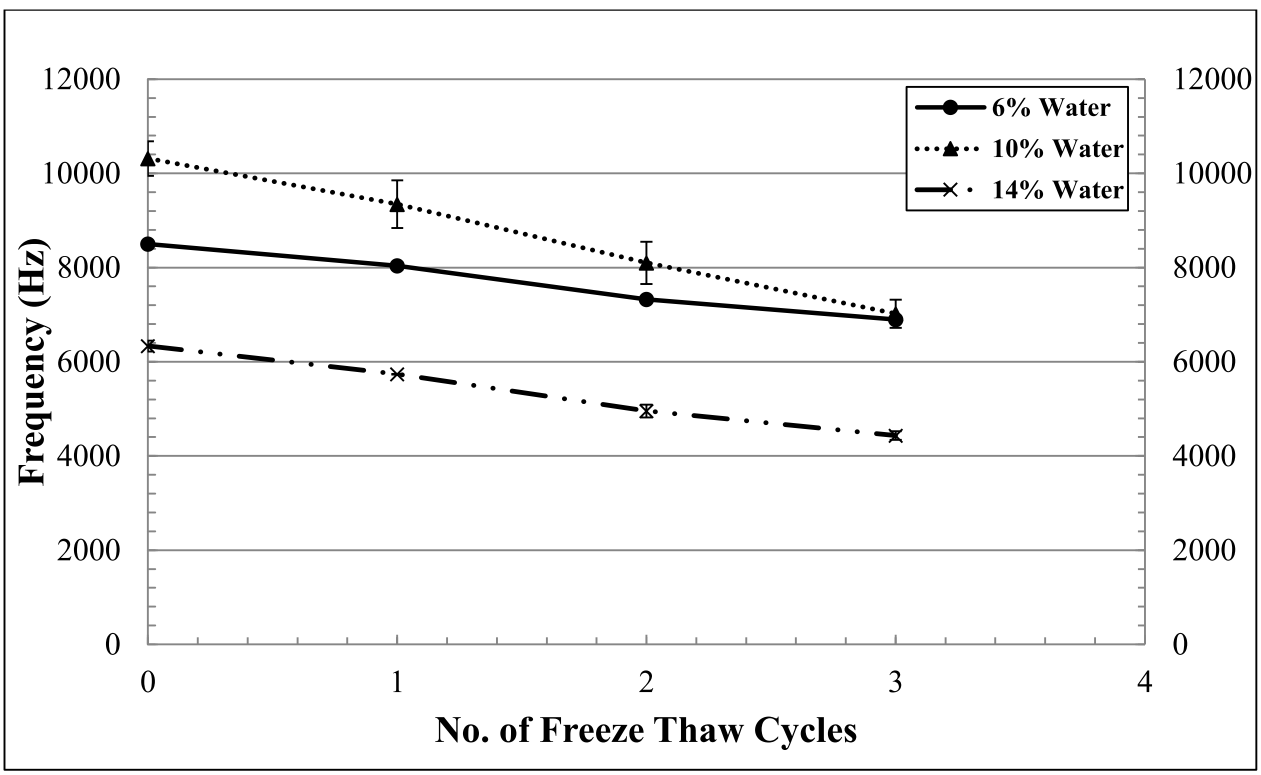


Figure 4.10 Resonant frequency variance over three f/t cycles at 6% cement content

Figures 4.9 and 4.10 show that for both the 3% and 6% cement contents, specimens at 10% moisture content had the highest RF readings while 6% moisture content specimens showed second highest RF readings and the 14% moisture content specimens showed the lowest RF reading. These specimens compacted at optimum water contents exhibited the highest UCS values at each cement content prior to freeze/thaw. A second observation is that the RF readings are considerably higher at 6% cement content compared to readings at 3% cement content (i.e. similar to UCS values). Also RF readings show a decrease with increased number of f/t cycles which indicates a progressive increase in damage. Figures 4.9 and 4.10 also show that most change in RF with increase in f/t cycles occurred at optimum water content (i.e. compared to wet and dry of optimum conditions) which is similar to the observation in the hydraulic conductivity results. Figure 4.11 shows generally that more hydraulic damage lead to more resonant frequency damage both after three f/t cycles.

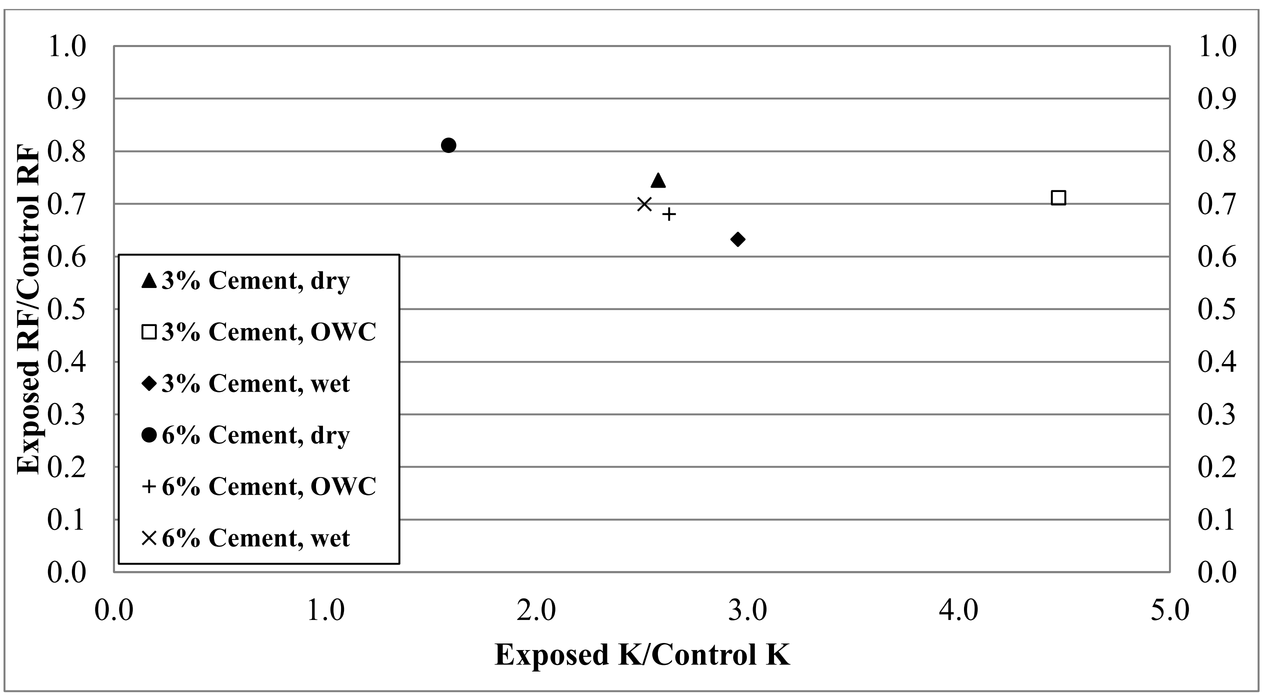


Figure 4.11 Variation of resonant frequency ratio compared to the hydraulic conductivity ratio after three f/t cycles

Jamshidi (2014) compared resonant frequency ratio to hydraulic conductivity ratio both after 12 f/t cycles and showed a similar trend to Figure 4.11. Figure 4.12 shows that resonant frequency ratio generally followed the same trend as unconfined compressive strength ratio.

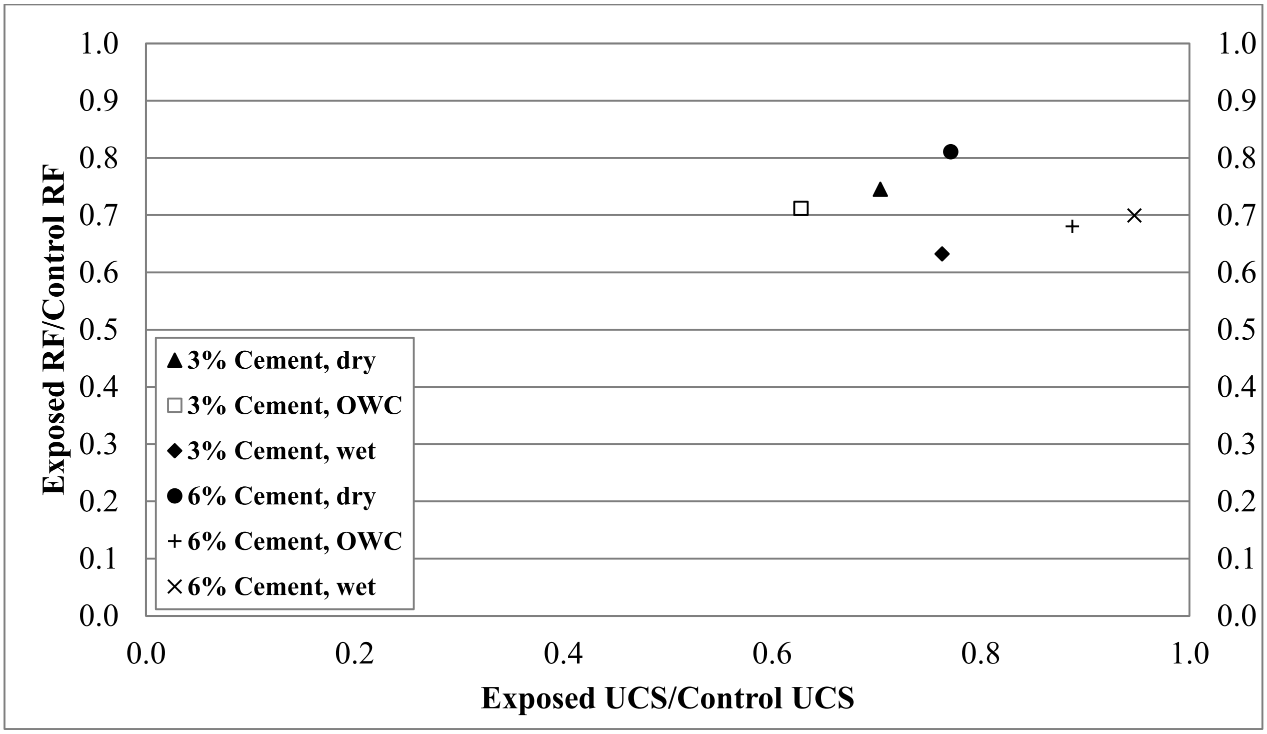


Figure 4.12 Variation of resonant frequency ratio compared to the unconfined compressive strength ratio after three f/t cycles

4.4 Optical Microscopy

An optical microscope was used at different magnifications (see Figure 4.13) in an attempt to investigate the macro and micro-structural changes of the soil-cement specimens due to exposure to three f/t cycles. Figure 4.13 shows an example for the available magnifications in the optical microscope used in this research. The blue color in the microscopic photograph represents the pore structure that was filled with the resin used during preparation of thin sections. A 0.25mm magnification did not give a clear picture.

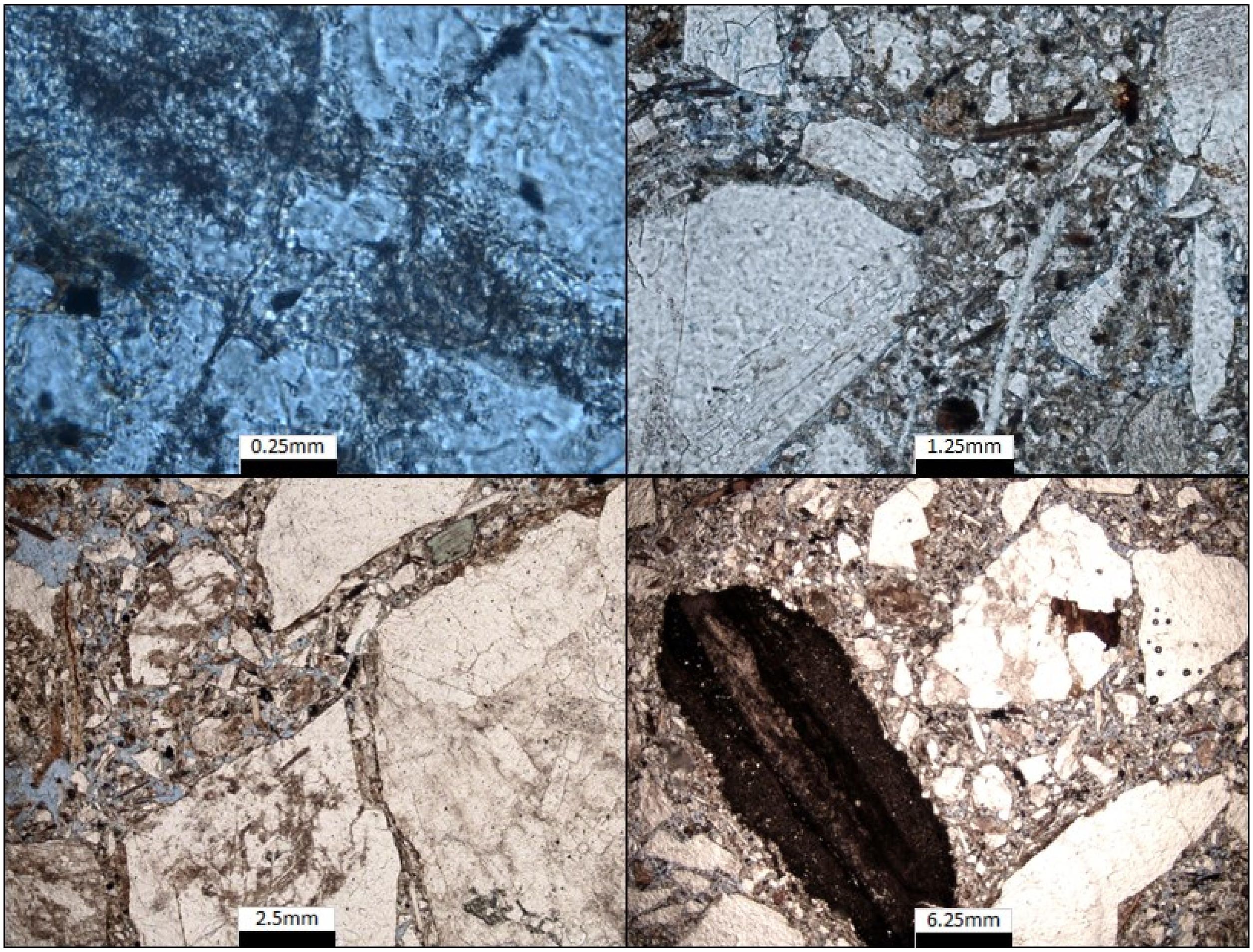


Figure 4.13 Optical micrographs at different magnifications for specimen mixed at 3% cement content and dry of optimum water condition

Figures 4.14 and 4.15 show differences in structure of the pores under various moisture contents at 3% and 6% cement content respectively. Mixes A, B and C were all mixed at 3% cement content while D, E and F were all mixed at 6% cement content as discussed in Chapter 3. In figure 4.14, mix A (dry of optimum condition) and mix C (wet of optimum condition) tended to show more visible presence of the blue resin than mix B (optimum moisture content) due to the existence of more voids in the soil at wet and dry conditions. Similar observations was made of the 6% cement content samples in Figure 4.15. Those observations agree with higher hydraulic conductivity results at wet and dry of optimum conditions when compared to optimum water condition as discussed in section 4.1. More voids at dry and wet conditions represent less density which could verify the observation of low UCS compared to optimum condition where maximum density and maximum UCS achieved. The right side of Figures 4.14 and 4.15 represent the soil-cement mixes exposed to three f/t cycles. Exposed specimens generally showed more blue resin concentration than control specimens which could be due to the expansion of water inside the pores during freezing. In exposed specimens, blue resin generally appeared between the aggregates and cement paste which represents the cracks that occurred when excess pore water pressure due to freezing exceeded the bursting strength of the material. These optical observations verify the damage observed in hydraulic and strength performance. On the right side of Figures 4.14 and 4.15, blue resin is more visible at optimum and wet of optimum water conditions which suggests more damage than at dry of optimum water condition which tends to agree with hydraulic conductivity results. Figure 4.15 generally show less blue resin concentration than Figure 4.14 due to existence of more cement that fills more voids and improves resistance to f/t exposure which is likely the reason behind improved strength and hydraulic performance.

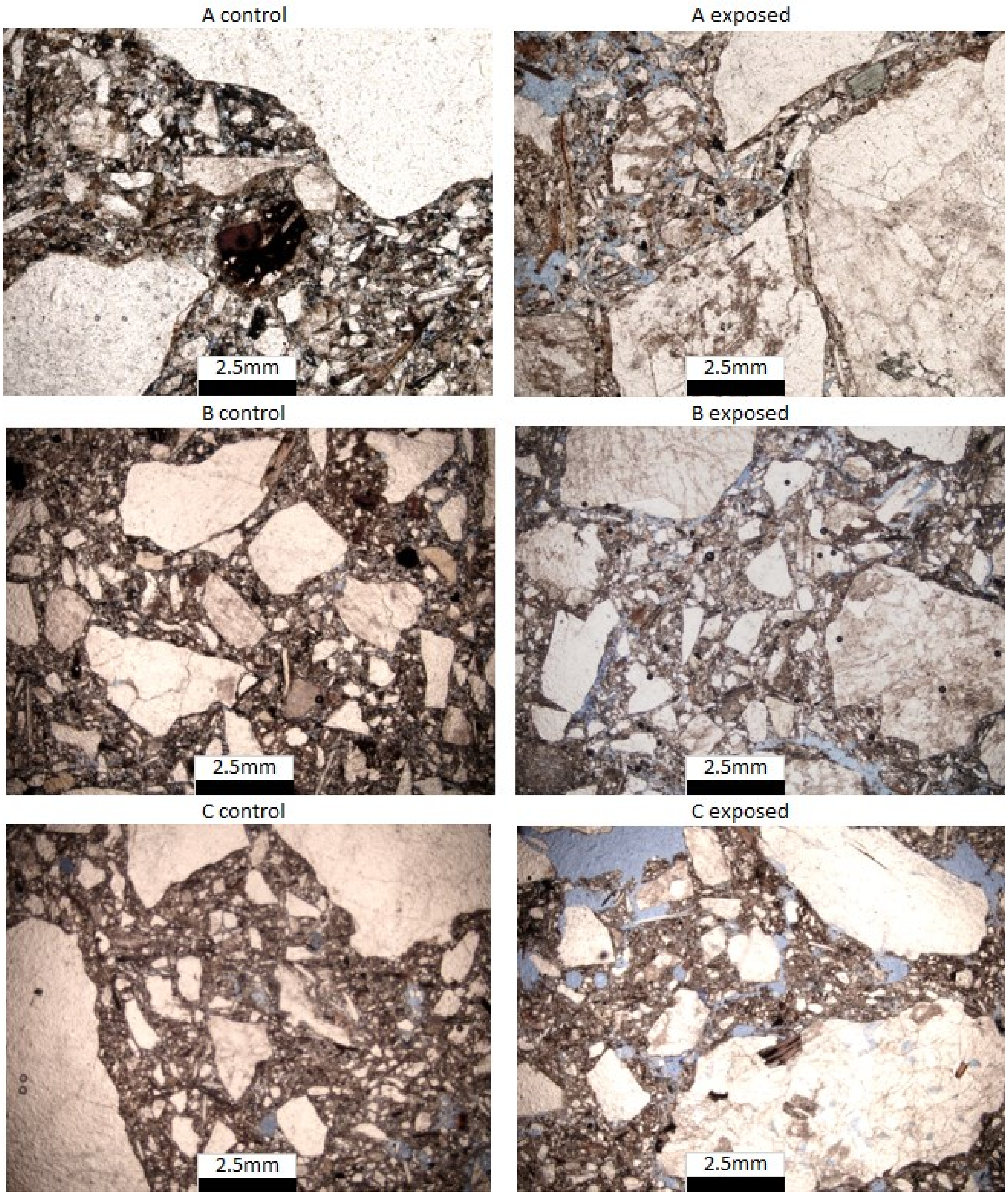


Figure 4.14 Typical micrographs from vertical planes of specimens at low cement content of 3% and under various moisture contents: A (dry), B (optimum), C (wet)

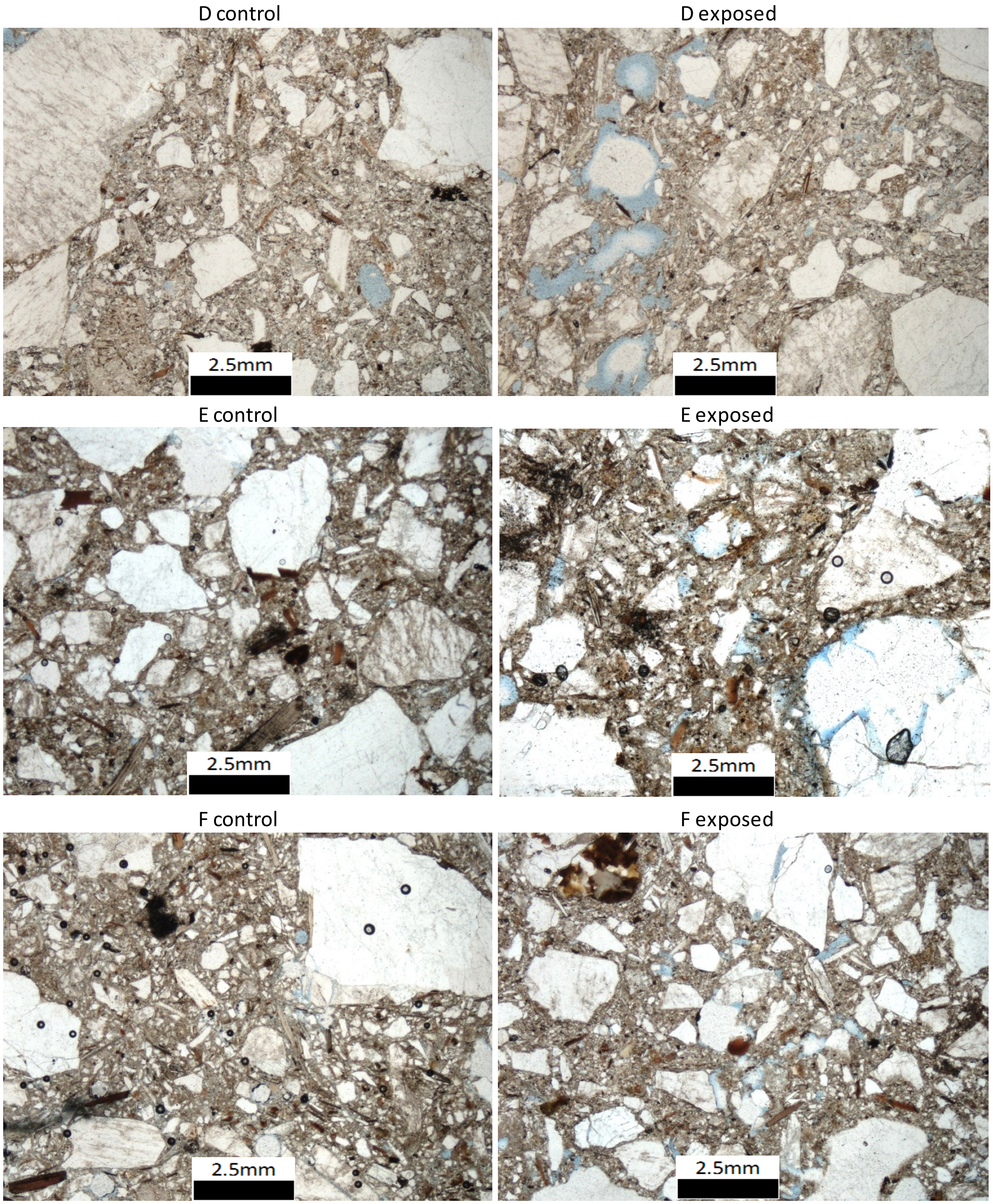
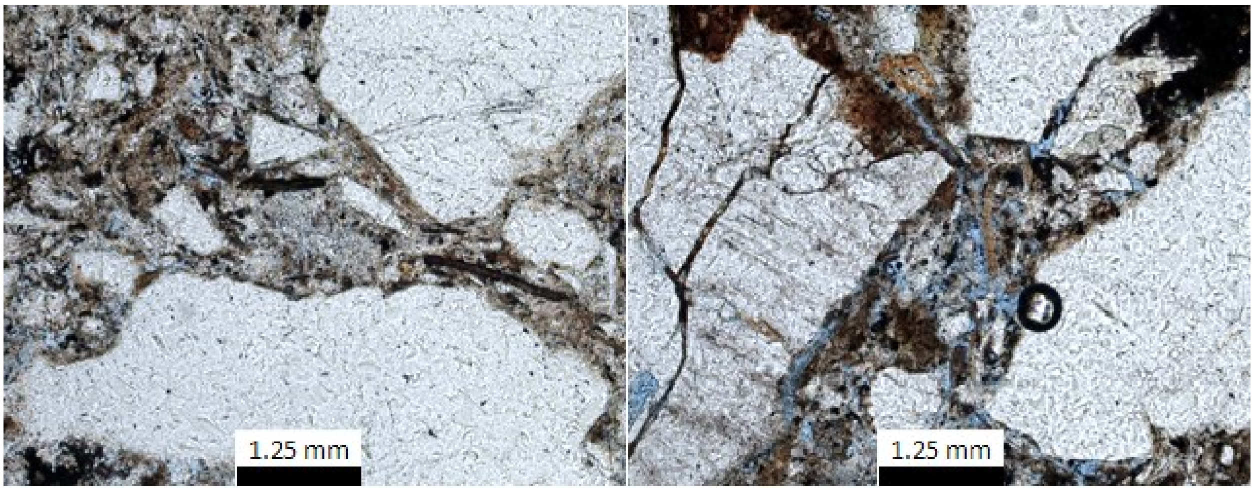


Figure 4.15 Typical micrographs from vertical planes of specimens at 6% cement content and under various moisture contents: A (dry), B (optimum), C (wet)

At higher magnification for control and exposed specimens from mixture A, it was noticed that cracks occurred not only in the soil-cement matrix but also on the surface of the soil particles due to f/t exposure (see Figure 4.16).



Control f/t exposed

Figure 4.16 Matrix damage and cracks in soil-cement specimen A (3% cement, 6% water) due to exposure to three f/t cycles

At times, exposure to f/t conditioning resulted in matrix damage in the soil-cement specimen as shown in Figure 4.17. Figure 4.17 also points to the cracks that occurred between aggregates and cement paste due to damage of three f/t cycles.

To examine the presence/absence of frost lenses in the soil cement samples, the entire thin sections were compared before and after freeze thaw using a light table and front light source, as was done by Othman and Benson (1992) for compacted clays. An example is shown for the 3% cement and 14% water sample in Figure 4.18. The continuous blue color might represent ice lens formation due to f/t exposure. Ice lens formation is observed at the optimum and wet of optimum conditions of 3% cement which is likely due to the existence of more pores and sufficient water. The dry of optimum condition of 3% cement and the dry, wet and optimum water conditions of 6% cement have not shown ice lens formation. Additional photos are provided in Appendix A.

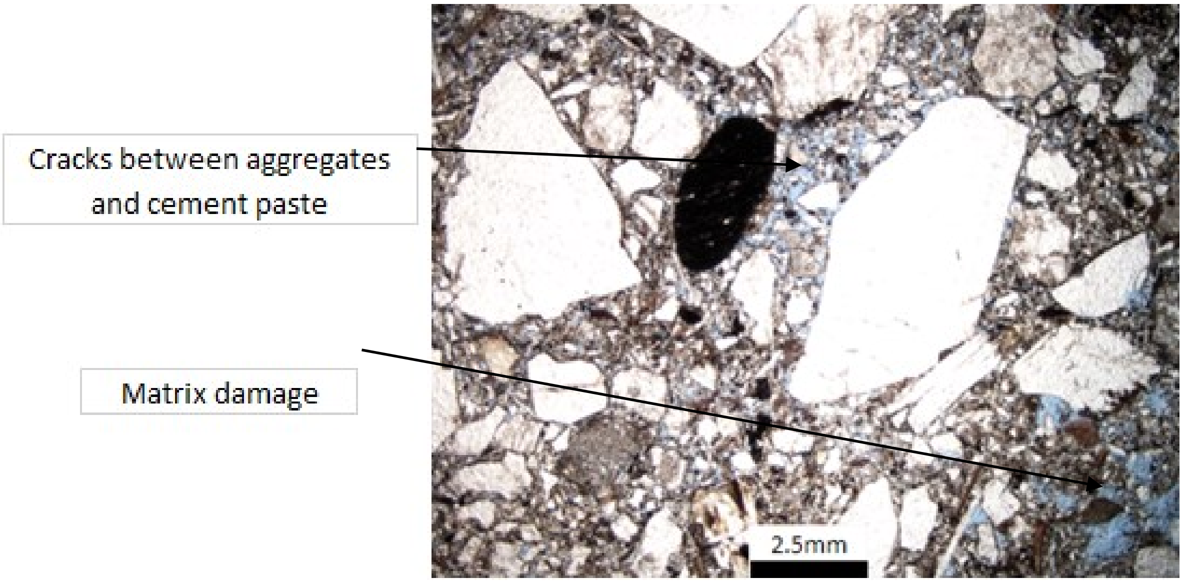


Figure 4.17 Cracks and matrix damage in soil-cement specimen A1 due to exposure to three f/t cycle

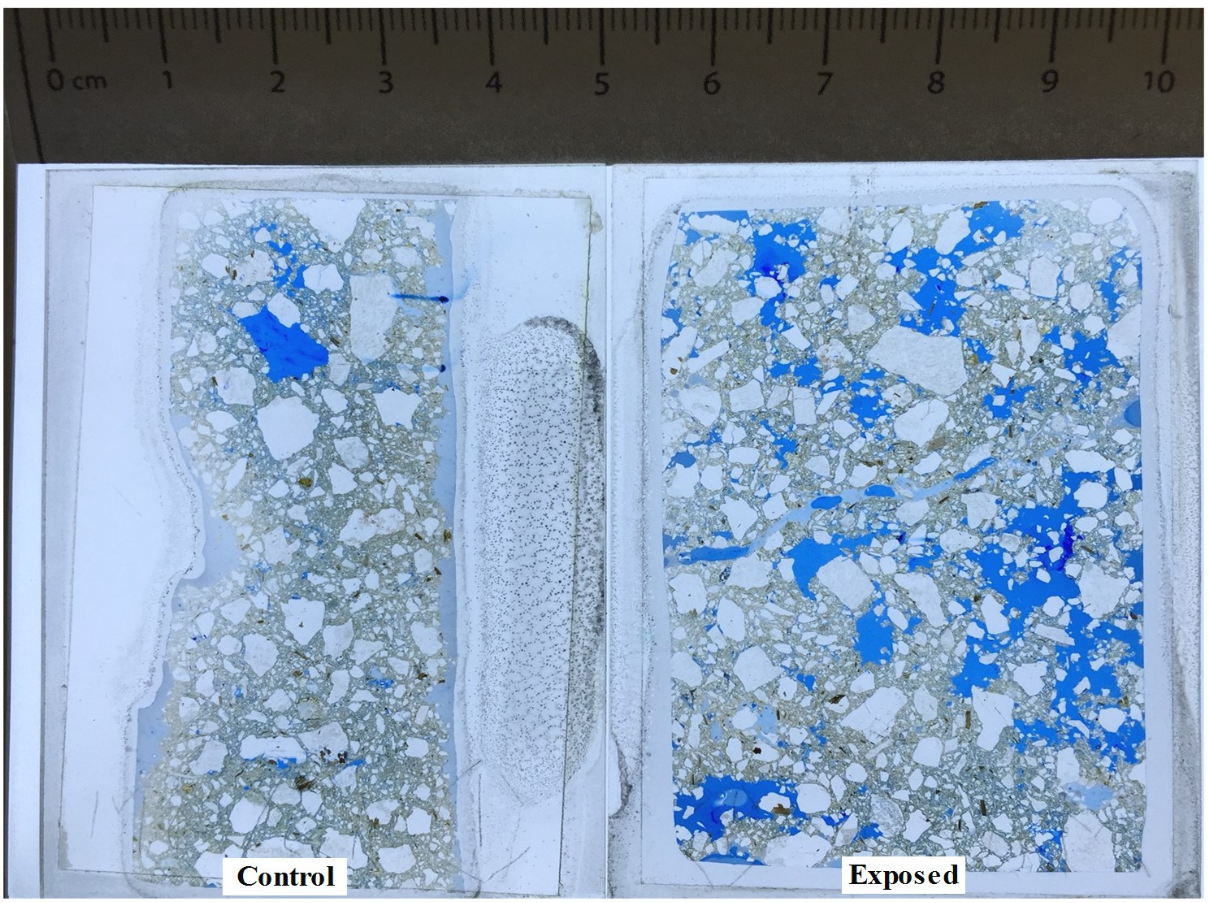


Figure 4.18 Ice lens formation shown on a thin section of soil-cement specimen A (3% cement and 14% water content) after exposure to three f/t cycles

4.5 Mercury Intrusion Porosimetry

Five MIP tests were initially performed on the same soil cement specimen (3% cement and 10% water content) to examine the variability of the test (see Figure 4.19). Figure 4.19 shows some variability throughout the same soil-cement mix. However, even though the magnitude of pore volume is variable amongst the samples, Figure 4.19 shows consistently a peak at 1µm where significant pore volume is recorded. The consistency is likely due to random sampling of the samples and having several samples in the MIP holder at the same time during testing (averaging). Any dominant peaks that are present in this process would expected to be consistent, as noted in Figure 4.19.

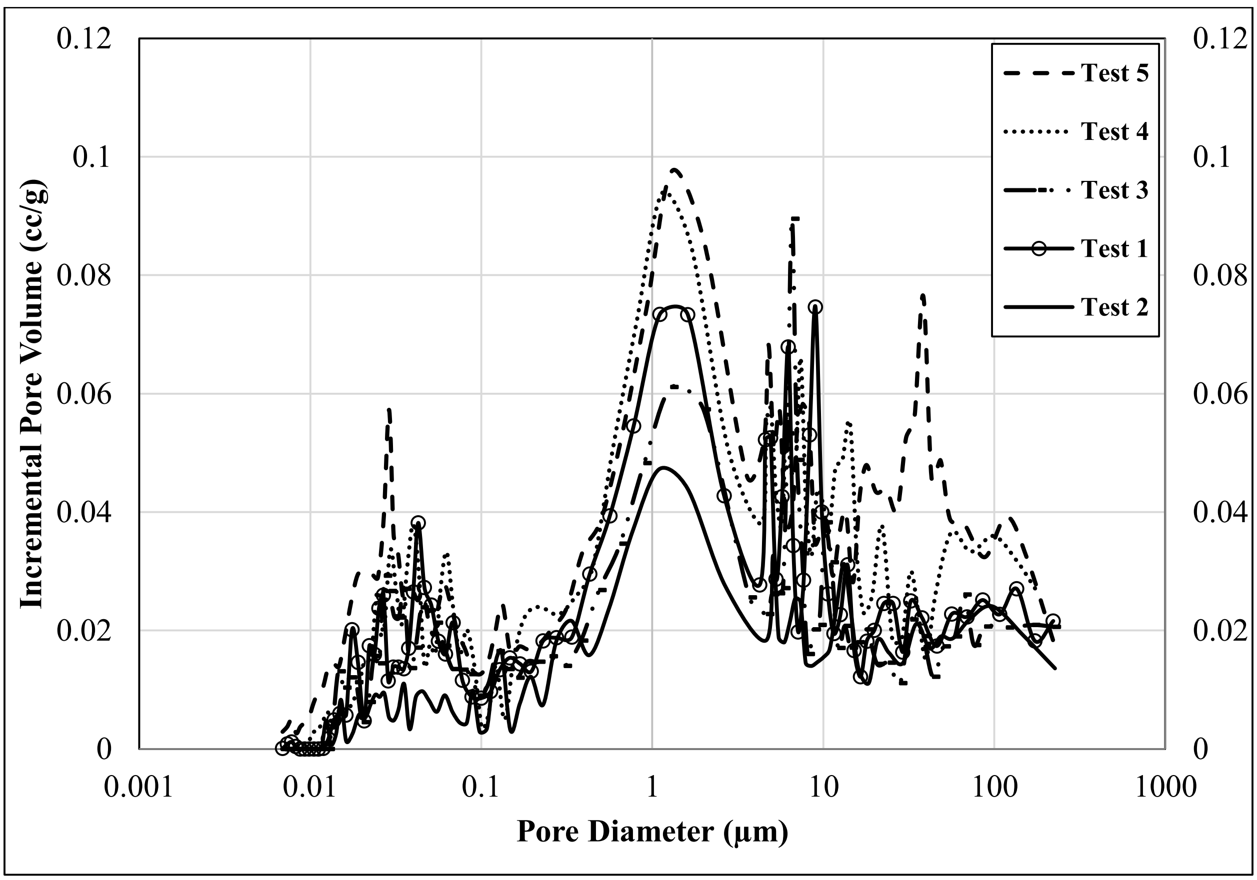


Figure 4.19 Variation of incremental pore volume with pore size for exposed specimen at 3% cement and 10% water content

4.5.1 Effect of Water and Cement Content on Pore Size Distribution

MIP tests were performed under all compaction moisture contents and cement contents to examine the effect of water and cement content on pore size distribution of the soil-cement specimens. At dry of optimum water condition (specimen A9 in Figure 4.20), larger peaks were observed at 10μm diameter than at optimum and wet of optimum water conditions. Larger peaks at dry of optimum condition represent more large pores that allow more space for water to go through which agrees to hydraulic conductivity results. At optimum water content (specimen B9 in Figure 4.20), pore sizes found to be the smallest which agrees to minimum hydraulic conductivity observed at optimum water condition. In Figure 4.20, less difference was observed in pore sizes between optimum and wet conditions than between optimum and dry conditions which also agrees with the magnitude of difference between hydraulic conductivity results. A similar trend is shown on Figure 4.21 for 6% cement content. Less accurate results appear to be recorded below 0.1 μm which is likely due to deformation in the specimen caused by pressure required to intrude the mercury.

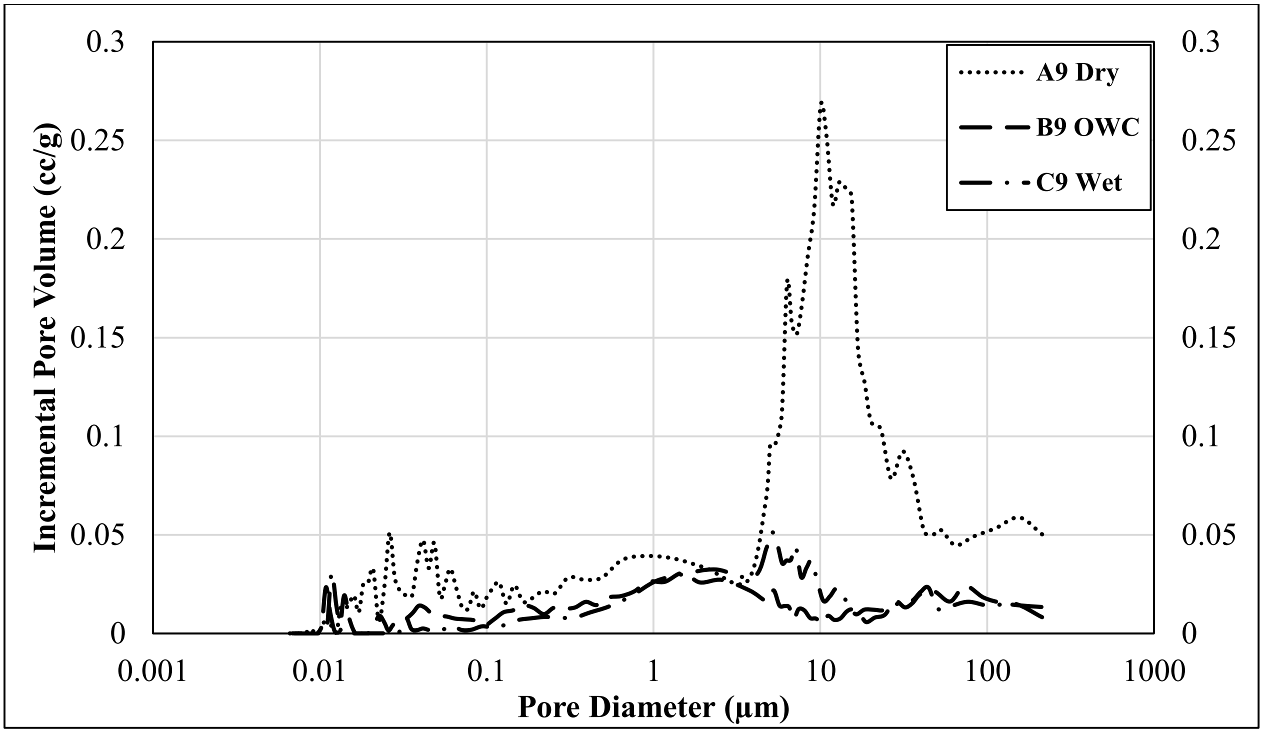


Figure 4.20 Comparison of incremental pore volume with pore diameter due to change in compaction moisture contents (i.e. A9: 6%, B9: 10%, C9: 14%) at 3% cement content

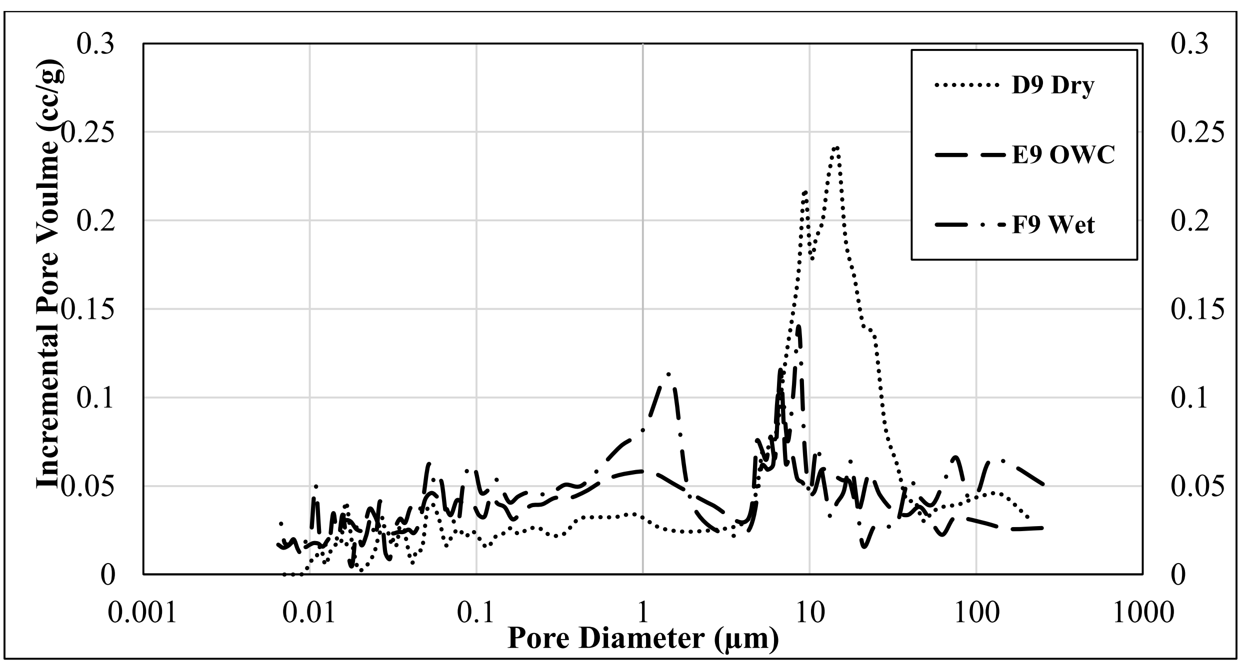


Figure 4.21 Comparison of incremental pore volume with pore diameter due to change in compaction moisture contents (i.e. A9: 6%, B9: 10%, C9: 14%) at 6% cement content

Results in Figures 4.20 and 4.21 are re-plotted in Figures 4.22, 4.23 and 4.24 to show the changes in incremental pore volume due to change in cement content for the same compaction water content. Similar trends are observed at 6% cement to 3% cement content. However, more pores are observed at 1µm and 10µm at 6% than at 3% cement content. The influence of more cement was not captured because probably some of the pore volume is isolated within the cement matrix (i.e. inaccessible pores).

Based on hydraulic conductivity testing, one would expect more difference in MIP plots. However, macro pores are probably dominating flow for 3% cement content and macro structure could be the reason behind higher hydraulic conductivity at 3% cement content than at 6% cement content.

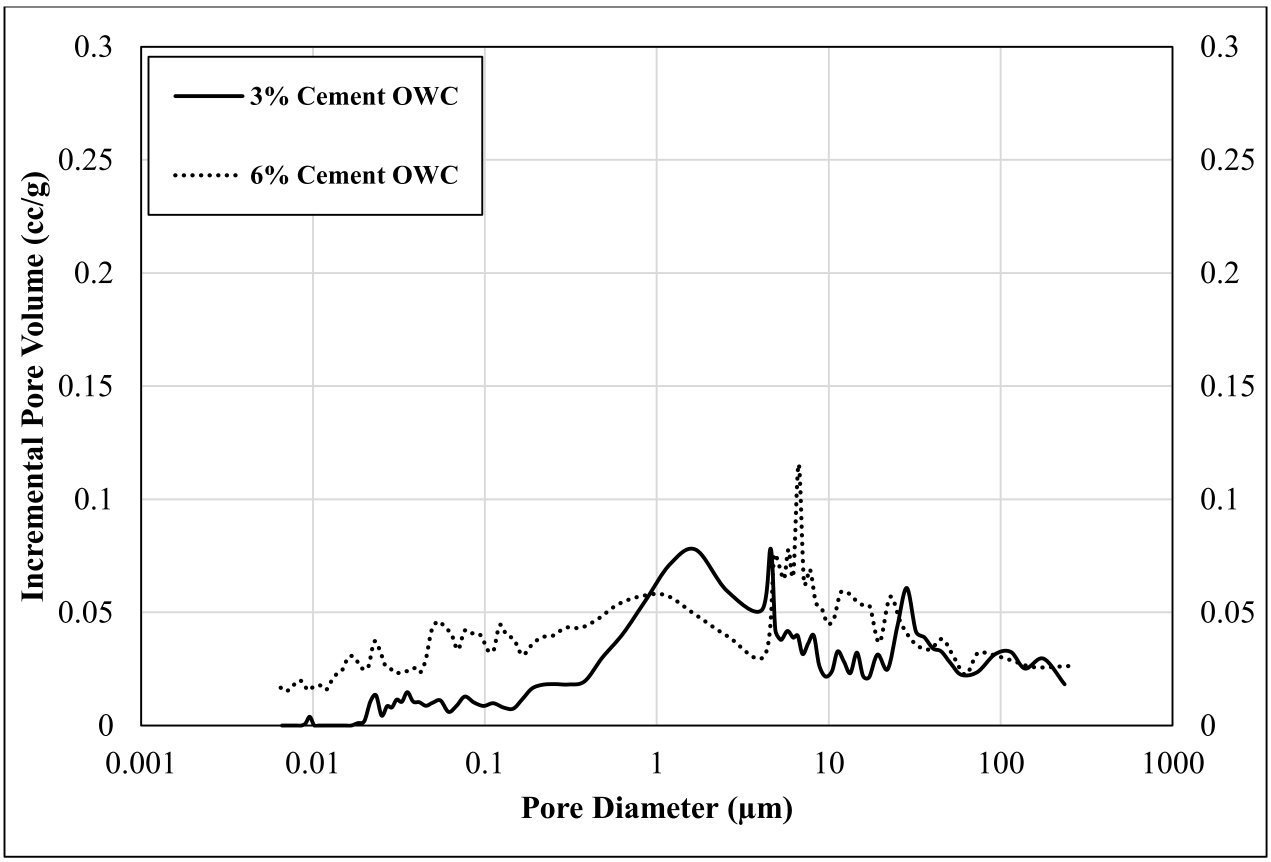


Figure 4.22 Comparison of incremental pore volume with pore diameter due to change in cement content (3% & 6%) at optimum water content

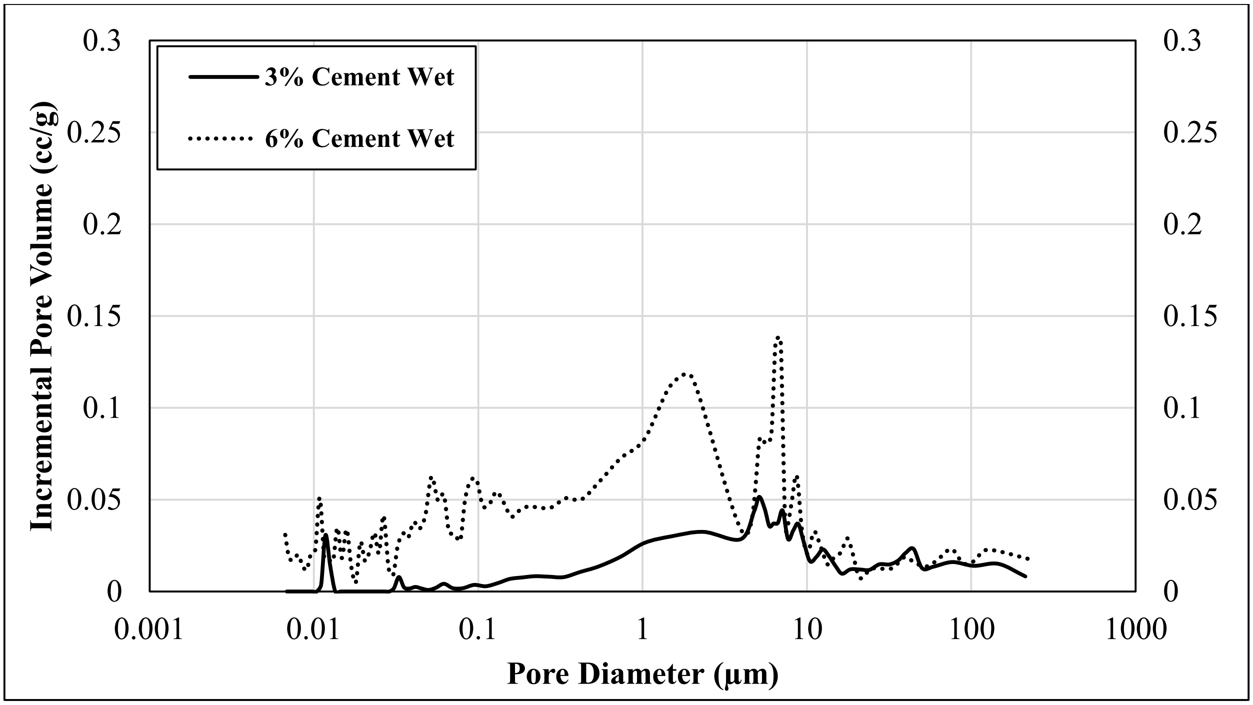


Figure 4.23 Comparison of incremental pore volume with pore diameter due to change in cement content (3% & 6%) at wet of optimum water content

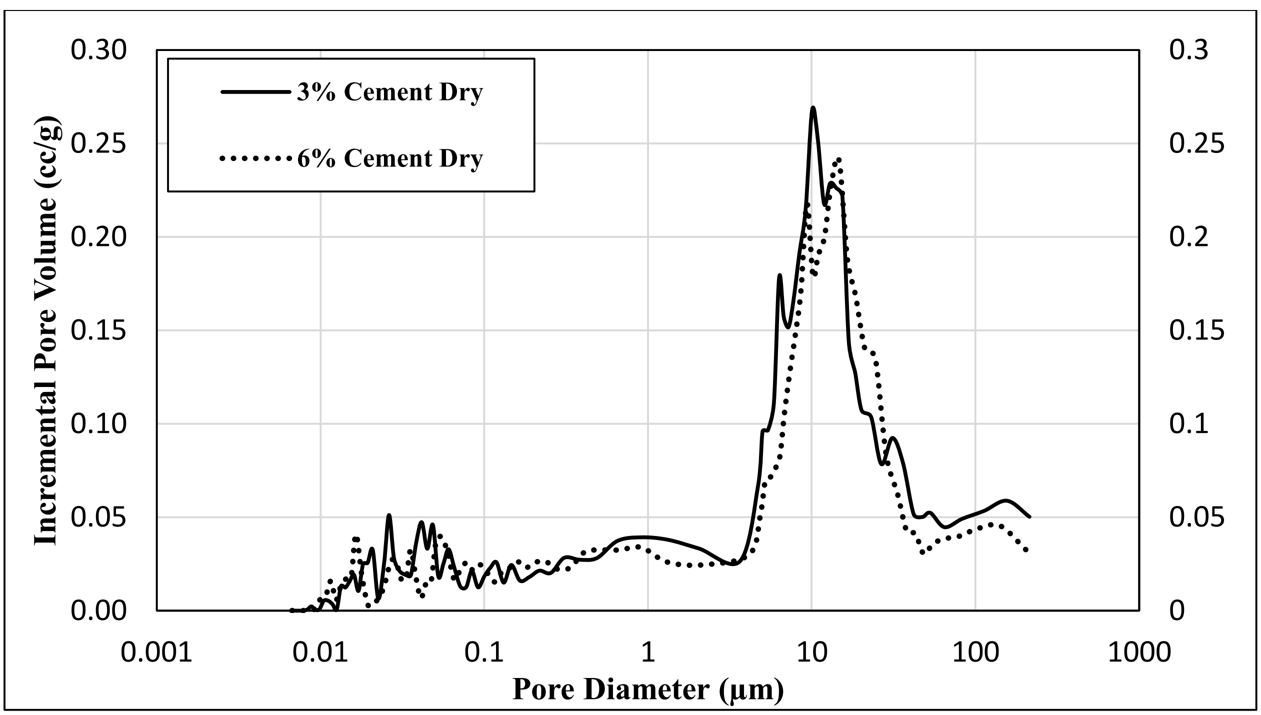


Figure 4.24 Comparison of incremental pore volume with pore diameter due to change in cement content (3% & 6%) at dry of optimum water content

4.5.2 Effect of Freeze/Thaw Damage on Pore Size Distribution

MIP testing was also performed on soil-cement specimens after exposure to three f/t cycles (Figures 4.25 – 4.30). At first examination, there appears to be only minimal changes in the poresize distributions when comparing MIP results before and after three f/t cycles. At 3% cement content, there appears to be shifting of pore size to larger sizes in the control samples after f/t. Mixture A shows a loss at the 10µm to 100µm sizes, probably the creation of macropores is not measurable by the MIP. In this thesis, macropores defined as pores that are larger than 200µm. Mixture B shows the loss of poresize in the 0.01µm and 10µm sizes and some development of poresize in the 10 µm range. Mixture C shows some loss of poresize at the range of 0.01µm to 1µm range as well as at the 8µm to 80µm range. There is no significant gain in poresize volume suggesting the creating of macropores. For the 6% samples Mixture D shows only minor gains in poresize at the 0.02µm to 0.06µm range after freezing and some gain in poresize at 1-7µm range and 30µm to 200µm range. The general distribution of the curves did not appear to change and some loss at the 0.01µm and 0.1µm range observed. Mixture E shows a consistent loss of pore size between 0.01µm and 1µm as well as between 5µm and 180µm without any corresponding increase in poresize in the f/t exposed samples. This is likely the development of macropores. Mixture F shows the loss of poresize between 0.3µm and 10µm and some increase between 16µm and 200µm.

Although MIP results show some shifting of poresize in the 0.01µm to 200µm range, there is no consistent poresize range that develops after f/t which suggests that the main mechanism responsible for hydraulic damage due to f/t exposure is cracking which is similar to the mechanism responsible for damage of soil-cement at high cement contents (>10%) that was reported by Jamshidi (2014).

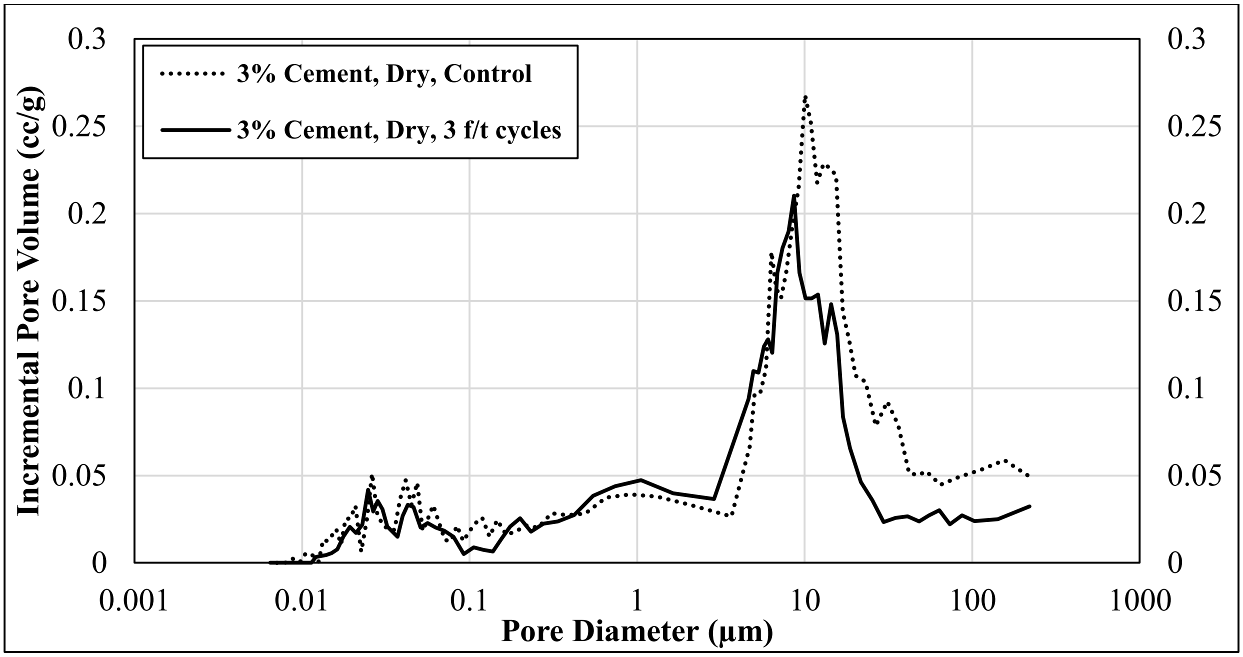


Figure 4.25 Comparison of incremental pore volume with pore diameter due to exposure to three f/t cycles at dry of optimum condition and 3% cement content

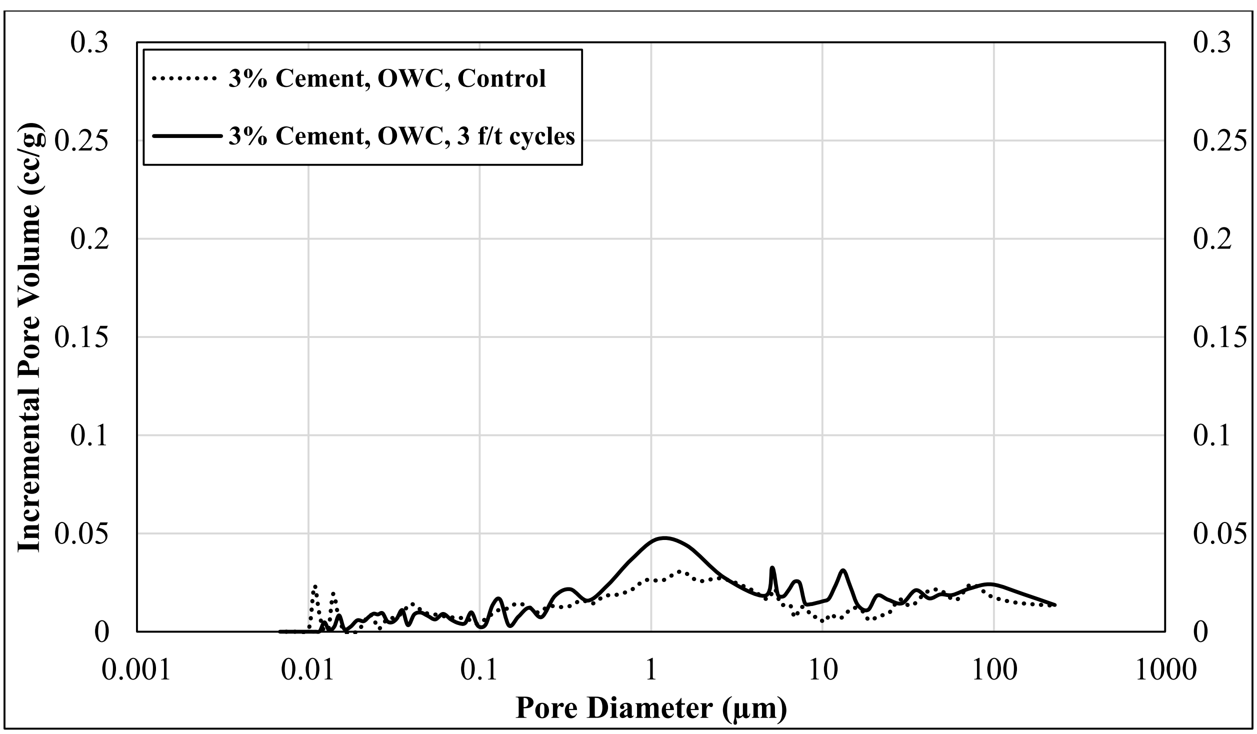


Figure 4.26 Comparison of incremental pore volume with pore diameter due to exposure to three f/t cycles at optimum water condition and 3% cement content

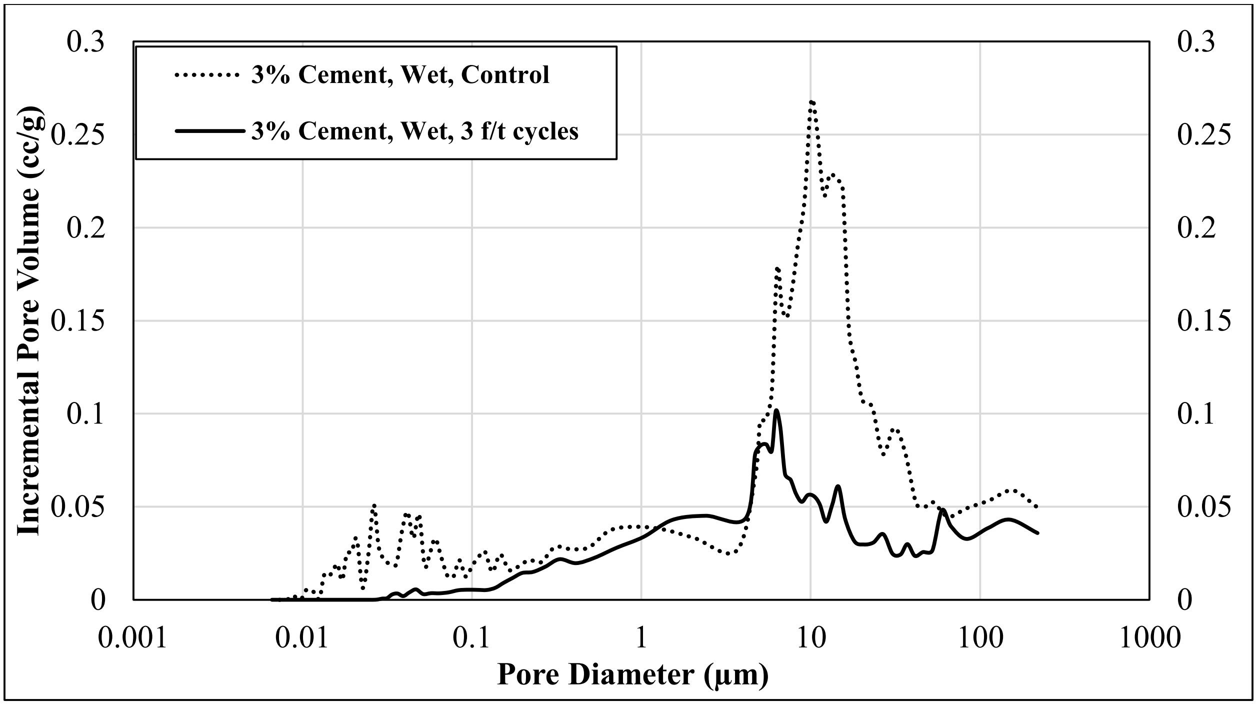


Figure 4.27 Comparison of incremental pore volume with pore diameter due to exposure to three f/t cycles at wet of optimum water condition and 3% cement content

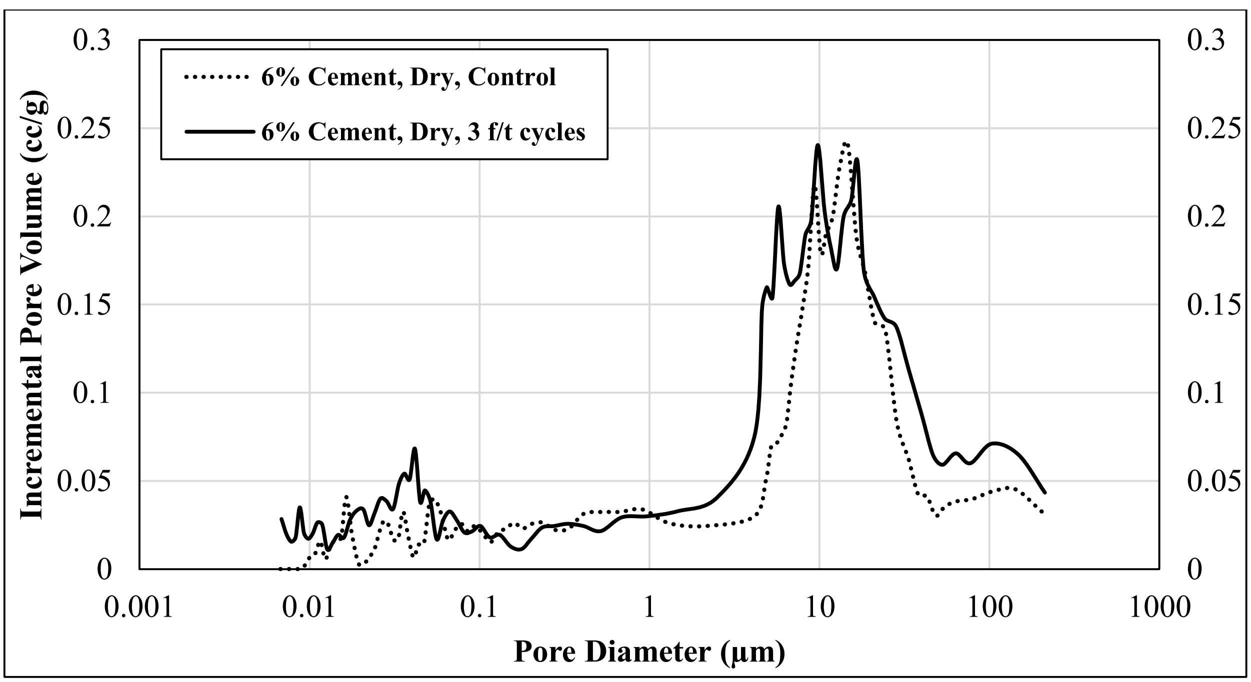


Figure 4.28 Comparison of incremental pore volume with pore diameter due to exposure to three f/t cycles at dry of optimum water condition and 6% cement content

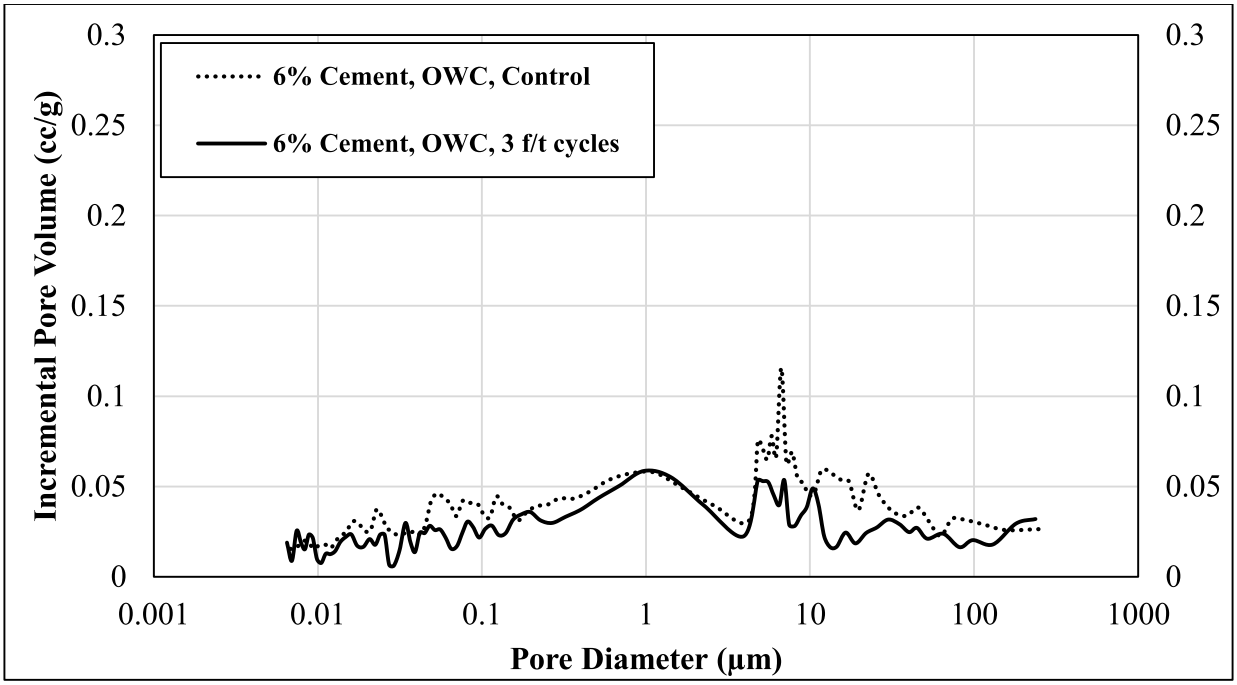


Figure 4.29 Comparison of incremental pore volume with pore diameter due to exposure to three f/t cycles at optimum water condition and 6% cement content

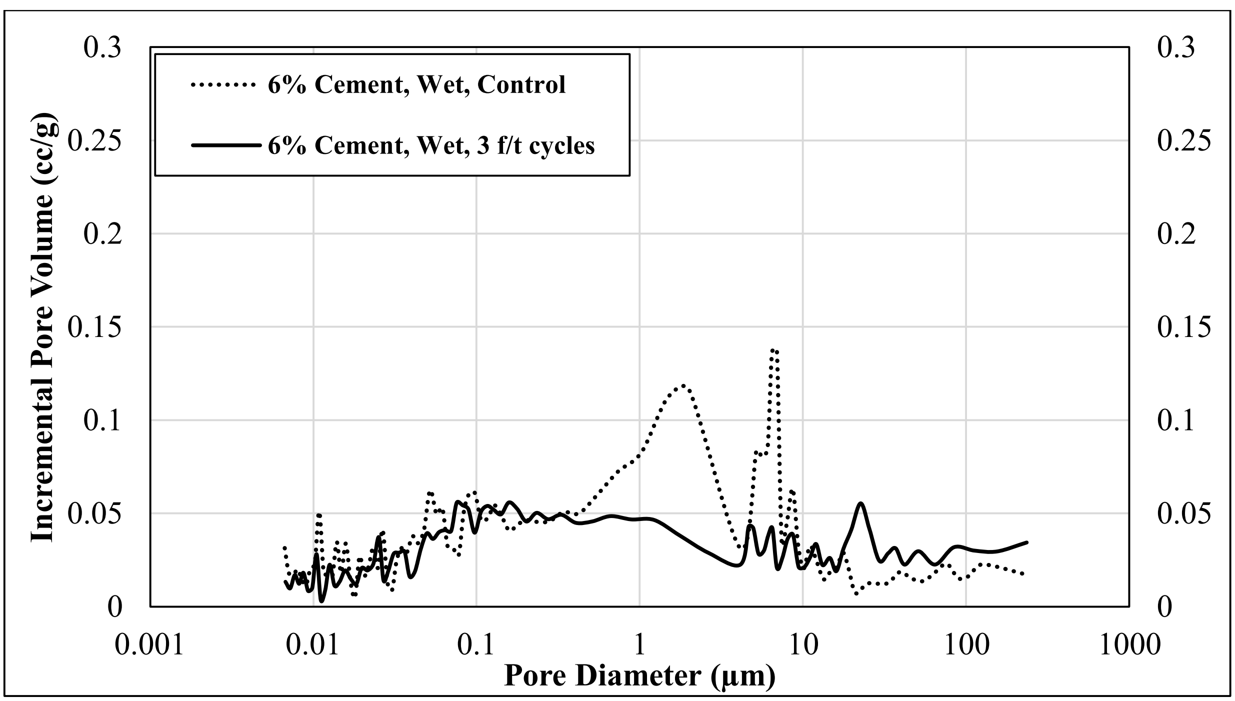


Figure 4.30 Comparison of incremental pore volume with pore diameter due to exposure to three f/t cycles at optimum water condition and 6% cement content

Chapter 5: Conclusions

As discussed in Chapter 1, the goals of this research was to enhance the knowledge of hydraulic and mechanical performance of soil-cement mixes and assess its durability. More specifically, the main objectives of this research were to:

1) Assess the impact of using low cement content (3% to 6%) on the hydraulic and mechanical performance of soil cement by performing laboratory testing on soil-cement mixes at different cement contents and moisture contents under standard proctor compaction conditions. In addition, the freeze/thaw damage on these soil cement samples were evaluated in terms of hydraulic and strength performance testing.
2) Examine the mechanisms responsible for the freeze/thaw damage observed by studying the micro and macro-scale changes in the samples using thin section and microscope imaging as well as mercury intrusion porosimetry testing to assess possible changes in poresize distributions.

The materials used to assess these objectives were a silty sand soil of a defined gradation, General use Portland-limestone cement (CSA type GUL) and tap water. Prior to soil-cement testing being performed, two standard proctor moisture density tests (ASTM-D558 (2011)) were performed to provide information on the compaction characteristics of the soil-cement mixture. Based on these results, six soil-cement mixtures were selected to represent a range of moisture conditions (below optimum water content, at optimum water content and above optimum water content) that might be expected in the field.

5.1 Summary of Hydraulic Conductivity Testing

Flexible wall hydraulic conductivity testing (ASTM-D5084 (2000)) was used to evaluate the influence of cement and water content on the hydraulic performance of soil-cement both before and after f/t cycling. The minimum hydraulic conductivity values were observed at optimum water conditions for each cement content. A lower hydraulic conductivity was observed at wet of optimum water condition than at dry of optimum water condition with the following order of hydraulic conductivity observed: $K_{owc} < K_{wet} < K_{dry}$. It was generally found that increasing cement content improves (decreases) hydraulic conductivity of soil-cement ($K_{6\%cement} < K_{3\%cement}$) at all the water contents. It was hypothesized that the addition of more cement fills more voids in the soil-cement mixture, leaving less space for water flow. The least improvement in hydraulic conductivity occurred at dry of optimum conditions where water is insufficient for the cement to react. Jamshidi (2014) observed that wet of optimum water content gave the lowest hydraulic conductivity, which is likely due to the use of high cement content that requires more water for hydration process in addition to the water required to achieve maximum compaction.

Exposure to three freeze/thaw cycles showed general increase in the hydraulic conductivity values. The least amount of damage was recorded at dry of optimum water conditions where more space is available for the water to expand when frozen, compared to optimum and wet of optimum water conditions. At higher cement content, better resistance to stresses created by water freezing and subsequent expansion was observed.

5.2 Summary of Unconfined Compressive Strength (UCS) Testing

Unconfined compressive strength (UCS) test was performed in general accordance to ASTM-D1633 (2007) to evaluate the influence of changing cement and water contents and freeze/thaw conditioning on the strength performance of soil-cement. Maximum UCS values were observed at optimum water conditions where maximum density is achieved. Higher UCS values were observed at dry of optimum water condition than at wet of optimum water condition (UCS_{owc} > UCS_{dry} > UCS_{wet}). Reduction of UCS values at dry of optimum water content conditions is likely due to limited water leading to inadequate lubrication of soil-cement mixture during compaction. Reduction of UCS values at wet conditions is likely due to excessive water content in the soil-cement mixture that could lead to bleeding in the paste and consequent decrease in density. Significant improvement in UCS was observed after increasing cement content from 3% to 6% ($UCS_{6\%cement}$ > $UCS_{3\%cement}$) due to improved bonding between soil-cement particles. All cement and water contents examined showed a general decrease in UCS values after exposure to three f/t cycles, however residual strengths of 6% cement specimens were still high. More damage was observed at dry of optimum and optimum water conditions than at wet of optimum water condition which is likely due to the ductile behavior of wet specimens. Less damage was observed at 6% cement content than at 3% cement content which is likely because 6% cement content provides better resistance to stresses created by water freezing and thawing.

5.3 Summary of Resonant Frequency (RF) Measurements

RF measurements method was used according to ASTM C215 (2008) to evaluate the macro-porosity changes due to freeze/thaw exposure and variation in cement and water contents of the soil-cement mixtures. Testing the soil-cement specimens at different water contents shows the highest RF measurements at optimum water content and the lowest RF measurements at wet of optimum water condition ($RF_{optimum}$ > RF_{dry} > RF_{wet})which agrees to UCS results. Higher RF measurements are obtained at higher cement content ($RF_{6\%cement}$ > $RF_{3\%cement}$) which also agrees with UCS results. RF measurements show gradual decrease with increased number of freeze/thaw cycles which indicates a progressive increase in damage. Most change in RF measurements with increase in f/t cycles occurred at optimum water content compared to wet and dry of optimum water conditions which agrees to the observation in the UCS results.

5.4 Summary of Examination of Mechanisms Responsible for Freeze-Thaw Changes of Soil- Cement Samples at 3% and 6% Cement Content

After studying the strength and hydraulic performance of soil cement under different conditions, an attempt was made to examine the mechanisms responsible for the obtained results. Longitudinal thin sections were obtained from soil-cement specimens and examined under the microscope to investigate the structural changes in the soil-cement mixture due to f/t conditioning and variations in cement and water contents. Ice lens formation is observed at the optimum and wet of optimum conditions of 3% cement which is likely due to the existence of more pores and sufficient water. The dry of optimum condition of 3% cement and the dry, wet and optimum water conditions of 6% cement have not shown ice lens formation. Generally, more voids and cracks were observed in freeze/thaw exposed specimens which verified the higher hydraulic conductivity results after freeze thaw exposure. Soil-cement specimens with higher cement content showed less pores which verified the lower hydraulic conductivity results at 6% cement content compared to 3% cement. Least pores were observed at optimum water condition compared to dry and wet of optimum water conditions which also verified the hydraulic conductivity results.

MIP testing method was used to evaluate the microstructural changes of soil-cement specimens due to changes in water and cement contents and exposure to three f/t cycles. Pore size distribution curves obtained by MIP testing show more pores at dry of optimum water condition than at optimum and wet of optimum water condition which agrees to hydraulic conductivity results. At optimum water content, minimal pores were observed which also agrees to hydraulic conductivity results ($K_{owc} < K_{wet} < K_{dry}$). Similar trend observed at 6% cement to 3% cement content. However, more pores observed at 1μm and 10μm at 6% than at 3% cement content. The influence of lower hydraulic conductivity with increasing amounts of cement was not captured because probably some of the pore volume trapped in the cement matrix (i.e. inaccessible pores). This implies that macro pores are probably dominating for 3% cement content and macro structure could be the reason behind higher hydraulic conductivity at 3% cement content than at 6% cement content. After exposure to three f/t cycles, results did not show a significant change in the pore volumes. This suggests that the main mechanism responsible for hydraulic and strength damage due to f/t exposure is cracking which is similar to the mechanism responsible for damage of soil-cement at high cement contents (>10%) that was reported by Jamshidi (2014).

5.5 Conclusions

The results of soil-cement testing in this thesis demonstrated the following:

- Cement content, water content and exposure to freeze/thaw cycles are key parameters in controlling the hydraulic and strength performance of soil-cement
- Mixing soil-cement at optimum water condition results in maximum UCS and minimum hydraulic conductivity because at optimum water condition, soil-cement particles are closest to each other leaving less space for water flow and achieving maximum density.
- At dry and wet conditions, more voids are created inside the soil-cement specimen leading to increase in hydraulic conductivity and decrease in UCS values
- Addition of cement increases UCS and decreases hydraulic conductivity by filling more voids and providing better linking and bonding between soil particles.
- Exposure to freeze/thaw conditioning lead to increase in hydraulic conductivity and decrease in UCS of the soil-cement

- It was observed that the main source of f/t damage is the cracks occur in the soil-cement mixtures that was observed by the optical microscope while minimal changes in the microstructure was observed by performing mercury intrusion porosimetry testing. The formation of ice lens observed by thin section photos at the wet and optimum water conditions of 3% cement could be another source of f/t damage.

5.7 Recommendations for Future Work

Future work that could be performed to improve the knowledge base of this work includes:

- Perform similar testing methods at dry and wet conditions but closer to optimum water condition to get a more accurate hydraulic and strength curve
- Evaluate the influence of changing the curing age at low cement contents
- Addition of common soil-cement admixtures and examine its influence on the hydraulic and strength performance due to freeze/thaw damage
- Performing field-scale experiments to examine the reliability of observations under controlled laboratory conditions
- Examine the validity of the observations on different types of soils
- Evaluate the influence of the interaction of contaminants in the soil structure with the changes in soil-cement performance due to freeze/thaw exposure

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Appendix A Photos of Longitudinal Soil-cement Thin Sections: Before and After Exposure to Three f/t Cycles

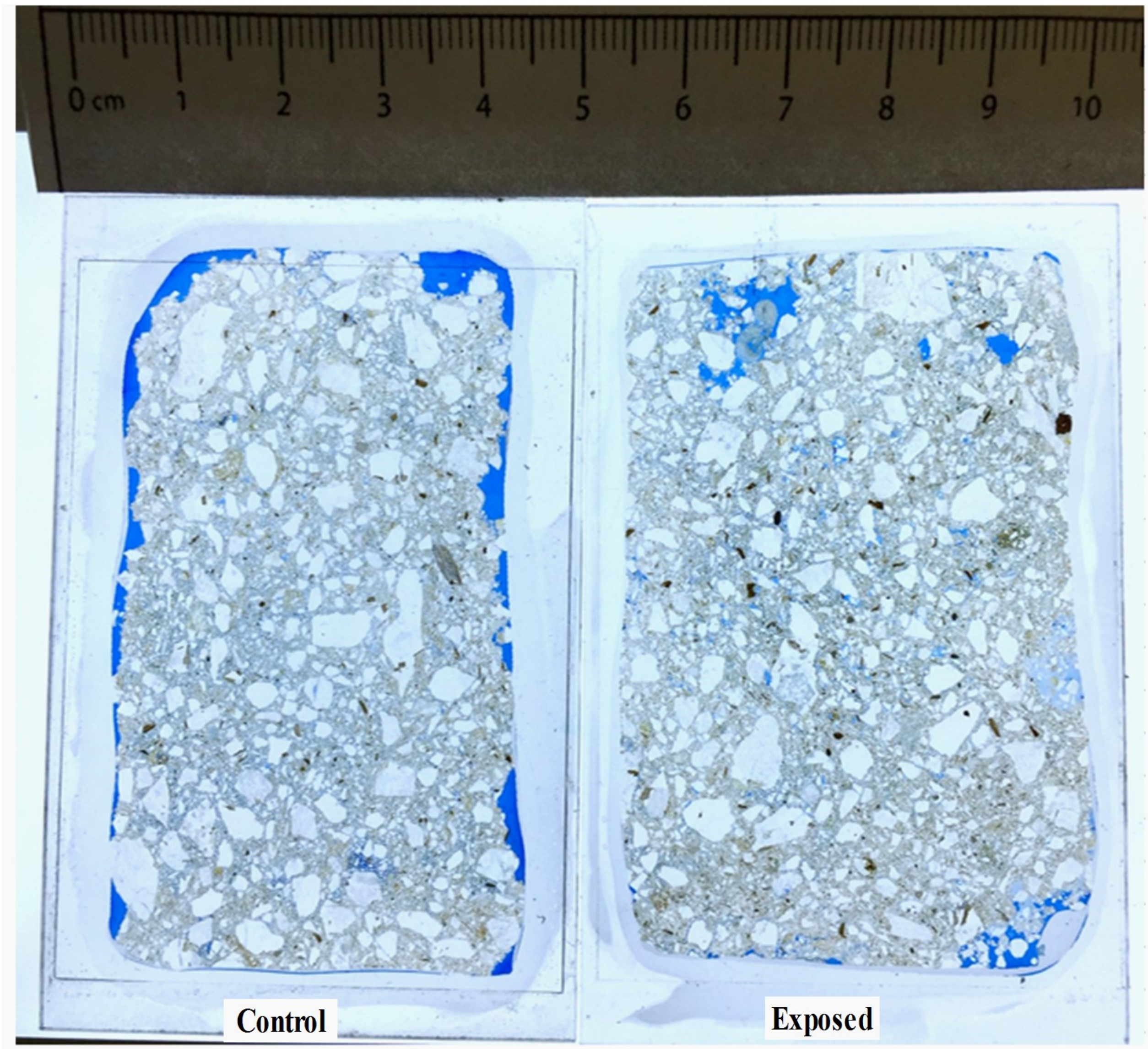

Figure A.1 No ice lens formation shown on a thin section of soil-cement specimen C (3% cement and 6% water content) after exposure to three f/t cycles

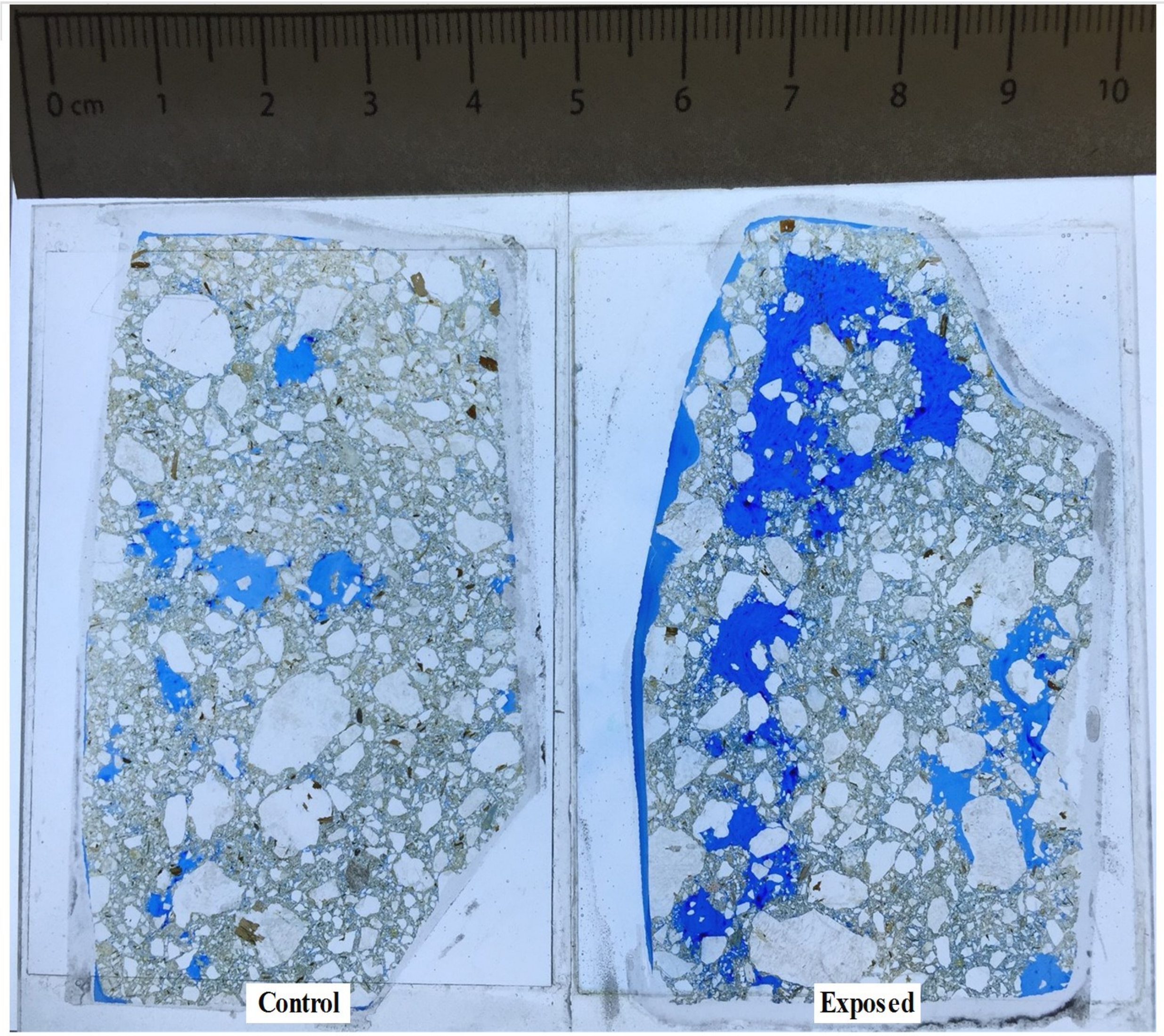

Figure A.2 Ice lens formation shown on a thin section of soil-cement specimen B (3% cement and 10% water content) after exposure to three f/t cycles

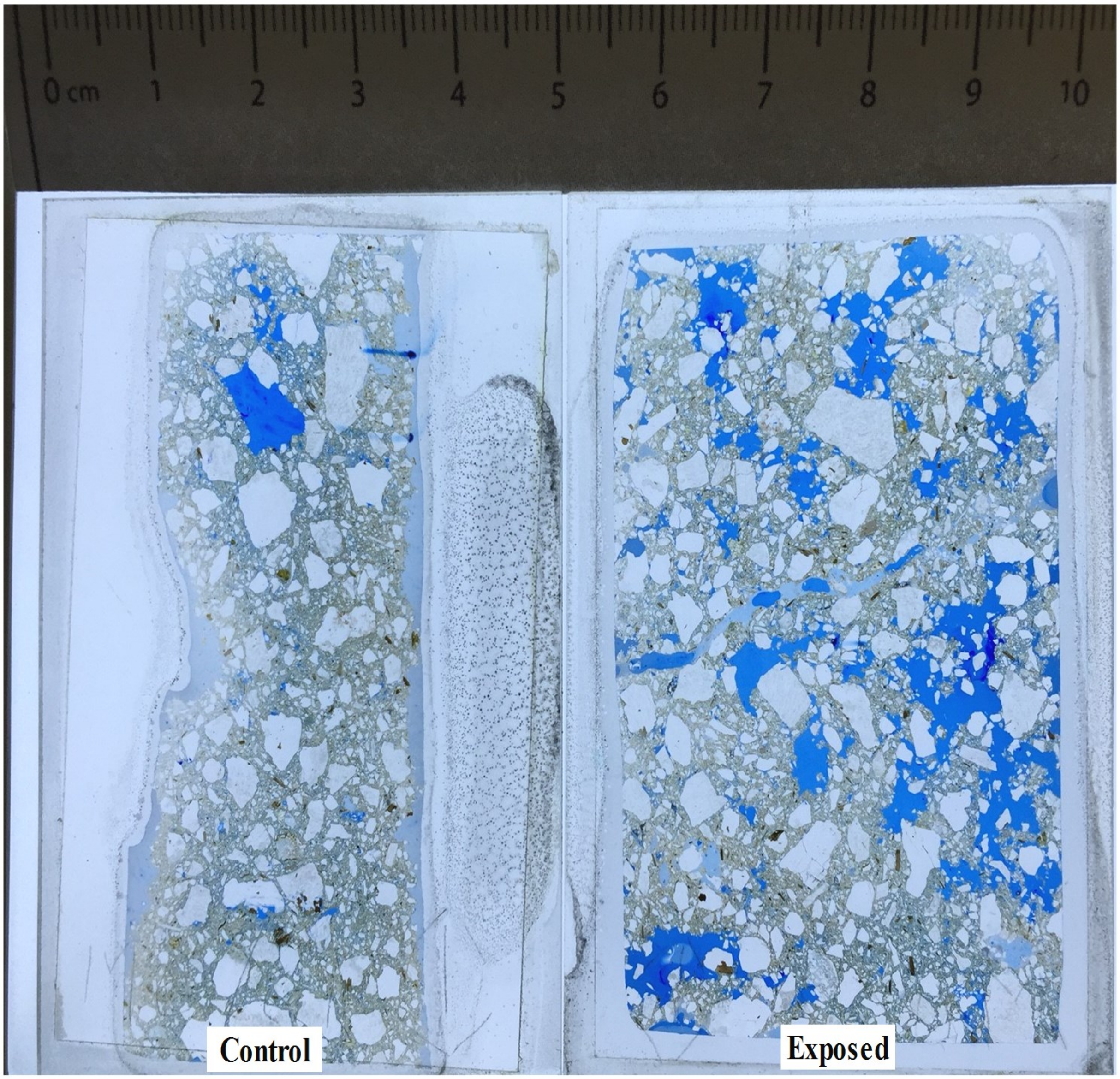

Figure A.3 Ice lens formation shown on a thin section of soil-cement specimen C (3% cement and 14% water content) after exposure to three f/t cycles

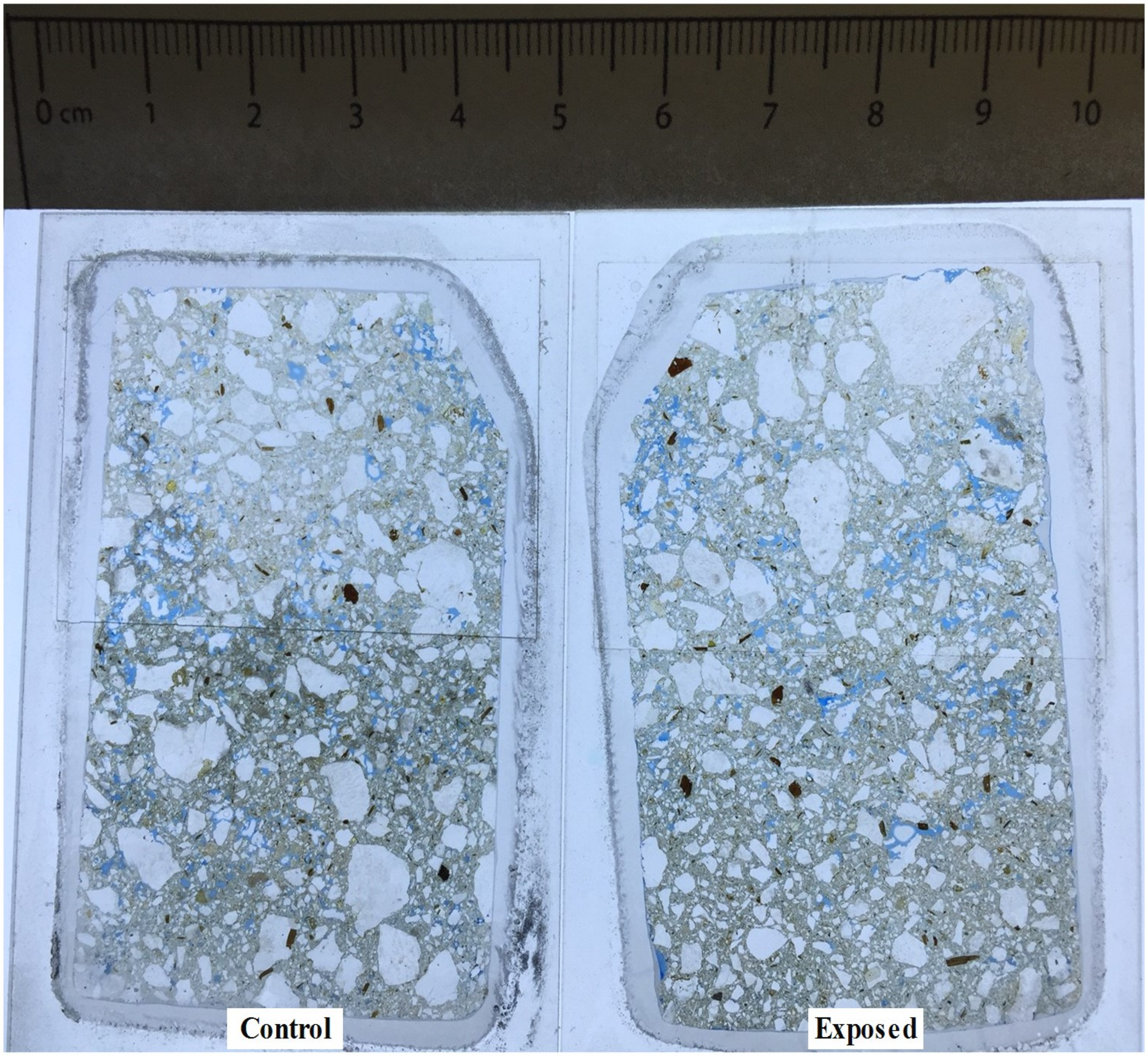

Figure A.4 No ice lens formation shown on a thin section of soil-cement specimen D (6% cement and 6% water content) after exposure to three f/t cycles

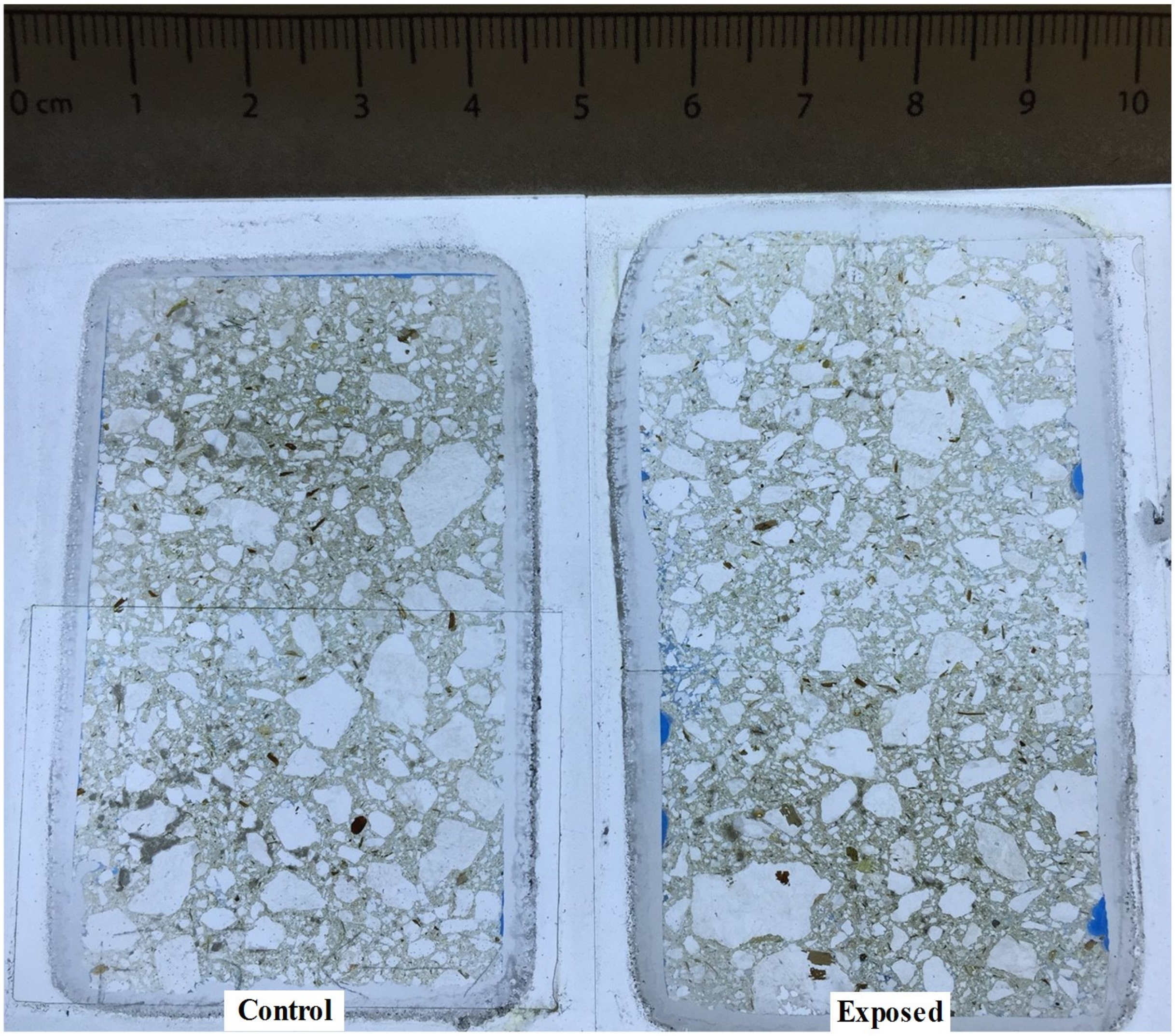

Figure A.5 No ice lens formation shown on a thin section of soil-cement specimen E (6% cement and 10% water content) after exposure to three f/t cycles

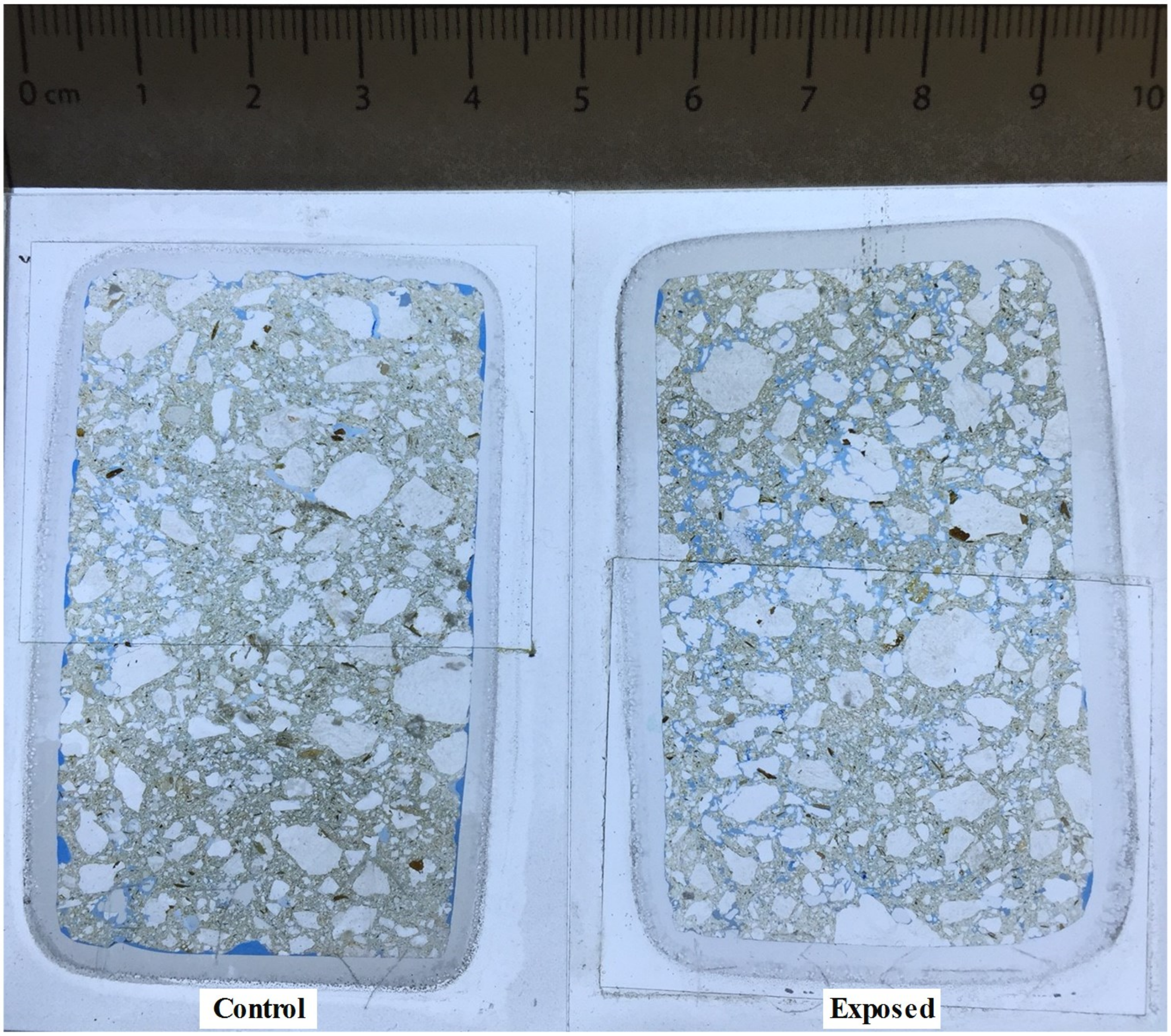

Figure A.6 No ice lens formation shown on a thin section of soil-cement specimen F (6% cement and 14% water content) after exposure to three f/t cycles