SECTION 1

INTRODUCTION

A considerable amount of research has been devoted to the stabilization of soils, aggregate, and recycled pavement materials using fly ash in highway applications, which demonstrated improvement in shear strength, compressibility and stiffness. However, how there is limited amount of research regarding how these materials stabilized with fly ash behave after exposed to winter conditions in the field.

Aging pavements, increasing wheel loads, and traffic frequency, combined with the effects of seasonal frost action are the main factors responsible for the rapid degradation of the highways in the northern regions of the United States. Pavements subjected to seasonal frost, experience freezing in the winter and thawing in the spring. During winter, an increase in strength and stiffness of the base and subgrade is observed. When spring comes, the base and subgrade become nearly saturated as the soils thaws and the snow and ice melt; which produce a reduction in strength and stiffness, often to values lower than prefreezing conditions. The recovery of the soil takes a long time and is partial. As a result, the weakened pavement cannot support the load for which it was originally designed, and damage occurs.

The objective of this study was to determine the freeze-thaw cycling effects on the engineering properties of soils stabilized using fly ash. The specimens of various coarse and fine-grained natural earthen and recyclable pavement materials stabilized using different fly ashes were subjected to freeze-thaw cycles at selected temperatures and number of freeze-thaw cycles. Subsequently, resilient modulus and unconfined compressive strength tests were performed on these specimens. The results are presented in terms of modulus and strength as a function of freeze-thaw cycles as well as the percent reduction in these properties relative to the properties of the unfrozen specimens.

SECTION 2

BACKGROUND

2.1 FROST ACTION

Seasonally frozen grounds involve temperatures below 0°C only during winter season and undergo many cycles of freeze-thaw. Frost action is used to describe the detrimental process of frost heaving in soil during the freezing period followed by thaw weakening or decrease in bearing strength when seasonally frozen soils thaw.

Three basic requirements for frost action are: (1) Freezing Temperatures, (2) Water Availability, and (3) Frost-Susceptible Soils. If one of any of these factors can be controlled, frost action can be prevented. Frost action is usually prevented by replacing the fine grained soil with a coarser granular material. Soil moisture can also be controlled by careful attention to drainage, so that the extent of frost heaving is greatly reduced.

2.1.1 Frost heaving and thaw weakening

When the air temperature at the surface is lower than the temperature of the soil, heat is extracted from the soil. The removal of heat from the soil causes its temperature to drop. If the surface temperature is below 0°C, a freezing plane advances into the soil. Ice crystals begin to form along the freezing plane and water migration starts from the lower unfrozen soil part toward the freezing front due to suction pressures which results from the freezing action. This water migration produces higher moisture contents at the top portion of the soil than before freezing, drying the unfrozen soil mass. Higher moisture content increments result with an open freeze-thaw system (having external source of water) than with a closed system (no external source of water). Water is available from underlying ground water table, through infiltration, from water held within the voids of soil, or even from a perched water source.

Clayey soil shows shrinkage cracks below and perpendicular to the freezing front as a result of the suction pressures. As the freezing front advances into the unfrozen soil mass, these cracks become filled with ice. Ice crystals continue to grow and join the freezing front, if the freezing is slow and fed mostly by capillary water, forming ice lenses. The formation of ice lenses produces a vertical pressure that heaves the surface. The conversion of water in the soil pores to ice, produces an increase in volume by about 9%. The increase in the volume of the soil due to the formation and growth of ice lenses is known as frost heave.

Ice lenses frequently develop in the soil under road surface and cause it to heave (see Fig. 2.1) The frost heave varies over a wide range, but vertical movements of 100 mm (4 in) to 200 mm (8 in) are not unusual and as much 610 mm (24 in.) has been reported (NRC-CIRC, 1962). For soils where the only supply of water is that held within the pores of the freezing soil, the frost heave is limited to the change in volume of the in situ pore water upon freezing. External water sources produce larger ice lenses and increases the severity of frost action. Studies performed with base-course and subbase course material frost heave increased linearly with increasing fine contents and increasing kaolinite fraction (Konrad et al.

2005, Tester et al. 1996). Heaving pressures also vary within quite wide limits and depend mainly on the type of soil and its moisture content. A saturated soil will develop the maximum heaving pressures; as moisture content drops; the heaving pressure drops also and is reduced to zero in a soil with low moisture content. Clay soil develops higher pressures than silts. Pressures in excess of 14 psi have been measured, and in a laboratory experiment a pressure of 213 psi was developed in clay (NRC-CIRC, 1962). Pressures of this order are much in excess of the pressures found under roadways and roads can heave quite readily when conditions are appropriated for ice lens formation. For frost heaving to occur, heaving pressure should exceed the load on the soil.

As thawing proceeds downward from the surface in the spring, the ice lenses thaw and contribute water to the soil. In some cases the accumulated water as a result of the ice lens formation and subsequent melting saturate the soil sufficiently to cause it lose strength. In roads during thaw weakening the action of traffic may cause the paved road surface to break, through loss of support. When the clayey soils thaw, a network of cracks is left behind which consists of vertical shrinkage cracks and horizontal cracks formed by ice lenses. A study performed with crushed limestone (granular soil) containing from 2 to 14% non plastic fines showed a linear increment in the rate of frost heave as fine content increased but the bearing strength or thaw weakening was not significantly affected by freezing and thawing (Tester et al. 1996).

Janoo et al. 1997 reported that for the frost susceptibility of stabilized lime and Portland cement soils a minimum of 3% lime or cement is required to reduce frost heave by about 50%. The addition of a pozollith to lime or cement appeared to reduce frost heave significantly in ML and CL soils. In cohesionless soils, about 3 to 8% of cement is required to reduce frost heave. For frost-susceptible gravel soils, 2% cement is required to change it a non-frost susceptible material.

2.1.2 Frost penetration depth

Frost penetration depth depends on the type of soil, its moisture content, its thermal properties, the freezing temperature (its magnitude, intensity and duration), insulating effect of snow, and many other factors. The density, conductivity of the soil particles and water content all influence the over-all thermal conductivity of soil. Among all the factors, probably the most important is the amount of water to be frozen, since it requires 151,898 Joules [144 heat units (Btu)] to freeze each 453 grams (1 pound) of water and only about 211 Joules [0.20 heat units(Btu)] to change the temperature of 453 grams (1 pound) of dry soil by (-17ºC) 1ºF (NRC-CIRC, 1962). In general, because clay particles have a higher insulation value than silt or sand particle and since clay soils normally hold more moisture than silt and sands, the depth of frost penetration is usually greater in silts and sand soils (light-textured soils) than in clays and silty clays (heavy-textured soils). Also insulating effect of snow deserves special mention. It has been shown that each decimeter of undisturbed snow reduces the depth of soil freezing by approximately the same amount. Also the USDA has shown that vegetation in agricultural fields can reduce the depth of frost by 50%.

Among the meteorological factors such as air temperature, sunshine, precipitation, and wind velocity, air temperature is probably the most significant. Frost action depends on the temperatures of the soil (source and amount of heat given to or available in the soil). However in most analysis frost action is correlated with air temperature, since records of air temperature are available for most locations. Based on air temperature, the soil freezing depends on the temperature below freezing and duration of freezing. These two factors are measured with the freezing index with units of degree-day. One degree-day represent one day with a mean air temperature one degree below freezing. For example, 10 degree-days may result when either the air temperature is -1°C for 10 days or -10°C for 1 day. The "freezing index" is the total number of degree-days of freezing for a given winter. Air-freezing index (in ºF days) an estimate of the 100 years return period for the United States is shown in Fig. 2.2.

The use of freezing index to predict the depth of frost penetration is based only on air temperature and other factor like soil type, snow cover, and local climatic are not considered; therefore, it should be used with caution. The freezing index is a useful guide in areas where no actual frost penetration information is available. The freezing index against the depth of frost penetration determined from an analysis of many records of frost penetration in any type of soil and for any moisture content is shown in Fig. 2.3. Of all the design curves and field observation presented in Fig. 2.3 Brown's curve has a better correlation with all the data points. Therefore, Brown's design curve is recommended for practical use. The general tendency of the effect of varying type of soils and moisture content on frost depth, as illustrated in Fig. 2.3, is that the higher the moisture content and finer the soil grain size the lower is the frost depth. The curves presented in Fig. 2.3 including Brown's curve are based on homogenous soils with favorable conditions for frost penetration. Therefore, Brown's design curve in some situations may be considered conservative.

2.1.3 Frost susceptibility of soils

Frost susceptibility of soils is defined in terms of frost-heaving and thaw-weakening behavior. Frost heave is not necessary for thaw weakening. Some clay soils develop segregated ice (and hence thaw weakening) while exhibiting little or no heave. Shrinkage or consolidation of layers adjacent to an ice lens cancels the heave normally associated with ice segregation, particularly where the water supply is limited and soil permeability is low. Frost-susceptible soils have the permeability and capillarity required to move the water from the unfrozen soil to the freezing front. Clays has the required capillarity but lack the permeability, coarse grained soil has the permeability but lack the capillarity. Silts have both characteristic and are the most frost-susceptible soils.

Frost-susceptible soils can be classified in six categories: negligible, very low, low, medium, high, and very high using the U.S. Army Corps of Engineering frost design and soil classification system (see Table 2.1). Three steps are involved: (1) the percentage of particles smaller than 0.02 mm; (2) soil type based in USCS; (3) a laboratory freezing test. The classification of negligible frost susceptibility is given to gravel with less than 1.5% finer than 0.02 mm and sand with less than 3%. For soils that do not fulfill these specifications, soil classification based in USCS is required (step 2). The range of possible degrees of frost susceptibility is very wide for most soils (see Fig. 2.4), that is why a laboratory freezing test in step 3 is recommended when precise information is required. Fig. 2.4 shows that the higher the plasticity and the more homogenous the clay is, it becomes less frost susceptible.

U.S Army Corps of Engineers Cold Region and Engineering Laboratory (CRREL) recommended a freezing test which measures frost heave and thaw weakening. This test takes five days. Four samples of 150 mm diameter by 150 mm are tested. The test involves two freeze-thaw cycles. Samples are compacted to field density and moisture conditions. Computer-controlled temperatures are applied at the top, bottom and sides of the sample by heat exchangers connected to refrigerated baths. Freezing is from the top down and the sample is enclosed in split rings and a rubber membrane to allow frost heave. Free access of water is allowed (open system) with the water level at the freezing boundary. The freeze heave rate at the end of the first 8 hours of each 2 days of freeze-thaw cycle is used as an index of frost-heave susceptibility. After 2 freeze-thaw cycles when the sample is completely thawed and a California Bearing Ratio (CBR) test is performed and results are used as an index of thaw-weakening susceptibility. The preliminary frost susceptibility criteria for this freezing test are shown in Table 2.2.

2.2 EFFECTS OF FREEZE AND THAW CYCLING ON THE ENGINEERING PROPERTIES OF SOILS

2.2.1 Shear Strength (CBR)

The strength of clayey soils can be either increased or decreased by freeze-thaw cycling. Moisture redistribution, consolidation condition, and particle reorientation are the major factors which determine either increase or decrease. The main cause for changes in strength of soils subjected to freeze-thaw cycles is the moisture redistribution. In general, because of the moisture redistribution caused by the migration of moisture during freezing, the upper portion of the ground would experience loss of strength while the lower portions may actually see strength increase. The magnitude of such changes depend on at least the rate of change in temperature, the rate of drainage upon thawing and the amount of moisture involved in the process. Upon completion of thawing and drainage, the soils regain most of their strength but it takes several days.

Field data reported by Hans Kok (1989) show a decrease in the strength of clayey soils upon thawing of 50% or less in comparison with the pre-freezing strength. Large strength losses are observed in sensitive clays with high initial water content (above liquid limit) after freeze-thaw cycles because of the breakdown in the original cementation bonding between clay particles. The thixotropic strength of clay soils is affected by freezing and thawing and the rate regain of strength with time is greatly reduced by freeze-thaw action. Based on different tests performed on clays, Chamberlain (1989), concluded that strength increases after freeze-thaw cycles can be expected when increased consolidation and density occur. The special cases where strength decrease can be expected include highly cemented clays and clay soils that are highly overconsolidated before freezing. Alkire (1981) reports that freeze-thaw affects shear strength by altering the pre-shear history and consolidaton conditions. Test conditions that simulated fast or constant water content freezing (undrained, closed system) caused an effect similar to that caused by increasing the overconsolidation ratio and reducing the effective consolidation stress. This caused a slight reduction in the post-thaw shear strength. Slow freezing or freezing with increasing water content (drained, open system) not only caused an increase in overconsolidation ratio and a reduction in effective consolidation stress but also caused an increase in water content, which produced a substantial reduction in post-thaw shear strength. He also stated that the main reason for strength loss in a soil subjected to freeze-thaw cycles is due to the change in water content that accompanies freeze-thaw cycles. Freeze-thaw cycles can have a beneficial effect if water content is held constant. Based on these studies, it is clear moisture redistribution cause by freeze-thaw cycling is an important factor in the change of shear strength. At constant water content increases in strength were observed. Loss of strength was observed for saturated conditions.

Frost action in granular material used as base and subbase layers is often ignored, because these materials are usually considered non-frost susceptible. However studies demonstrate that frost susceptibility increases with increasing fine content. Also the mineralogy of the fines is also an important factor to consider and has been investigated. Yet the specifications used by pavement engineers are only based on grain size distribution and allowable fines content. In Wisconsin and

Minnesota, a base and subbase material is considered frost susceptible if the amount passing the No. 200 sieve exceeds 5% and 10%, respectively. The reliability of these values for predicting lack of frost susceptibility was in the range of 40%-80%, which is attributed to the influence of mineralogy of the fines, which is not taken into account (Konrad et al. 2005).

Based on laboratory tests on thousand of samples and field observation Brandl (1977, 2000) proposed to add a mineral criterion to the criterion of maximum allowable amount of particles smaller than 0.02 mm to account for the influence of mineralogy. In this research, it was demonstrated that a high relative amount of fines was admissible in the case of carbonates or quartz, whereas chlorites and their weathered products led to severe frost damage. The proposed criterion considered frost heave and thaw weakening. Granular mixtures with large chlorite and muscovite content cause primarily excessive heave but, in general do not reduce significantly the bearing capacity during thawing. Montmorillonite fines increase the potential for thaw weakening, whereas kaolinite causes mainly frost heaving.

2.2.2 Stiffness (M_r)

Simonsen et al. 2002 performed resilient modulus on five different types of soil after one freeze-thaw cycle with temperatures of 20°C to -20°C. Soil are classified as glacial till (silty sands, A-4); silty fine sand (silty sands, A-2-4); coarse gravelly sand (poorly graded sands, A-1-b); fine sand (poorly graded sands, A-1-a); and marine clay (very fine clay, A-7-5). Samples were compacted at optimum water content in five layers by means of kneading compactor. The samples are exposed to closed-system freeze-thaw (no in or outflow of moisture is allowed during freezing or thawing) and freezing-thawing in 3D. They were tested following AASHTO TP46-94. A resilient modulus reduction after 1 freeze-thaw cycle was observed for all soils. Percentages in resilient modulus reduction were as follows: glacial Till (27%), silty fine sand (19%), coarse gravelly sand (23%), fine sand (50%), and marine clay (57%). In general, resilient modulus reduction ranges between 20 to 60%. Coarse gravelly sand with only 0.6% of 0.075 mm fraction shows a net volume increase after 1 freeze-thaw cycle, resulting in a looser soil structure and inevitably causes a decrease in resilient modulus. These observations are consistent with the results of Viklander (1998) who investigated permeability and volume changes in a non-cohesive till during cyclic freeze-thaw. He observed volume reduction in an initially loose soil and volume increase in an initially dense soil after the freeze-thaw cycles. Independent of the initial soil density a constant residual void ratio was obtained after 1-3 freeze-thaw cycles. The author presented data indicating that the void ratio in a very dense soil might increase due to freeze-thaw, because during thawing the soil particles may not fall back to exactly the same position. The result is a net volume increase of the soil, making the soil structure slightly looser than it was prior to freezing. Silty fine sand with 18% 0.075-mm fraction took a long time to increase the temperature to 20°C giving sufficient time for the soil to partially recover from the effect of freeze-thaw. Fine sand with only 2.5% of 0.075-mm fraction, which would generally not be regarded as highly frost susceptible soil, showed a resilient modulus decrease of 50% also attributed to volume increase due to freeze-thaw. In this study resilient modulus tests were performed immediately after reaching the target temperature, and no attempt to fully recover the specimen was made.

A study of the effect of freeze-thaw cycle on the resilient characteristics of compacted clay till was performed by Culley (1971). Samples were compacted by static compaction at 93%, 95% and 100% standard Proctor and 100% modified Proctor. Each specimen was compacted at optimum water content and ±1.5% and ±2.5% of the optimum, except for 100% modified Proctor. The specimens were subject to 3 freeze-thaw cycles, each cycle consisting of 8 hours at a constant temperature of -17.7 ºC and 8 hours of 21.1ºC. A closed-system was used and approximately 50,000 load repetitions were placed on each specimen. Resilient modulus after 3 freeze-thaw cycles indicated that at water contents lower than the optimum resilient modulus decreased as density increased. At optimum water content or wet of optimum resilient modulus decreased as the density increased. At optimum water content, the resilient modulus decreased 59% (at 93% standard Proctor) and 66% (at 100% standard Proctor). For 100% modified Proctor only a resilient modulus reduction on 41% after 3 freeze-thaw cycles was observed.

Lee et al. (1995) studied the effect of 3 freeze-thaw cycles with temperatures ranging from -7ºC to 7ºC on the resilient modulus of a compacted fine-grained subgrade soil. Resilient modulus test was performed on undisturbed specimens from an in service pavement. The highest resilient modulus reductions (30% to 50%) were observed after 1 freeze-thaw cycle, subsequent freeze-thaw cycles shows insignificant effects.

Three secondary highways with flexible pavements in Wisconsin were instrumented for 18 months to record the resilient modulus changes in the base and subgrade because of the seasonal changes by Jong et al. (1998). When the bases were completely thawed, the resilient modulus values were about 35% of the pre-freezing value. When the subgrades were completely thawed, the resilient modulus values were about 65% of the pre-freezing value. Complete recovery to the pre-freezing values took 4 months. Similar percentages were reported by Mahoney et al. (1985) from Washington field data. Resilient modulus of the base and the subgrade reduces by 23% and 52% on the average after thawing.

2.2.3 Stiffness (Mr) and Compressibility (qu) of soils stabilized with fly ash

There are very limited data on the engineering properties (required for mechanistic design) of stabilized soils with fly ash subjected to freeze-thaw cycling.

Zaman et al. 2003 focused on evaluating the effect of freeze-thaw cycles on Class C fly ash stabilized aggregate base specimens cured for 3 and 28 days. The aggregate used in this study had a liquid limit of 18%, a plastic index of 5%, specific gravity of 2.67, absorption values of 4.5% and Los Angeles abrasion value of 34%. The Class C fly ash had a moisture content of 0.33%, specific gravity of 2.69, loss of ignition (LOI) of 0.23%, Calcium Oxide (CaO) of 25%, Silica (SiO_2) of 20%, and Ferric Oxide (Fe_2O_3) of 7%.

Eighteen specimen 15.24 cm in diameter and 30.48 cm in height were prepared at a dry density of 2.21 kg/m^3, moisture content of 7.0% and stabilized with 10% of fly ash. Specimens were cured in a humidity room at 21ºC and 90% relative humidity for different durations. Eight specimens were cured for 3 days, subjected to freeze-thaw cycles and then tested for resilient modulus. Another 8 specimens were cured for 28 days, subjected to freeze-thaw cycles and then tested for resilient modulus. Two specimens were cured for 90 days, not subjected to freeze-thaw cycles and tested for resilient modulus. All of the specimens were subjected to unconfined compressive strength test following the resilient modulus test. The numbers of freeze-thaw cycles were 0, 4, 12, and 30 at temperatures of -25ºC for 24 hours and thawing in the humidity room for another 24 hours.

The resilient modulus after 28 days of curing increased with increasing freeze-thaw cycles up to 12 cycles and then started to drop. The resilient modulus after 3 days of curing increased with freeze-thaw cycles up to 30 cycles. The unconfined compressive strength after 3 and 28 days of curing increased as the numbers of freeze-thaw cycles increased. The resilient modulus reduction after 12 freeze-thaw cycles of the samples cured for 28 days was attributed to a deceleration of the cementitious/pozzolanic reactions caused by the freezing temperatures. The moisture content in the samples remained constant after the freeze-thaw cycles because of the closed-system freeze-thaw cycling. Visual observation of specimens revealed no cracks. The damage caused by the formation of ice lenses in the pores of stabilized specimens was found to have a negligible effect.

Another study was performed to investigate the use of Class F fly ash amended soil-cement or soil-lime as base layers in highways (Arora et al. 2005). Lime or cement was added to Class F fly ash as activators to produce cementitious/pozzolanic reactions. The freeze-thaw effect was examined by performing unconfined compressive strength test. A sandy soil classified as silty sand (SM and A-2-4) with 18% particles passing the U.S. 200 sieve and a specific gravity of 2.68 was used in this study. The Class F fly ash had low calcium content with a pH of 7.9, insoluble in water, and a dark grayish color indicating medium to high amount of carbon. The fly ash had an specific gravity is of 2.24, fines content of 85%, moisture content of 22.4%, loss of ignition (LOI) of 8%, SiO_2 of 54.9%, AlO_2 of 31.7%, K_2O of 3.4%, Fe_2O_3 of 3.4, SO_2 and SO_4 of 0.8%, CaO of 0.74 and MgO of 0.43. Type I Portland cement and high calcium (95%) quicklime were used as activators. Also kaolinite was added to some mixtures to investigate the effect of cohesion on engineering parameters.

Specimens with varying cement, lime, and fines contents were compacted at optimum moisture content following ASTM D 698. Fly ash percentage used was 40%, samples were cured for 7 days, frozen at -23±1ºC for 24 hours and thawed in a humidity chamber for 23 hours following ASTM D 560. Unconfined compressive strength test was performed following ASTM D 1633 and D 5102 for the cement and lime treated specimens, respectively, with a strain rate of 0.85%/min after 2, 4, 8, and 12 freeze-thaw cycles. For the cement–treated specimens, cement percentage used were 4, 5, and 7%.

For cement-treated specimens, the unconfined compressive strength increased with increasing number of freeze-thaw cycles. Higher increase in unconfined compressive strength was obtained with mixtures with 7% cement than with mixtures with 4 and 5% cement. The damage caused by the formation of ice lenses in the pores was negligible. For lime-treated specimens, strength decreased with increasing number of freeze-thaw cycles. The presence of kaolinite in the sample reduced the strength.

2.3 BACKGROUND SUMMARY

2.3.1 Frost Action

Frost susceptibility of soils is defined in terms of frost heaving and thaw weakening behavior. When soils are subjected to freeze-thaw using a closed system (no external sources of water available), the only supply of water is that held within the pores of the freezing soil and therefore the frost action is limited to the change in volume of the in situ pore water upon freezing. An open–system (with external water sources) produces ice lenses and thus increases the severity of frost action. If moisture content in the sample stays approximately constant during the freeze-thaw cycles and visual observation of specimens reveals no cracks and degradation, it is assumed that the volume of the pores are large enough to accommodate the formation of ice lenses without causing any noticeable damage. A saturated soil will develop the maximum heaving pressures; as moisture content drops the heaving pressure drops and reduces to zero in a soil with low moisture content. For granular soil, frost heave increase linearly with increasing fine content and increasing kaolinite fraction.

Frost penetration is usually greater and faster in silts and sand soils (light texture soils) than clay and silty clays (heavy textured soils) because clay particles have higher insulation values and normally hold more moisture than silts or sands. Frost heave is not necessary for thaw weakening. Some clay soils develop segregated ice (and hence thaw weakening) while exhibiting little or no heaves. Shrinkage or consolidation of layers adjacent to an ice lens cancels the heave normally associated with ice segregation, particularly where the water supply is limited and soil permeability is low.

Silts are considered the most frost susceptible soils because has the permeability and capillarity required to move water from the unfrozen soil to the freezing soil. Negligible frosts susceptible are gravels with less than 1.5% finer than 0.02 mm and sands with less than 3%.

2.3.2 Shear Strength (CBR) after freeze-thaw cycling

The main cause for changes in strength of soils due to freeze-thaw cycles is the moisture redistribution. During the moisture redistribution the water content of the soil portion above the freezing plane increase and soil would experience loss of strength (specially when ice start melting and water can not drain because soils below are still frozen) while the soil portion below the freezing plane experience strength increase (because of capillary movement of water to the freezing plane). Field data reported indicate a decreased of 50% in the strength of a clayey soil after one freeze-thaw cycle right after the target temperature is reached, without allowing time for strength recuperation. Upon thawing and drainage, soils regained most of their strength but it takes some time up to 4 months. Strength recovery can be expected when consolidation and increased density occur. Special cases where not recuperation in strength can be expected include highly cemented clays and clay soils that are highly overconsolidated before freezing. Closed-freeze thaw system (no water from external sources) and fast freezing cause an effect similar to that caused by increasing the overconsolidation ratio and reduce the effective consolidation stress; which causes a slight reduction in the post-thaw shear strength. Thaw weakening (or reduction in bearing capacity) of granular soils is influence by the mineralogy of clay fraction. Montmorillonite fines increase the potential for thaw weakening.

2.3.3 Stiffness (M_r) after freeze-thaw cycling

When fine-grained and coarse grained soil are freezing and thawing using a closed system, similar resilient modulus reduction percentages are experienced. Field data (open system) show higher resilient modulus reduction in fined-grained soils after freeze-thaw (as the percentage of fines increase, the resilient modulus decreases). Fined-grained soil shows the higher resilient modulus reduction after the first freeze-thaw cycle, subsequent cycles show insignificants effects. A volume increase in a sample after freeze-thaw cycle increases the void ratio and causes reduction in resilient modulus. As longer the soil takes to reach room temperature partial stiffness can be recovered and lower resilient modulus reduction can be experienced.

2.3.4 Stiffness (M_r) / Compressibility (qu) of stabilized soil with fly ash after freeze-thaw cycling

Aggregate base stabilized with a 10% Class C fly ash prepared at a water content of 7% after 28 days of curing and freezing-thawing using a closed system, showed a resilient modulus increase of 23% after 12 freeze-thaw cycles and a lower increase (15%) after 30 freeze-thaw cycles compared with a sample cured for 28 days and not subjected to freeze-thaw. A resilient modulus reduction of 8% was observed after 30 freeze-thaw cycles. The resilient modulus reduction is attributed to the fact that freezing temperatures (-25ºC) reduces the cementitious/ pozzolanic reactions.

The fact of freeze-thaw on resilient modulus of soil-fly ash mixtures is dominated by the amount of water available within void space and the detrimental effects on the cemetitious/pozzolanic reactions due to higher number of freeze-thaw cycles. Higher temperatures accelerate the pozzolanic reactions and lower temperatures reduce the pozzolanic reactions.

Visual observation of treated specimens (cement-treated, cement-treated with kaolinite, lime-treated, and lime-treated with kaolinite) did not reveal cracks or degradation. In summary, qu of stabilized samples after freeze-thaw cycles behavior depends on the temperature. Lower temperature reduce pozzolanic reactions, conversely thawing temperature accelerates the pozzolanic reactions. Also cohesive fines cause a reduction in the qu of the stabilized samples.

SECTION 3

MATERIALS AND METHODS

3.1 SOILS

3.1.1 Sources

Two subgrade soils, one riding surface gravel, and two recycled pavement materials (RPM) were used in the testing program. The two subgrade soils are an organic clay named Lawson from Hwy 11 Green County, WI and a clayey sand from USH 12 STA 614 in Fort Atkinson, WI. Locations where the subgrade soils were obtained are shown in Figs 3.1 and 3.2.

The riding surface gravel is a well-graded mixture of sand and gravel with fines and it is obtained from County Road 53 Rush City, MN. This material is a composite of riding surface gravel from a reconstruction project conducted along County Road 53 where the gravel road is converted to an asphalt paved road. Locations where the samples of County Road 53 gravel were collected (i.e., STA 10+00, 20+00, 27+30, 40+00, 50+00, 60+00, 70+00, and 104+00) are shown in Figs. 3.3.and 3.4.

One of the recycled pavement material is a mixture of pulverized asphalt, base course, and subgrade soil. Three samples of it were collected from 7th Avenue (STA 2 and STA 8) and 7th Street (STA 9) in the Waseca, MN. These are coarse-grained materials with varying amounts of fines. Locations where the recycled pavement materials were obtained are shown in Figs 3.3.and 3.5.

The second RPM consisting of pulverized existing asphalt surface layer mixed with the underlying base material was sampled from State Trunk Highway 144 West Bend, WI by Bloom Consultants LLC (See Fig. 3.1) and called STH 144 RPM. It is a granular material with negligible amount of fines.

3.1.2. Index Properties of Test Materials

Index properties, compaction properties, and classifications of the test soils are summarized in Table 3.1. Properties of the recycled pavement materials are summarized in Table 3.2. The soils classified as organic clay (Lawson), clayey sand with gravel (USH 12 STA 614 and Waseca STA 2 RPM), well graded sand with clay and gravel (County Road 53, Waseca STA 8 and Waseca STA 9 RPMs), and well-graded sand with gravel (STH 144 RPM).

Particle size distributions for the soils and RPMs are shown in Fig. 3.6 and Fig. 3.7. STH 144 RPM represents the coarsest material with only 4% of fines and Lawson represents the finest soil.

Compaction curves corresponding to standard Proctor effort were determined following the procedure in ASTM D 698. STH 144 RPM compaction curve was determined with the modified Proctor test following ASTM D 1557. The optimum water contents and maximum dry unit weight are summarized in Table 3.1 and 3.2. Bell-shaped compaction curves were obtained (Fig. 3.8). Among the soil, Lawson clay has the highest optimum water content (28%) and the lowest dry unit weight (13.3 kN/m^3) because of its organic content and high plasticity. County Road 53 gravel has the lowest water content (8.4%) and the highest dry unit weight (21.4 kN/m^3), which reflects the large fraction of coarse particles in the material. Among the recycled material, Waseca STA 9 RPM has the highest optimum water content (8.6%) and the lowest dry unit weight (18.6 kN/m^3). STH 144 RPM has the lowest water content (6.5%) and the highest dry unit weight (21.2 kN/m^3), which also reflects the large fraction of coarse particles in the material.

3.2 FLY ASHES

3.2.1 Sources

Five different fly ashes were used in this study: Columbia, Dewey, King, Riverside 7, and Riverside 8. Columbia fly ash is from Columbia Power Plant Unit #2 in Portage, WI. Dewey fly ash is from the Nelson Dewey Power Plant in Cassville, WI and King fly ash is from the Allen S. King Power Plant in Cassville, Wisconsin. Riverside 7 fly ash is from Riverside Power Plant Unit #7 in St. Paul, MN and Riverside 8 fly ash is from Riverside Power Plant Unit # 8 in St. Paul, MN. The Columbia Nelson Dewey Power Plants are operated by Alliant Energy. The Allen S. King Power Plant and the Riverside Unit #7 and Unit #8 Power Plants are operated by Xcel Energy. All five fly ashes (Columbia, Dewey, King, Riverside 7 and Riverside 8) are derived from combustion of sub-bituminous coal, were collected using electrostatic precipitators, and stored in dry silos. Columbia fly ash is from a pulverized boiler. The other four fly ashes (Dewey, King, Riverside 7 and Riverside 8) are from cyclone boilers.

Columbia, Dewey, and King fly ash were used to stabilize Lawson subgrade soil (20% by mass) based on Tastan et al. (2005). The USH 12 STA 614 subgrade soil (12% Columbia), County Road 53 composite gravel (10% Riverside 8), and

Waseca STA 2, STA 8, and STA 9 RPM (10% Riverside 7) were stabilized with the fly ashes and percentages used in the highway construction projects from which the soils were obtained. STH 144 RPM was stabilized with 10% and 14% of King fly ash. Fly ash percentages were based on dry weight of the soil.

3.2.2 Physical Properties and Chemical Composition

A photograph of the five fly ashes used in this study is shown in Fig. 3.9. Columbia, King, Riverside 7, and Riverside 8 fly ashes have a powdery texture. Dewey fly ash has a more granular texture. Both Columbia and Riverside 7 fly ashes are light brown in color. Dewey fly ash is dark gray and the King and Riverside 8 fly ashes are dark brown, which indicates higher amounts of carbon (Acosta et al. 2002).

Physical properties, chemical composition, and classification of the five fly ashes are summarized in Table 3.3, along with the typical chemical composition of Class C and Class F fly ash and the criteria in ASTM C 618 used to classify the fly ashes as Class C or Class F. Columbia and Riverside 7 fly ashes are classify as Class C following ASTM C 618, whereas Dewey, King, and Riverside8 fly ashes are referred to as "off-specification" fly ashes because they do not meet the Class C or Class F criteria described in ASTM C 618.

The CaO/SiO_2 ratio, which is indicative of cementing potential (Edil et al. 2006), fall between 0.75 (Columbia and Riverside 7) and 1.2 (King). The CaO/SiO_2 ratio can also be interpreted as $CaO/(SiO_2 + Al_2O_3)$ (Tastan, 2003). The $CaO/(SiO_2 + Al_2O_3)$ fly ash ratio are the following: Columbia (0.47), Dewey (0.61), King (0.66), Riverside 7 (0.47), and Riverside 8 (0.67). The pozzolanic activity or strength activity at 7 days minimum is for Columbia (95.8%), Dewey (82.7%), King (77.7%), Riverside 7 (108%), and Riverside 8 (87%). The loss on ignition, which is indicative of the amount of unburned coal in the fly ash, varies between 0.7% (Columbia) and 42.0% (Dewey).

3.3 TEST PROCEDURES

3.3.1 Sample Preparation

All of the soils and soil-fly ash mixtures tested in this study were compacted in cylindrical molds with a diameter of 101.6 mm and a height of 203.2 mm using a standard Proctor hammer. Specimens of soil alone were prepared as follows: (1) air-dried, (2) sieved or scalped, (3) blended with the corresponding amount of water until obtaining a uniform color, (4) sealed in a plastic bag and stored for 24 hours for water content equilibration, (5) compacted, (7) resilient modulus testing, and (8) unconfined compressive strength testing using the same specimen used for resilient modulus testing. The resilient modulus testing procedure followed AASHTO T 292-97(2000). Specimens of soil-fly ash mixtures were prepared as follows: (1) air-dried soil after sieved or scalped, was blended with the required percentage by weight of fly ash until the mixture had uniform color, (2) the soil-fly ash mixture was moistened with water to the target water content and blended until uniform, (3) the soil-fly ash mixture was allowed to sit for 2 hours to simulate the delay that typically occurs in the field and then were compacted, (4) after half of the layers were compacted a thermocouple was placed at the center of the sample and the compaction of the remaining layers was completed, (5) the specimens were sealed with plastic wrap and cured at 25°C in a 100% relative humidity room, (6) the freeze-thaw cycling was performed, (7) resilient modulus testing was performed, and (7) unconfined compressive strength test was performed on the same specimen.

The granular material-fly ash mixtures prepared with gravel from County Road 53, Waseca RPM, and STH 144 RPM were cured in the mold to provide an opportunity for cementation to form between the particles prior to extrusion since these materials lacked inherent cohesion prior to fly ash stabilization. After extrusion these granular material-fly ashes mixtures were soaked for 5 hours before the freeze-thaw cycling start. Most of the soil-fly ash mixtures were cured for 7 days (Lawson, USH 12 STA 614, County Road 53, and STH 144 RPM) (Tastan et al. 2005). However, the specimens from Waseca were cured for 14 days (Edil et al. 2006).

Soils and soil-fly ash mixtures from Lawson and USH 12 STA 614 were prepared using the same compactive effort as specimens prepared using the standard Proctor procedure. The compactive effort was matched by adjusting the number of blows per layer to provide the same energy per volume as standard Proctor compaction (600 kN-m/m^3). Specimens were compacted in 6 layers with 22 blows/layer (see Appendix A for calculations).

Specimens prepared with the Lawson and USH 12 STA 614 soils with and without fly ash were compacted at 7% wet of optimum water content (34.4% and 22.5% respectively) to simulate the natural wet conditions observed in the field in Wisconsin. These soils were sieved through No. 4 sieve before sample preparation. Soil-fly ash mixtures with the Lawson soil were prepared using three fly ashes: 20%

Columbia fly ash, 20% Dewey fly ash, and 20% King fly ash. The fly ash content was relatively high because of the organic content of Lawson soil. Soil-fly ash mixtures with soil from USH 12 STA 614 were prepared with 12% Columbia fly ash to match the field application. The fly ash percentages used with these soils were based on dry weight of the soil.

Composite gravel from County Road 53 was scalped using 3/4 in. sieve to fulfill with the maximum particle size criterion in AASHTO T 292-97(2000). Gravel and gravel-fly ash specimens prepared with base gravel from County Road 53 were compacted at a w% = 6.4% and γ_d = 19.3 kN/m^3 (averages for the stabilized gravel at County Road 53 after 7 days of curing time, see Appendix B for details) and stabilized with 10% Riverside 8 fly ash. Soil and soil-fly ash mixtures prepared with the RPM from Waseca were compacted at field conditions (w% = 8.5% and γ_d = 18.5 kN/m^3) (see Appendix C for details) and stabilized using 10% Riverside 7 fly ash. RPM-fly ash mixtures prepared with STH 144 RPM were compacted at optimum water content and maximum dry unit weight corresponding to modified Proctor effort based on all solids including the fly ash soils. STH 144 RPM + 10% King fly ash (w_{opt}% = 5.8% and γ_{dmax} = 21.2 kN/m^3) and STH 144 RPM + 14% King fly ash (w_{opt}% = 5.7% and γ_{dmax} = 21.1 kN/m^3).

3.3.2 Freeze-Thaw Procedure

3.3.2.1 Freezing-Point Depression Test

A freezing-point depression test following ASTM 5918 was performed on each material and their fly ash mixture to determine the temperature at which to freeze the specimens, except for STH 144 RPM. The freezing-point depression test identifies the temperature at which the water in the material begins to freeze. To be sure that the chosen temperature completely freezes the sample, a lower temperature than the freezing-point depression temperature was used in freezing cycles. The freezing-point depressions and selected freezing temperatures for each soil and soil-fly ash mixture are summarized in Table 3.4. All freezing-point depression graphs are in Appendix D. During the freezing-point depression test, the temperature was recorded every 1 minute.

3.3.2.2 Freeze-Thaw Cycles

The soil-fly ash mixtures were subjected to 0, 1, 3, 5, 10 and 12 cycles of freeze-thaw. Before the fly ash mixtures prepared with granular soil from County Road 53, and RPMs from Waseca and STH 144 were subject to the freeze-thaw cycling samples were soaked for 5 hours. Ten freeze-thaw cycles were applied only to Lawson + 20% Columbia fly ash. Twelve freeze-thaw cycles were applied only to STH 144 RPM + 10% King fly ash and STH 144 RPM + 14% King fly ash. Specimens prepared without fly ash were not subjected to freeze-thaw cycling. A flowchart describing the freeze-thaw cycles is presented in Fig. 3.10. Specimens were initially cooled to 5ºC so that all specimens would begin the freezing phase at uniform temperature. A thermocouple was placed at the center of each specimen and the temperature was recorded every 30 minutes using a data logger. Freeze-thaw cycle graphs are presented in Appendix E.

3.3.2.3 One-Dimensional (1-D) and Three-Dimensional (3-D) Freeze-Thaw Cycling

Freeze-thaw cycling of the soil-fly ash mixtures prepared with the Lawson and USH 12 STA 614 soils was conducted one-dimensionally (1-D) with the ends open and 100 mm of insulation around the specimen (see Fig. 3.11.). Specimens were placed horizontally to allow heat flow through the end. Before wrapping the samples with the insulation, samples were completely wrapped with plastic to prevent water content changes.

To verify that freezing and thawing was occurring in 1-D, a specimen was instrumented with 15 thermocouples. Three thermocouples were placed between each compacted layer (at the center and edges). The specimen was prepared at 7% wet of optimum moisture content and compacted in 6 layers and 22 blows per layer. During the test, temperatures were recorded every 1 minute. A drawing of the 1-D freeze-thaw test sample is shown in Fig. 3.12.

Graphs showing temperatures during the cooling, freezing, and thawing phases are in Fig. 3.13., 3.14., and 3.15. Cooling and thawing begins at the edges and moves toward the center until the temperatures are uniform. A different behavior was observed during freezing (from 5°C to -5°C ~ -20°C) (Fig. 3.14), with one half of the specimen reaching colder temperatures than the other half.

Samples prepared with materials from County Road 53, Waseca RPMs, and STH 144 RPMs were subjected to three-dimensional (3-D) freeze-thaw cycling. No insulation was used for the specimens frozen and thawed in 3-D. All other conditions were the same.

3.3.3. Resilient Modulus Test

The procedures described in AASHTO T 292-97 (2000) was followed for the resilient modulus tests. All specimens were tested using the loading sequence for cohesive soils, except for the unstabilized specimens prepared with material from County Road 53, which were tested using the loading sequence for granular materials. The test sequence for cohesive specimens is summarized in Table 3.5. The loading sequence for granular specimens is summarized in Table 3.6. A photograph of the resilient modulus cell in the loading frame is shown in Fig. 3.16. Detailed information about the procedure to calibrate the resilient modulus machine is described in Appendix F.

Several resilient modulus tests were conducted with a medium sand to assess the repeatability of the procedure. The sand is a medium, uniformly-graded quartz sand classified as SP (USCS classification) and A-3 (0) (AASHTO classification). It has a uniformity coefficient (C_u) of 1.3 and an effective grain size (D_{10}) of 0.45 mm. This sand has maximum and minimum units weights of 17.86 and 15.25 kN/m^3, respectively and a solid specific gravity of 2.65. It has an average grain roundness of 0.85 and an internal friction angle of 35º in the loose state. The procedure followed for these resilient modulus calibrations tests are described at Sawangsuriya et al.2003. Repeatability among the medium sand resilient modulus calibration tests was obtained as described in Appendix G.

3.3.4. Unconfined Compressive Strength Test

Unconfined compressive strength tests were conducted following the procedure in ASTM D 5102. ASTM D 5102 recommends that the strain rate should be between 0.5% and 2%/min. However, a strain of 0.21%/min was used because stabilized soils with fly ash were expected to be stiffer than un-stabilized soils (Acosta et al. 2002). This reduction in strain rate is consistent with Note 7 in ASTM D 5102, which suggests that stiffer specimens be tested at lower strain rates. The unconfined compression tests were performed on the resilient modulus specimens after the resilient modulus test. Resilient moduli test does not fail specimens because the applied stresses are typically are lower than the strength of the materials. However, the unconfined compression test results should be viewed as a relative measure of strength because there may be some degradation of specimens due to resilient modulus test loading.

SECTION 4

ANALYSIS AND RESULTS

4.1. VOLUME AND MOISTURE CHANGES DUE TO FREEZE-THAW CYCLING

4.1.1 Volume Change

The volume changes observed on the soil-fly ash and granular material-fly ash mixtures after freezing and thawing when soils are completely thawed before the resilient modulus test are summarized in Table 4.1. Additional details on the volume change data are in Appendix H.

Volume change is shown as a function of freeze-thaw cycling in Fig. 4.1. A general trend of volume increase in response to freeze-thaw cycling is observed with all the soil-fly ash and granular material-fly ash mixtures. An exception is the mixture prepared with Lawson and 20% Dewey fly ash which shows a volume decrease in respond to freeze-thaw cycling. All soil-fly ash and material-fly ash mixtures show a nearly linear volume change trend. The maximum volume increase was 2.7% showed by Waseca STA 9 RPM + 10% Riverside 7 fly ash after 5 freeze-thaw cycles. The maximum volume reduction was -2.7% showed by Lawson + 20% Dewey fly ash after 5 freeze-thaw cycles.

In Fig. 4.2 volume change of coarse grained material-fly ash mixtures and fine grained soil-fly ash mixtures are identified with different symbols. All coarse grained material-fly ash mixtures show an increase in volume with the freeze-thaw cycling. This volume increase is between 2-3% after 5 freeze-thaw cycles. All coarse grained material-fly ash mixtures show similar volume change after freeze-thaw cycles.

All the fine-grained soil-fly ash mixtures (with the exception of Lawson + 20% Dewey fly ash) show a volume reduction after 1 freeze-thaw cycle thereafter the volume starts increasing but still stays lower than the volume of the sample before freeze-thaw cycling. The fine-grained soil-fly ash mixtures volume change after 5 freeze-thaw cycles range between 1 and -3%. The higher volume increase is obtained when the fine-grained soils are stabilized with Columbia fly ash. Fig. 4.3 identifies fly ash content and shows that higher volume increment after freeze-thaw cycling was obtained with the lower percentage of fly ash.

Fig. 4.4 shows the volume change after freeze-thaw cycles of Lawson soil stabilized with three different fly ashes. The highest volume increase occurs when Lawson is stabilized with Columbia fly ash. Lawson stabilized with Dewey fly ash shows a completely different behavior, i.e., volume reduction with freeze-thaw cycles. Lawson soil stabilized with King fly ash like Columbia fly ash shows a reduction in volume after 1 freeze-thaw cycle thereafter the volume starts increasing.

Relatively low volume changes were observed because specimens were freezing and thawing in a closed system (no external sources of water available), with the only supply of water being that held within the pores of the materials and therefore frost action is limited to the change in volume of the in situ pore water upon freezing.

Cruz (2002) reported volume loss less than 4% after freeze-thaw cycles of fine grained soils stabilized with fly ash. Volume was measured after samples were completely thawed. A tendency of volume reduction as the number of freeze-thaw cycles increase was observed. On the other hand, Culley (1971) reported an increase in volume of a typical till subgrade as number of freeze-thaw increase in a range of 0.5% and 1.5% measured when samples are completely thawed. For samples at optimum water content or higher; the height of the sample did not return to its original value after initial thawing, a subsequent freeze-thaw cycling added to the increase in specimen height. Also Simonsen (2002) reported a net volume increase for coarse gravelly sand and fine sand after freeze-thaw cycles when samples were completely thawed resulting from a loose soil structure, because during thawing the soils particles do not fall back to exactly the same position which produces an increase in void ratio.

4.1.2. Moisture Content Change

A summary of the changes in moisture content of the soil-fly ash and granular-material fly ash mixtures is given in Table 4.2. Additional information about the changes in moisture contents are in Appendix I. An important factor to mention is that the granular material-fly ash mixtures were soaked before the start of the freeze-thaw cycles (County Road 53 + 10% Riverside 8 fly ash, and Waseca STA 8 RPM + 10% Riverside 7 fly ash). For these samples the water content change reported in Table 4.2 or Appendix I represent the water content increase resulting from soaking. Water content measurements were taken after sample preparation before soaking and after the freeze-thaw cycles where sample was initially soaked. For this reason an accurate moisture content change due to freeze-thaw cycles can not be calculated. Therefore, only the fine-grained soil-fly ash specimens data showed in Table 4.2 can be used to analyzed the effect moisture content change due to freeze-thaw cycling. In Fig. 4.5 shows the moisture content change vs. number of freeze-thaw cycles for the fined-grained soil-fly ash mixtures. Either no change or some drop in moisture content is observed. Lawson + 20% Columbia fly ash show a water content reduction of 2% after 10 freeze-thaw cycles and USH 12 STA 614 + 12% Columbia showed a constant water content, which demonstrated no effect on moisture content due to freeze-thaw cycling. The changes, when measured, are small and likely to reflect drying of the samples during testing, which is consistent with the fact that the specimens were subjected to freeze-thaw using a closed system (no external source of water available).

4.2. FREEZING POINT DEPRESSION

The depression of the freezing point of the water in soil is due to the effects of particle size, mineralogy, and chemistry. Generally, the temperature at which water begins to freeze decreases with an increasing amount of fine-grained particles as described in ASTM D 5918. The addition of salts also decreases the freezing point of soil water. The freezing point temperatures for the tested materials are summarized in Table 4.3. Freezing point depression graphs are in Appendix D.

Comparison of the freezing point temperatures in Table 4.3 shows a general trend of decrease when soils and granular materials are mixed with fly ash (See Fig. 4.6). Only Waseca STA 9 RPM show an increase of 0.4ºC in the freezing point when it is mixed with 10% Riverside 7 fly ash. In general, when fine- grained soils are mixed with fly ash an average reduction of 2 ºC in the freezing point was observed. When coarse-grained material are mixed with fly ash an average reduction of 3.2 ºC in the freezing point was observed. The fact that the freezing point decrease with an increasing amount of fines was not observed. The reduction of freezing point does not exhibit any relationship to particle size, fly ash content, and fly ash type (See Fig. 4.7 to 4.10).

4.3 RESILIENT MODULUS AFTER FREEZE-THAW CYCLES

An objective of this research was to study how the resilient modulus and unconfined compressive strength of soils stabilized with fly ash change after freeze-thaw cycling. To reach this objective, resilient modulus and unconfined compression tests were conducted on a range of fly ash stabilized materials after freeze-thaw cycling (0, 1, 3, 5, 10, and 12 cycles). The stabilized materials tested included fine-grained soil, coarse-grained soil, and recycled pavement material. Five different fly ashes were used [Columbia and Riverside 7 (classified as Class C); Dewey, King and Riverside 8 (classified as off-specification)] at different percentages (10%, 12%, 14% and 20%) and at three different water contents (7% wet of optimum, optimum, and at field water content). Tests were also conducted on soil alone (0% fly ash) without freeze-thaw cycling to define the reference condition. A summary of the test conditions is given in Table 4.4, also indicating the replicate tests performed.

4.3.1 RESILIENT MODULUS

The resilient modulus of soils is a non-linear relative to stress conditions and depends on the stress level. Typically, the non-linear behavior is modeled as a power function of the deviator stress (σ_d) (for cohesive soils) or the bulk stress (σ_b) (for granular soils). For cohesive soils, the relationship between resilient modulus (M_r) and deviator stress (σ_d) is characterized as:

$$M_r = K_1(\sigma_d)^{K_2} \tag{4.1}$$

where K_1 and K_2 are coefficients. For granular soils, the relationship between resilient modulus (M_r) and bulk stress (σ_b) is characterized as:

$$M_r = K_1(\sigma_b)^{K_2} \tag{4.2}$$

where K_1 and K_2 are coefficients and σ_b is the sum of the principal stresses. The bulk stress is equal to:

$$\sigma_b = \sigma_d + 3\sigma_c \tag{4.3}$$

where σ_c is the confining pressure applied during the resilient modulus test.

The coefficients K_1 and K_2 for all of the tests are summarized in Table 4.4. The resilient modulus at a deviator stress of 21 kPa is often used for comparison. A summary of the resilient moduli at 21 kPa is given in Table 4.5. Normalized resilient moduli after the freeze-thaw cycling (using the unfrozen modulus for normalization) are shown in Table 4.6. The resilient modulus of the cohesive soils and soil-fly ash mixtures shown in Tables 4.4, 4.5, and 4.6 corresponds to the first deviator stress (21 kPa) applied to the specimens, which also corresponds to the deviator stress expected in situ. The resilient modulus of the granular materials shown in Tables 4.4 and 4.5 corresponds to the first bulk stress applied to the specimens. For some specimens the bulk stress is 70 kPa and for other 89 kPa, the applied bulk stress is identified in the tables. .

4.3.2. General behavior of soil-fly ash and granular material-fly ash mixtures after freeze-thaw cycles

The effect of freeze-thaw cycling on resilient modulus is shown in Fig. 4.11 in terms of the normalized residual modulus. For all of the mixtures, the resilient modulus decreases in response to freeze-thaw cycling and then appears to level off in approximately 1 to 5 cycles. The drop in modulus ranges between 7% and 50%. The average drop in resilient modulus is 28.5%. From this graph it can be concluded that for highway design, the safest way to represent the effect of freeze-thaw cycling on the resilient modulus of the soil-fly ash or granular material- fly ash mixtures is dividing the value by 2. Similar behavior showing resilient modulus reduction after freeze-thaw cycles have been reported based on laboratory tests and field data. For example, Simonsen et al. (2002) reported resilient modulus reduction between 20-60% on various unstabilized coarse and fine-grained soils after 1 freeze-thaw cycle. Culley (1971) also reported resilient modulus reduction on unstabilized clay till after 3 freeze-thaw cycles at different water contents and standard densities in a range between 41-66%. Lee et al. (1995) also reported resilient modulus reduction between 30 – 50% on a compacted fine-grained soil after 1 freeze-thaw cycle, the subsequent two freeze-thaw cycles showed insignificant effects. Also Jong et al. (1998) reported field data after a winter (18 months) showing a resilient modulus reduction in the base of 35% and of 65% on the subgrade compare with the pre-freezing value. In summary, based on the studies mentioned above, resilient modulus reduction of various unstabilized coarse and fine-grained soils after 1-3 freeze-thaw cycles range between 20 – 66%. In this study, soil-fly ash or granular material-fly ash mixtures showed lower resilient modulus reduction after higher number of freeze-thaw cycles (7%-50%) which can be attributed to the addition of fly ash.

A different behavior showed the mixture prepared with soil from USH 12 STA 614 and 12% Columbia fly ash. The modulus of this mixture increased nearly linearly as the number of freeze-thaw cycles increased. This is the only mixture where the modulus increased in response to freeze-thaw cycling. Similar behavior was reported by Zaman et al. 2003. In his study, evaluating the effect of freeze-thaw cycling on aggregate base specimen stabilized with 10% Class C fly ash, an increase is reported in resilient modulus after 28 days of curing up to 12 freeze-thaw cycles. For samples cured only for 3 days, resilient modulus increased up to 30 freeze-thaw cycles. They reported an increase in resilient modulus in a range between 15%-80% compared the same soil-fly ash mixture without being subject to freeze-thaw cycling. The resilient modulus increase is attributed to the thawing temperature (above 4ºC) which accelerates the cementitious/pozzolanic reactions as reported by NCHRP (1976). NCHRP (National Cooperative Highway Research) (1976) reported that higher temperatures (above 4ºC) accelerate the pozzolanic/cementitious reactions and lower temperatures retard the reactions. For the USH 12 STA 614 and 12% Columbia fly ash mixtures this attribution can be applied. These mixtures were exposed longer time to 5ºC (thawing phases) because they were the samples that took longer time to change its temperature from freezing to thawing 5ºC (see Appendix E Fig. E.2). Given the soil-fly ash mixture more time than the other mixtures in the study to recover strength and for additional pozzolanic reactions between soils and fly ash particles to develop.

4.3.2.1. Effect of freeze-thaw cycles on soil type

In Fig. 4.12 the resilient modulus behavior of fine-grained soils are represent by open circles and of coarser materials are represent by solid circles. With the exception of USH 12 STA 614 and 12% Columbia fly ash, the drop in resilient modulus of the fine-grained soils mixtures ranges between 9% and 50% with an average of 29.5%. The resilient modulus decreased in response to freeze-thaw cycling then appears to level off in approximately 1 to 3 cycles. In based on unstabilized fine-grained soil data reported by Simonsen et al. (2002), Culley (1971), Lee et al. (1995), and Jong et al. (1998) resilient modulus reduce in a range between 27% - 66% after 1 to 3 freeze-thaw cycles, with an average of 47%. In this study, less resilient modulus reductions were obtained after more numbers of freeze-thaw cycles than the reported values and can also be attributed to the stabilization with fly ash. The reduction on stiffness during the first 1-3 freeze-thaw cycles can be attributed to that the freezing temperatures dominate and retard the cementitious reaction, thereafter both freezing and thawing temperatures compensates each other and the variation in stiffness becomes minimal. As discussed earlier, USH 12 STA 614 and 12% Columbia fly ash shows a different behavior the modulus increased nearly linearly as the number of freeze-thaw cycles increased, showing an increment of 156% after 5 freeze-thaw cycles.

For coarse material-fly ash mixtures, the drop in modulus ranges between 7 to 42%, with an average of 24.5% For all the coarse material mixtures, the resilient modulus decreases in response to freeze-thaw cycling and then appears to level off in approximately 1 to 5 cycles. Data reported by Simonsen et al. (2002) on unstabilized coarse-grained soils indicated a reduction in resilient modulus after 1 freeze-thaw cycle between 19% - 50%, with an average of 34%. Less reduction in stiffness were obtained in this study after more freeze-thaw cycles, which can be attributed to the stabilization with fly ash. The reduction of stiffness during the first 1-3 freeze-thaw cycles can be attributed to the retardation of pozzolanic reactions. The freezing temperatures dominate and retard the cementitious reaction, thereafter both freezing and thawing temperatures compensate each other and the variation in stiffness becomes minimal.

Fine-grained soil-fly ash mixtures demonstrated an average drop in resilient modulus with freeze-thaw cycles of 5% more than coarse material mixtures, which is not a big difference. On the other hand, resilient modulus reduction of fine-grained soil-fly ash mixtures level off in less freeze-thaw cycles (1-3) than the coarse materials mixtures (1-5).

The RPM–fly ash mixtures show a reduction in M_r as the number of fines increases (See Fig. 4.13). STH 144 RPM + 10% King fly ash (4% of fines), STH 144 RPM + 14% King fly ash (4% of fines), Waseca STA 8 RPM + 10% Riverside 7 fly ash (7% of fines), and Waseca STA 9 RPM + 10% Riverside 7 fly ash (9% of fines) show the following resilient modulus reduction after 5 freeze-thaw cycles of approximately 19%, 29%, 33%, 43%, respectively.

4.3.2.2. Effect of freeze-thaw cycles on fly ash type

The resilient modulus reduction of the three Lawson (organic clay)-fly ashes mixtures ranges between 9 to 50%, with an average reduction of 29.5%. For all mixtures the resilient modulus decreased in response to freeze-thaw cycling then appears to level off in approximately 1 to 3 cycles. Increased reduction in resilient modulus with freeze-thaw cycles in response to lower lime (CaO) in the fly ash content is evident. A drop in resilient modulus of about 50% was obtained when Lawson is mixed with Dewey fly ash which contains the lowest CaO content of 9.2%. And only a drop in resilient modulus of 12% was obtained when Lawson is mixed with King fly ash which contains the highest CaO content of 25.8% (see Fig. 4.14).

In Fig. 4.15, it is demonstrated that the resilient modulus reduction with freeze-thaw cycles depends on the amount of lime (CaO) percentage in the fly ash. Among the fine grained soil–fly ash mixtures and coarse material-fly ash mixtures where the resilient modulus reduction level off in approximately 1-5 freeze-thaw cycles showed lower resilient modulus reduction with freeze-thaw cycling when the fly ash has higher lime (CaO) percentage. Mixtures stabilized with King fly ash

(CaO = 25.8%) showed a drop in resilient modulus between (11-31%), mixture stabilized with Riverside 7 fly ash (CaO = 24.0%) a resilient modulus drop 32%, mixture stabilized with Columbia fly ash (CaO = 23.3%) a resilient modulus drop 42% and the mixture stabilized with Dewey fly ash (CaO = 9.2%) a resilient modulus drop of about 50%.

Also RPM-fly ash mixtures showed less resilient modulus reduction after freeze-thaw cycles when are stabilized with a fly ash having a higher CaO content. RPM stabilized with King fly ash (CaO = 25.8%) shows a resilient modulus decrease between 19-29% and RPM stabilized with Riverside 7 fly ash (CaO=24.0%) showed a decrease in the range of 33-43% (see Fig. 4.13). STH 144 RPM stabilized with Dewey fly ash (CaO = 9.2%) disintegrated before testing, only one sample stabilized with 14% of Dewey fly ash survived 12 freeze-thaw cycles given a resilient modulus of 41.7 MPa resulting in 46% lower than the resilient modulus of the specimen stabilized with 14% King fly ash (CaO = 25.8%) after 12 freeze-thaw cycles.

To see if the classification of the fly ash affects the resilient modulus after freeze-thaw cycles in Fig. 4.16 the mixtures stabilized with Class C fly ash were identified with closed symbols and mixtures stabilized with off-specification fly ashes were identified with opened symbols. Through this figure clearly it can be observed that no relation exists between the freeze-thaw effect and fly ash classification

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4.3.2.3. Freeze-thaw cycles effect on the fly ash content

Fig. 4.17 demonstrated that change in the resilient modulus after freeze-thaw cycles does not depend on the amount of fly ash. Granular materials mixed with fly ash percentages between 10 - 14% showed a resilient modulus variation between 7 – 42%. The resilient modulus of these mixtures decreases in response to freeze-thaw cycling and then appears to level off in approximately 1 to 5 cycles. Organic soil mixed with 20% of fly ash showed a resilient modulus variation between 9- 50%. The resilient modulus of this organic soil-fly ash mixtures decreases in response to freeze-thaw cycling and then appears to level off in approximately 1 to 3 cycles.

In Fig. 4.18 is shown the normalized resilient modulus vs. freeze-thaw cycles for the STH 144-King fly ash mixtures with different fly ash percentages (10% and 14%). For both mixtures the resilient modulus decreased in response to freeze-thaw cycling then appears to level off in 5 cycles. With this coarse grained material a difference of about 10.5% in resilient modulus was obtained. Higher resilient modulus was obtained with 10% of King fly ash. But basically similar resilient modulus values were obtained can be explained by the small difference in fly ash percentage that was used. More tests using same soil and same fly ash only changing the fly ash percentages are necessary to clarify if change in resilient modulus after freeze-thaw cycles depends on the fly ash content.

4.3.3. Resilient modulus of stabilized samples after freeze-thaw cycles compared to the unfrozen soils without fly ash

In Fig. 4.19 resilient modulus after the last freeze-thaw cycle compared to the resilient modulus of unfrozen soils without fly ash are presented. A general trend of higher resilient modulus when soils are stabilized with fly ash even after freeze-thaw cycles compared to unstabilized soils without freeze-thaw cycles is clearly observed.

The increase in resilient modulus of the stabilized soil and granular materials after freeze-thaw cycles range between 168% and 56% compared to unfrozen but unstabilized material. Recycled pavement material from Waseca STA 9 and STH144 showed decrease in resilient modulus after freeze-thaw in a range of -2% and – -15%.

4.4 UNCONFINED COMPRESSIVE STRENGTH AFTER FREEZE-THAW CYCLES

4.4.1. General behavior of soil-fly ash and granular soil/material-fly ash mixtures after freeze-thaw cycles

The unconfined compressive strengths are summarized in Table 4.7. Normalized unconfined compressive strengths of the soil-fly ash mixtures are summarized in Table 4.8. Normalization consisted of dividing the strength of the mixture after freeze-thaw cycling by the strength before freeze-thaw cycling.

Normalized unconfined compressive strength is shown vs. number of freeze-thaw cycles in Figs. 4.20, 4.21, and 4.22. Different responses are observed due to freeze-thaw cycles. The unconfined compressive strength remains essentially unaffected by freeze-thaw cycling (Lawson + 20% King fly ash, County Road 53 + 10% Riverside 8 fly ash, Waseca STA 9 RPM + 10% Riverside 7 fly ash, and STH 144 + 14% King fly ash), remains unaffected until 3 to 5 freeze-thaw cycles and then starts dropping (USH 12 STA 614 + 12% Columbia fly ash, Lawson + 20% Dewey fly ash, and STH 144 + 10% King fly ash), decreases until 3 freeze-thaw cycles and then levels off (Lawson + 20% Columbia fly ash), and decreases as freeze-thaw cycles starts (Waseca STA 8 + 10% Riverside 8 fly ash). In general, a reduction in unconfined compressive strength after freeze-thaw cycles up to 70% was observed.

4.4.1.1. Effect of freeze-thaw cycles on soil type

In Fig. 4.23 fine-grained soil are identified with open symbols and coarse-grained soils or materials are identified with closed symbols. The fine-grained soils show the following behavior (no effect with freeze-thaw cycling, no effect with freeze-thaw cycling until 3 cycles then strength starts dropping, or drop in strength until 3 freeze-thaw cycles and then level off). The coarse grained soils show the following behavior (no effect with freeze-thaw cycling, no effect with freeze-thaw cycling until 5 cycles then strength starts dropping, or strength continue decrease since the first cycle). This indicates that there exist some differences in the behavior trends between fine-grained soils and coarse-grained soils/materials. Fine-grained soils show an increase in strength up to 20% and a reduction up to 40%. Coarse-grained soils/materials show an increase in strength up to 20% and a decrease up to 70%. Higher reductions were obtained with coarse grained soils.

Different unconfined compressive strength behaviors on soil-fly ash mixtures after freeze-thaw cycling have been published. Coarse-grained soil stabilized with class C fly ash cured for 28 and 3 days show an increase in unconfined compressive strength as number of freeze-thaw cycles increased (Zaman et al. 2003). Different behaviors trends have been also observed with sandy soil (Arora et al. 2003). Sandy soil stabilized with Class F fly ash and cement show qu gain gradually up to 4 freeze-thaw cycles and nearly constant between 4 to 12 cycles. Sandy soil stabilized with Class F fly ash and lime strength qu reduce gradually 17% up to 12 freeze-thaw cycles. Sandy soil stabilized with Class F fly ash, lime, and kaolinite qu pick up 47% after 2 cycles, then gradually reduce 59% up to 12 cycles. Sandy soil stabilize with Class F fly ash, cement, and kaolinite qu reduced approximately 27% after 2 freeze-thaw cycles and then stays constant until 12 freeze-thaw cycles. In these studies, reduction in qu are attributed to the freezing action which retards the cementitious reaction, conversely increase in qu is attributed to the thawing action which accelerates the cementitious reaction. And when no or minimal variation in qu is observed, which is attributed to that both the freezing and thawing action compensated each other

4.4.1.2. Effect of freeze-thaw cycles on fly ash type

Fine-grained soils showed increase or less reduction in qu with freeze-thaw cycles when are stabilized with high CaO content fly ashes. Also Lawson showed this same pattern see Fig 4.23. Lawson stabilized with three different fly ashes. With each fly ash a different behavior was observed. Strength increases 5% or no effects with freeze-thaw cycle are observed when the soil was stabilized with King fly ash, which has the higher CaO content 25.8%. The largest reduction after 5 freeze-thaw cycles of 40% was experienced when soil was stabilized with Dewey fly ash which has the lowest CaO content (9.2%).

Coarse-grained soils and/or RPMs showed increase or less reduction in qu after freeze-thaw cycles when are stabilized with high $CaO/(SiO_2+Al_2O_3)$ content (King and Riverside 8 fly ashes, $CaO/(SiO_2+Al_2O_3)$ = 0.7%). The RPM stabilized with the fly ash with the lowest $CaO/(SiO_2+Al_2O_3)$ content of 0.5 showed the highest qu reduction and a continue decrease since the first freeze-thaw cycle.

4.4.1.3. Freeze-thaw cycles effect on the fly ash content

Fig. 4.25 showed stabilized STH 144 RPM behavior after freeze-thaw cycles with different percentages of King fly ash. Two different behaviors where observed, with 14% of fly ash qu increased 5% and stayed constant after the freeze-thaw cycles. With 10% of fly ash no changes in qu were observed up to 5 freeze-thaw cycles thereafter start to reduce decreasing 40%.

In Fig. 4.26 all soil/materials stabilized with 10-14% fly ash are represented with open symbols, and with 20% fly ash are representing by closed symbols. No relation was observed between fly ash percentage and qu behavior after freeze-thaw cycles.

4.4.1.4 Q_u of stabilized samples after freeze-thaw cycles compared to the unfrozen samples without fly ash

USH 12 STA 614 showed a qu 157% higher, when is stabilized with fly ash and after 5 freeze-thaw cycles. Qu of Lawson stabilized with Columbia fly ash increased 9.3%, with King fly ash increased 6.6%, and with Dewey fly ash decreased 13%, when is stabilized with fly ash and after 5 freeze-thaw cycles (see Fig. 4.27). Qu test could not be performed with the coarse soil/material because of loose soils particles. Basically stabilizing the fine soils with fly ash of high CaO content increases the soil strength making it resistant to freeze-thaw cycles.

4.5. ISSUES FOR FREEZE-THAW CYCLING

4.5.1. One-Dimensional vs. Three Dimensional Freeze-Thaw Cycling

On Figs. 4.28 and 4.29 samples freezing-thawing in one direction (1-D) and in 3 directions (3-D) are identified. In this study only the fine-grained soil (Lawson samples and USH 12 STA 614) were subject to freeze thaw cycle in 1-D. In Table 4.9 is summarize each sample freeze-thaw cycle time. Freezing-thawing samples in 1-D take longer time than freezing and thawing in 3-D. No difference in the trends of normalized M_r and normalized q_u vs. freeze-thaw cycles attributed to dimensionality is observed. Then for future testing it is recommended to freeze-thaw samples in 3-D to save time.

SECTION 5

CONCLUSIONS

The objective of this research was to study how the resilient modulus and unconfined compressive strength of soils stabilized with fly ash change after freeze-thaw cycling. To reach this objective, resilient modulus and unconfined compression tests were conducted on a range of fly ash stabilized materials after freeze-thaw cycling (0, 1, 3, 5, 10, and 12 cycles). The stabilized materials tested included fine-grained soil, coarse-grained soil, and recycled pavement material. Five different fly ashes were used [Columbia and Riverside 7 (classified as Class C); Dewey, King and Riverside 8 (classified as off-specification)] at different percentages (10%, 12%, 14% and 20%) and at three different water contents (7% wet of optimum, optimum, and at field water content). Tests were also conducted on soil alone (0% fly ash) without freeze-thaw cycling to define the reference condition.

5.1 VOLUME CHANGE AFTER FREEZE-THAW CYCLES

All coarse-grained soils and recycled pavement materials (RPMs) showed an increase in volume with freeze-thaw cycles in a range of 1.6% to 2.8%. Most of the fine-grained soils shrink after the first freeze-thaw cycle showing a reduction in volume of 1% to 2% and subsequent cycles added to the increase in specimen volume. This increase in volume with the freeze-thaw cycles results from a loose soil structure produced when soils thawed (ice melts) and the soil particles do not fall back to the exactly same position. This particle redistribution increases the specimen void ratio producing a reduction in strength and stiffness of the specimen when pressure is applied.

Low volume changes were observed because specimens were freezing and thawing in a closed system (no external sources of water available), with the only supply of water being that held within the pores of the soils or material and therefore frost action (frost heave and thaw weakening) is limited to the change in volume of in the in situ pore water upon freezing.

5.2 MOISTURE CONTENT CHANGE AFTER FREEZE-THAW CYCLES

The moisture content reductions after freeze-thaw cycling are small (1% to 3% lower) and likely to reflect drying of the samples during testing, which is consistent with the fact that the specimens were subject to freeze-thaw cycling using a closed system (no external source of water available).

5.3 FREEZING POINT DEPRESSION

Freezing point temperatures shows a general trend of decrease when soils or granular materials are mixed with fly ash. When fine-grained soils are mixed with fly ash an average reduction of 2ºC in the freezing point was observed. When coarse-grained soils or materials are mixed with fly ash an average reduction of 3.2ºC in the freezing point was observed. The fact that the freezing point decrease with an increase amount of fines was not observed.

5.4 RESILIENT MODULUS AFTER FREEZE-THAW CYCLES

For all the mixtures, with an exception of USH 12 STA 614 + 12% Columbia fly ash, the resilient modulus (M_r) decreases in response to freeze-thaw and then appears to level off in approximately 1 to 5 cycles. The drop in modulus ranges between 7 and 50%; with an average of 28.5%. From these results can be concluded that for highway design, the safest way to represent the freeze-thaw cycling on the resilient modulus of the soil-fly ash or granular-material fly ash mixtures is dividing the value by 2.

Recycled pavement materials (RPMs)-fly ash mixtures show a M_r reduction after freeze-thaw cycling as the percentage of fines increased. Lower M_r reduction after freeze-thaw cycling were obtained when soil-fly ash mixtures and coarse-grained material – fly ash mixtures are stabilized with high CaO content fly ashes.

Previous research published that reduction on stiffness after freeze-thaw cycles can be attributed to that the freezing temperatures dominate and retard the cementitious/ pozzolanic reactions. When no variation or minimal variation in stiffness is observed after freeze-thaw cycles is attributed to that the freezing and thawing temperatures compensates each other producing a balance in the cementitious/pozzolanic reactions. And increase on stiffness after freeze-thaw cycles is attributed to that the thawing temperatures dominates and accelerates the cementitious/pozzolanic reactions.

A general trend of higher resilient modulus (156% to 56% higher) when soils are stabilized with fly ash even after freeze-thaw cycles is clearly observed.

5.5 UNCONFINED COMPRESSIVE STRENGHT AFTER FREEZE-THAW CYCLES

In general, a reduction in unconfined compressive strength (qu) after freeze-thaw cycles up to 70% was obtained. Different qu behavior trends were observed:

- unaffected with freeze-thaw cycling
- unaffected up to 3 to 5 freeze-thaw cycles then strength starts dropping
- drop in strength up to 3 freeze-thaw cycles and then levels off (fine-grained soils only)
- continue decrease in strength since the first freeze-thaw cycle (RPM only)

Higher qu reduction after freeze-thaw cycles were experienced by RPMs.

Fine-grained soils showed increase or less reduction in qu with freeze-thaw cycles when are stabilized with high CaO content fly ashes. Coarse-grained soil and RPMs showed increase or less reduction in qu after freeze-thaw cycles when are stabilized with $CaO/(SiO_2+Al_2O_3)$ content fly ashes.

Previous research published that reduction on stiffness after freeze-thaw cycles can be attributed to that the freezing temperatures dominate and retard the cementitious/ pozzolanic reactions. When no variation or minimal variation in stiffness is observed after freeze-thaw cycles is attributed to that the freezing and thawing temperatures compensates each other producing a balance in the cementitious/pozzolanic reactions. And increase on stiffness after freeze-thaw cycles is attributed to that the thawing temperatures dominates and accelerates the cementitious/pozzolanic reactions.

A general trend of higher unconfined compressive strength (between 157% and 9.3%) when fine-grained soils are stabilized with high CaO content fly ashes even after freeze-thaw cycles is clearly observed. Qu test could not be performed on coarse- grained soil and RPMs because are loose material.

5.6. ISSUES FOR FREEZE-THAW CYCLING

Freezing-thawing samples in 1-D take longer time than freezing-thawing samples in 3-D. No difference in trends of normalized M_r or normalized qu vs. freeze-thaw cycles attributed to dimensionality was observed. Then for future testing it is recommended to freeze-thaw samples in 3-D to save time.

SECTION 6

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TABLES

Table 2. 1. U.S. Army Corps of Engineers frost design soil classification system

Frost-Susceptibility[a]	**Frost Group**	**Kind of Soil**	**Amount finer than 0.02 mm (wt%)**	**Typical soil type under USCS**[b]
Negligible to low	NFS[c]	Gravels	0 - 1.5	GW, GP
		Sands	0 - 3	SW, SP
Possible	PFS[d]	Gravels	1.5 - 3	GW, GP
		Sands	3 - 10	SW, SP
Low to medium	S1	Gravels	3 - 6	GW, GP, GW-GM, GP-GM
Very low to high	S2	Sands	3 - 6	SW, SP, SW-SM, SP-SM
Very low to high	F1	Gravels	6 -10	GM, GW-GM, GP-GM
Medium to high	F2	Gravels	10 - 20	GM, GM-GC, GW-GM, GP-GM
Very low to very high	F2	Sands		SM, SW-SM, SP-SM
Medium to high	F3	Gravels	> 20	GM, GC
Low to high	F3	Sands except very fine silty sands	> 15	SM, SC
Very low to very high	F3	Clays, I_p>12	---	CL, CH
Low to very high	F4	All silts	---	ML, MH
Very low to high	F4	Very fine silty sands	> 15	SM
Low to very high	F4	Clays, I_p>12	---	CL, CL-ML
Very low to very high	F4	Varved clays and other fine-grained banded sediments	---	CL and ML; CL, ML and SM; CL, CH, and ML; CL, CH, ML, and SM

[a] Based on laboratory frost-heave test
[b] G-gravel, S-sand, M-silt, C-clay, W-well graded, H high plasticity, L - low plasticity.
[c] Non-frost susceptible
[d] Requires laboratory frost-heave test to determine frost susceptibility.

Table 2. 2. Preliminary frost-susceptibility criteria for the new freezing test

Frost-susceptibility classification	**Heave rate (mm/day)**	**Thaw CBR (%)**
Negligible	< 1	> 20
Very low	1 - 2	20 - 15
Low	2 - 4	15 – 10
Medium	4 - 8	10 - 5
High	8 - 16	10 - 5
Very high	> 16	< 2

Table 3. 1. Index properties of the soils

Soil Name	Sampling Location	LL	PI	Percent Fines	G_s	OC (%)	Classification		pH		γ_d (kN/m³)	w_{OPT} (%)
							USCS	AASHTO	ASTM D 4972	ASTM D 2976		
Lawson	Hwy 11 Green County, WI	50	19	97	2.58	5	OH	A-7-5	6.9	6.8	13.3	27.4
USH 12 STA 614+00	Fort Atkinson, WI	26	11	34	2.72	---	SC	A-2-6	---	---	16.5	15.5
Composite County Road 53	City of Chisago, MN	---	---	11	2.75	---	SW-SC	A-1-b	---	---	21.4	8.4

Notes: LL = Liquid limit, PI = Plasticity index, Percent Fines = percentage passing No. 200 sieve, G_s = Specific gravity, OC = Organic content, γ_d = Maximum dry unit weight, w_{OPT} = Optimum water content.

Table 3. 2. Recycled pavement material and recycled asphalt pavement properties.

Soil Name	Location	Gravel %	Sand %	Fines %	Classification		w_N (%)	γ_d (kN/m^3)
					USCS	AASHTO		
Waseca STA 2 RPM	7th avenue 700 Waseca, MN	34	54	12	SC	A-1-a	6.6	19.9
Waseca STA 8 RPM	7th avenue 508 Waseca, MN	30	63	7	SW-SC	A-1-b	---	---
Waseca STA 9 RPM Lysimeter Location	7th street 701 Waseca, MN	35	56	9	SW-SC	A-1-b	8.6	18.6
STH 144 RPM	State Hwy 144 West Bend, WI	46	51	4	SW	A-1-a	6.5[a]	21.2[a]

Notes: w_N = in situ water content and γ_d = in situ dry unit weight. [a] Determined in accordance with ASTM D-1557, modified proctor test, by Bloom Consultants, LLC.

Table 3. 3 Physical properties and chemical composition of the fly ashes.

Parameter	Percent of Composition							Specifications	
	Columbia*	Dewey*	King*	Riverside 7**	Riverside 8**	Typical Class C***	Typical Class F***	ASTM C 618 Class C	ASTM C 618 Class F
SiO_2 (silicon dioxide), %	31.1	8.0	24.0	32.0	19.0	39.9	54.9		
Al_2O_3 (aluminum oxide), %	18.3	7.0	15.0	19.0	14.0	16.7	25.8		
Fe_2O_3 (iron oxide), %	6.1	2.6	6.0	6.0	5.9	5.8	6.9		
$SiO_2 + Al_2O_3 + Fe_2O_3$, %	55.5	17.6	45.0	57.0	38.8	62.4	87.6	50 Min	70 Min
CaO (calcium oxide), %	23.3	9.2	25.8	24.0	22.4	24.3	9.7		
MgO (magnesium oxide), %	3.7	2.4	5.3	6.0	5.5	4.6	1.8		
SO_3 (sulfur trioxide), %	3.7	-----	-----	2.0	5.4	3.3	0.6	5.0 Max	4.0 Max
CaO/SiO_2	0.75	1.15	1.08	0.75	1.18	0.61	0.18		
pH	12.8	9.9	10.9	-----	-----	-----	-----		
Loss on Ignition, %	0.7	42.0	12.0	0.9	16.4	6.0	6.0	6.0 Max	10.0 Max
Moisture Content, %	-----	-----	-----	0.17	0.32	-----	-----	3.0 Max	4.0 Max
Specific Gravity	2.63	2.00	2.66	2.71	2.65	-----	-----	5 Max	5 Max
Fineness, amount retained on # 325 sieve, %	12.0	27.0	41.0	12.4	15.5	-----	-----	34 Max	34 Max
Classification	***C***	***Off-Spec***	***Off-Spec***	***C***	***Off-Spec***	***C***	***F***	*C*	*F*

* Tastan et al. (2005) ** Lafarge North America ***Bin-Shafique et al. (2004)

Table 3. 4. Freezing-point depressions and freezing temperatures.

Soil Combination	**Freezing Point Depression (ºC)**	**Freezing Temperature (ºC)**
Lawson	0	---
Lawson + 20% Columbia Fly Ash	-1.9	-5 ~ -12
Lawson + 20% Dewey Fly Ash	-0.1	-5 ~ -12
Lawson + 20% King Fly Ash	-1.6	-5 ~ -12
USH 12 STA 614	-0.8	---
USH 12 STA 614 + 12% Columbia Fly Ash	-1.2	-12
County Road 53	-11	---
County Road 53 + 10% Riverside 8 Fly Ash	-12	-15
Waseca STA 2 RPM	-8.6	---
Waseca STA 2 + 10% Riverside 7 Fly Ash	-8.7	-15
Waseca STA 8 RPM	-0.9	---
Waseca STA 8 RPM + 10% Riverside 7 Fly Ash	-9.4	-15
Waseca STA 9 RPM	-10	---
Waseca STA 9 RPM + 10% Riverside 7 Fly Ash	-9.6	-15
STH 144 RPM	---	---
STH 144 RPM + 10% King Fly Ash	---	-19 ~ -20
STH 144 RPM + 14% King Fly Ash	---	-19 ~ -20

Table 3. 5. AASHTO T 292-97 (2000) test sequence for cohesive soils.

Phase	Sequence Number	Deviator Stress (kPa)	Number of Repetitions
Specimen Conditioning	1	21	1000
Testing	2	21	50
	3	34	50
	4	48	50
	5	69	50
	6	103	50

Note: The confining pressure for deviator stresses is 21 kPa.
A seating load of 13.8 kPa was used.

Table 3. 6. AASHTO T 292-97 test sequence for granular specimens of base/subbase material.

Conditioning	Sequence Number	Deviator Stress (kPa)	Confining Pressure (kPa)	Number of Repetitions
Specimen Conditioning	1	103	138	1000
Testing	2	69	138	50
	3	138	138	50
	4	207	138	50
	5	276	138	50
	6	69	103	50
	7	138	103	50
	8	207	103	50
	9	276	103	50
	10	34	69	50
	11	69	69	50
	12	138	69	50
	13	207	69	50
	14	34	34	50
	15	69	34	50
	16	103	34	50
	17	34	21	50
	18	48	21	50
	19	62	21	50

Note: A seating load of 13.8 kPa was used.

Table 4. 1. Soil-fly ash granular-material fly ash mixture volume change after freeze-thaw cycles

Soil-Fly Ash Mixture	Freeze-Thaw Cycles	Volume Change (%)
USH 12 STA 614 + 12% Columbia Fly Ash	1	-1.2
	3	0.3
	5	1.0
Lawson + 20% Columbia Fly Ash	1	-2.0
	3	-1.2
	5	-1.2
	10	0.6
Lawson + 20% Dewey Fly Ash	1	0.1
	3	-1.4
	5	-2.7
Lawson + 20% King Fly Ash	1	-2.1
	5	-1.3
County Road 53 + 10% Riverside 8 Fly Ash	1	2.0
	3	1.6
	5	2.4
Waseca STA 8 RPM + 10% Riverside 7 Fly Ash	1	1.7
	3	1.6
	5	2.6
Waseca STA 9 RPM+ 10% Riverside 7 Fly Ash	1	---
	3	1.9
	5	2.7
STH 144 + 10% King Fly Ash	5	---
	12	---
STH 144 + 14% King Fly Ash	5	---
	12	---

Table 4. 2. Change in moisture content of soil-fly ash granular material –fly ash mixtures

Soil or soil-fly ash mixtures samples	**Soaked or Unsoaked**	**Moisture State**	**Target wc (%)**	**F-T Cycles**	**Water Content Change (%)**
USH 12 STA 614 + 12% Columbia Fly Ash	Unsoaked	7% wet of optimum	22.5	0	-1.6
				1	-1.4
				3	-1.5
				5	-1.4
Lawson + 20% Columbia Fly Ash	Unsoaked	7% wet of optimum	34.4	0	-1.0
				10	-2.9
County Road 53 + 10% Riverside 8 Fly Ash	Soaked	w field %	6.4	0	4.6
				1	4.0
				3	3.8
				5	5.0
Waseca STA 9 RPM + 10% Riverside 7 Fly Ash	Soaked	w field %	8.5	0	5.6
				3	1.8
				5	1.3

Table 4. 3. Freezing Point Depression

Soil Combination	Freezing Point Depression (ºC)	Freezing Temperature (ºC)
Lawson	0	---
Lawson + 20% Columbia Fly Ash	-1.9	-5 ~ -10
Lawson + 20% Dewey Fly Ash	-0.1	-5 ~ -10
Lawson + 20% King Fly Ash	-1.6	-5 ~ -10
USH 12 STA 614	-0.8	---
USH 12 STA 614 + 12% Columbia Fly Ash	-1.2	-10
County Road 53	-11	---
County Road 53 + 10% Riverside 8 Fly Ash	-12	-15
Waseca STA 2 RPM	-8.6	---
Waseca STA 2 RPM + 10% Riverside 7 Fly Ash	-8.7	-15
Waseca STA 8 RPM	-0.9	---
Waseca STA 8 RPM + 10% Riverside 7 Fly Ash	-9.4	-15
Waseca STA 9 RPM	-10	---
Waseca STA 9 RPM + 10% Riverside 7 Fly Ash	-9.6	-15

Table 4. 4(a). Resilient modulus model regression coefficients K_1 and K_2 for soils and soil-fly ash specimens after freeze-thaw cycles

Soil Name	Fly Ash Content (%)	Curing Time (days)	Freeze-Thaw Cycles	Moisture State	No Fly Ash		Columbia		Dewey		King	
					K1	K2	K1	K2	K1	K2	K1	K2
USH 12	0	0	0	W opt+7%	63.0	-0.376	-	-	-	-	-	-
STA 614	0	0	0	W opt+7%	57.2	-0.303	-	-	-	-	-	-
	12	7	0	W opt+7%	-	-	20.7	0.219	-	-	-	-
	12	7	0	W opt+7%	-	-	13.5	0.293	-	-	-	-
	12	7	1	W opt+7%	-	-	26.1	0.148	-	-	-	-
	12	7	1	W opt+7%	-	-	25.5	0.134	-	-	-	-
	12	7	3	W opt+7%	-	-	42.7	0.065	-	-	-	-
	12	7	3	W opt+7%	-	-	26.5	0.155	-	-	-	-
	12	7	5	W opt+7%	-	-	53.2	0.019	-	-	-	-
	12	7	5	W opt+7%	-	-	49.3	0.053	-	-	-	-
Lawson	0	0	0	W opt+7%	96.6[a]	-0.194[a]	-	-	-	-	-	-
	0	0	0	W opt+7%	47.4	-0.069	-	-	-	-	-	-
	20	7	0	W opt+7%	-	-	185.2[a]	-0.088[a]	169.1[a]	-0.179[a]	124.1[a]	-0.138[a]
	20	7	0	W opt+7%	-	-	344.3[a]	-0.258[a]	150.8[a]	-0.121[a]	136.9[a]	-0.172[a]
	20	7	0	W opt+7%	-	-	37.1	0.104	-	-	177.6[a]	-0.207[a]
	20	7	0	W opt+7%	-	-	34.0	0.131	-	-	-	
	20	7	1	W opt+7%	-	-	127.0[a]	-0.109[a]	137.7[a]	-0.132[a]	144.8[a]	-0.228[a]
	20	7	1	W opt+7%	-	-	217.4[a]	-0.243[a]	206.5[a]	-0.267[a]	141.8[a]	-0.192[a]
	20	7	3	W opt+7%	-	-	149.9[a]	-0.220[a]	79.3[a]	-0.170[a]	-	-
	20	7	3	W opt+7%	-	-	115.1[a]	-0.118[a]	120.8[a]	-0.265[a]	-	-
	20	7	5	W opt+7%	-	-	146.2[a]	-0.178[a]	115.8[a]	-0.225[a]	139.6[a]	-0.186[a]
	20	7	5	W opt+7%	-	-	141.0[a]	-0.178[a]	101.1[a]	-0.218[a]	138.4[a]	-0.208[a]
	20	7	10	W opt+7%	-	-	22.9[a]	0.122[a]	-	-	-	-
	20	7	10	W opt+7%	-	-	25.1[a]	0.078[a]	-	-	-	-
STH	0	0	0	w opt	5.0[b]	0.614[b]	-	-	-	-		
144	10	7	0	w opt	-	-	-	-	-	-	47.6	0.196
RPM	10	7	5	w opt	-	-	-	-	-	-	39.2	0.196
	10	7	12	w opt	-	-	-	-	-	-	45.8	0.153
	14	7	0	w opt	-	-	-	-	-	-	57.5	0.192
	14	7	5	w opt	-	-	-	-	-	-	54.0	0.089
	14	7	12	w opt	-	-	-	-	-	-	50.3	0.144

[a] Samples tested before changes to Mr machine [b] Sample tested as granular soil

Table 4. 4(b). Resilient modulus model regression coefficients K_1 and K_2 for soils and soil-fly ash specimens after freeze-thaw cycles (continued)

Soil Name	Fly Ash Content (%)	Curing Time (days)	Freeze-Thaw Cycles	Moisture State	No Fly Ash K1	No Fly Ash K2	Riverside 8 K1	Riverside 8 K2	Riverside 7 K1	Riverside 7 K2
County Road 53 Composite	0	0	0	w field	3.3[b]	0.650[b]	-	-	-	-
	0	0	0	w field	1.5[b]	0.831[b]	-	-	-	-
	10	7	0	w field	-	-	52.7	0.127	-	-
	10	7	0	w field	-	-	60.9	0.099	-	-
	10	7	1	w field	-	-	49.2	0.122	-	-
	10	7	1	w field	-	-	52.3	0.127	-	-
	10	7	3	w field	-	-	34.8	0.202	-	-
	10	7	3	w field	-	-	64.4	0.069	-	-
	10	7	5	w field	-	-	29.2	0.228	-	-
	10	7	5	w field	-	-	45.5	0.16	-	-
Waseca STA 2 RPM	0	0	0	w field	31.9	0.144	-	-	-	-
	10	14	0	w field	-	-	-	-	F	F
	10	14	1	w field	-	-	-	-	F	F
	10	14	3	w field	-	-	-	-	F	F
	10	14	5	w field	-	-	-	-	F	F
Waseca STA 8 RPM	0	0	0	w field	31.1	0.128	-	-	-	-
	10	14	0	w field	-	-	-	-	66.7	0.076
	10	14	1	w field	-	-	-	-	35.7	0.188
	10	14	3	w field	-	-	-	-	25.0	0.234
	10	14	5	w field	-	-	-	-	47.3	0.095
Waseca STA 9 RPM	0	0	0	w field	58.3	-0.033	-	-	-	-
	10	14	1	w field	-	-	-	-	57.5	0.101
	10	14	3	w field	-	-	-	-	-	-
	10	14	5	w field	-	-	-	-	34.6	0.154

[b] Samples tested as granular soil

Table 4. 5(a). Resilient moduli (MPa) of soils and soil-fly ash mixtures at a deviator stress of 21 kPa

Soil Name	Curing Time (days)	w_{OPT}	7% wet of optimum water content																	
		Unsoaked Samples																		
		Soil Alone	Soil Alone	Columbia									Dewey				King			
				Fly Ash Content (%)																
		0%	0%	12%				20%					20%				20%			
		1-D Freeze-Thaw Cycles																		
		0	0	0	1	3	5	0	1	3	5	10	0	1	3	5	0	1	3	5
USH 12 STA 614	7		20.1 22.7 ***21.4***	40.4 33.0 ***36.7***	41.0 38.4 ***39.7***	52.1 42.4 ***47.2***	56.3 58.3 ***57.3***	-	-	-	-	-	-	-	-	-	-	-	-	-
Lawson	7	53.6[a]	-	-	-	-	-	141.6[a] 156.8[a] ***149.2[a]***	91.2[a] 103.8[a] ***97.5[a]***	76.6[a] 80.4[a] ***78.5[a]***	85.0[a] 81.9[a] ***83.4[a]***	-	98.1[a] 104.4[a] ***101.2[a]***	92.3[a] 91.7[a] ***92.0[a]***	45.4[a] 53.9[a] ***49.6[a]***	58.4[a] 52.1[a] ***55.2[a]***	81.5[a] 81.2[a] 94.5[a] ***85.7[a]***	72.3[a] 79.1[a] ***75.7[a]***	-	79.2[a] 73.5[a] ***76.4[a]***
		38.4	-	-	-	-	-	50.8 50.7 ***50.8***	-	-	-	33.2 31.8 ***32.5***	-	-	-	-	-	-	-	-

Notes: All Mr at a deviator stress of 21 kPa. USH 12 STA 614 (wopt = 15.5 %) and Lawson (wopt = 27.4%). Numbers in bold and italic are average values.
[a] Data before fixing the M_r machine.

Table 4. 5(b). Resilient moduli (MPa) of soils and soil-fly ash mixtures at a deviator stress of 21 kPa (continued)

Soil Name	Curing Time (days)	Optimum water content							Field water content								
		Soaked Samples for 5 hours							Soaked Samples for 5 hours								
		Soil Alone	King						Soil Alone	Riverside 8				Riverside 7			
			Fly Ash Content (%)							Fly Ash Content (%)							
		0%	10%			14%			0%	10%				10%			
		3-D Freeze-Thaw Cycles							3-D Freeze-Thaw Cycles								
		0	0	5	12	0	5	12	0	0	1	3	5	0	1	3	5
County Road 53 Composite	7	-	-	-	-	-	-	-	52.2[c] 50.1[c] ***51.2[c]***	77.6 82.5 ***80.1***	71.5 77.0 ***74.2***	64.4 79.5 ***72.0***	58.5 74.2 ***66.4***	-	-	-	-
Waseca STA 2 RPM	14		-	-	-	-	-	-	49.6[d]	-	-	-	-	F	F	F	F
Waseca STA 8 RPM	14		-	-	-	-	-	-	46.0[d]	-	-	-	-	84.0	63.1	50.9	63.2
Waseca STA 9 RPM	14		-	-	-	-	-	-	52.8[d]	-	-	-	-	78.2	F	55.3	45.0
STH 144 RPM	7	79.5	86.4[e]	71.2[e]	73.0[e]	103.0[e]	70.9[e]	78.1[e]	-	-	-	-	-	-	-	-	-

Note: Mr reported at a deviator stress of 21 kPa, except for County Road 53 Composite (soil alone, 0 F-T cycles) which is reported at a bulk stress of 70 kPa and Bloom RAP (soil alone, 0 F-T cycles) which is reported at a bulk stress of 89 KPa. County Road 53 Composite + 10% Riverside 8 fly ash (w_N = 6.4%), Waseca STA 2, 8, and 9 Lysimeter + 10% Riverside 7 fly ash (w_N = 8.5%), Bloom RAP alone (w_{opt} = 6.5%), Bloom RAP + 10% King fly ash (w_{opt} = 5.8%) and Bloom RAP + 14% King fly ash (w_{opt} = 5.7%). Soil and soil-fly ash mixtures samples were compacted 2 hours after mixing, except for Bloom RAP which were compacted shortly after mixing. [c] Soils tested following test sequence for granular specimens of base/subbase material of AASHTO T 292-97 (2000) and values are reported at a bulk stress of 70 kPa. [d] Test performed by Jeremy Baugh, granular soils tested following test sequence for cohesive specimens of AASHTO T 292-97 (2000). [e] Test performed by Jeremy Baugh. F= sample failed before Mr test.

Table 4. 6(a). Resilient modulus of soil-fly ash mixtures after freeze-thaw cycles normalized to the resilient modulus without freeze-thaw cycles

Soil Name	Curing Time (days)	7% wet of optimum																
		Unsoaked samples																
		Columbia									Dewey				King			
		Fly Ash Content (%)																
		12%				20%					20%				20%			
		1-D Freeze-Thaw Cycles																
		0	1	3	5	0	1	3	5	10	0	1	3	5	0	1	3	5
USH 12 STA 614	7	1.00 1.00 ***1.00***	1.01 1.16 ***1.08***	1.29 1.28 ***1.29***	1.39 1.77 ***1.56***	-	-	-	-	-	-	-	-	-	-	-	-	-
Lawson	7	-	-	-	-	1.00[a] 1.00[a] ***1.00***[a]	0.64[a] 0.66[a] ***0.65***[a]	0.54[a] 0.51[a] ***0.53***[a]	0.60[a] 0.52[a] ***0.56***[a]	-	1.00[a] 1.00[a] ***1.00***[a]	0.94[a] 0.88[a] ***0.91***[a]	0.46[a] 0.52[a] ***0.49***[a]	0.60[a] 0.50[a] ***0.54***[a]	1.00[a] 1.00[a] ***1.00***[a]	0.89[a] 0.97[a] ***0.88***[a]	-	0.97[a] 0.91[a] ***0.89***[a]
		-	-	-	-	1.00 1.00 ***1.00***	-	-	-	0.65 0.63 ***0.64***	-	-	-	-	-	-	-	-

Note: Numbers in bold and italic are average values. [a] Sample tested before fixing Mr machine

Table 4. 6(b). Resilient modulus of soil-fly ash mixtures after freeze-thaw cycles normalized to the resilient modulus without freeze-thaw cycles (continued)

Soil Name	Curing Time (days)	Optimum water content						Field water content							
		Soaked Samples for 5 hours						Soaked Samples for 5 hours							
		King						Riverside 8				Riverside 7			
		Fly Ash Content (%)						Fly Ash Content (%)							
		10%			14%			10%				10%			
		3-D Freeze-Thaw Cycles													
		0	5	12	0	5	12	0	1	3	5	0	1	3	5
County Road 53 Composite	7	-	-	-	-	-	-	1.00 1.00 ***1.00***	0.92 0.93 ***0.93***	0.83 0.96 ***0.90***	0.75 0.90 ***0.83***	-	-	-	-
Waseca STA 2	14	-	-	-	-	-	-	-	-	-	-	F	F	F	F
Waseca STA 8	14	-	-	-	-	-	-	-	-	-	-	1.00	0.75	0.61	0.75
Waseca STA 9 Lysimeter	14	-	-	-	-	-	-	-	-	-	-	1.00	F	0.71	0.58
Bloom RAP	7	1.00[b]	0.82[b]	0.84[b]	1.00[b]	0.69[b]	0.76[b]	-	-	-	-	-	-	-	-

Note: Number in bold and italic are average values. F=Samples failed before Mr test. Soil-fly ash mixtures were compacted 2 hours after mixing except Bloom RAP samples which were compacted shortly after mix. Mr values reported at a deviator stress of 21 kPa.
[b] Data provided by Jeremy Baugh.

Table 4. 7(a). Soil and soil-fly ash mixture unconfined compressive strength (kPa) after freeze-thaw cycles

Soil Name	Curing Time (days)	w_{OPT}	7% wet of optimum water content																	
		Unsoaked Samples																		
		Soil Alone	Soil Alone	Columbia									Dewey				King			
				Fly Ash Content (%)																
		0%	0%	12%				20%					20%				20%			
		1-D Freeze-Thaw Cycles																		
		0	0	0	1	3	5	0	1	3	5	10	0	1	3	5	0	1	3	5
USH 12 STA614	7	-	106 125 ***115.5***	395 434 ***414.5***	544 480 ***512***	444 406 ***425***	267 326 ***296.5***	-	-	-	-	-	-	-	-	-	-	-	-	-
Lawson	7	-	-	-	-	-	-	508[a] 358[a] ***433***[a]	344[a] 373[a] ***358.5***[a]	232[a] 315[a] ***273.5***[a]	267[a] 281[a] ***274***[a]	-	-	354[a] 316[a] ***335***[a]	311[a] 346[a] ***328.5***[a]	249[a] 186[a] ***217.5***[a]	302[a] 228[a] ***265***[a]	216[a] 380[a] ***298***[a]	-	267[a] - ***267***[a]
		250.5	-	-	-	-	-	422 336 ***379***	-	-	-	334 301 ***317.5***	-	-	-	-	-	-	-	-

Note: Numbers in bold and italic are average values. [a] qu of the samples tested before the Mr machine was fixed

Table 4. 7(b). Soil and soil-fly ash mixture unconfined compressive strength (kPa) after freeze-thaw cycles (cont.)

Soil Name	Curing Time (days)	Optimum water content						Field water content							
		Soaked Samples						Soaked Samples							
		King						Riverside 8				Riverside 7			
		Fly Ash Content (%)						Fly Ash Content (%)							
		10%			14%			10%				10%			
		3-D Freeze-Thaw Cycles													
		0	5	12	0	5	12	0	1	3	5	0	1	3	5
County Road 53 Composite	7	-	-	-	-	-	-	209 156 ***182.5***	165 258 ***211.5***	415 424 ***419.5***	158 196 ***177***	-	-	-	-
Waseca STA 2 RPM	14	-	-	-	-	-	-	-	-	-	-	F	F	F	F
Waseca STA 8 RPM	14	-	-	-	-	-	-	-	-	-	-	219	100	121	70
Waseca STA 9 RPM	14	-	-	-	-	-	-	-	-	-	-	126	F	109	132
STH 144 RPM	7	542[b]	532[b]	320[b]	518[b]	618[b]	506[b]	-	-	-	-	-	-	-	-

Note: Bold and italics values are average values. F = Failed samples before test. Soil-fly ash mixtures were compacted 2 hours after mixing, except for Bloom RAP. [b] Data provided by Jeremy Baugh, soil-fly ash mixtures were compacted right after mixing.

Table 4. 8(a). Normalized soil-fly mixture unconfined compressive strength to the unconfined compressive strength at 0 freeze-thaw cycles

Soil Name	Curing Time (days)	w_{OPT}	7% wet of optimum water content														
		Unsoaked Samples															
		Columbia									Dewey			King			
		Fly Ash Content (%)															
		12%				20%					20%			20%			
		0	1	3	5	0	1	3	5	10	1	3	5	0	1	3	5
USH 12 STA 614	7	1.0 1.0 ***1.0***	1.4 1.1 ***1.2***	1.1 0.9 ***1.0***	0.7 0.8 ***0.7***	-	-	-	-	-	-	-	-	-	-	-	-
Lawson	7	-	-	-	-	1.0[a] 1.0[a] ***1.0[a]***	0.7[a] 1.0[a] ***0.8[a]***	0.4[a] 0.9[a] ***0.6[a]***	0.5[a] 0.8[a] ***0.6[a]***	-	1.0[a] 1.0[a] ***1.0[a]***	0.9[a] 1.1[a] ***1.0[a]***	0.7[a] 0.6[a] ***0.6[a]***	1.0[a] 1.0[a] ***1.0[a]***	0.7[a] 1.7[a] ***1.1[a]***	-	0.9[a] - ***1.0[a]***
		-	-	-	-	1.0 1.0 ***1.0***	-	-	-	0.8 0.9 ***0.8***	-	-	-	-	-	-	-

Note: Values in bold and italic are average values. Soil-fly ash samples mixtures compacted 2 hours after mixing. [a] Unconfined compressive strength values of samples tested before Mr machine was fixed.

Table 4. 8(b). Normalized soil-fly mixture unconfined compressive strength to the unconfined compressive strength at 0 freeze-thaw cycles (cont.)

Soil Name	Curing Time (days)	Optimum water content						Field water content							
		Soaked Samples						Soaked Samples							
		King						Riverside 8				Riverside 7			
		Fly Ash Content (%)						Fly Ash Content (%)							
		10%			14%			10%				10%			
		3-D Freeze-Thaw Cycles													
		0	5	12	0	5	12	0	1	3	5	0	1	3	5
County Road 53	7	-	-	-	-	-	-	1.0 1.0 ***1.0***	0.8 1.6 ***1.2***	2.0 2.7 ***2.3***	0.8 1.2 ***1.0***	-	-	-	-
Waseca STA 2 RPM	14	-	-	-	-	-	-	-	-	-	-	F	F	F	F
Waseca STA 8 RPM	14	-	-	-	-	-	-	-	-	-	-	1.0	0.4	0.6	0.3
Waseca STA 9 RPM	14	-	-	-	-	-	-	-	-	-	-	1.0	F	0.9	1.0
STH 144 RPM	7	1.0[b]	1.0[b]	0.6[b]	1.0[b]	1.2[b]	1.0[b]	-	-	-	-	-	-	-	-

Note: Number in bold and italic are average values. F = Sample failed before test. Soil-fly ash mixtures compacted 2 hours after mixing except Bloom RAP Data. [b] Data provided by Jeremy Baugh, soil-fly ash mixture compacted after mixing

Table 4. 9. Freeze-thaw cycles time

Soil-fly ash mixture	Freeze-Thaw Cycling	Freeze-thaw Cycles	Time (days)
USH 12 STA 614 + 12% Columbia fly ash	1-D	First[a]	22
		In between[b]	15
		Last[c]	10
Lawson + 20% Columbia fly ash	1-D	First[a]	11
		In between[b]	14
		Last[c]	21
Lawson + 20% Dewey fly ash	1-D	First[a]	9
		In between[b]	6
		Last[c]	8
Lawson + 20% King fly ash	1-D	First[a]	11
		In between[b]	6
		Last[c]	6
County Road 53 + 10% Riverside 8 fly ash	3-D	First[a]	5
		In between[b]	3
		Last[c]	7
Waseca STA 8 + 10% Riverside 7 fly ash	3-D	First[a]	4
		In between[b]	3
		Last[c]	4
Waseca STA 9 Lysimeter + 10% Riverside 7 fly ash	3-D	First[a]	2.5
		In between[b]	3
		Last[c]	5
Bloom RAP + 10% King fly ash	3-D	First[a]	1.45
		In between[b]	1.45
		Last[c]	1.45
Bloom RAP + 14% King fly ash	3-D	First[a]	1.42
		In between[b]	1.42
		Last[c]	1.42

[a]First cycle (from room temp., to freezing temp., and to thawing temp. (5ºC)). [b]In between (from thawing temp. (5ºC), to freezing temp., and again to thawing temp. (5ºC)). [c] Last (from thawing temp. (5ºC)), to freezing temp., and back to room temp.)

FIGURES

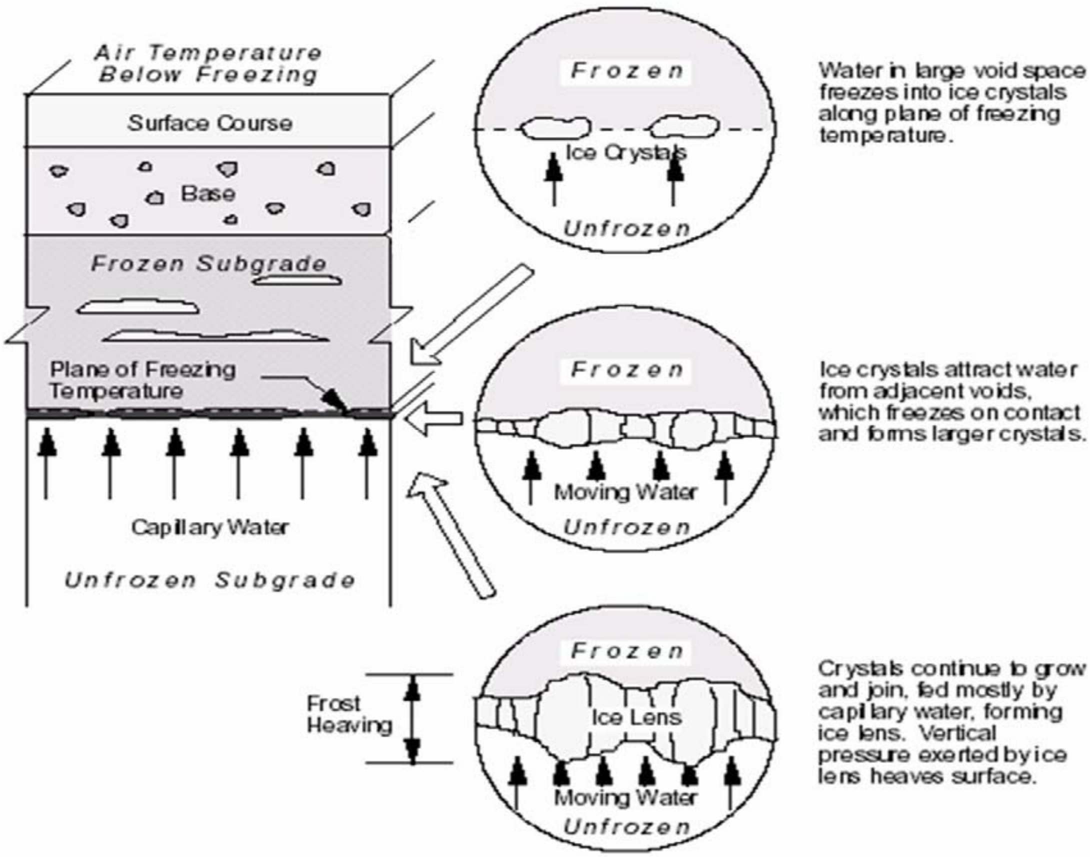


Figure 2. 1. Frost heaving in highways

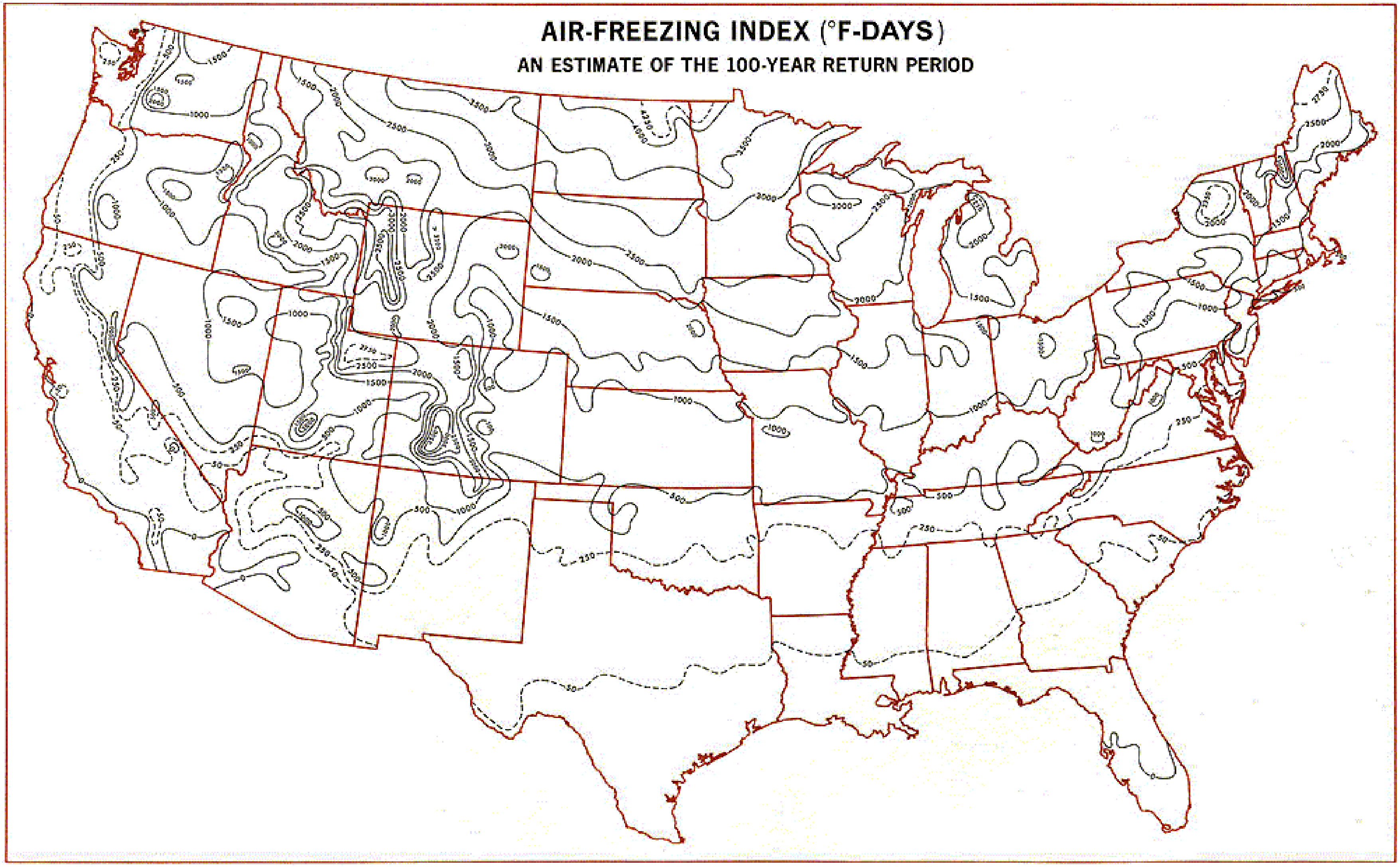


Figure 2. 2. Design freezing index value of USA

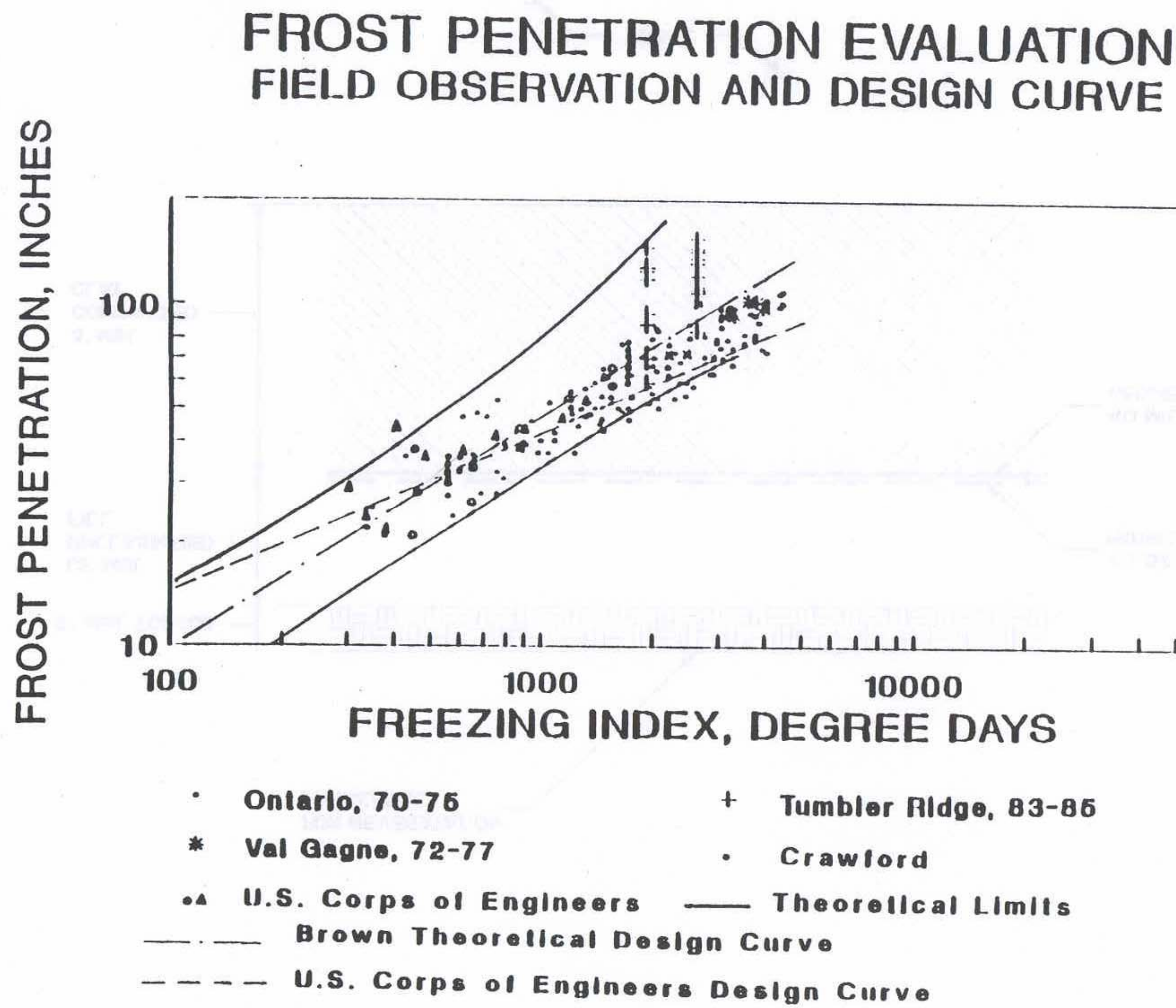
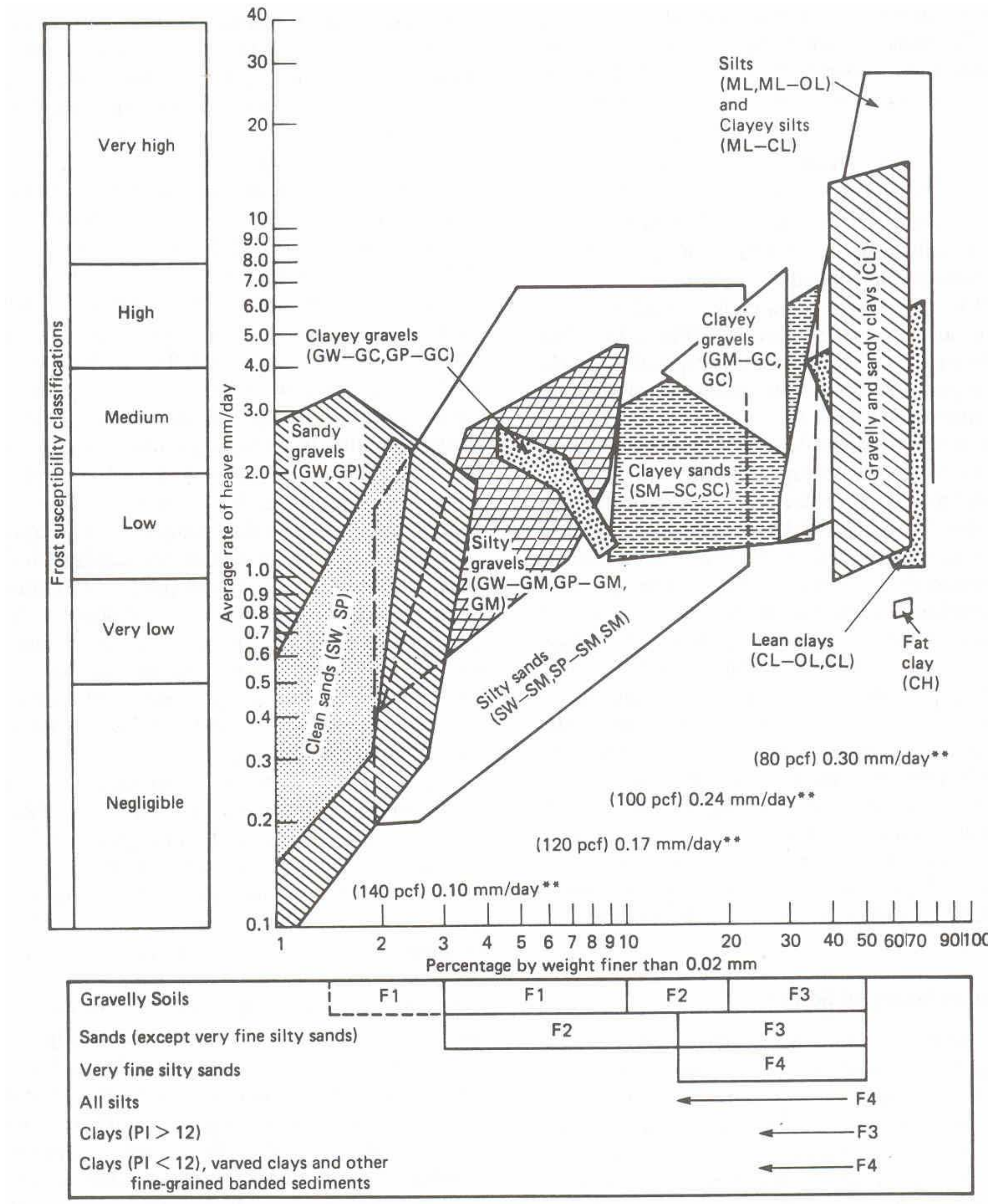


Figure 2. 3 Frost Penetration vs. Freezing Index



** Indicate heave rate due to expansion in volume if all original water in 100% saturated specimen were frozen, with rate of penetration 6.3 mm (0.25 in) per day. (Department of the Army, 1965)

Figure 2. 4 Frost susceptibility of soils

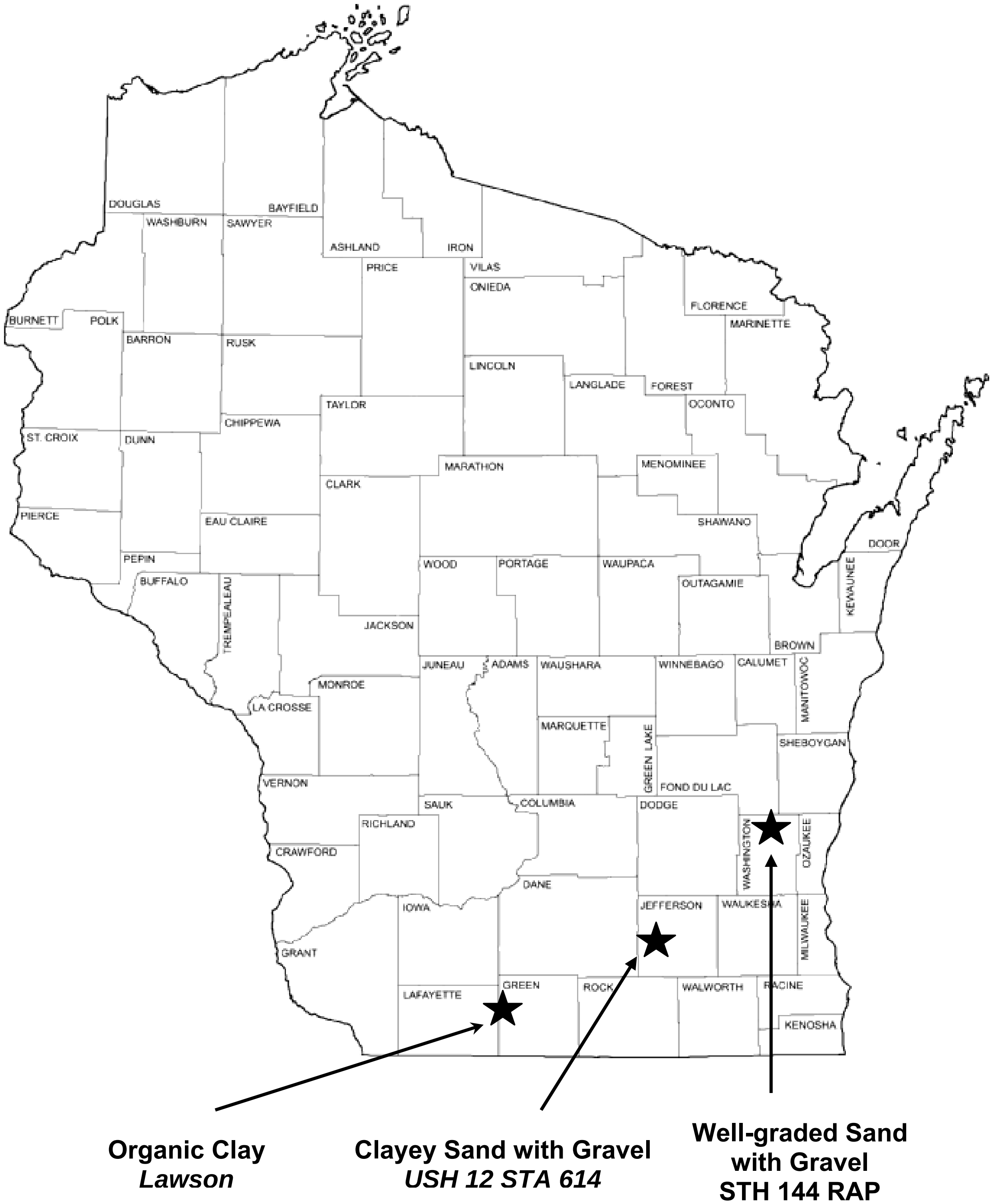


Figure 3. 1 Location where subgrade soils and the recycled asphalt pavement from WI were sampled

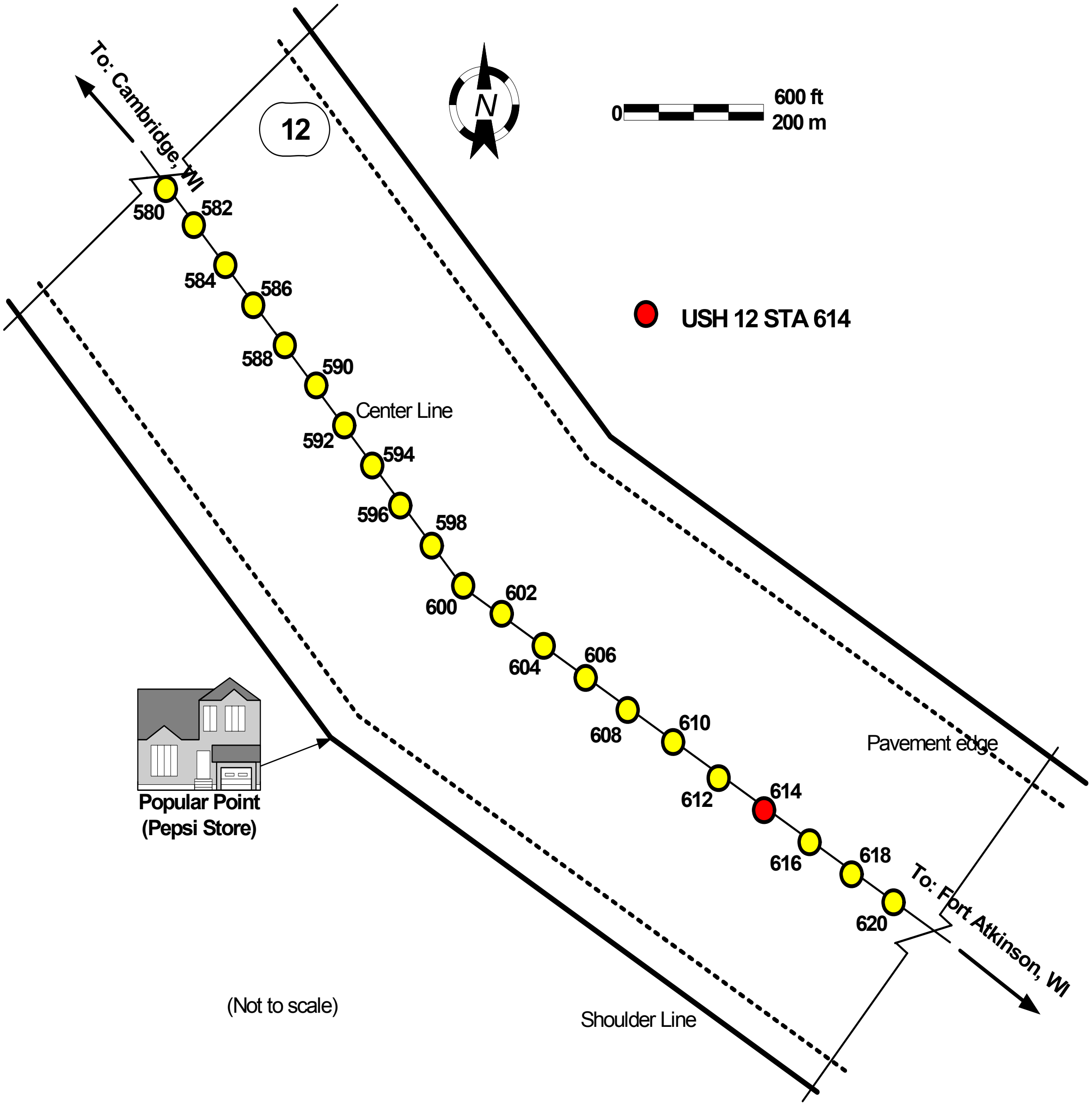


Figure 3. 2. Location where clayey sand from USH 12 STA 614 were sampled

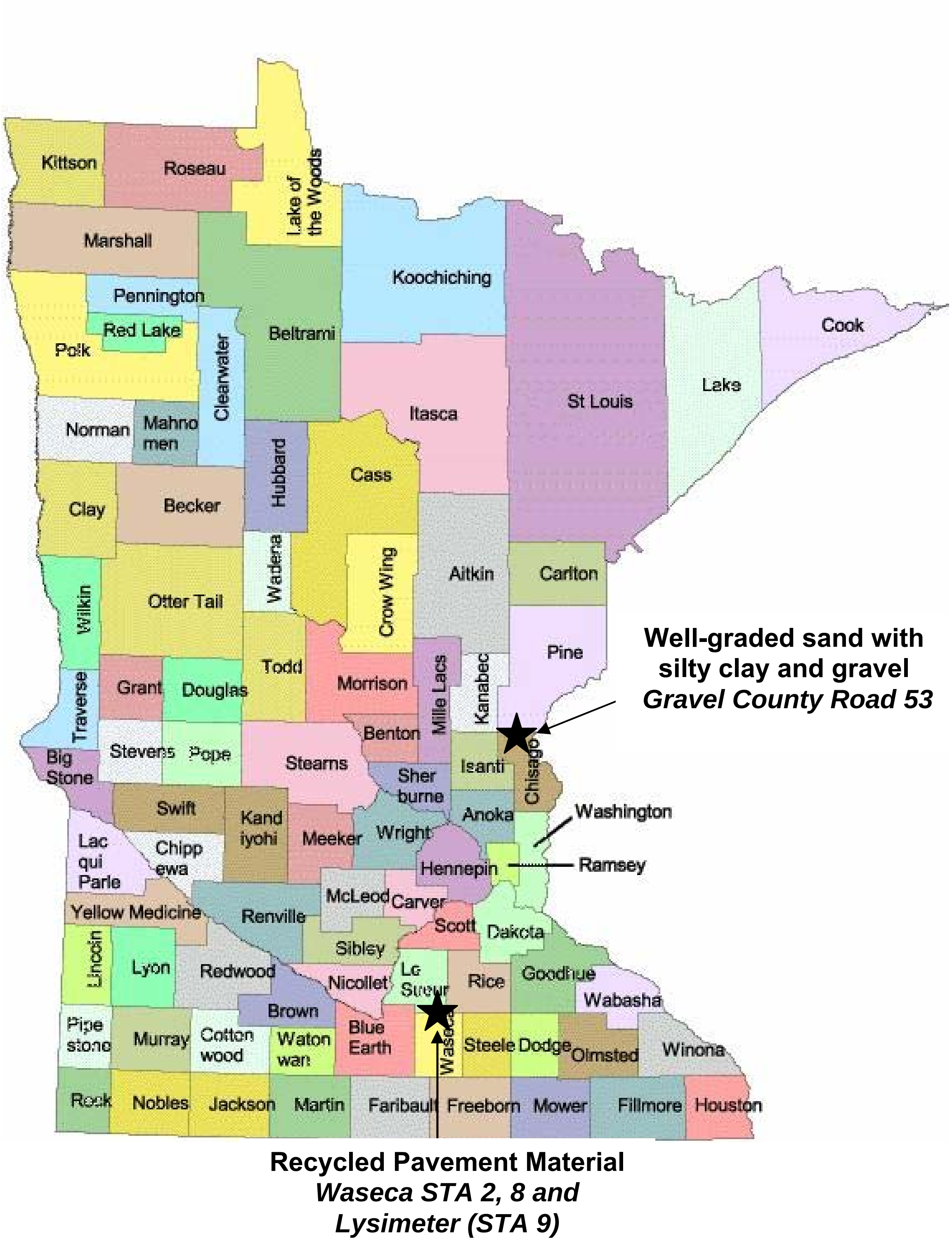


Figure 3. 3. Locations where the base soil and the recyclable pavement material from MN were sampled

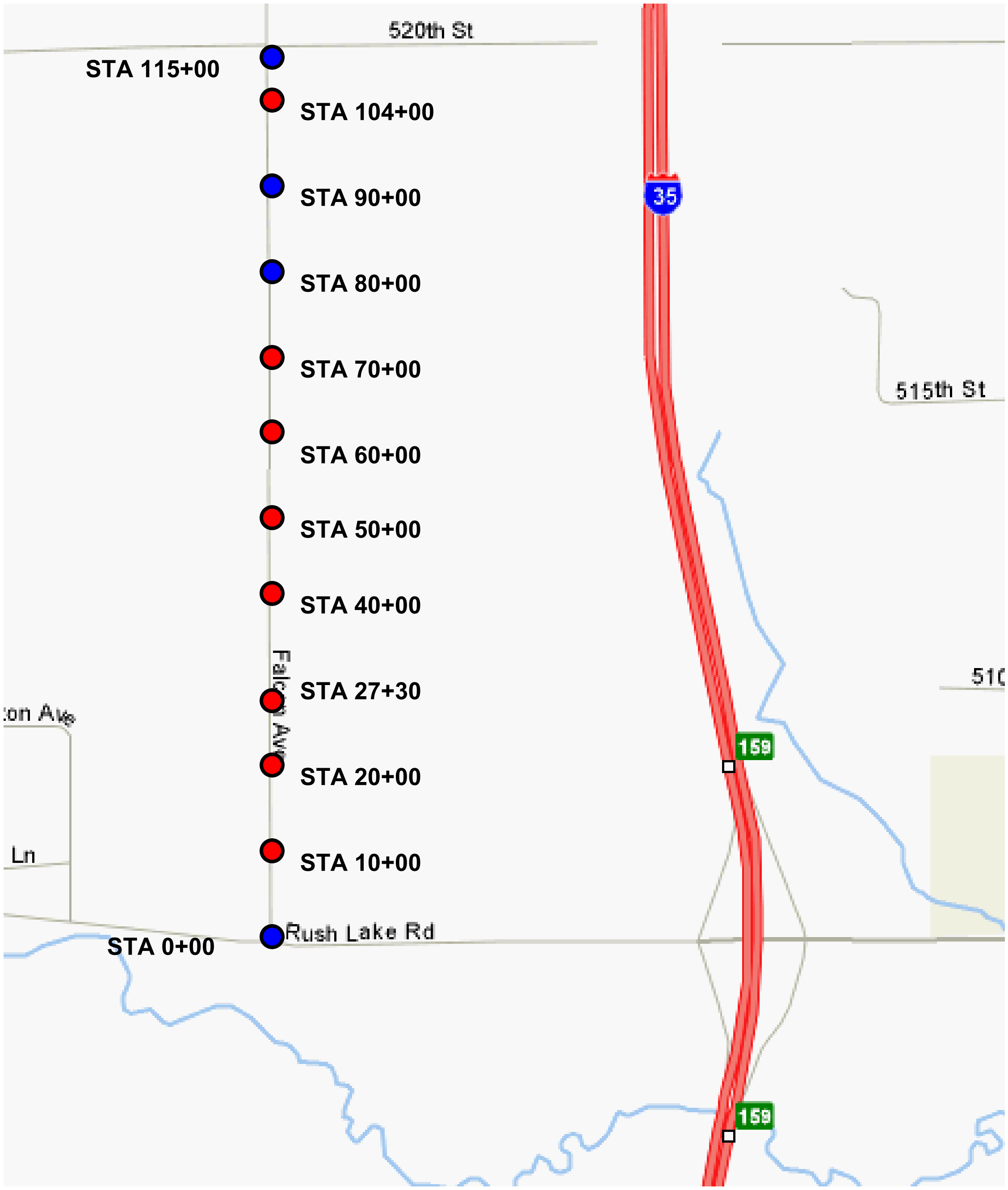


Figure 3. 4. Stations were Composite County Road 53 soil were sampled

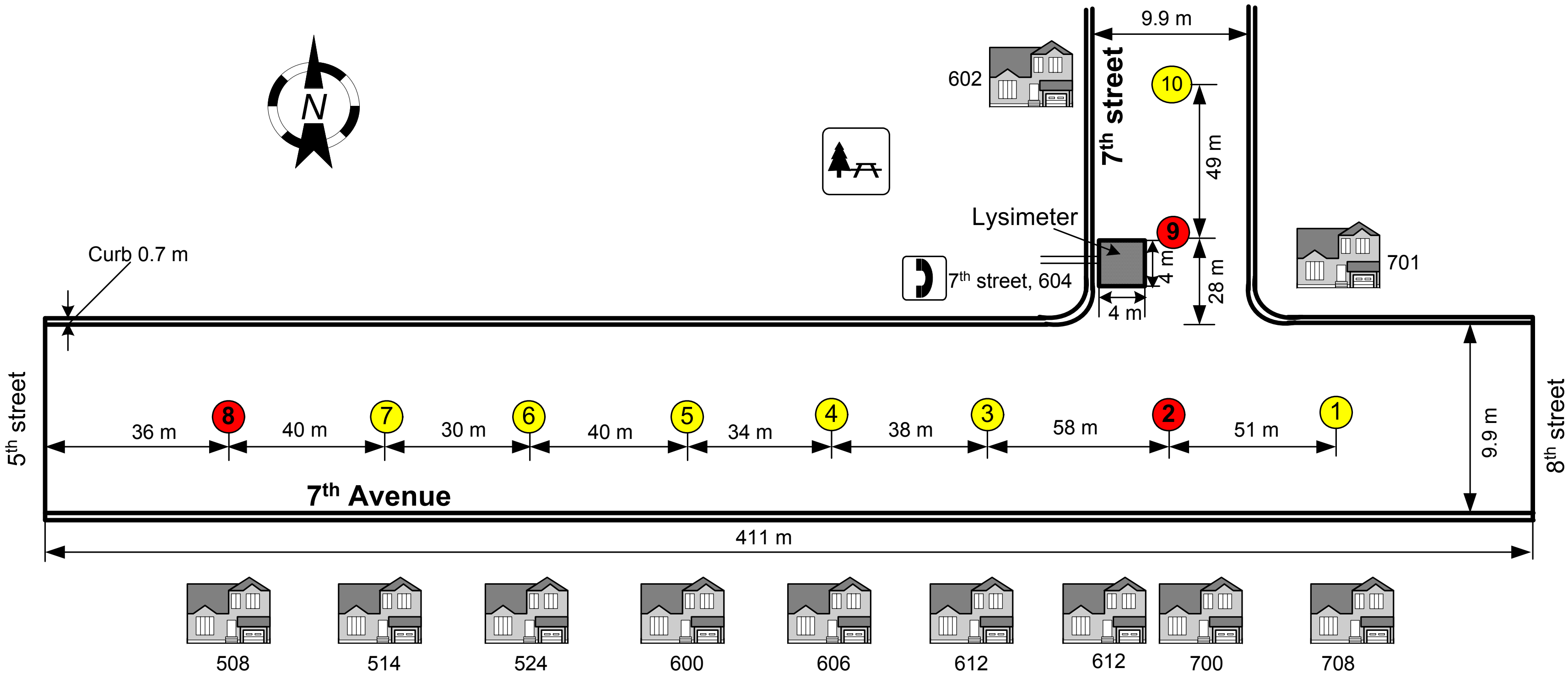


Figure 3. 5. Locations where recycled pavement material from Waseca STA 2, 8 and STA 9 (Lysimeter) were collected

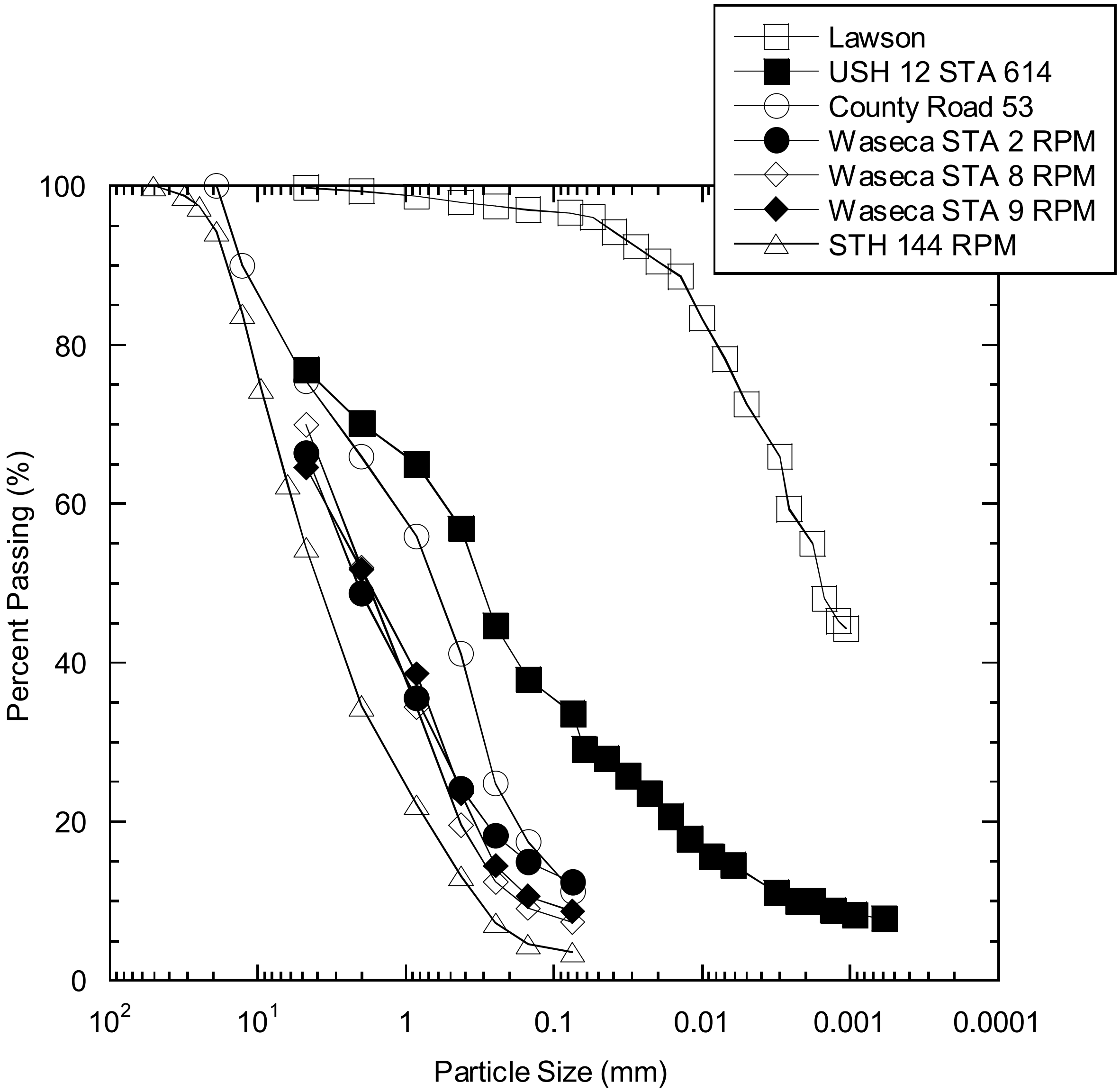


Figure 3. 6. Particle size distributions for the soils, Waseca and STH 144 recycled pavement materials

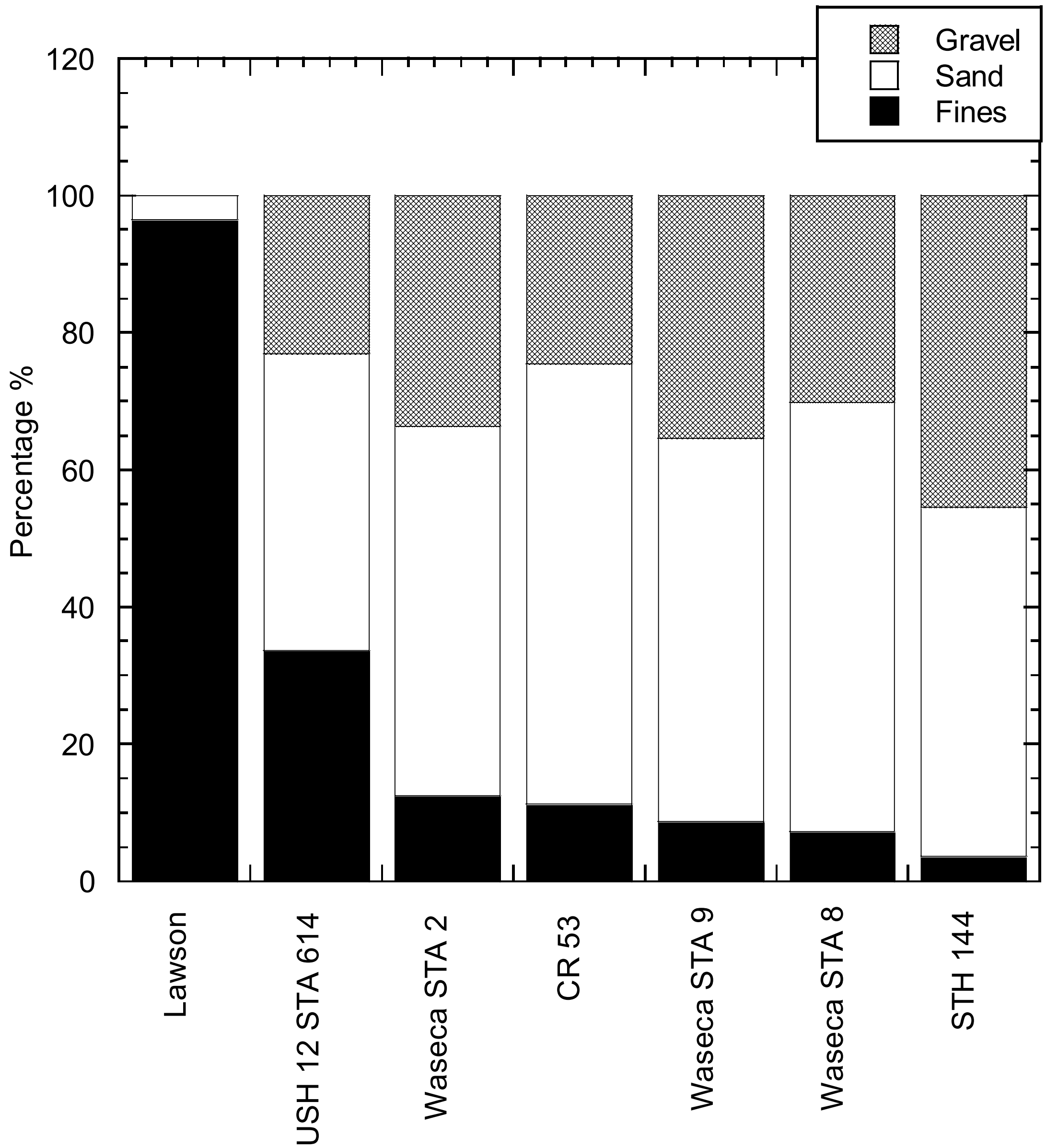


Figure 3. 7. Percentage of gravel, sand, and fines

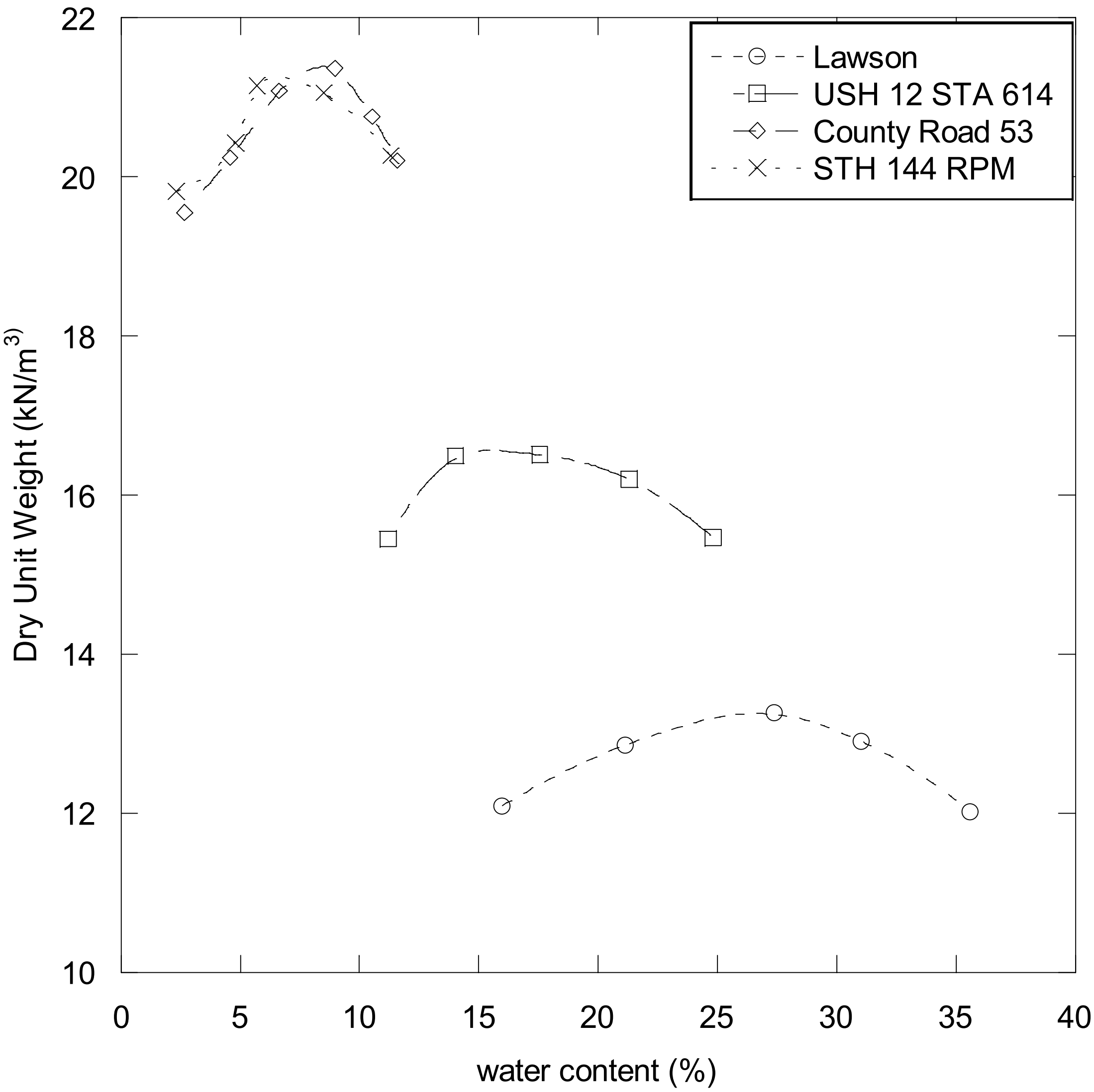


Figure 3. 8. Compaction curves

Figure 3. 9. Five fly ashes used in this study.

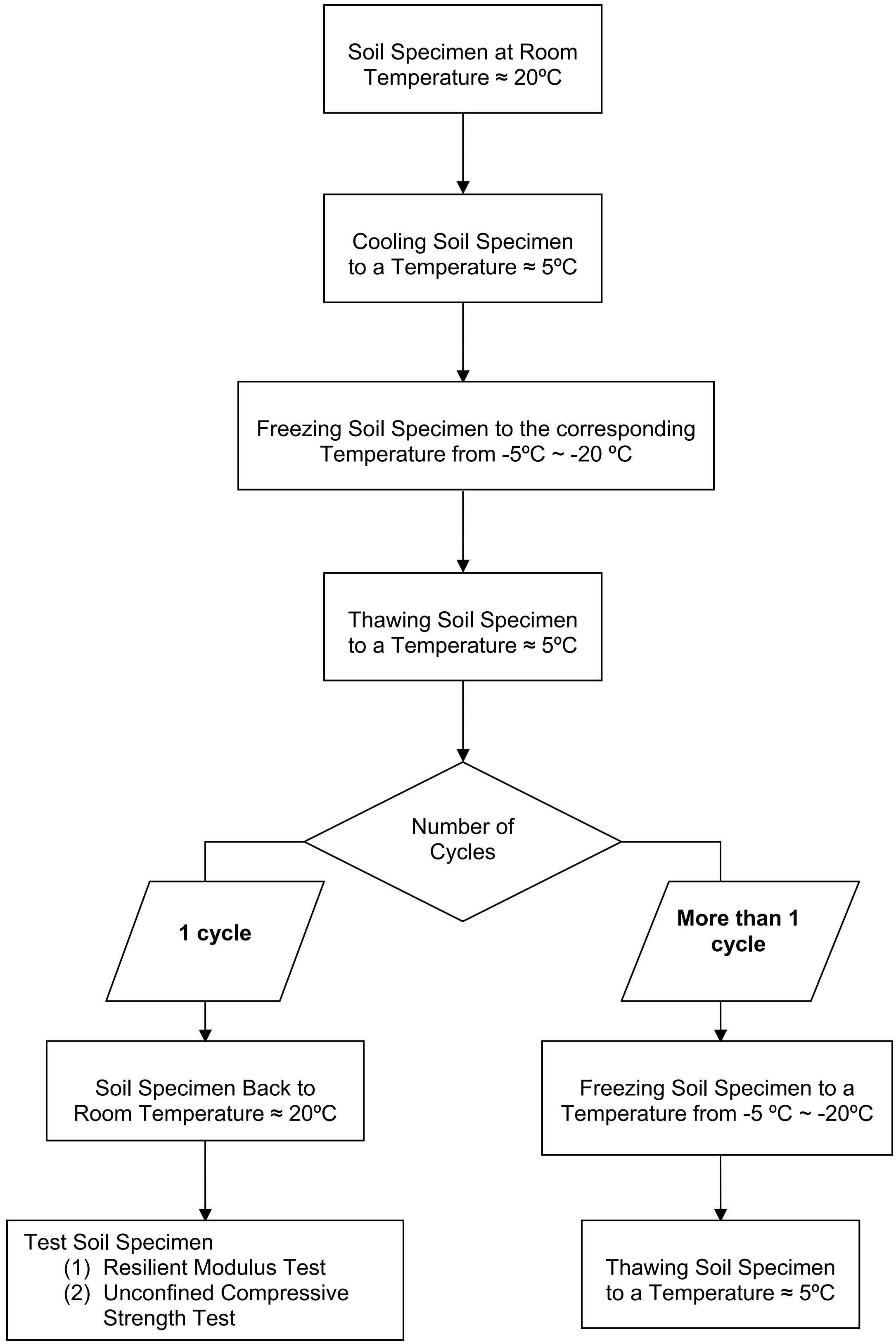


Figure 3. 10. Description of process used for freeze-thaw cycling

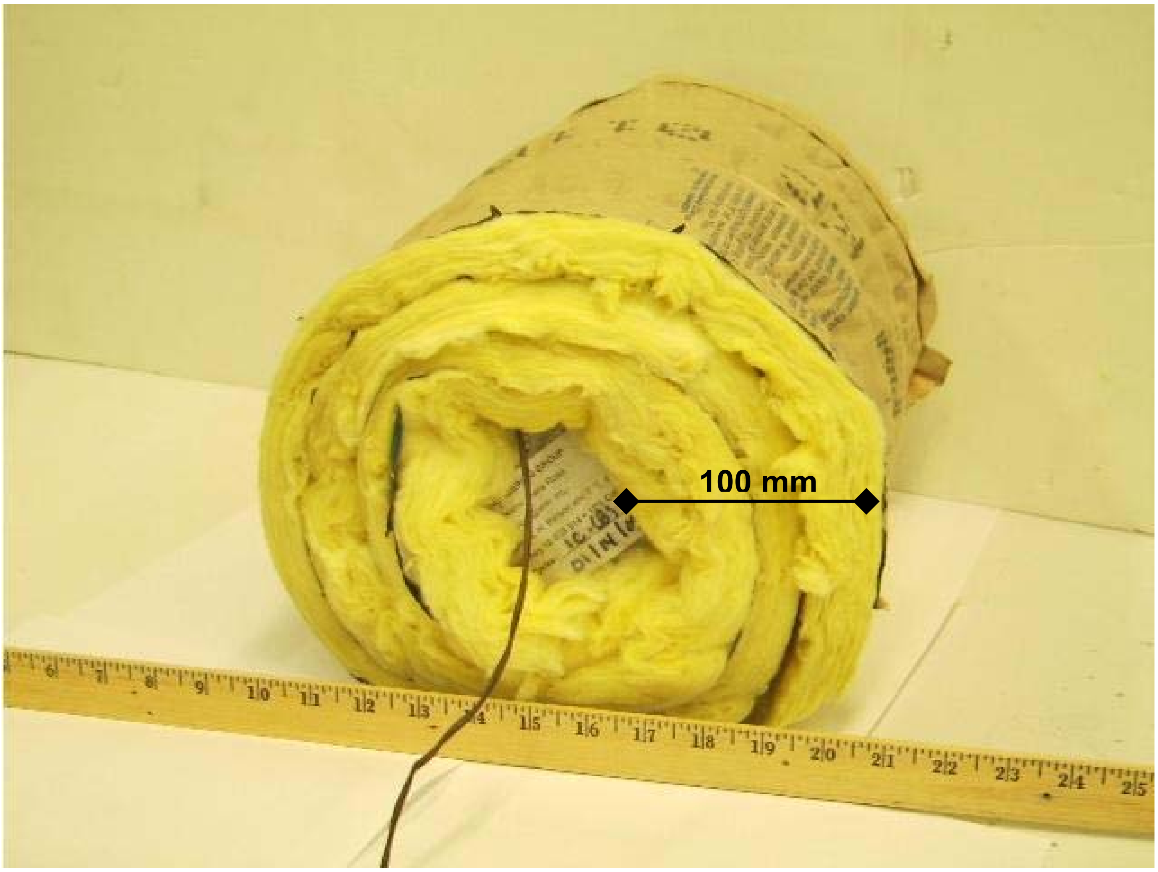


Figure 3. 11 Soil sample set up for 1-D freeze-thaw cycling.

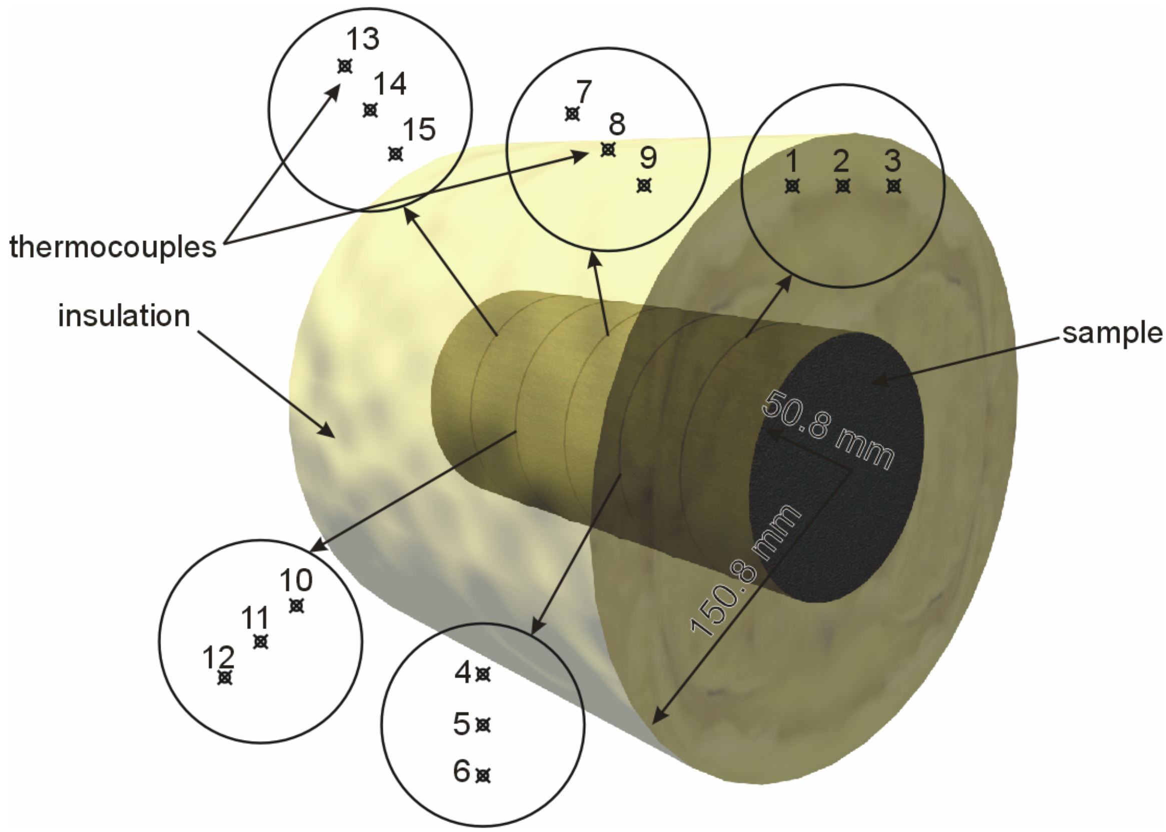
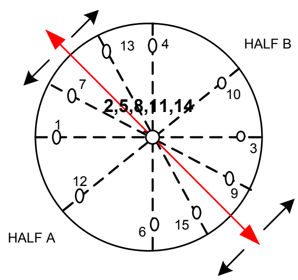


Figure 3. 12 1-D freeze-thaw cycle test.



(a)

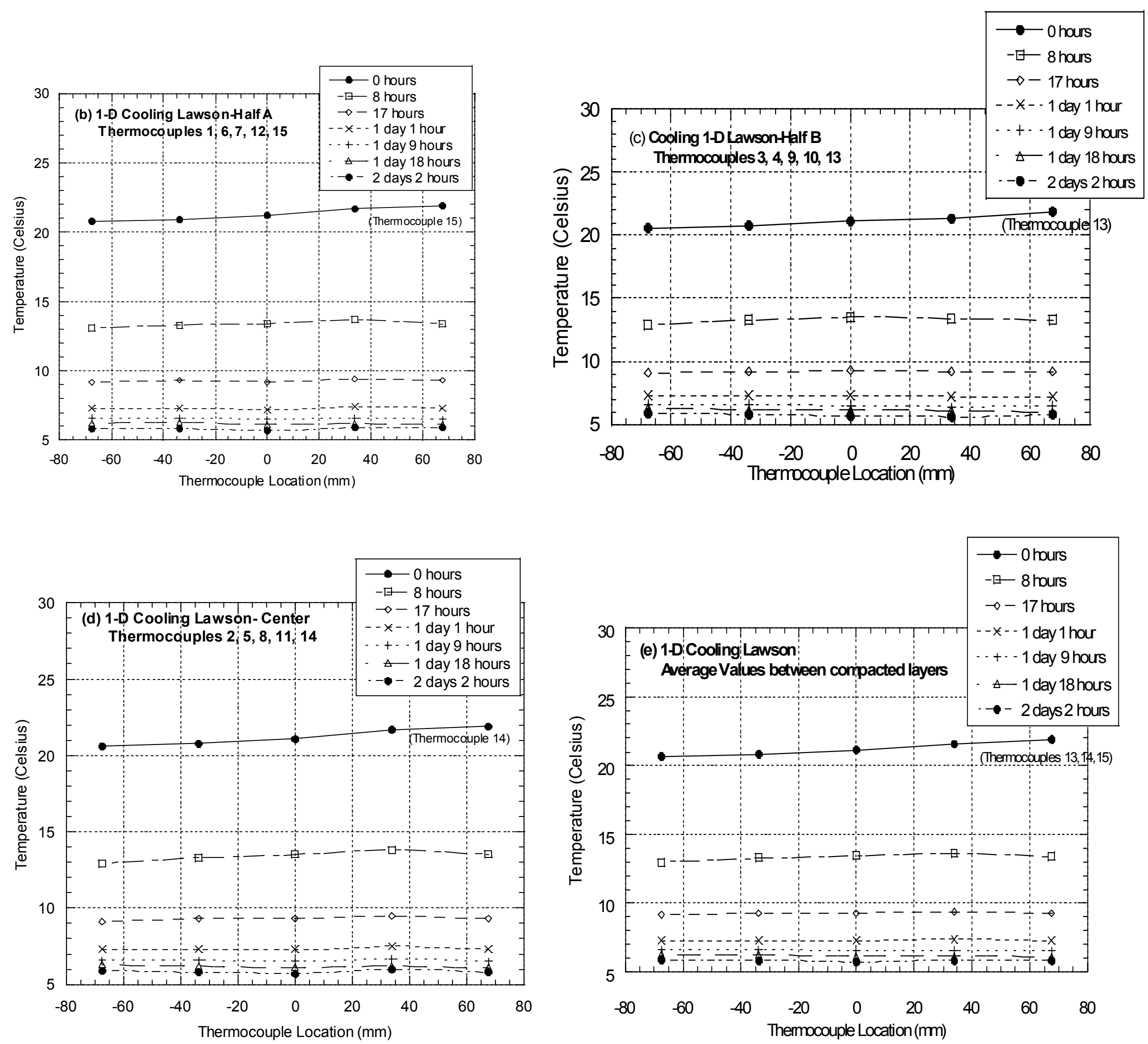


Figure 3. 13. Thermocouple Locations (a), 1-D Cooling Half A (b), 1-D Cooling Half B (c), 1-D Cooling at the center of the sample (d), and, 1-D Cooling average temperature values of the 3 thermocouples placed between the compacted layers (e).

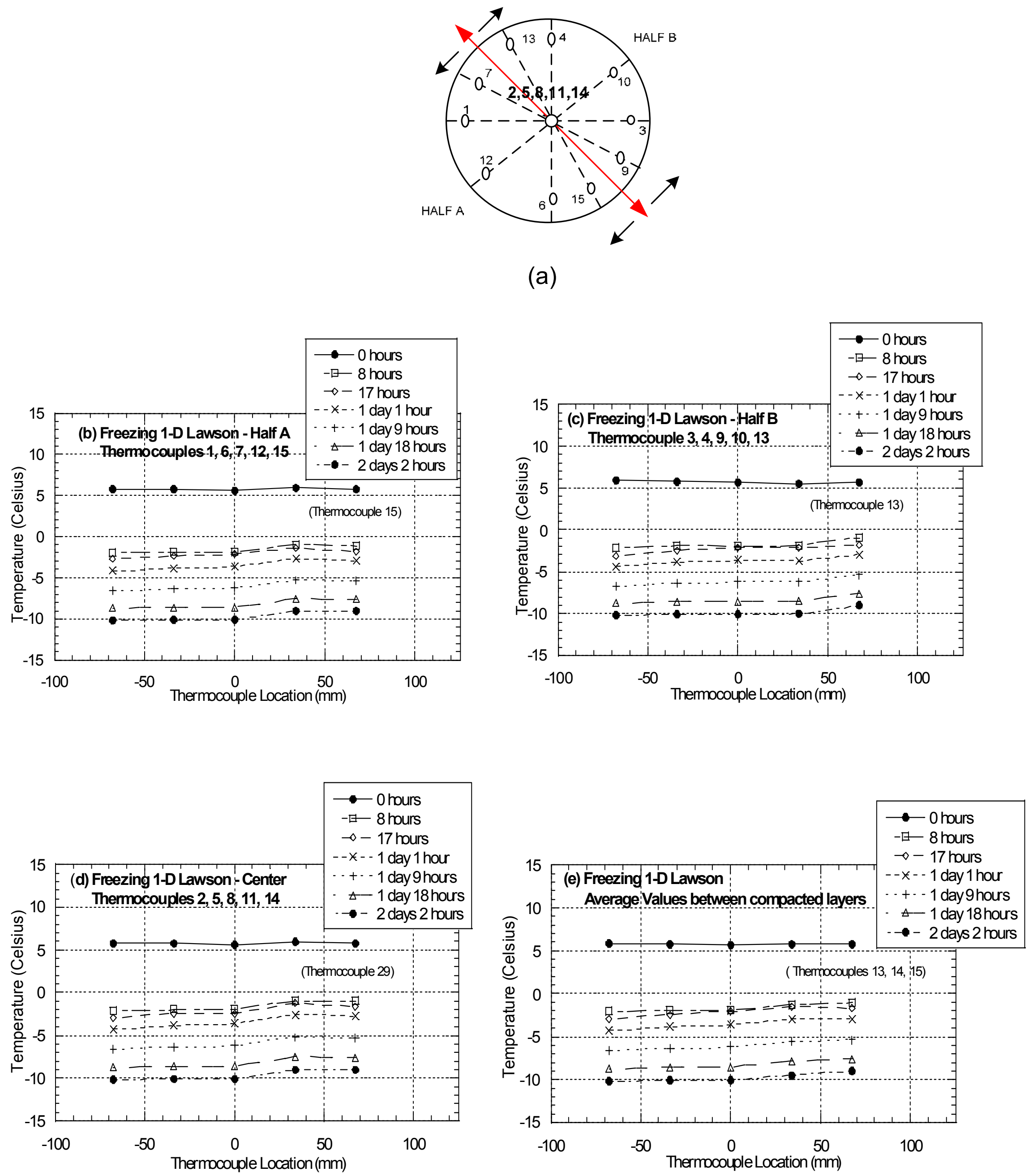


Figure 3. 14. Thermocouple Locations (a), 1-D Freezing Half A (b), 1-D Freezing Half B (c), 1-D freezing at the center of the sample (d), and, 1-D Freezing average temperature values of the 3 thermocouples placed between the compacted layers (e).

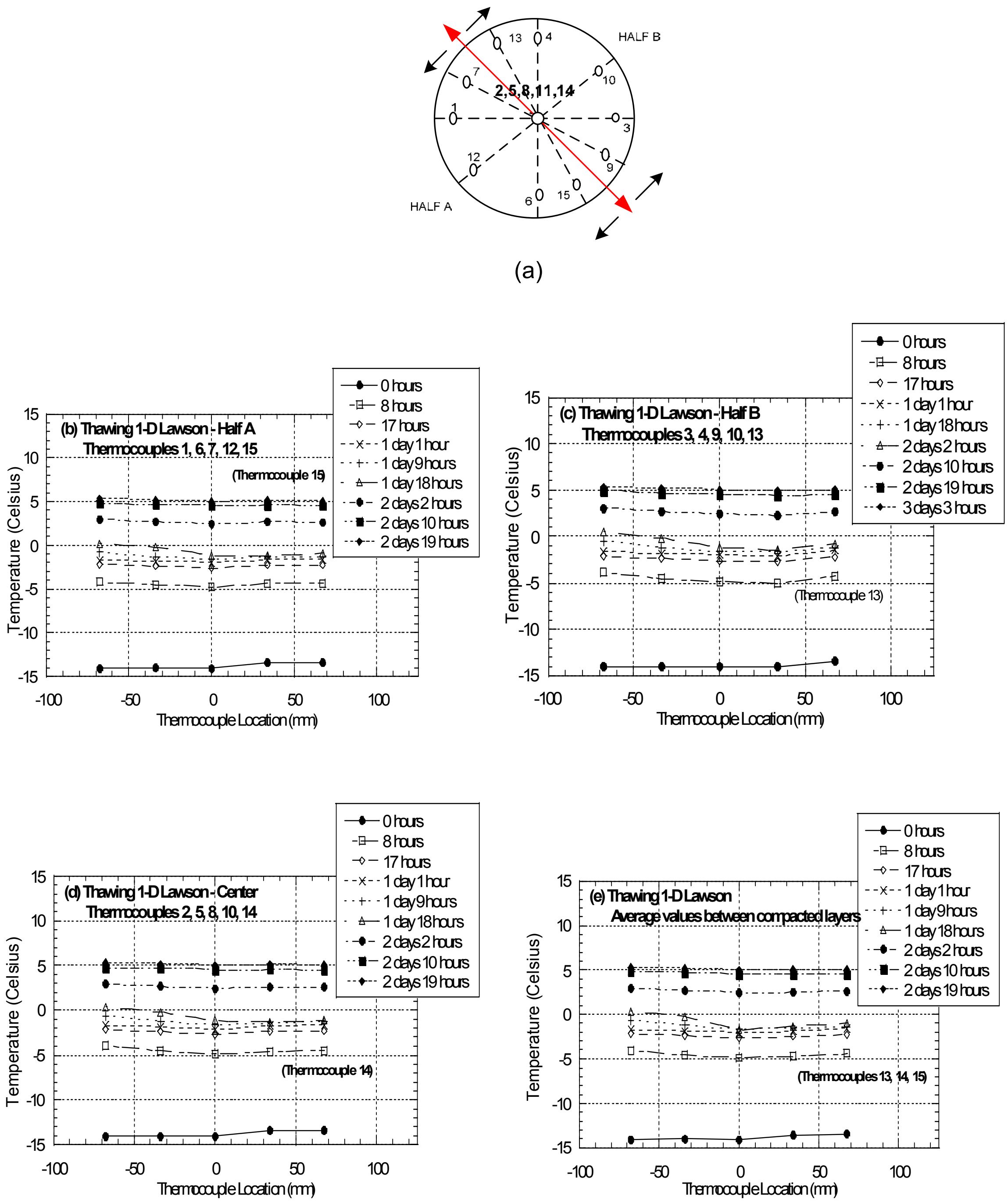


Figure 3. 15. Thermocouple Locations (a), 1-D Thawing Half A (b), 1-D Thawing Half B (c), 1-D thawing at the center of the sample (d), and, 1-D thawing average temperature values of the 3 thermocouples placed between the compacted layer (e).

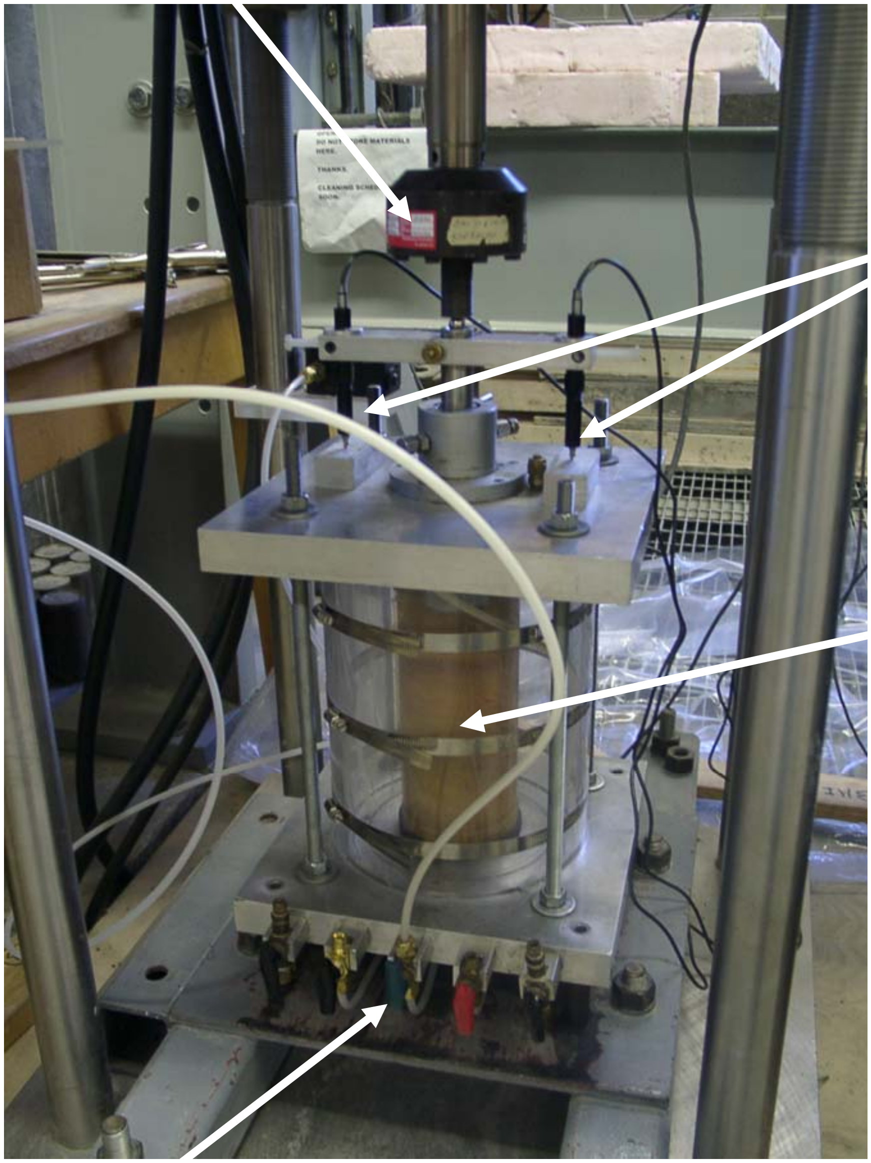


Figure 3. 16. Specimen under going resilient modulus test.

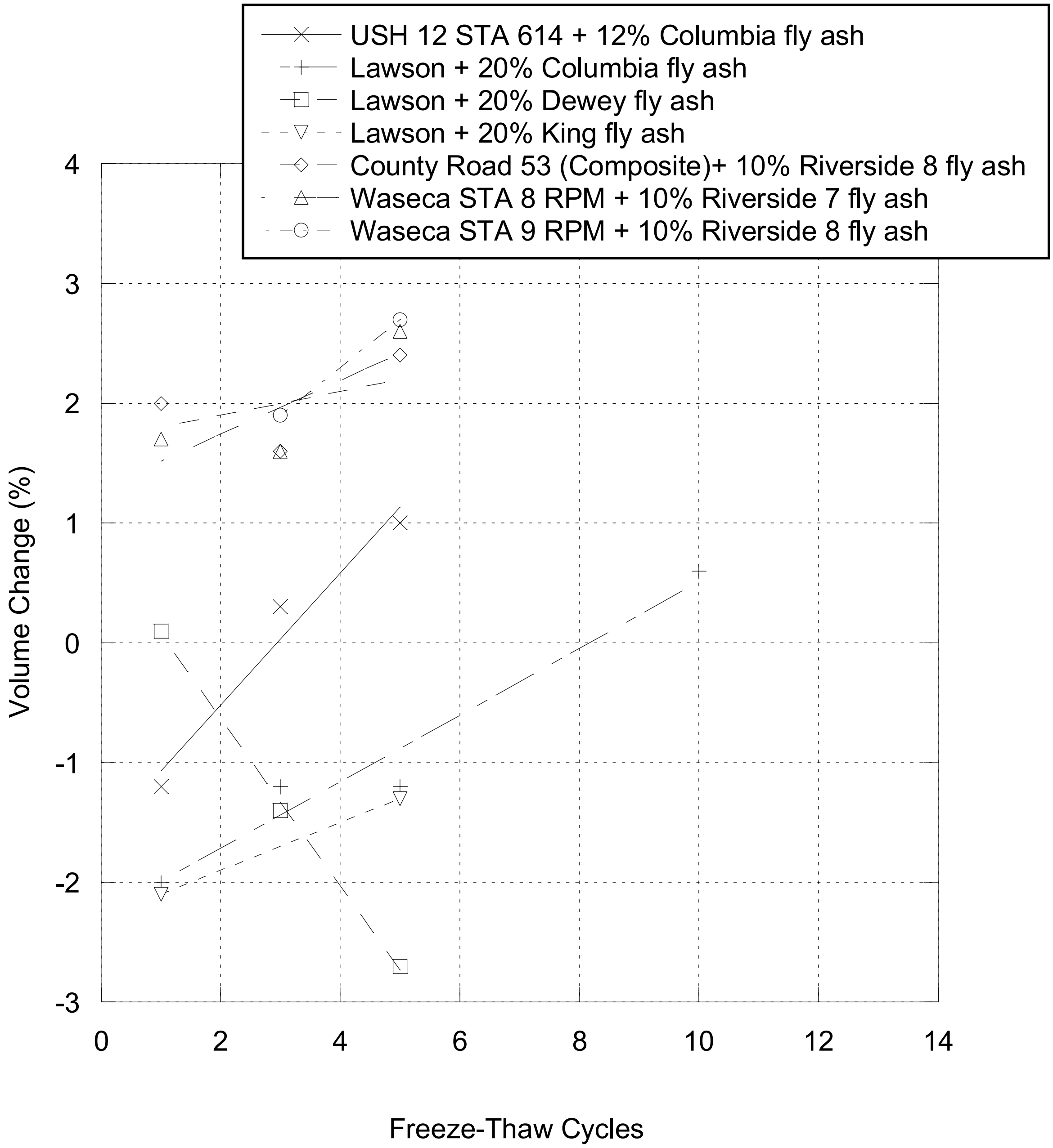


Figure 4. 1 Volume change vs. number of freeze-thaw cycles of all soil-fly ashes mixtures

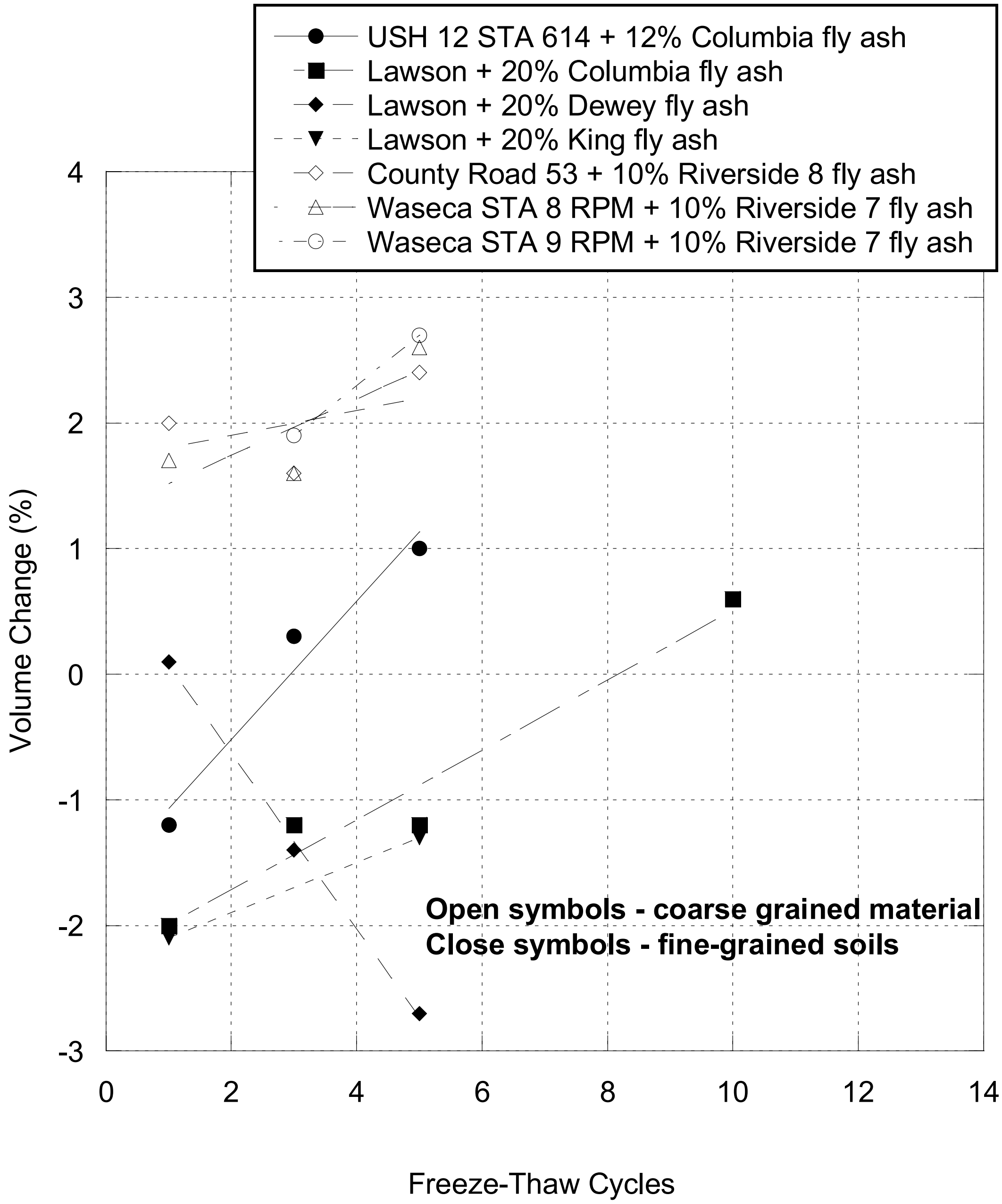


Figure 4. 2 Volume change (%) vs. number of freeze-thaw cycles of coarse grained material-fly ash mixtures and fine grained soil-fly ash mixtures.

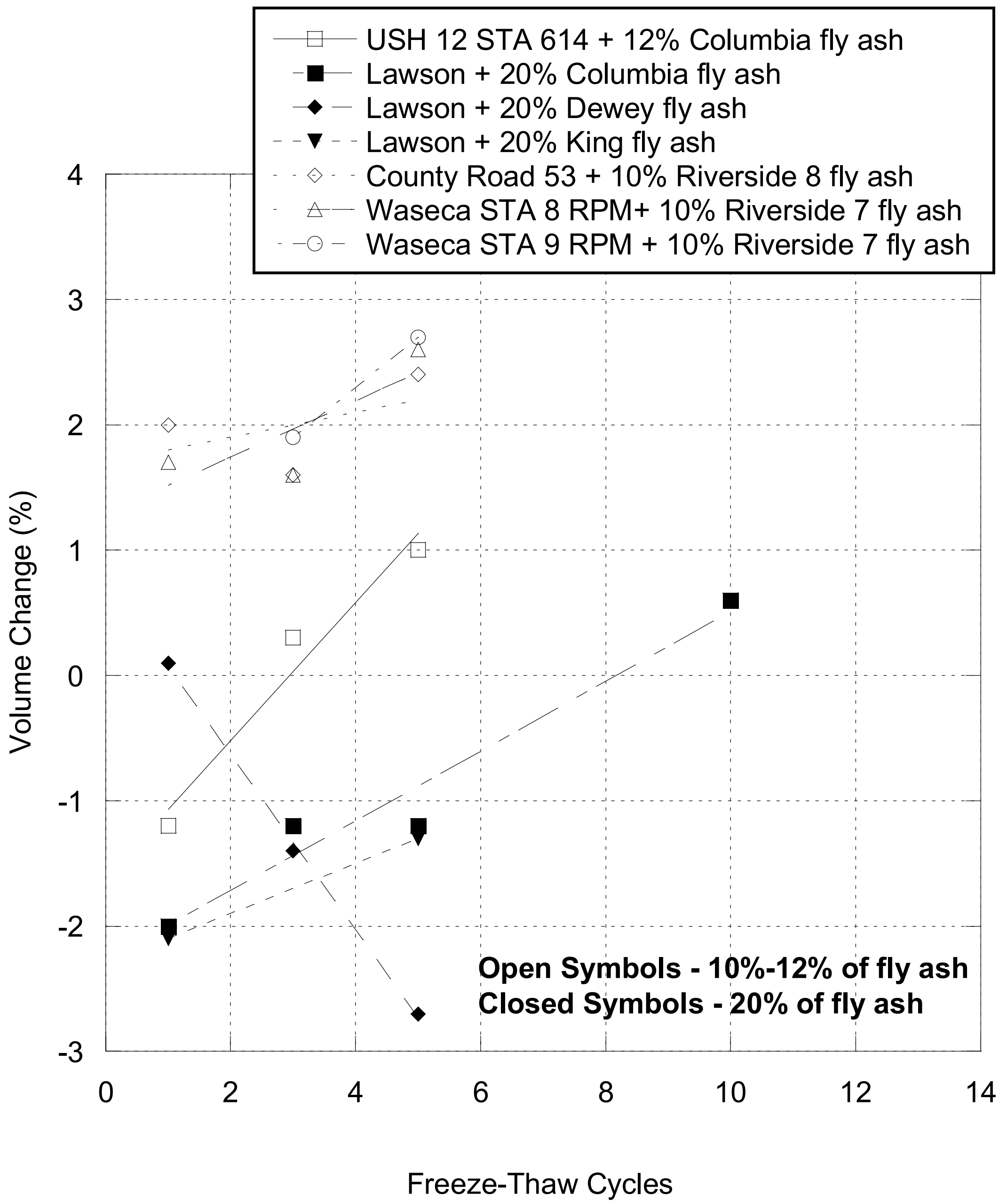


Figure 4. 3 Volume change (%) vs. freeze-thaw cycles, 10%-12% fly ash content and 20% fly ash content.

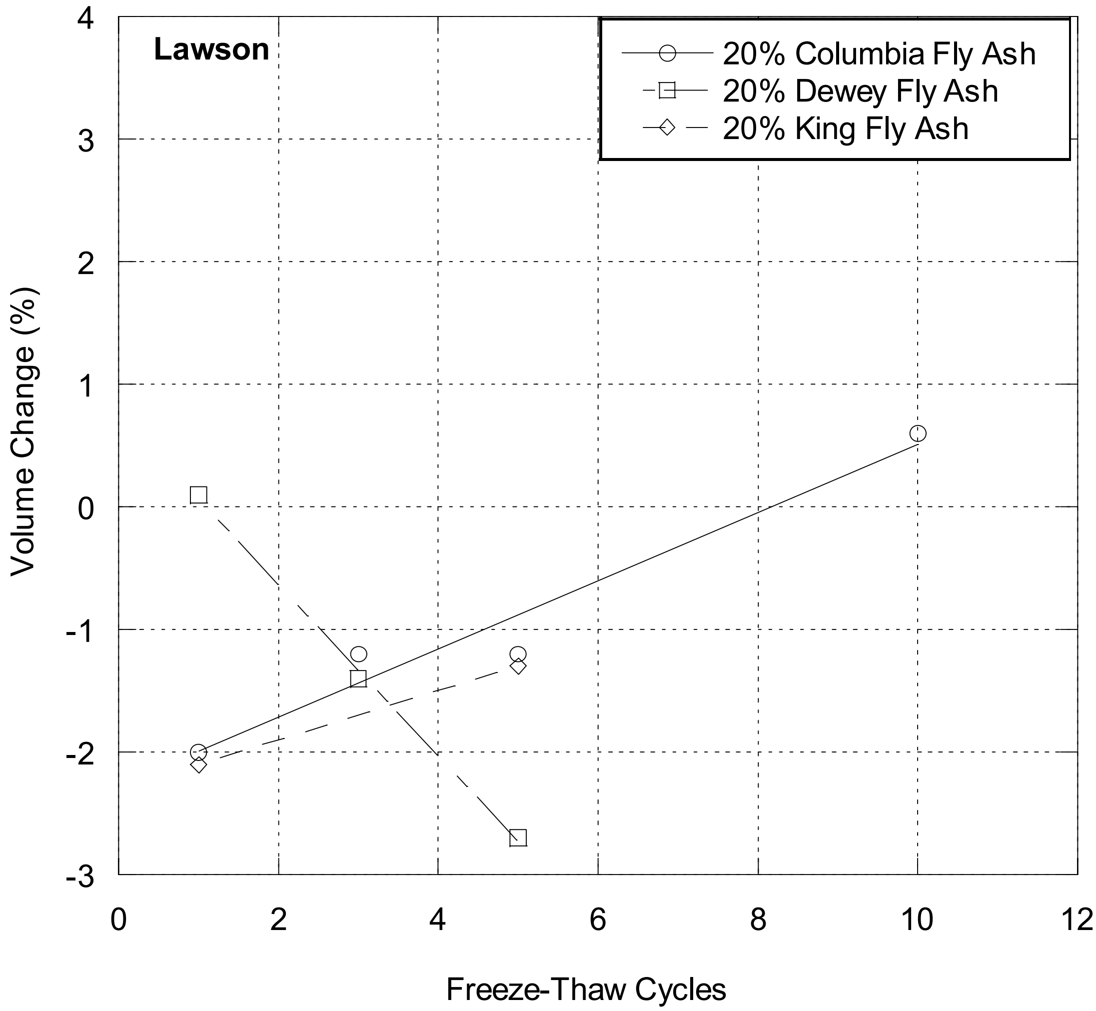


Figure 4. 4 Volume change after freeze-thaw cycles of Lawson stabilized with three different fly ashes

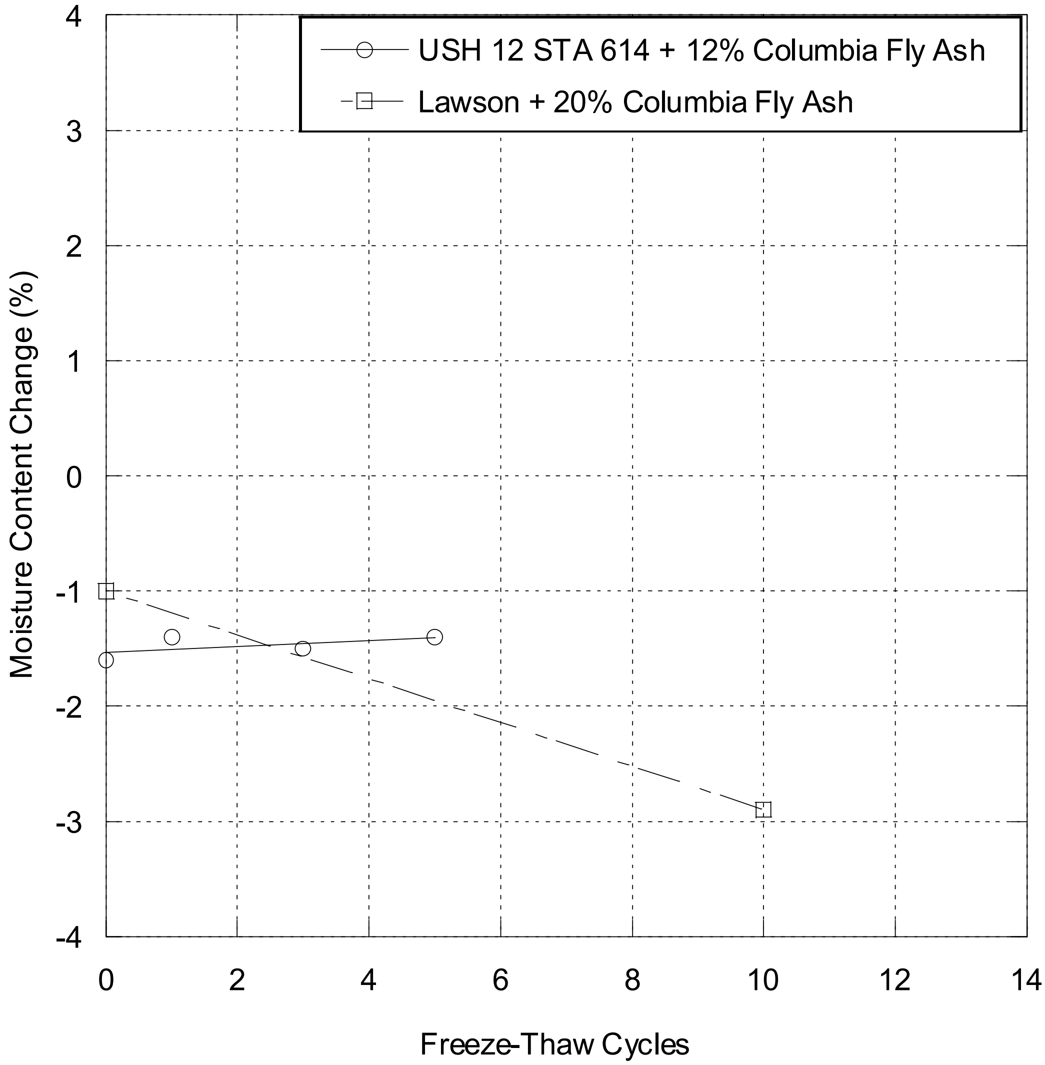


Figure 4. 5. Moisture Content Change vs. number of freeze-thaw cycles of the fined grained soil-fly ashes mixtures

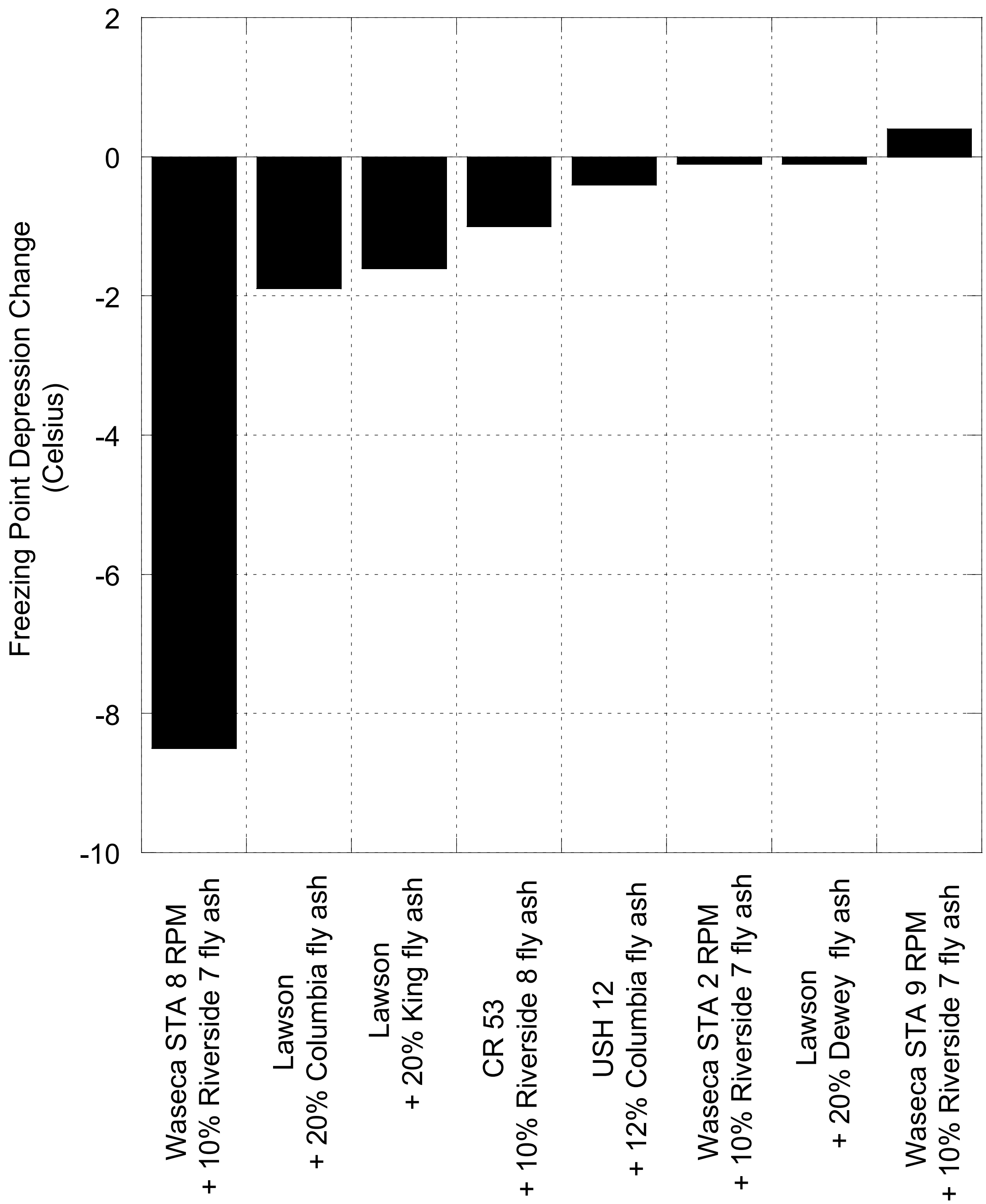


Figure 4. 6. Freezing point depression change after samples are mixed with fly ash

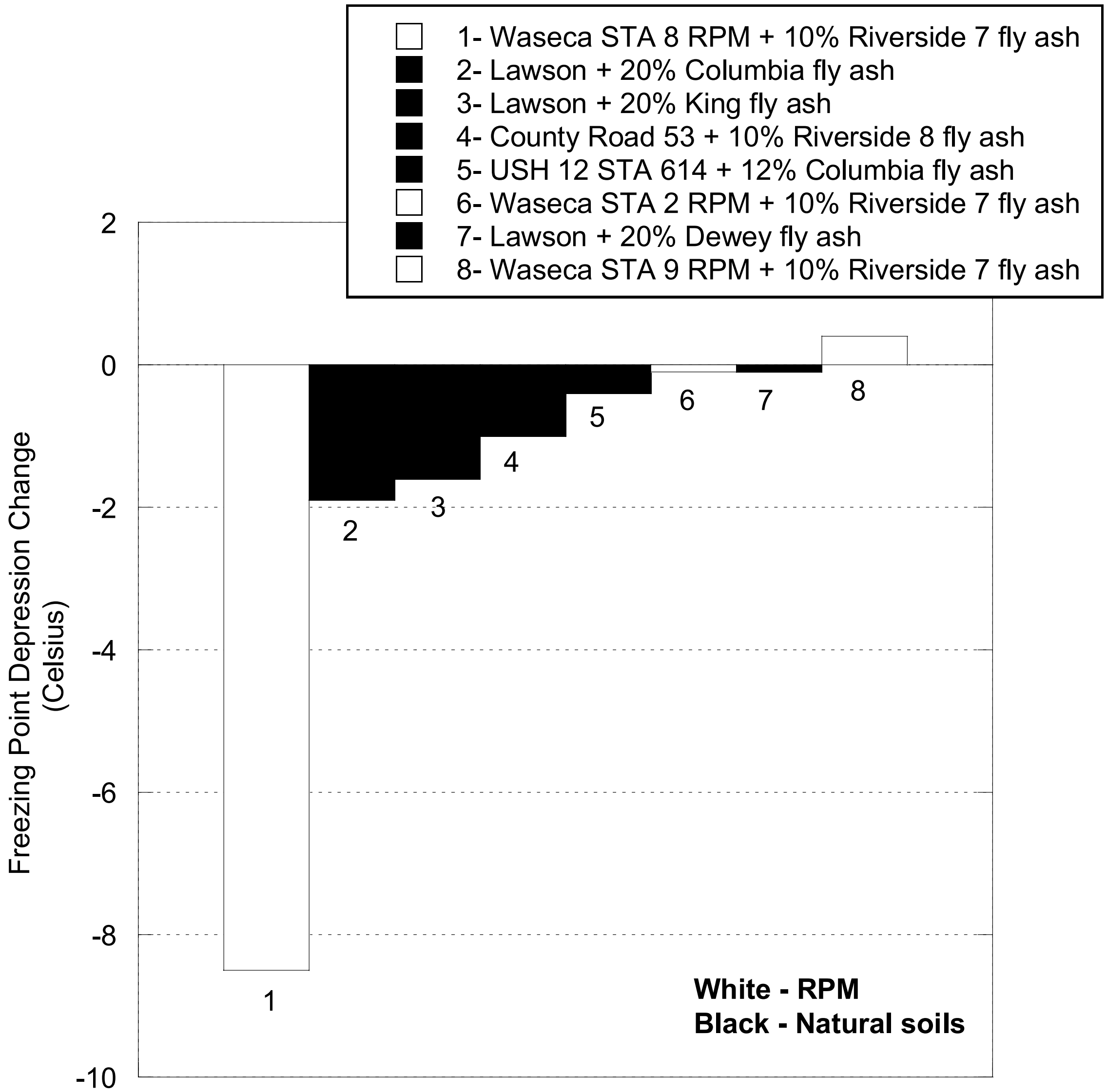


Figure 4. 7. Freezing point depression change of recycled pavement material vs. natural soils

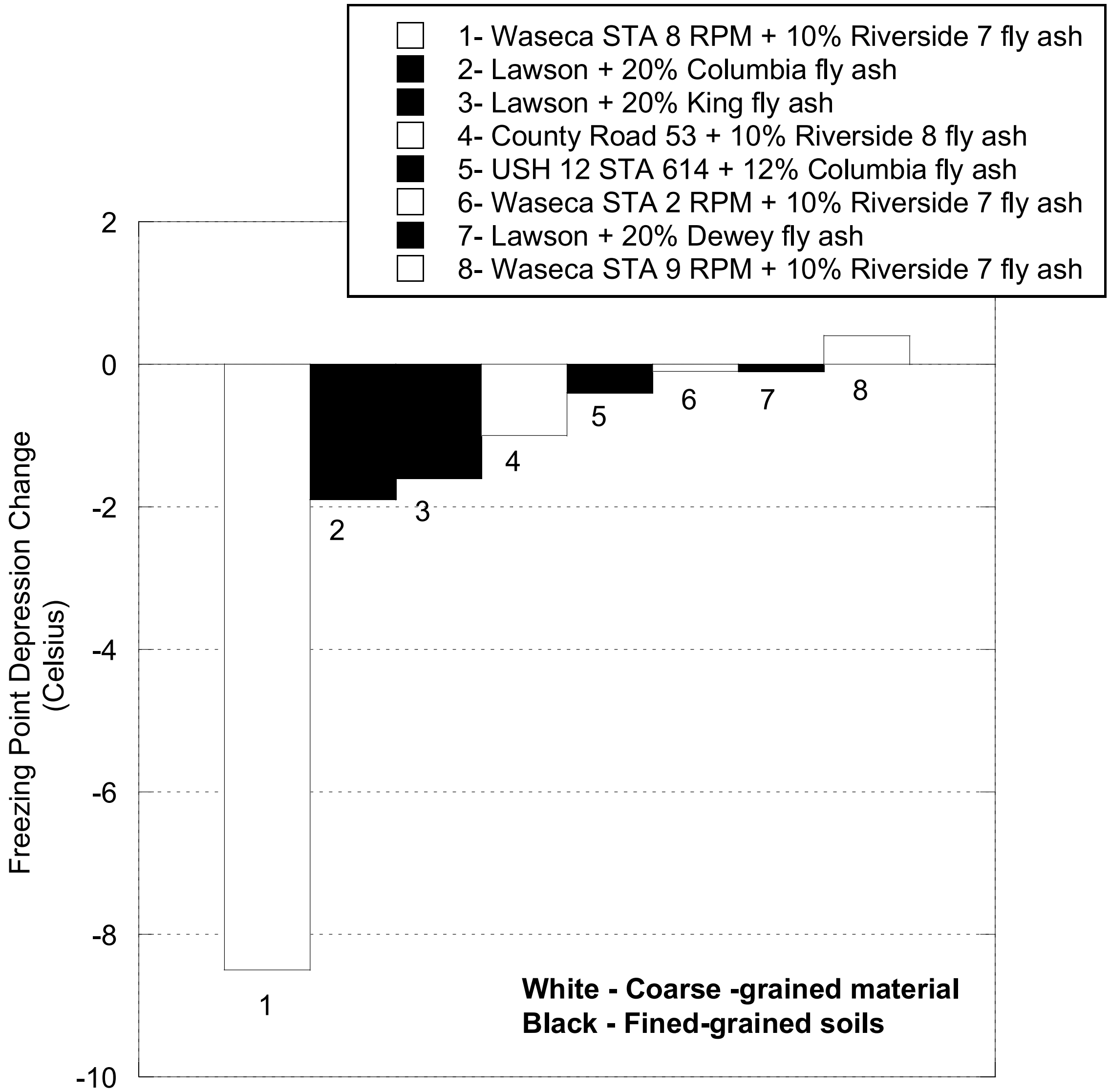


Figure 4. 8. Freezing point depression change of coarse grained soil vs. fine-grained soil

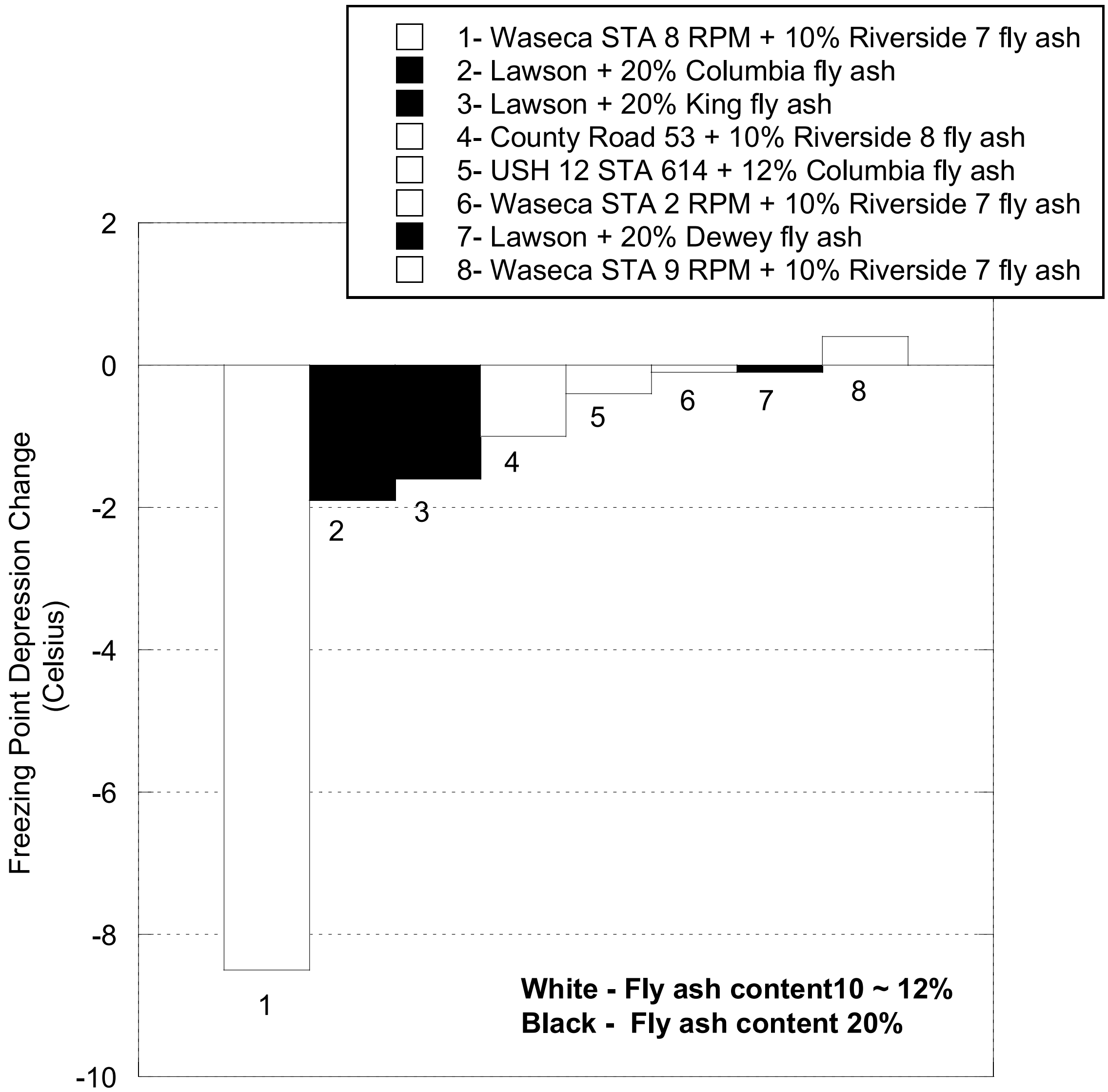


Figure 4. 9. Freezing point depression change of lower fly ash content vs. highest fly ash content

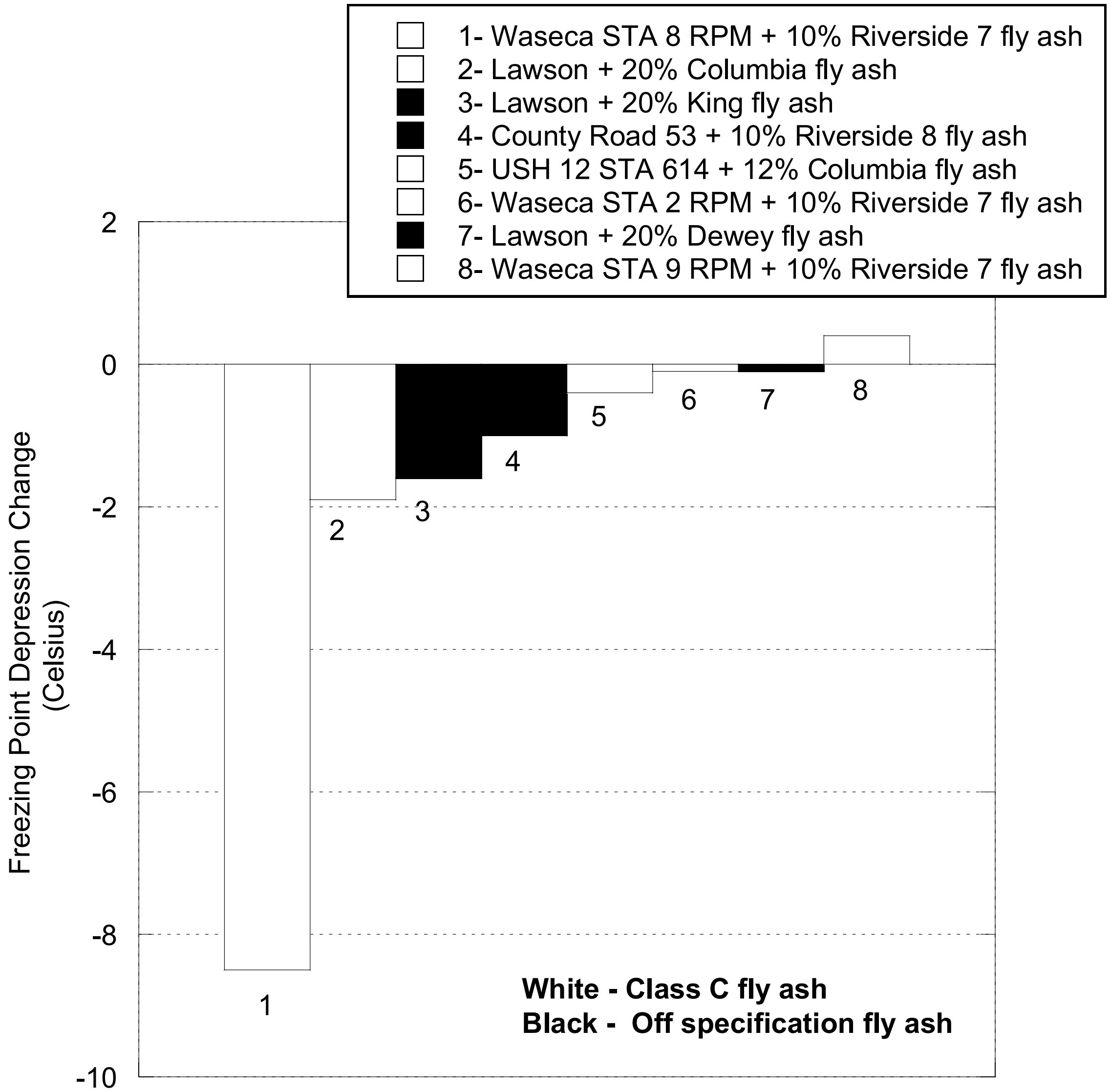


Figure 4. 10. Freezing point depression change of soils or materials stabilized with Class C fly ash vs. Off-specification fly ash

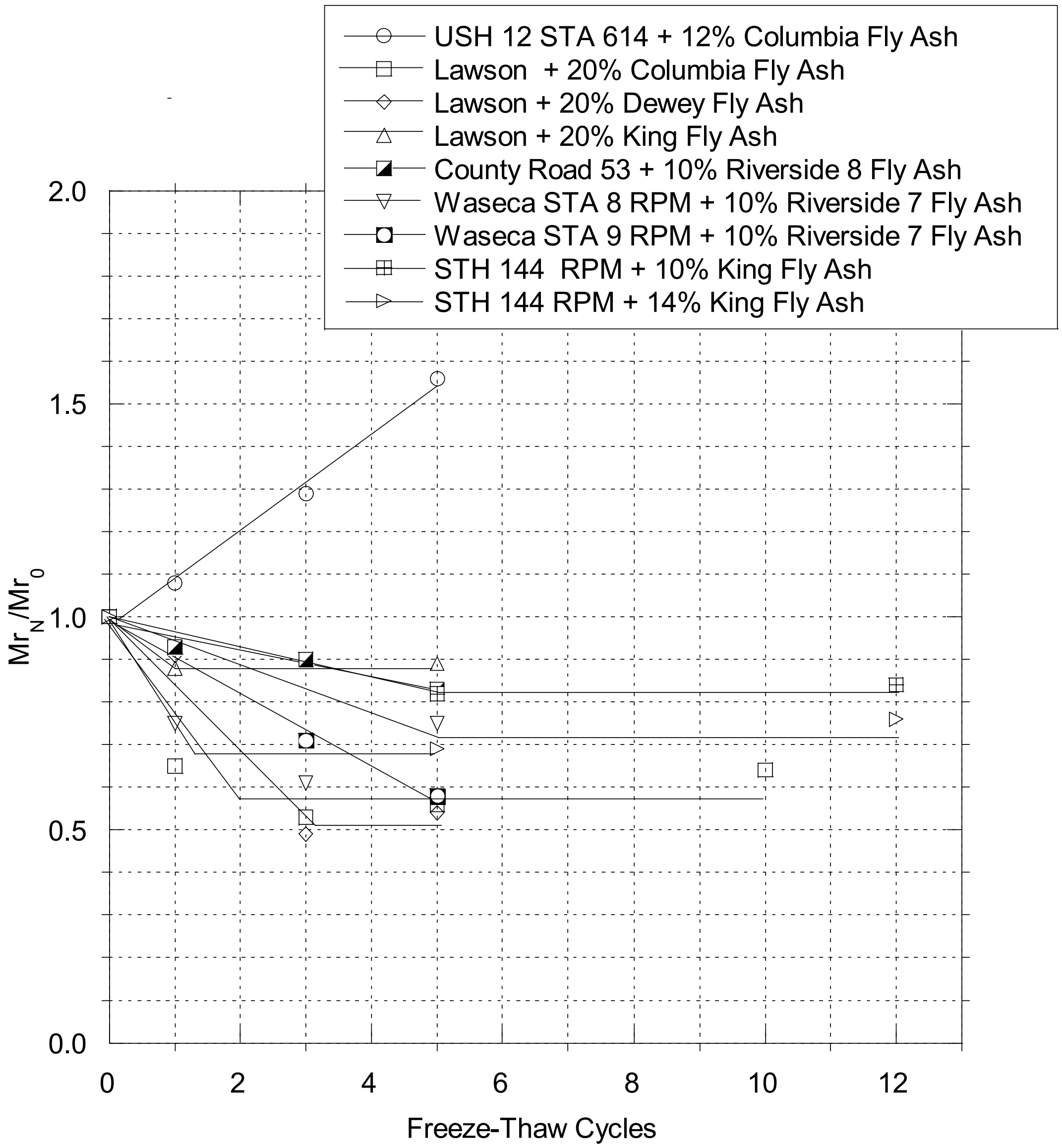


Figure 4. 11. Normalized resilient modulus vs. freeze-thaw cycles for all soil-fly ash mixtures

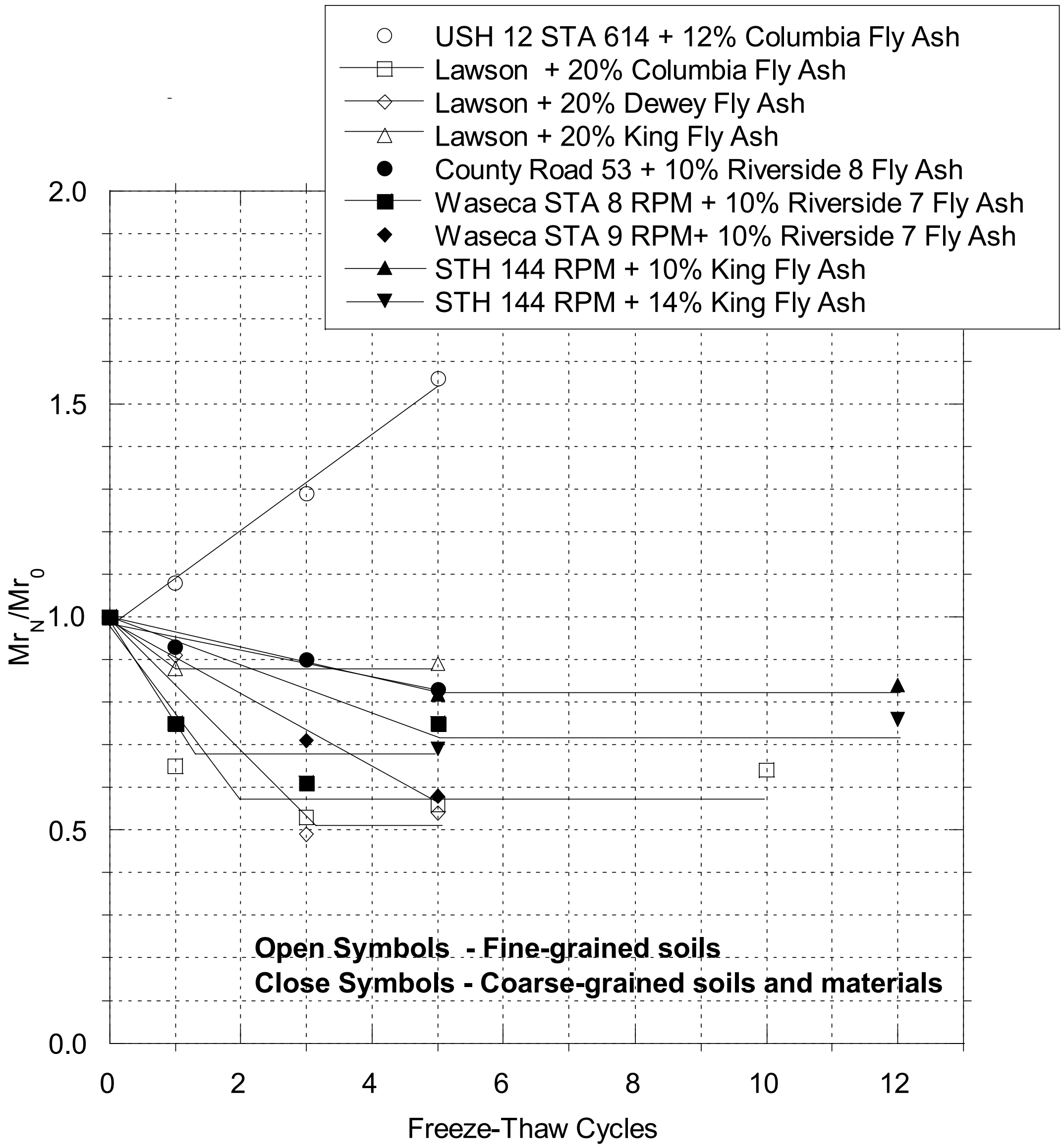


Figure 4. 12. Normalized resilient modulus vs. freeze-thaw cycles of fined-grained soils (open symbols) and coarse material (solid symbols)

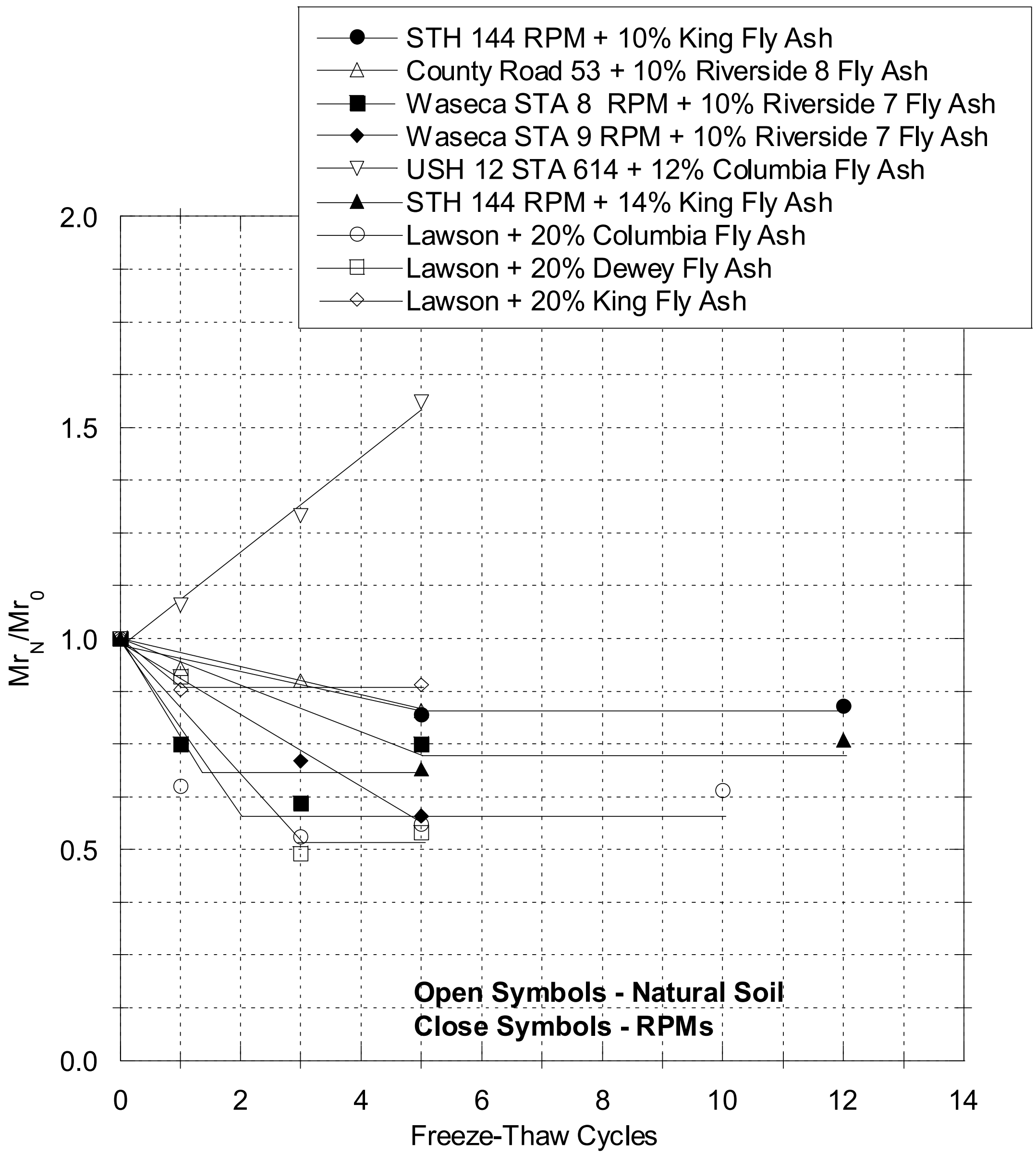


Figure 4. 13 Normalized resilient modulus natural soils vs. RPM's (Recycled Pavement Material)

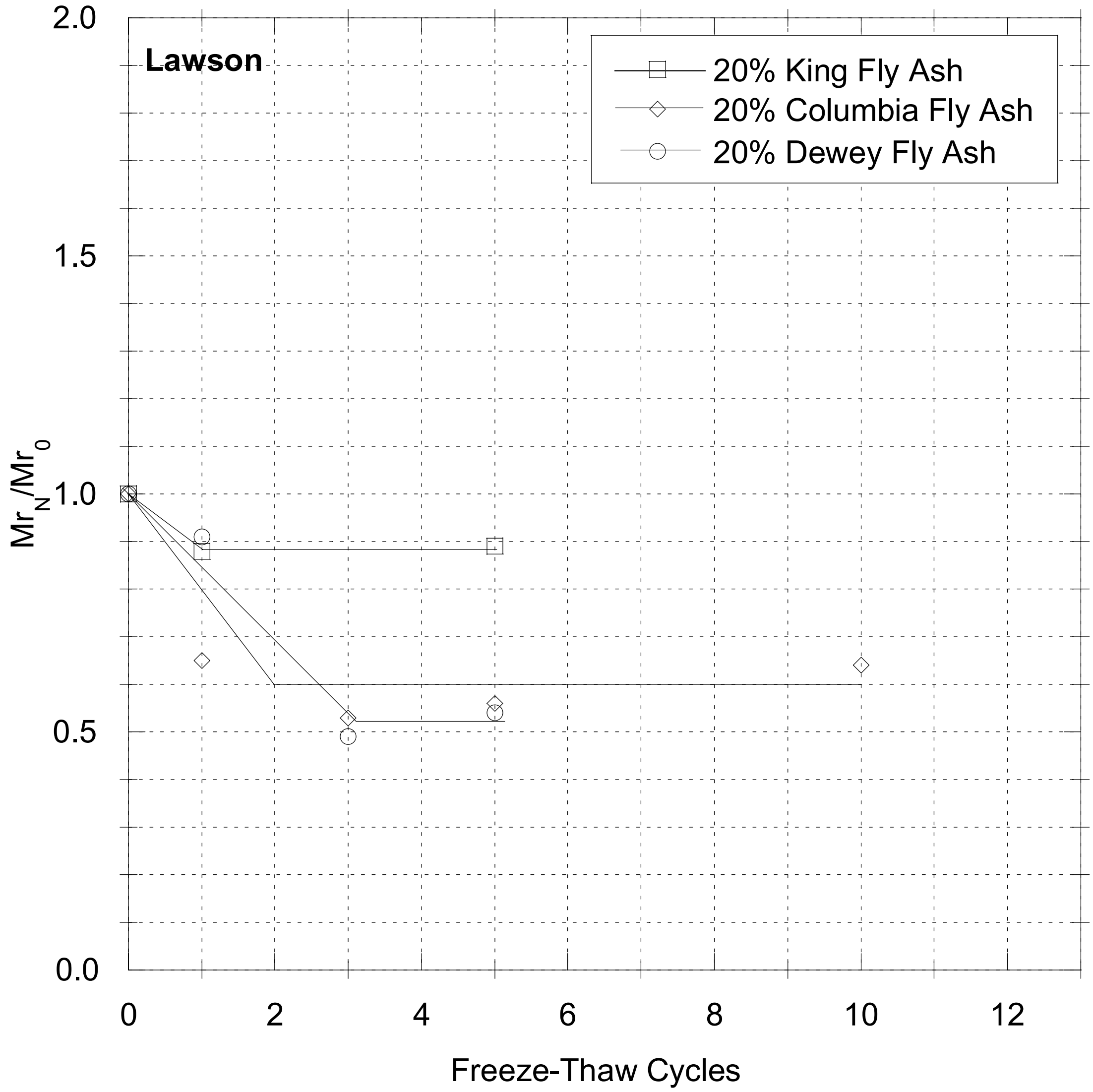


Figure 4. 14 Lawson normalized resilient modulus stabilized with three different fly ashes vs. freeze-thaw cycles

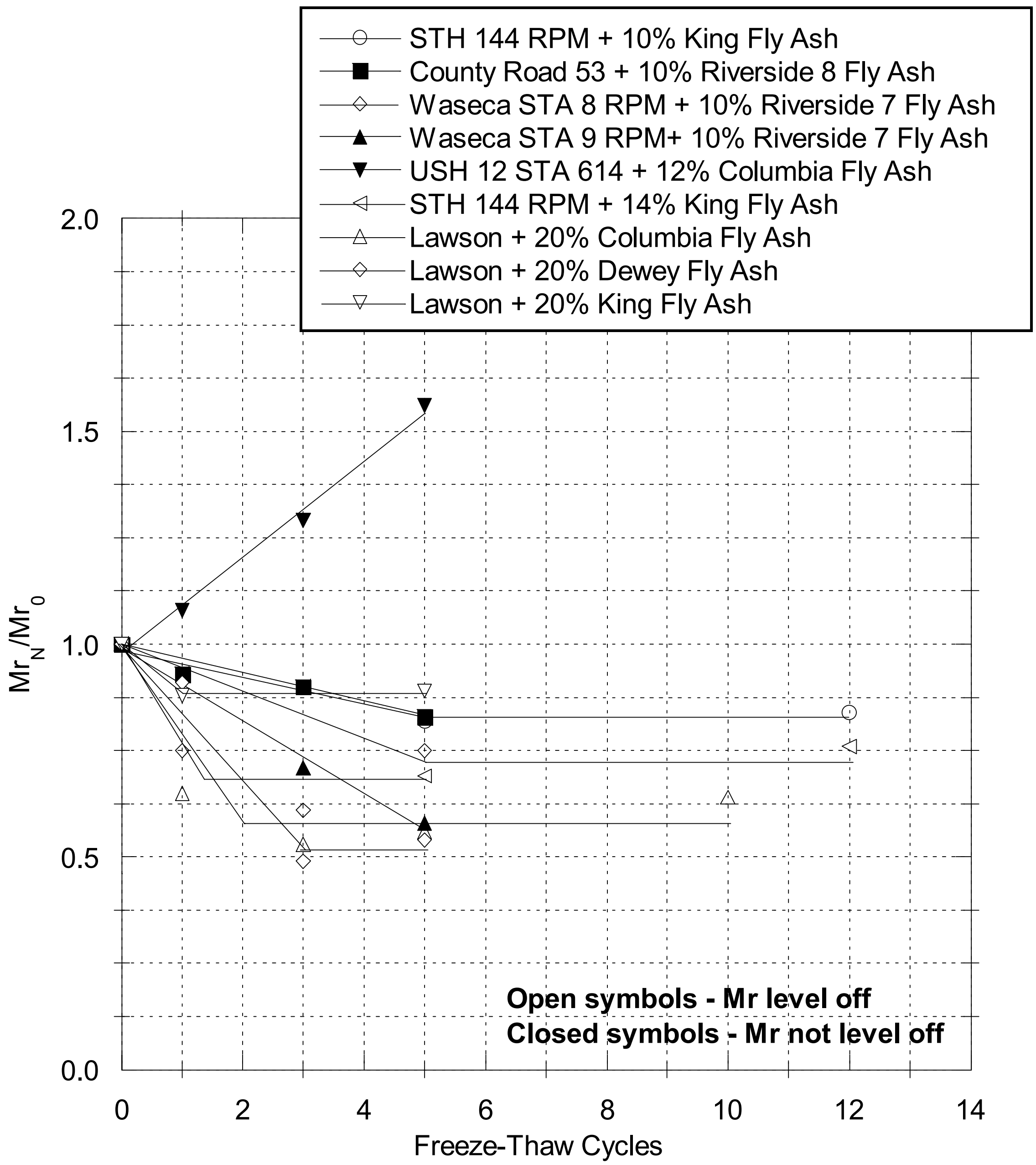


Figure 4. 15. Normalized resilient modulus vs. number of freeze-thaw cycles
Open symbols (level off) and closed symbols (not level off)

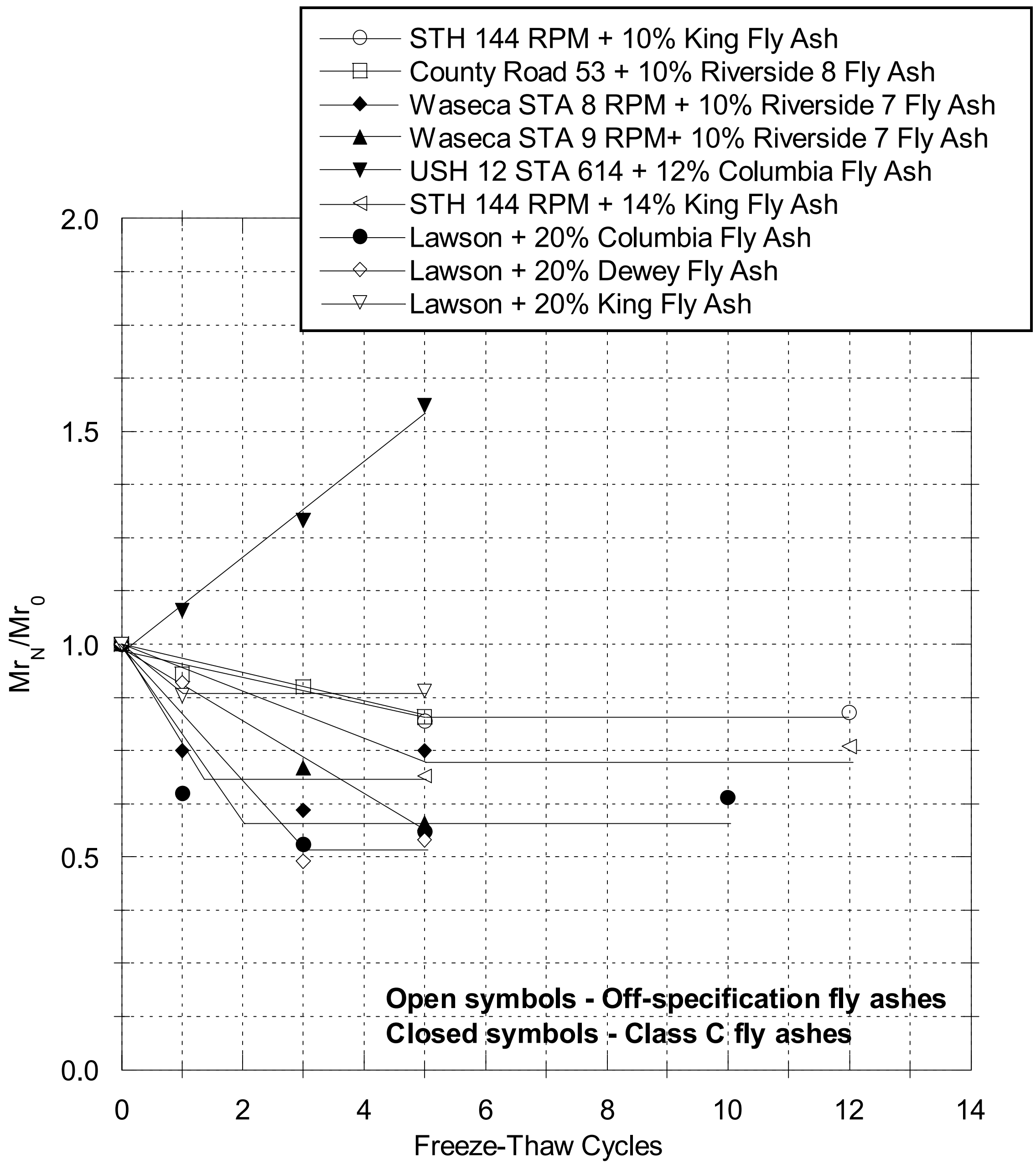


Figure 4. 16. Normalized resilient modulus vs. freeze-thaw cycles (Open symbols represents off-specification fly ashes and closed symbols represent Class C fly ash).

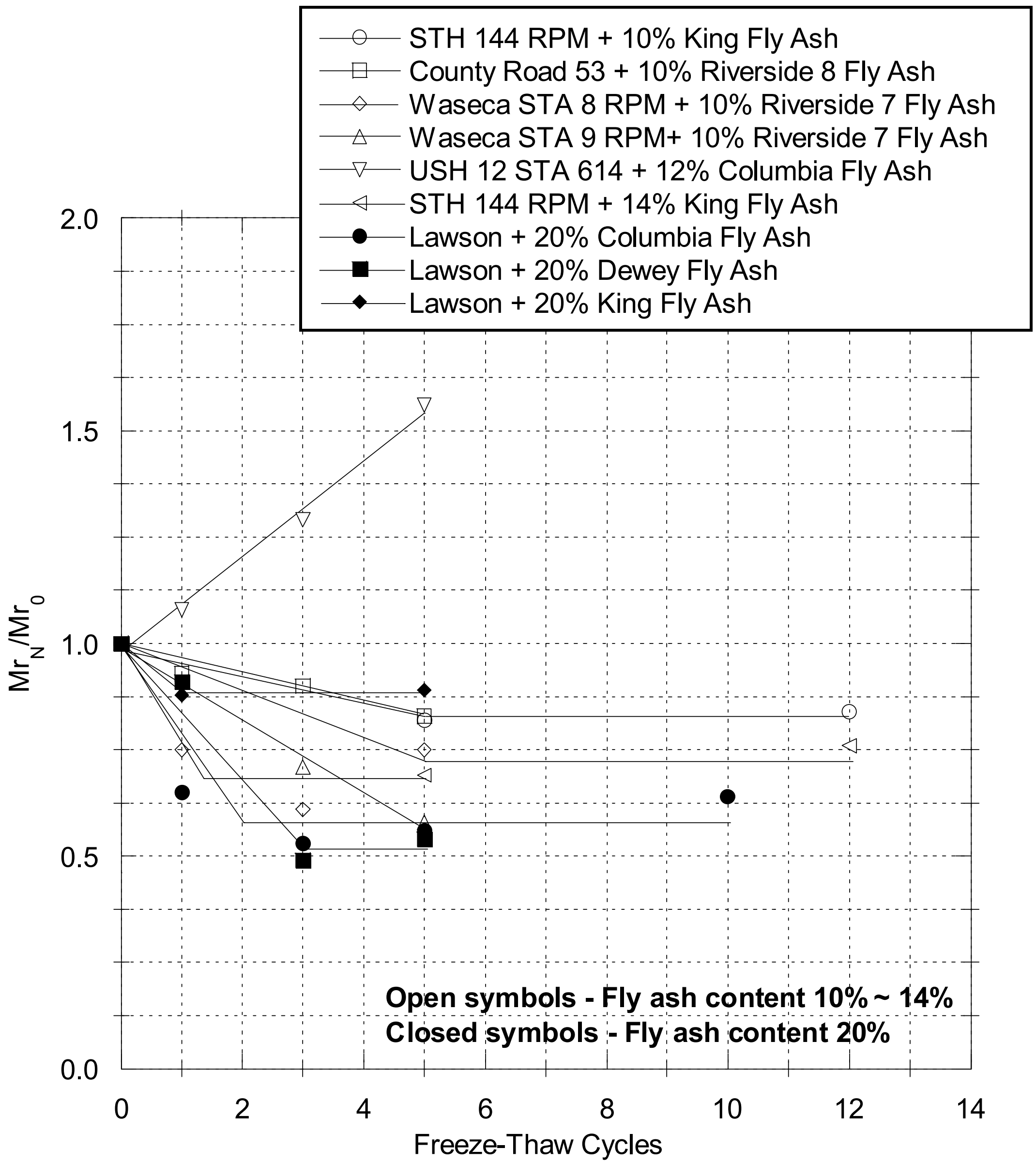


Figure 4. 17 Normalized resilient modulus vs. freeze –thaw cycles; opened symbols represents fly ash content between 10 to 14% and closed symbols represents fly ash content of 20%.

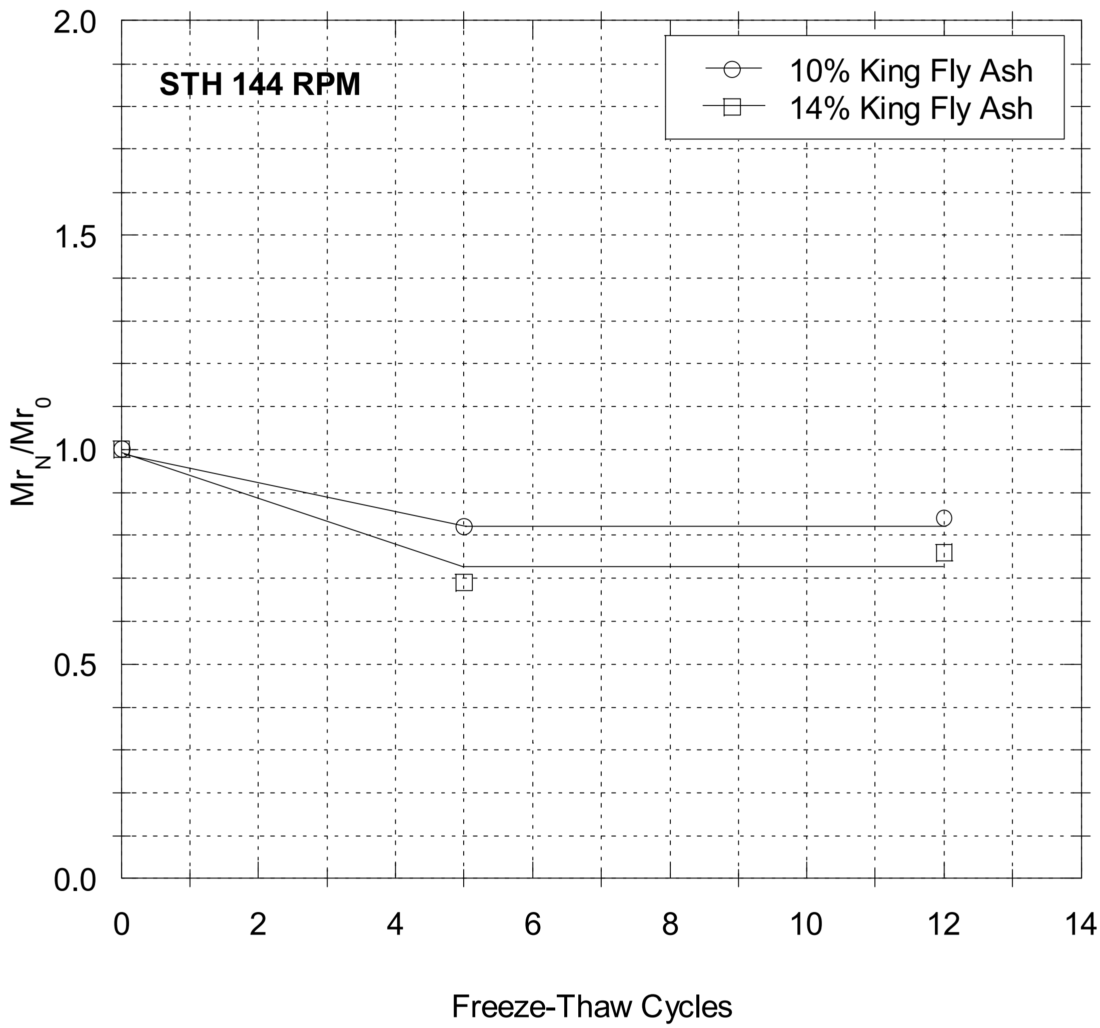


Figure 4. 18. STH 144 normalized resilient modulus vs. freeze-thaw cycles

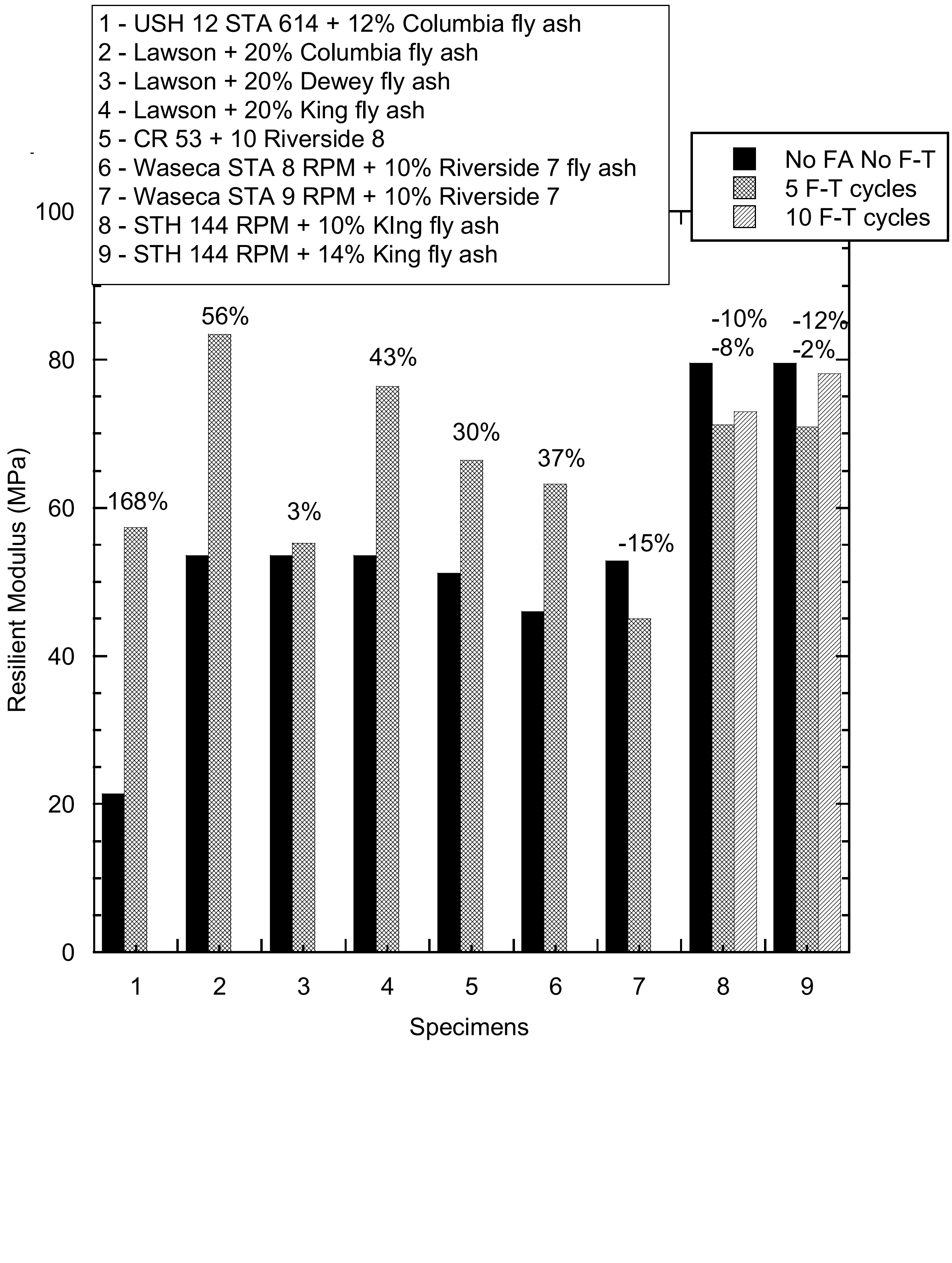


Figure 4. 19 Comparison of the resilient modulus values without fly ash and unfrozen with the resilient modulus of the soil-fly ash and granular material fly after 5 or 12 freeze-thaw cycles.

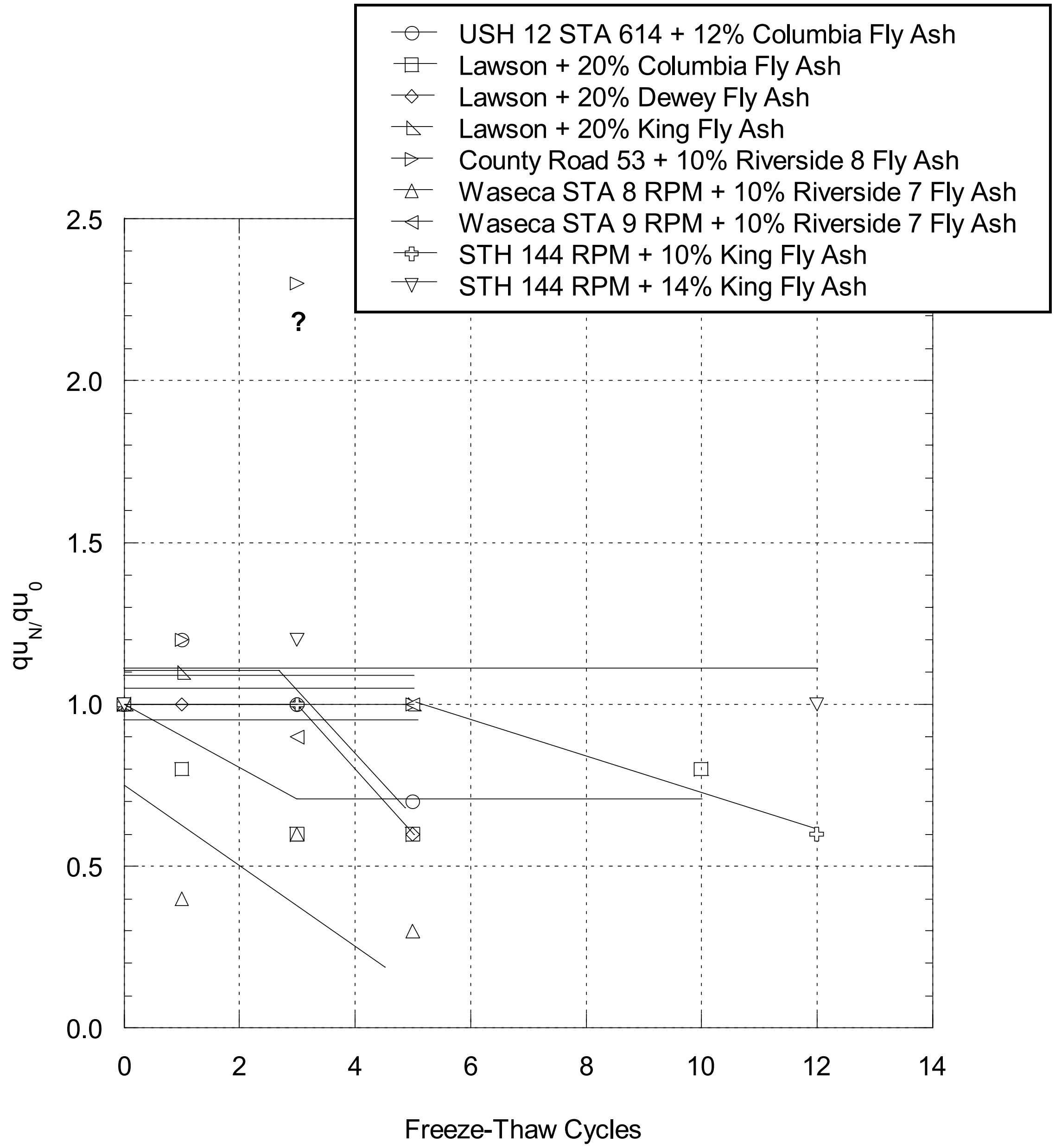


Figure 4. 20. Normalized unconfined compressive strength vs. freeze-thaw cycles

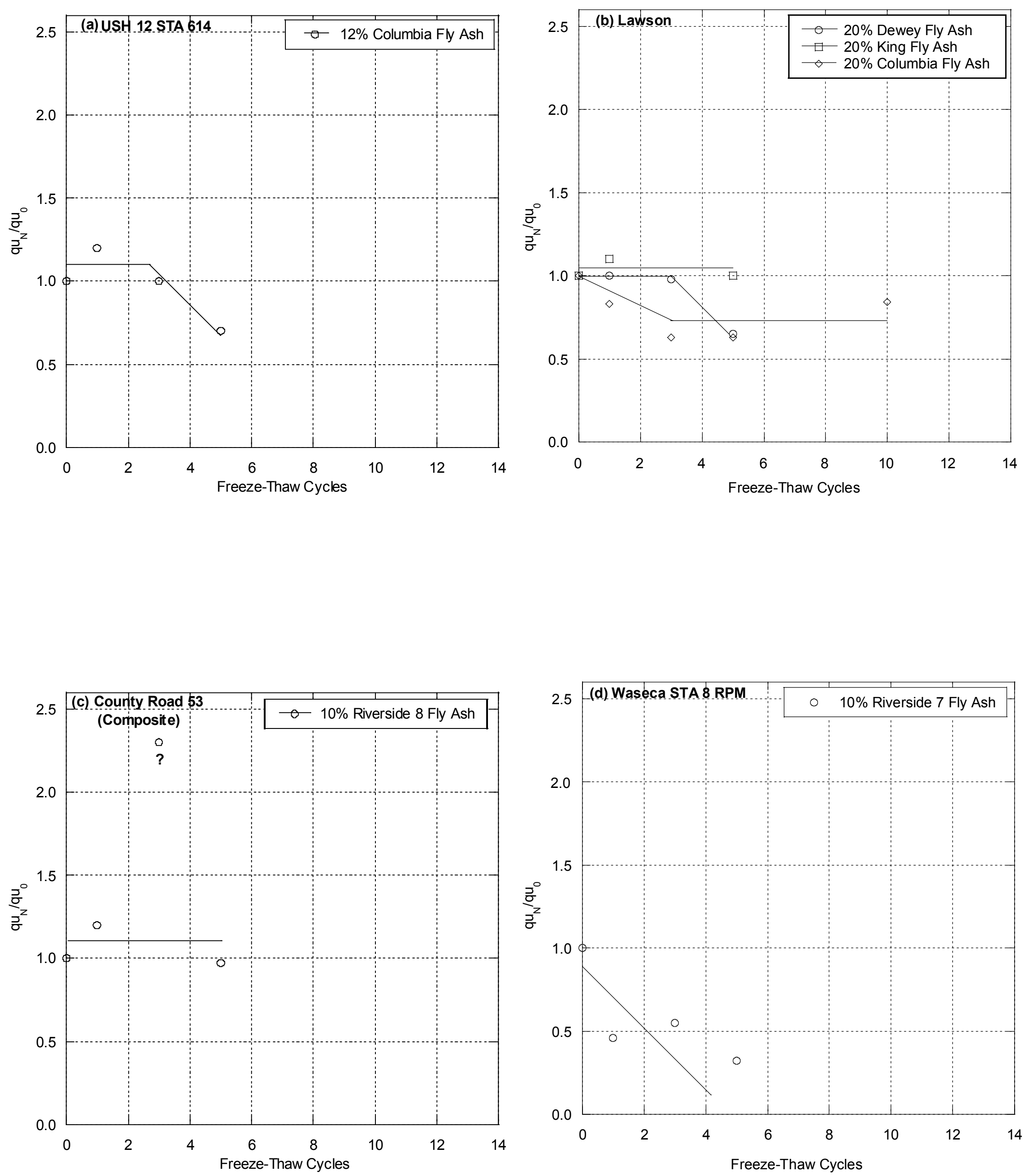


Figure 4. 21 Normalized unconfined compressive strength vs. freeze-thaw cycles; USH 12 STA 614 + 12% Columbia Fly Ash (a), Lawson + 20%Columbia, Dewey and King Fly Ash (b), County Road 53 + 10% Riverside 8 Fly Ash (c), and Waseca STA 8 RPM + 10% Riverside 7 Fly Ash (d).

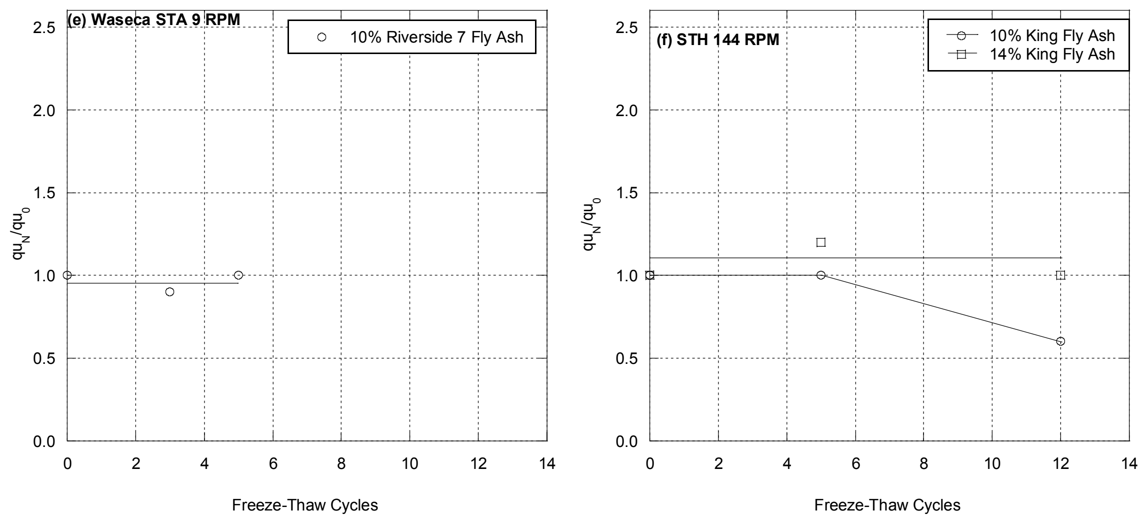


Figure 4. 22. Normalized unconfined compressive strength vs. freeze-thaw cycles; Waseca STA 9 RPM + 10% Riverside 7 fly ash (e), and STH 144 RPM + 10% and 14% King fly ash.

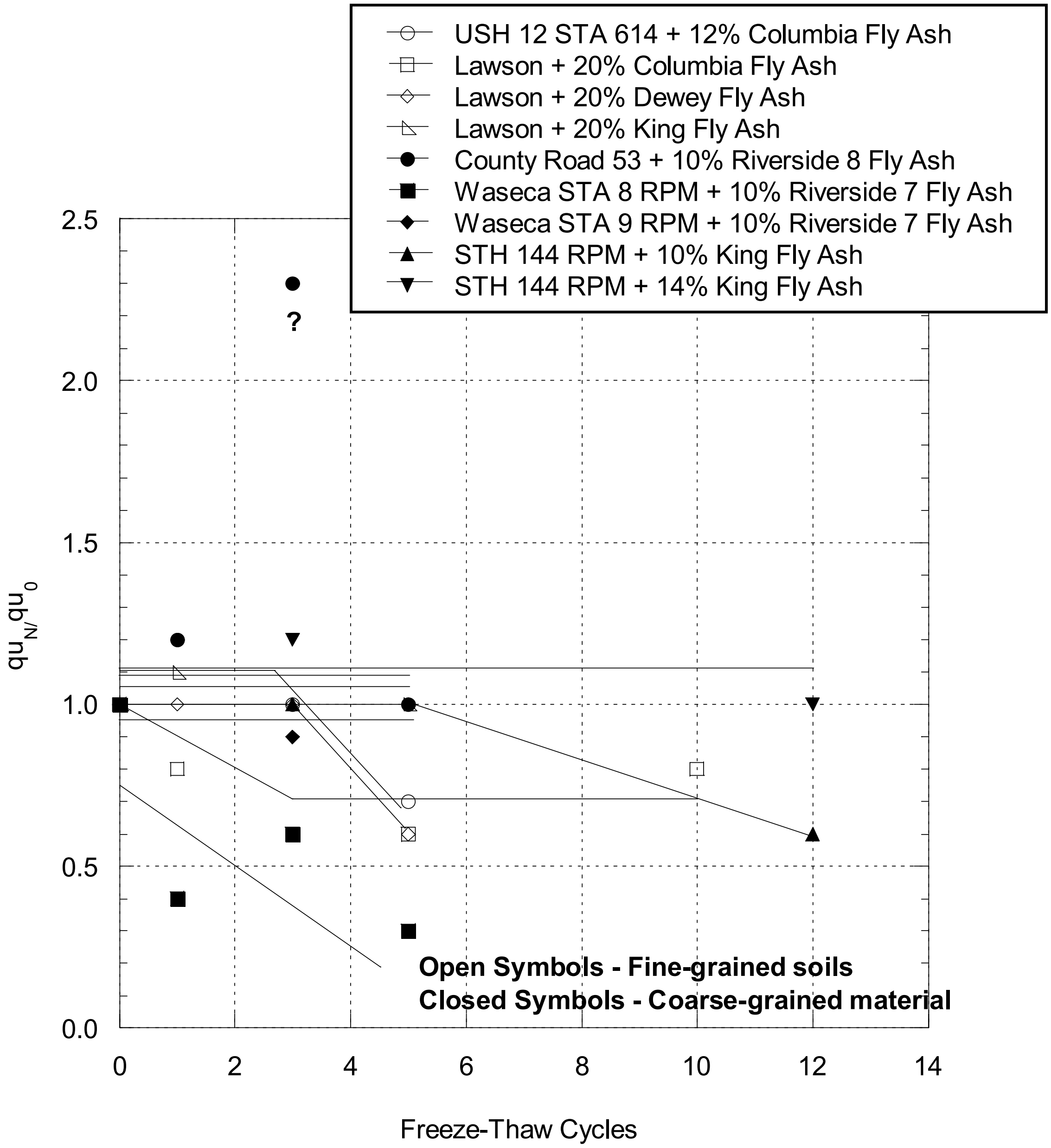


Figure 4. 23. Normalized qu vs. freeze-thaw cycles (fine-grained soils and coarse-grained material)

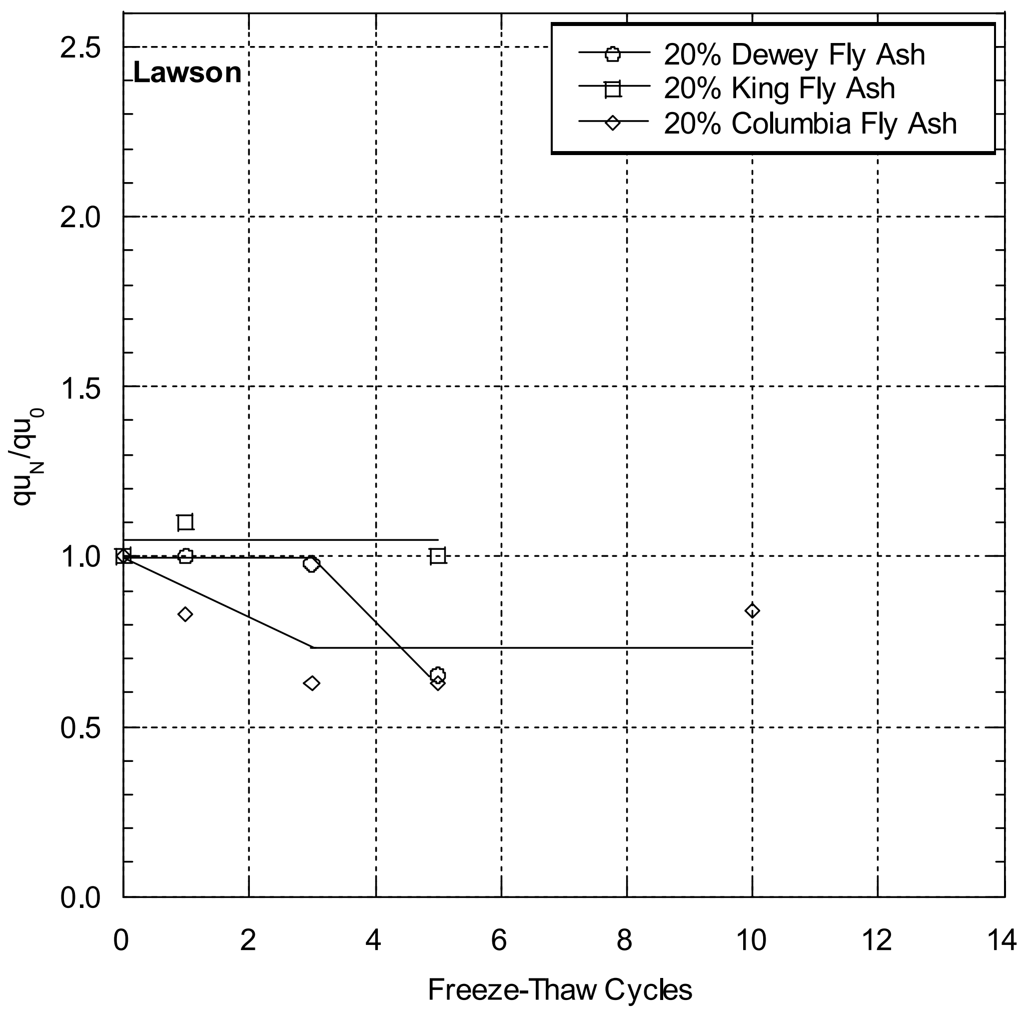


Figure 4. 24 Lawson stabilized with three different fly ashes

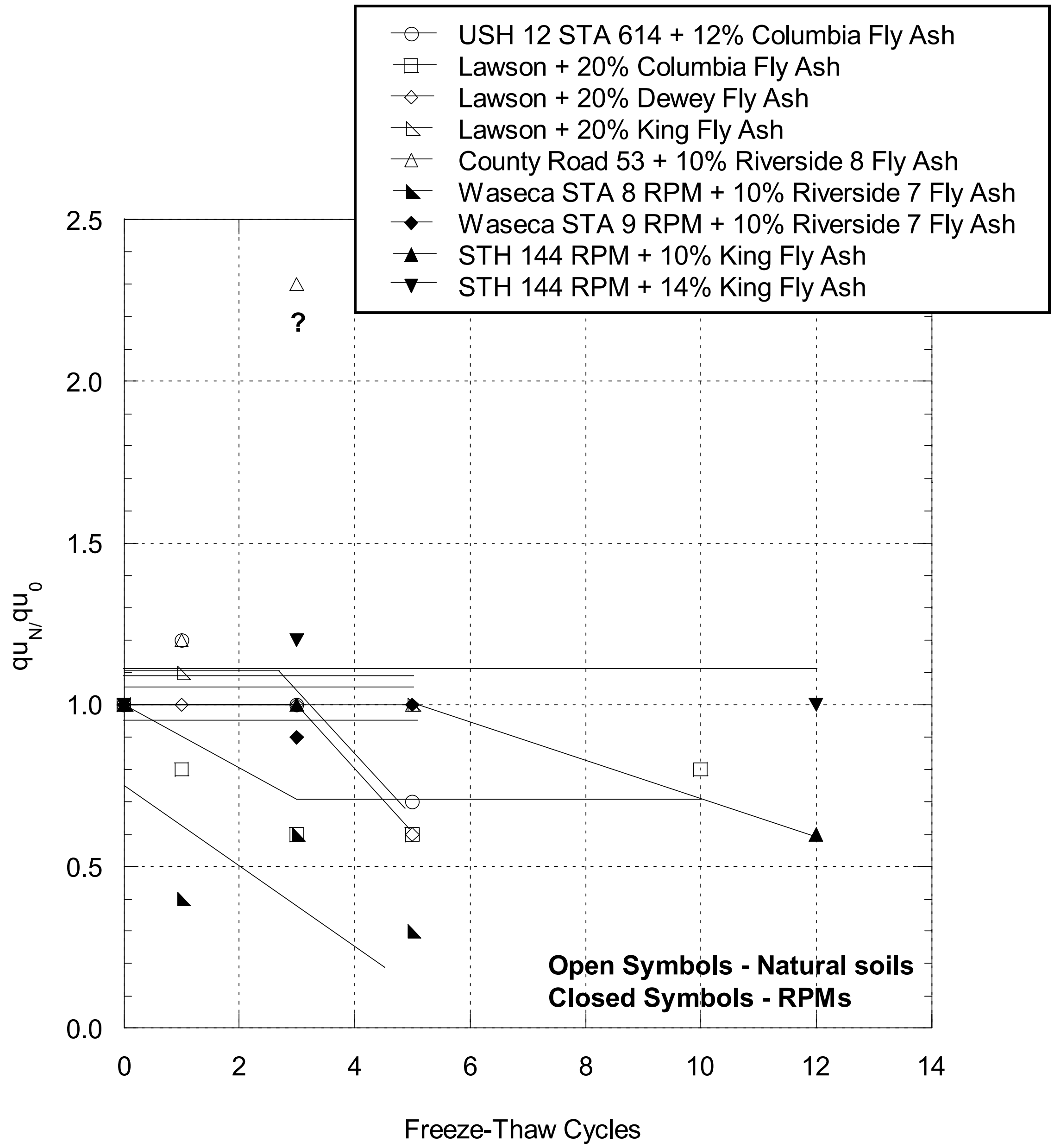


Figure 4. 25Normalized qu vs. freeze-thaw cycles (Normalized soils, RPMs)

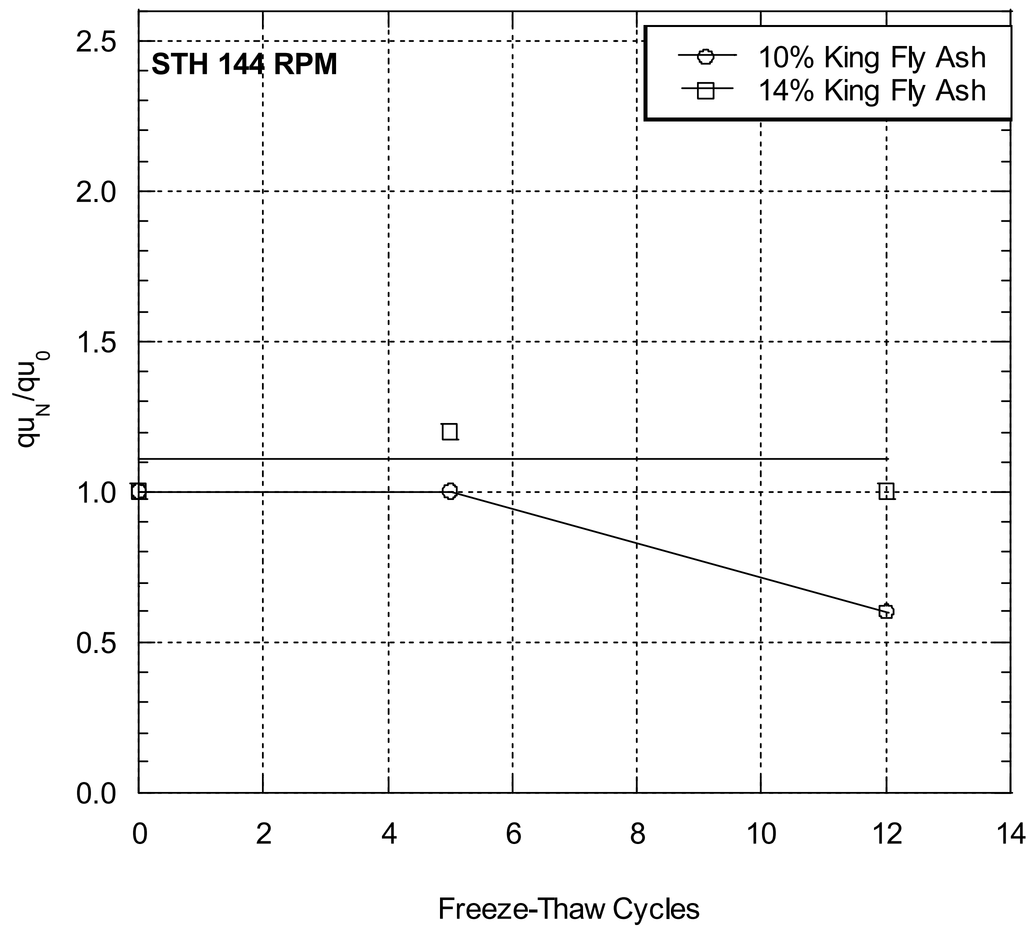


Figure 4. 26. Normalized qu vs. freeze-thaw cycles of STH 144 stabilized with different percentages of King fly ash

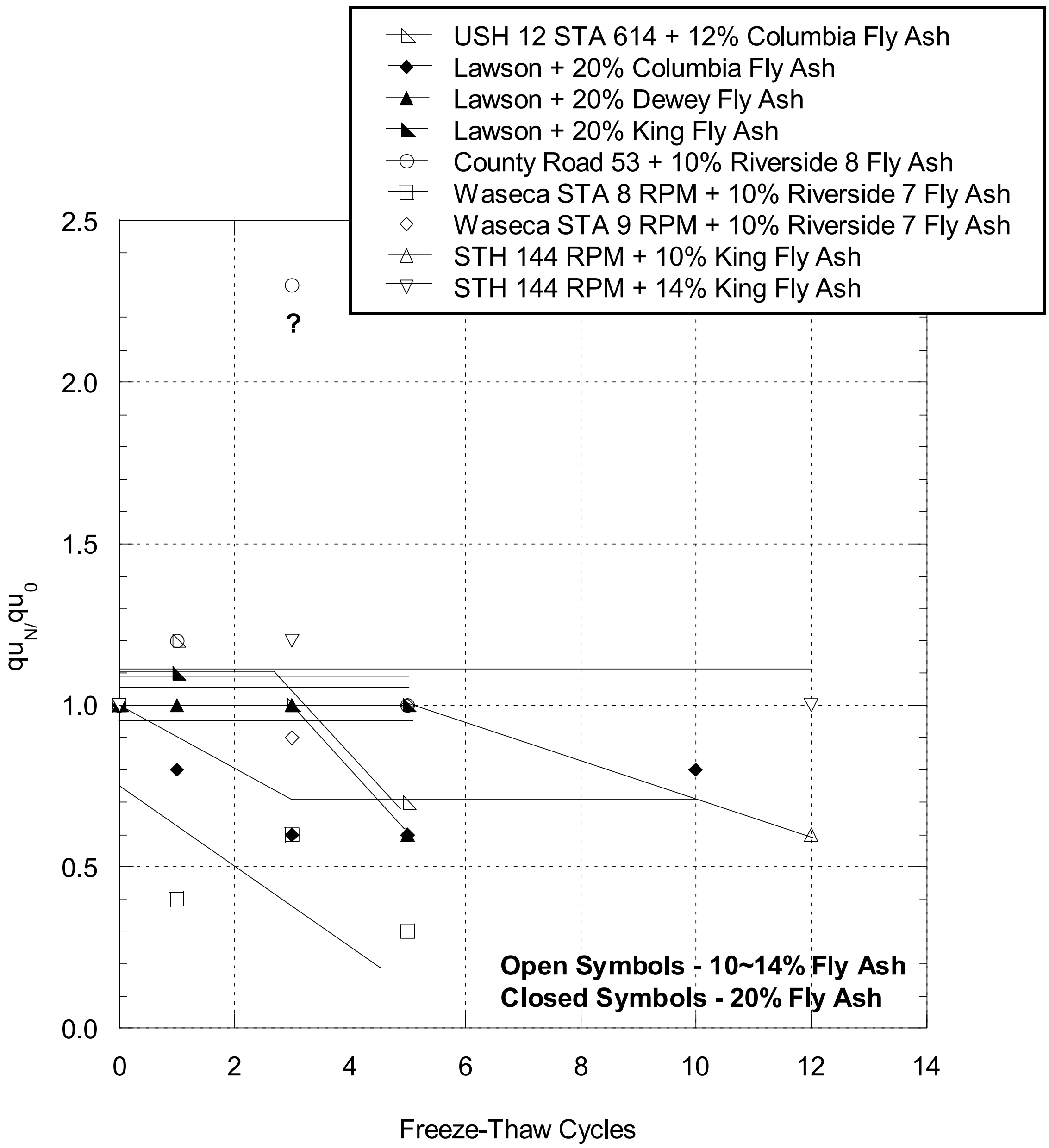


Figure 4. 27. Normalized qu vs. freeze-thaw cycles (10%~14% of fly ash and 20% of fly ash)

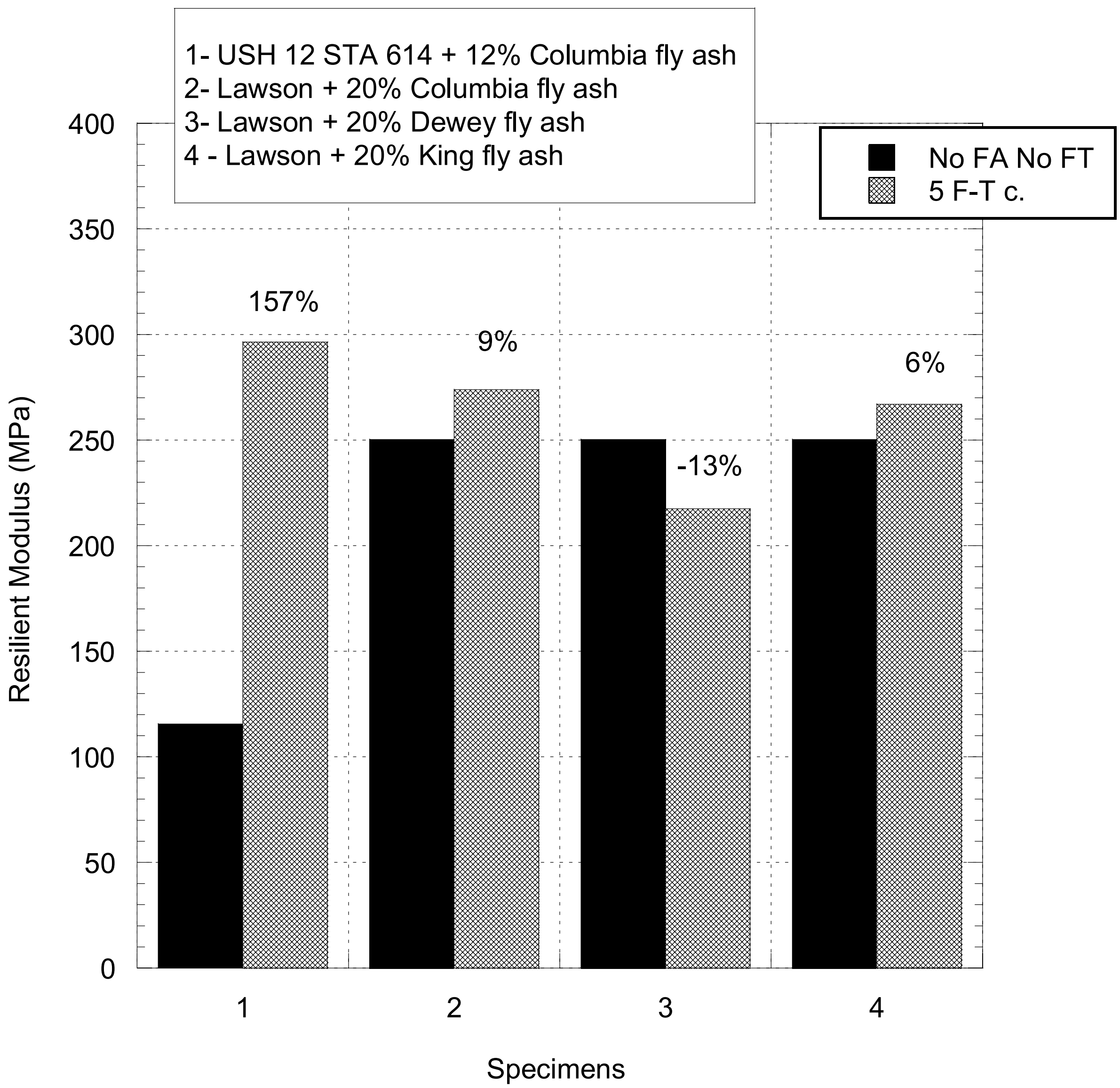

1- USH 12 STA 614 + 12% Columbia fly ash
2- Lawson + 20% Columbia fly ash
3- Lawson + 20% Dewey fly ash
4 - Lawson + 20% King fly ash
No FA No FT
5 F-T c.
157%
9%
-13%
6%
Resilient Modulus (MPa)
Specimens
0
50
100
150
200
250
300
350
400
1
2
3
4

Figure 4. 28. Q_u of unfrozen unstabilized soils compare with stabilized soil after freeze-thaw cycles

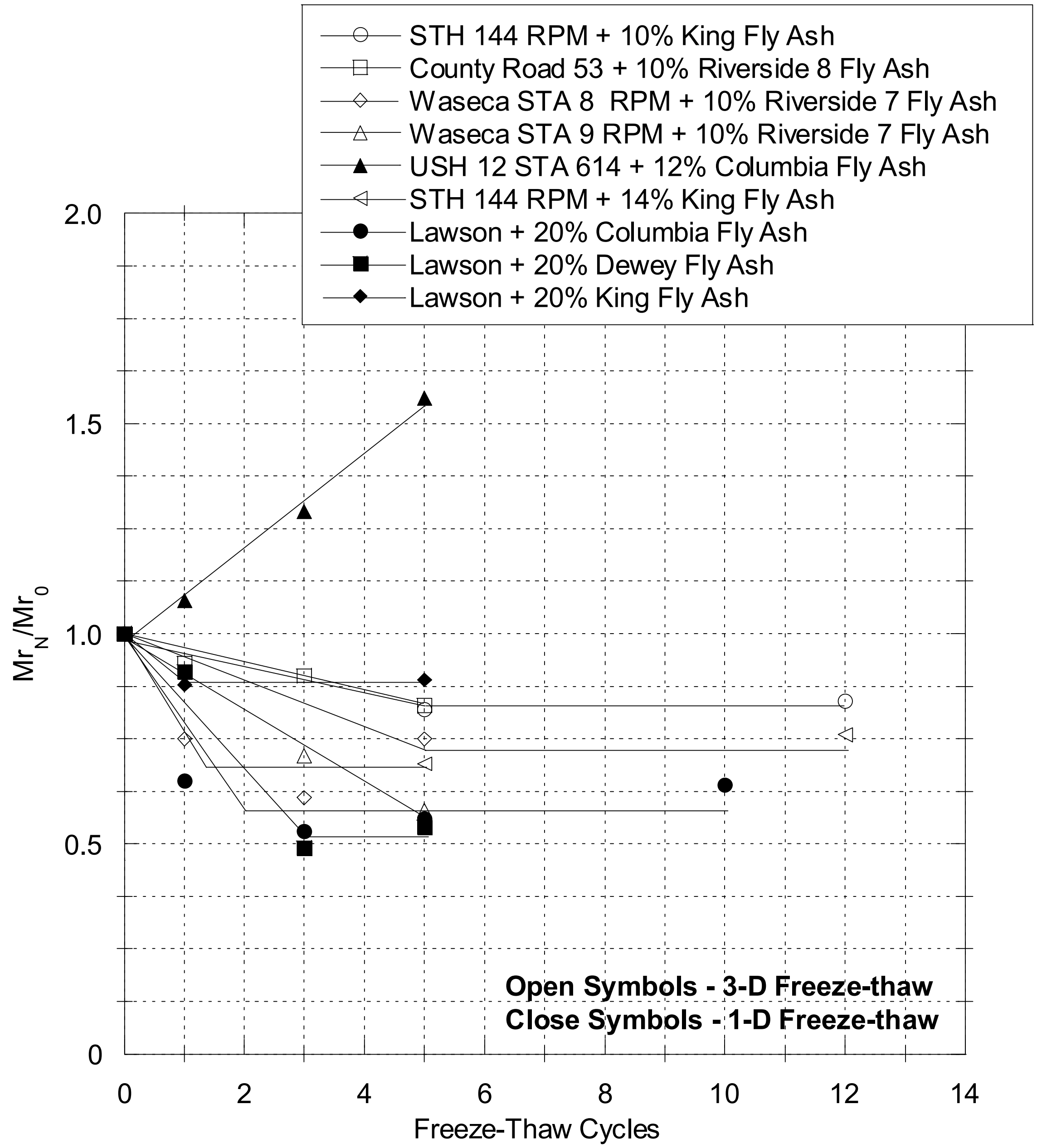


Figure 4. 29. Normalized M_r 1-D and 3-D freeze-thaw

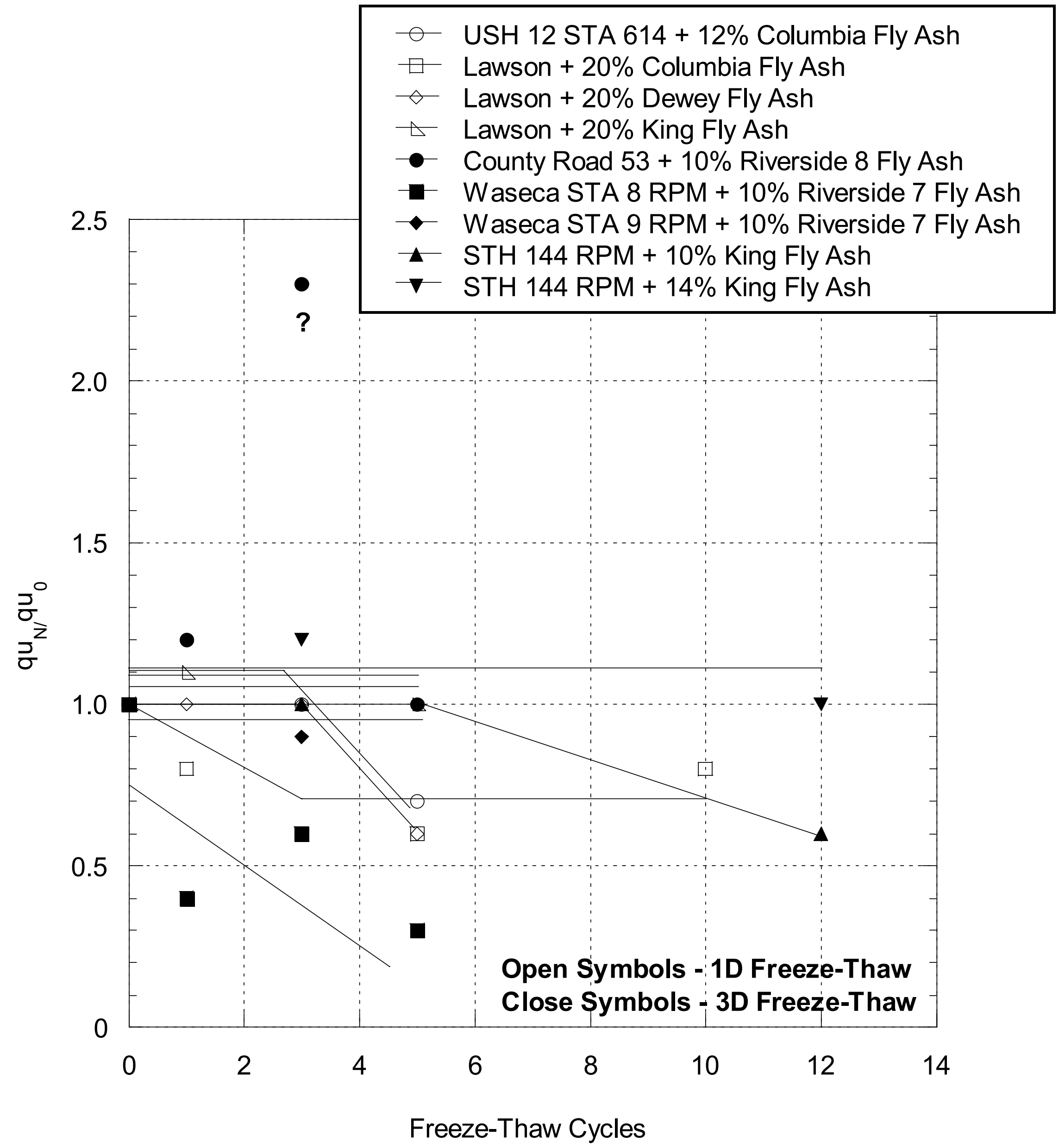


Figure 4. 30. Normalized qu 1-D and 3-D freeze-thaw

APPENDIX A
STANDARD PROCTOR COMPACTION EFFORT

$$E = \frac{\text{(Number of blows per layer)} \times \text{(Layers number)} \times \text{(Hammer weight)} \times \text{(Height of hammer drop)}}{\text{Volume of mold}}$$

where;

E = compaction effort per unit volume of soil ($600 \frac{\text{kN - m}}{\text{m}^3}$).

$$E = \frac{(22\ \text{blows/layer}) \times (6\ \text{layers}) \times (\frac{2.5 \times 9.81}{1000}\ \text{kN}) \times (0.305\ \text{m})}{1647.4\ \text{cm}^3 \times (\frac{1\text{m}^3}{1 \times 10^6\ \text{cm}^3})}$$

$$E = 599.4 \frac{\text{kN - m}}{\text{m}^3} \approx 600 \frac{\text{kN - m}}{\text{m}^3}$$

APPENDIX B

COUNTY ROAD 53 WATER CONTENT AND DRY DENSITY CALCULATIONS

Table B.1. Water content and dry density calculation for samples preparation using County Road 53 soil

Station	From Shelby Tubes	From Nuclear Gauge Tests						Soil Samples after M_r Test and Unconfined Compression Test	
	Subgrade	Subgrade (08/22/05)		Soil-FA Stab Layer After Compaction (8/23-25/05)		Soil-FA Stab Layer After 7 days Curing Time (09/01/05)		Soil-FA Field Mixture After 7 Days of Curing Time	
	w (%)	γ_d (kN/m^3)	w (%)	γ_d (kN/m^3)	w (%)	γ_d (kN/m^3)	w (%)	w (%)	γ_d (kN/m^3)
10+00	-	18.8	12.0	19.9	5.3	20.5	5.0	6.0	19.6
20+00	3.2	17.4	3.3	18.9	5.6	20.0	6.6	7.6	19.2
27+30	12.0	18.4	14.6	20.8	5.2	19.9	7.4	7.5	19.3
40+00	6.3	18.0	6.0	21.2	6.3	20.5	6.3	4.9	20.4
50+00	-	21.1	8.0	19.2	5.6	19.7	6.2	6.9	19.9
60+00	15.4	19.4	11.6	18.8	5.8	19.4	6.4	6.9	19.2
70+00	-	21.9	6.8	19.9	5.6	20.1	5.5	6.0	18
80+30	7.1	20.4	7.9	20.4	4.7	19.8	7.4	5.8	19.1
90+00	4.4	18.7	5.2	-	-	20.7	4.4	-	-
104+00	4.9	18.5	4.6	-	-	20.5	4.7	-	-
AVERAGE	***7.6***	***19.3***	***8.0***	***19.9***	***5.5***	***20.1***	***6.0***	***6.4***	***19.3***
STD DEV	***4.4***	***1.4***	***3.6***	***0.9***	***0.5***	***0.4***	***1.1***	***0.9***	***0.7***

APPENDIX C
WASECA WATER CONTENT AND DRY DENSITY CALCULATIONS

Table C.1. Water content and dry density calculation for sample preparation using soil from Waseca

Station	Waseca stabilized recyclable pavement material measurements after compaction	
	w_N (%)	γ_d (kN/m^3)
1	8	19.3
2	9.1	17.9
3	9.8	18.5
4	7	18.7
5	8.6	19.7
6	7.7	19.3
7	8.7	19.2
8	8.8	20
9	7.4	17.9
10	10.3	18.5
AVERAGE	***8.5***	***18.5***
Standard Deviation	1.0	0.7

Notes: w_N = in situ water content, γ_d = in situ dry unit weight

APPENDIX D

FREEZING-POINT DEPRESSION GRAPHS

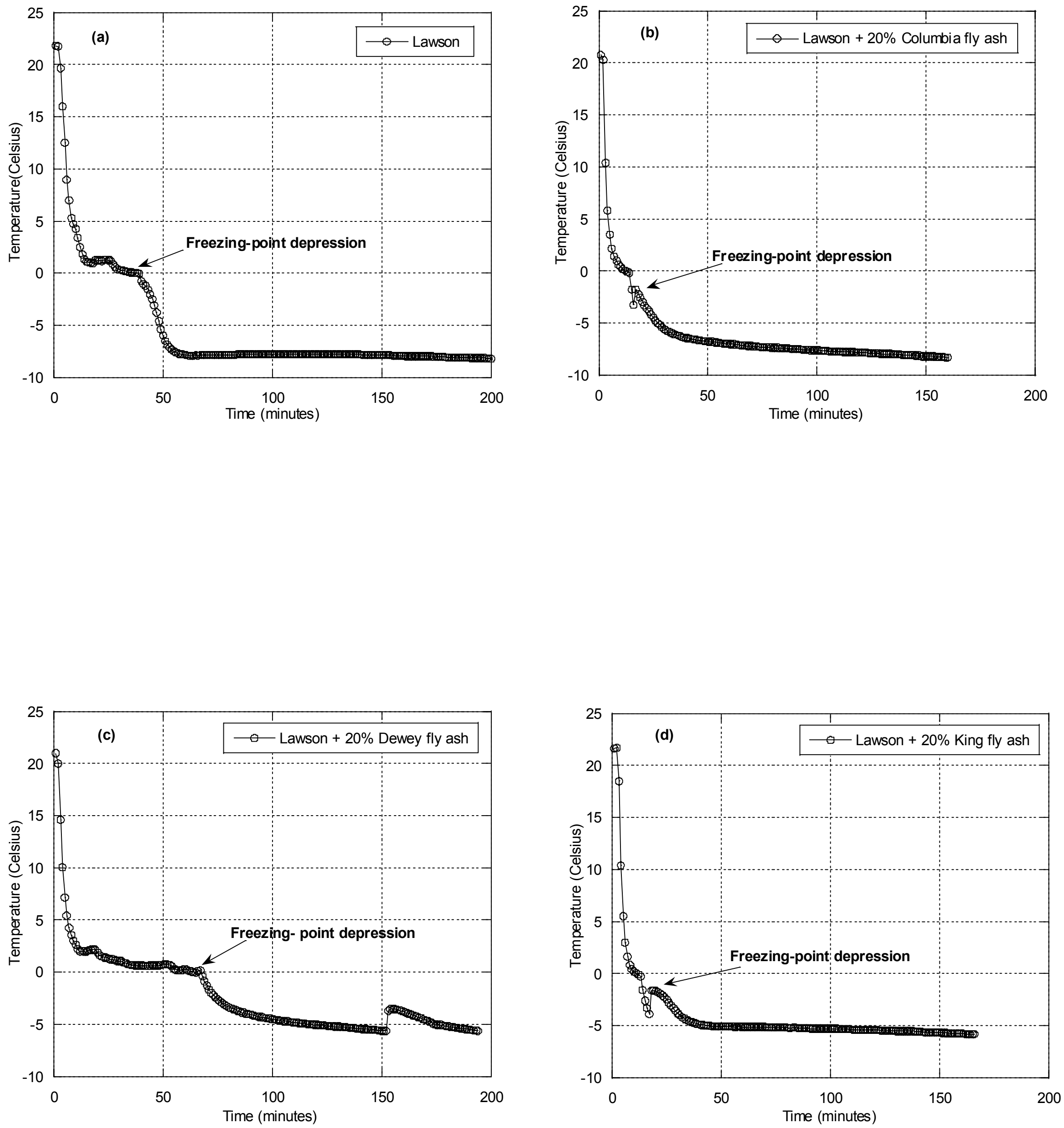


Figure D.1. Freezing-point depression (a) Lawson, (b) Lawson + 20% Columbia fly ash, (c) Lawson + 20% Dewey fly ash, (d) Lawson + 20% King fly ash

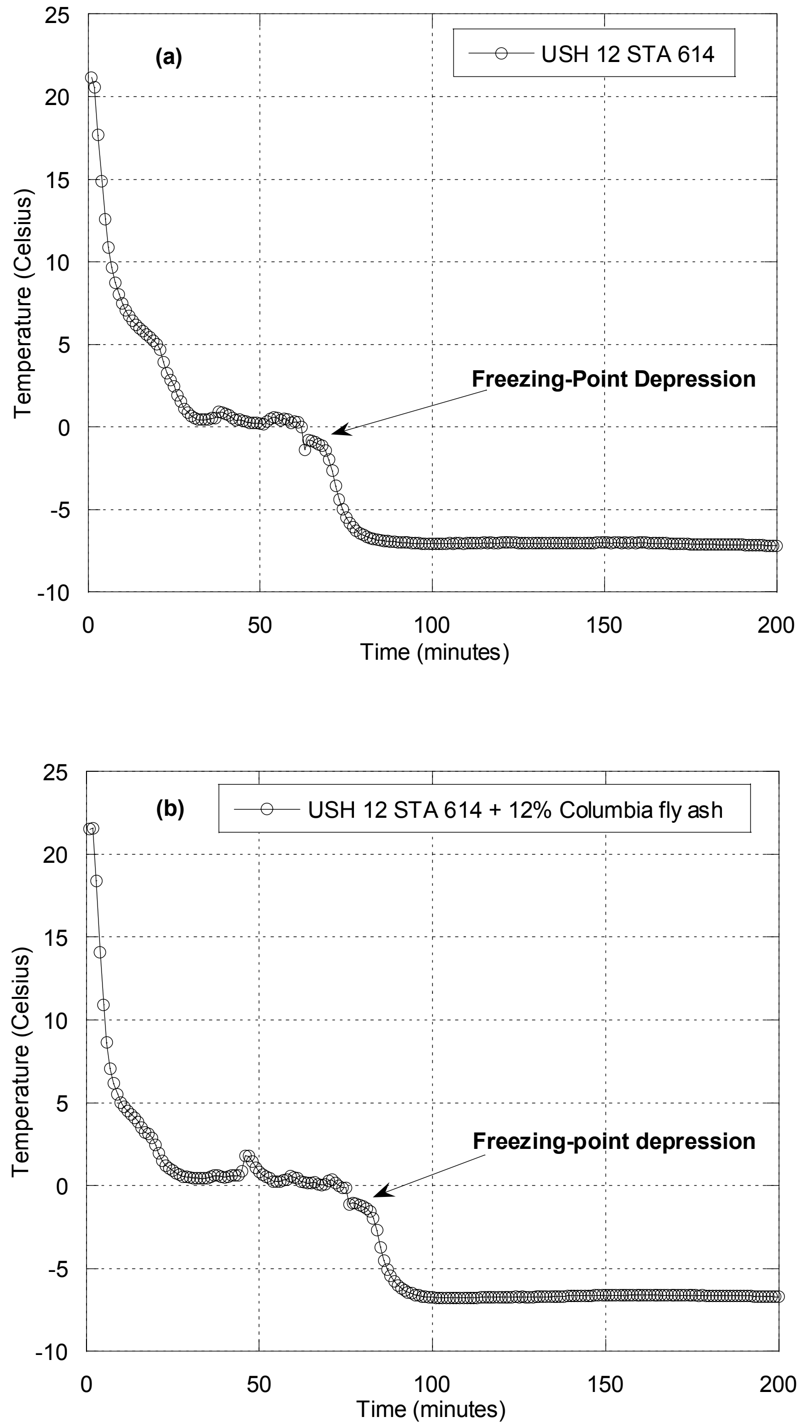
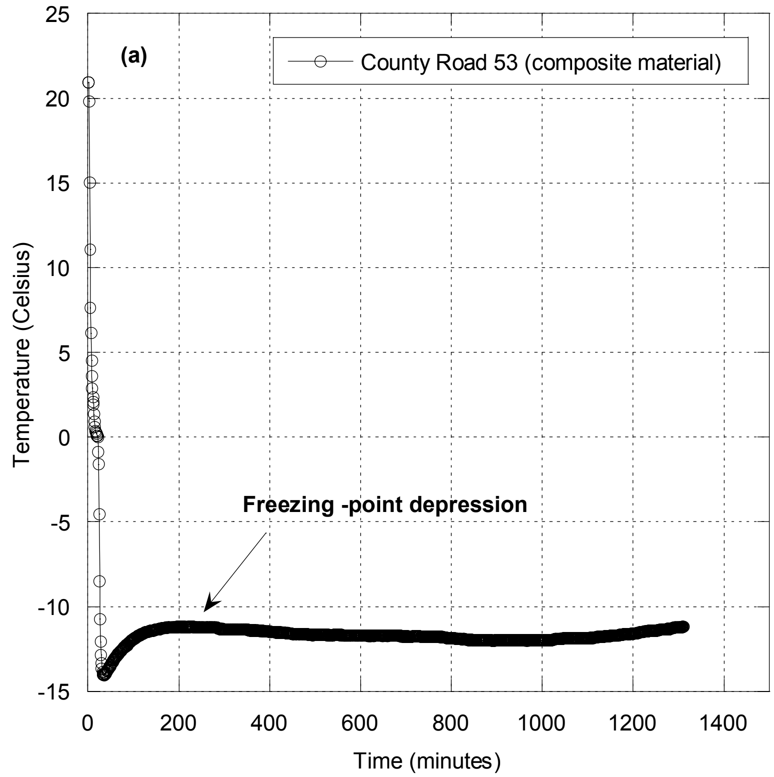


Figure D.2. Freezing-point depression (a) USH 12 STA 614, (b) USH 12 STA 614 + 12% Columbia fly ash



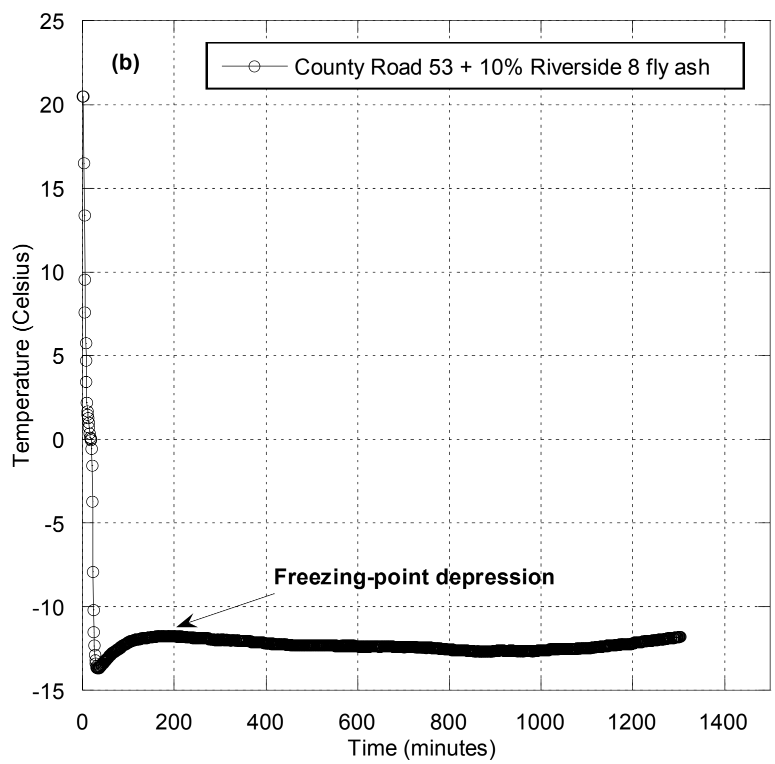


Figure D.3. Freezing-point depression (a) County Road 53 (b) County Road 53 + 10% Riverside 8 fly ash

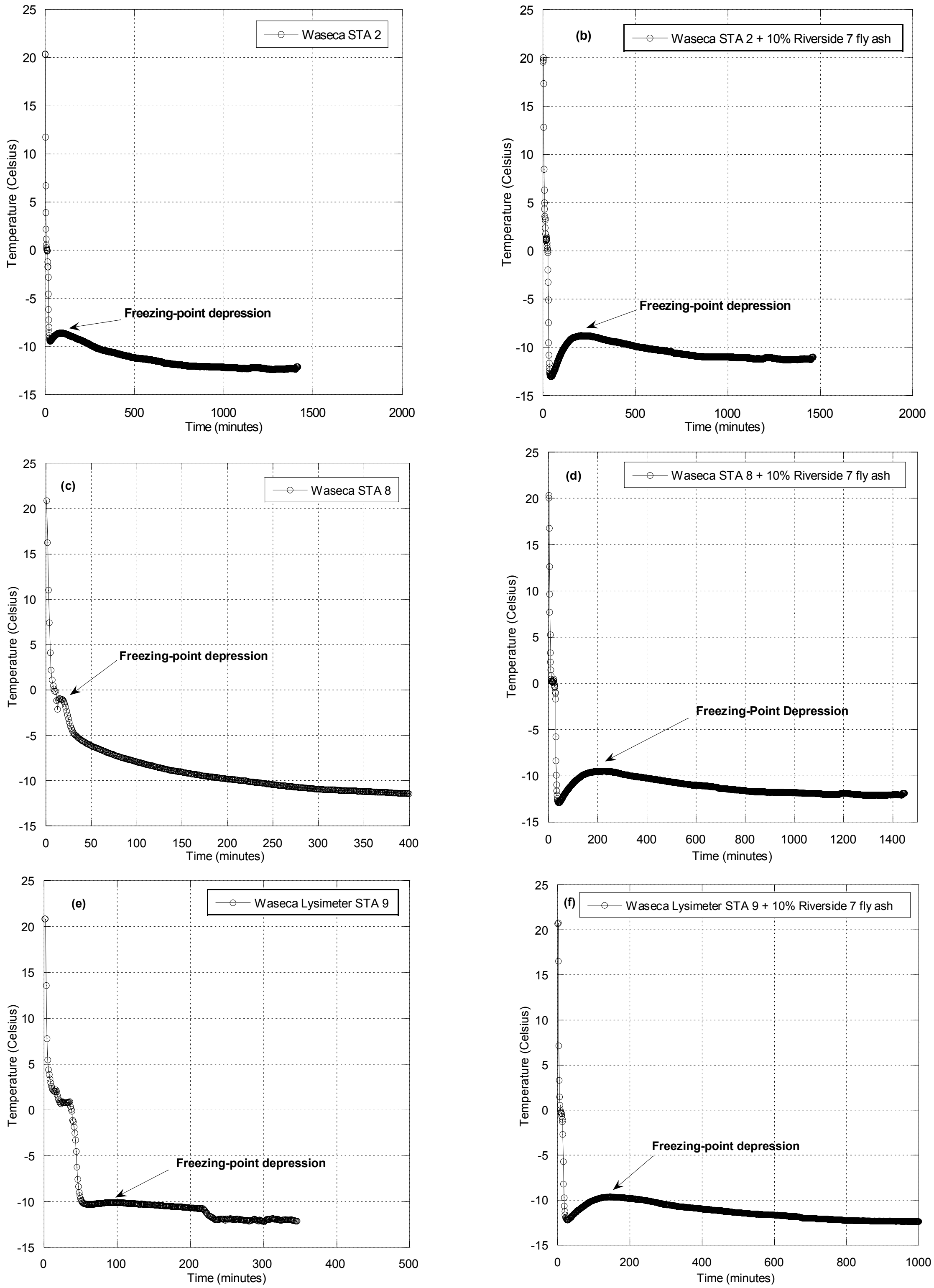


Figure D.5. Waseca freezing-point depression (a) STA 2, (b) STA 2 + 10% Riverside 7 fly ash, (c) STA 8, (d) STA 8 + 10% Riverside 7 fly ash, (e) Lysimeter STA 9, (f) Lysimeter STA 9 + 10% Riverside 7 fly ash

APPENDIX E

FREEZE-THAW CYCLES GRAPHS

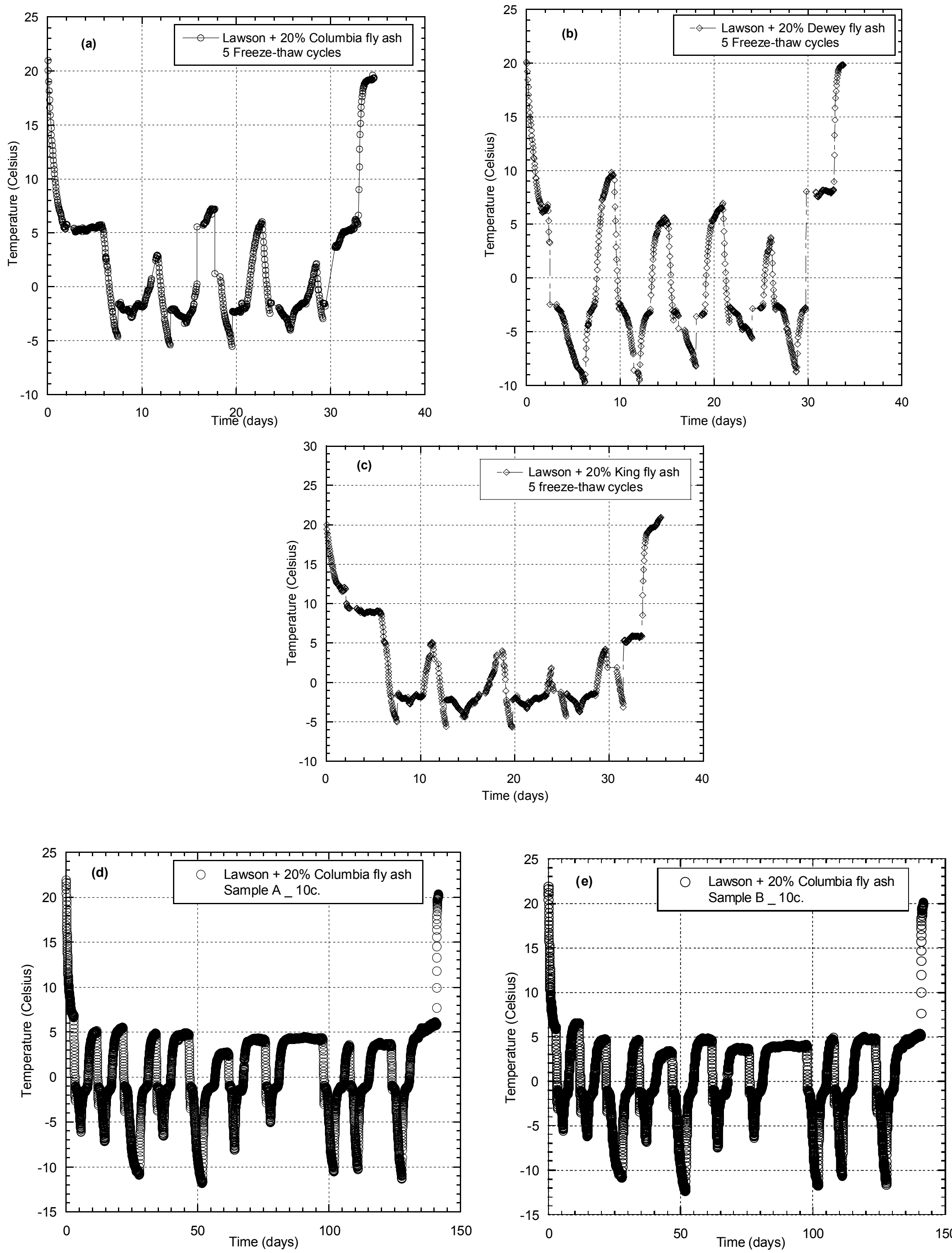


Figure E.1. Lawson freeze-thaw cycles (a) 20% Columbia fly ash (5 cycles), (b) 20% Dewey fly ash (5 cycles), (c) 20% King fly ash (5 cycles), (d), and (e) 20% Columbia fly ash (10 cycles)

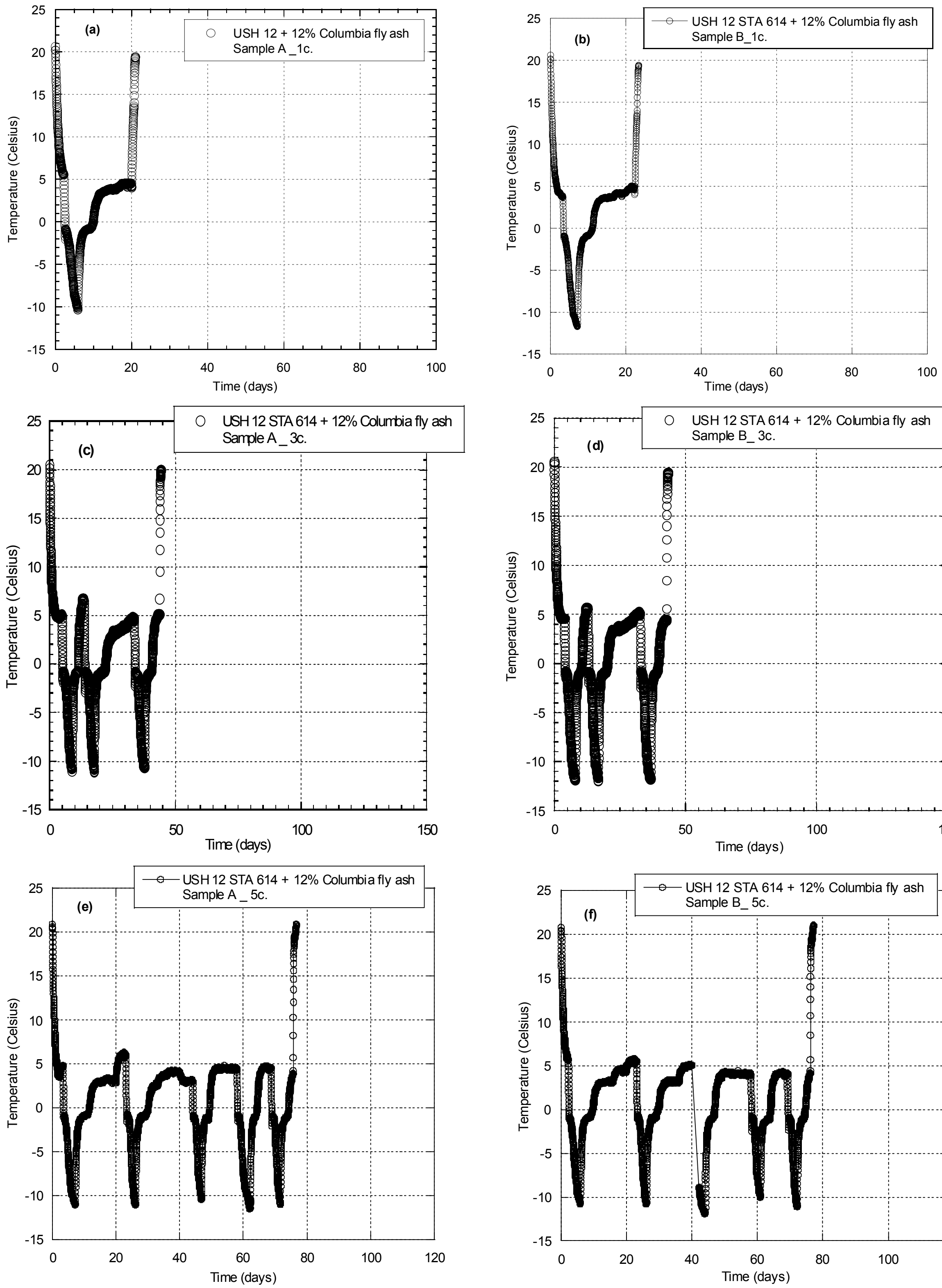


Figure E.2. USH 12 STA 614 + 12% Columbia fly ash freeze-thaw cycles (a) and (b) 1cycle, (c) and (d) 3 cycles and (e) and (f) 5 cycles

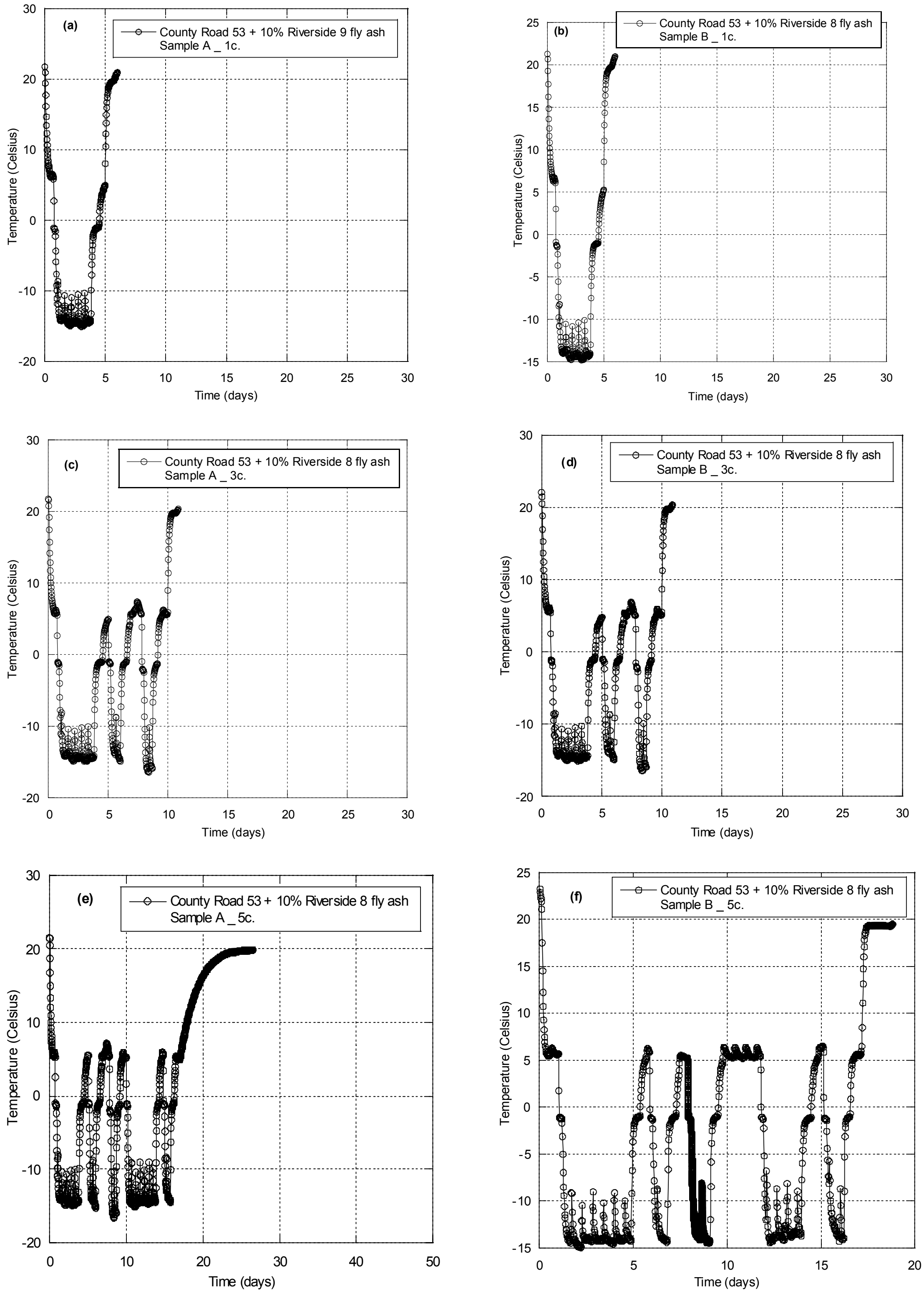
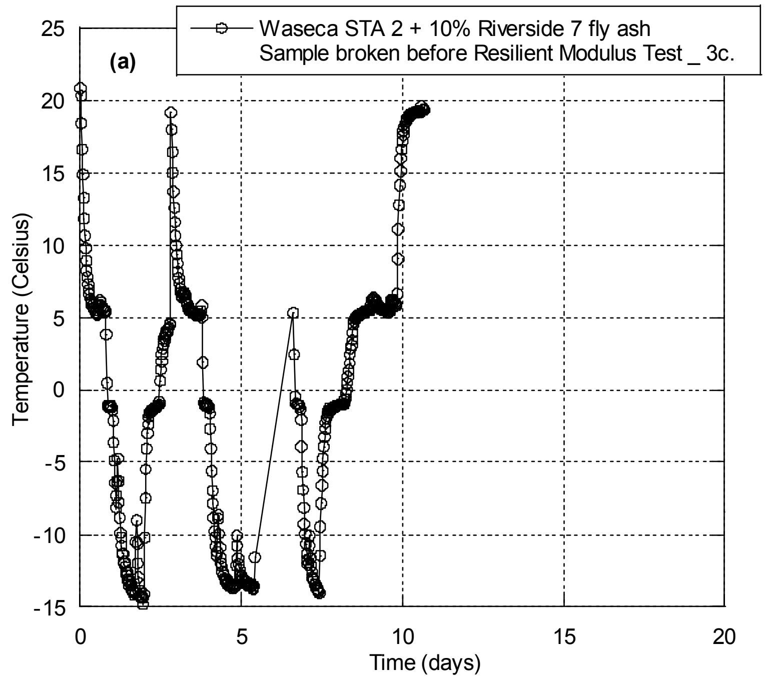


Figure E.3. County Road 53 + 10% Riverside 8 fly ash freeze-thaw cycles (a) and (b) 1 cycle (c) and (d) 3 cycles, and (e) and (f) 5 cycles



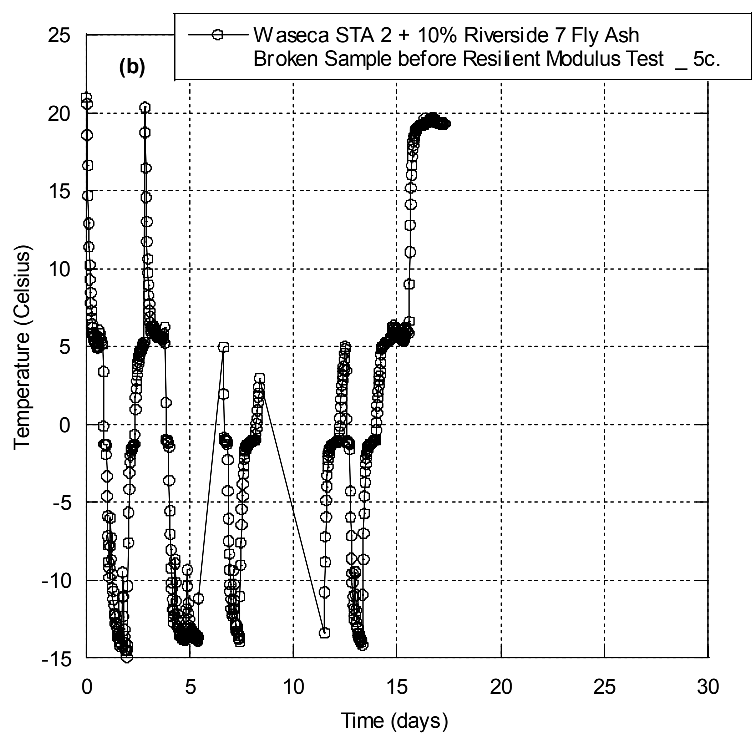


Figure E.4. Waseca STA 2 + 10% Riverside 7 fly ash freeze- thaw graphs (a) 1 cycle and (b) 3 cycles.

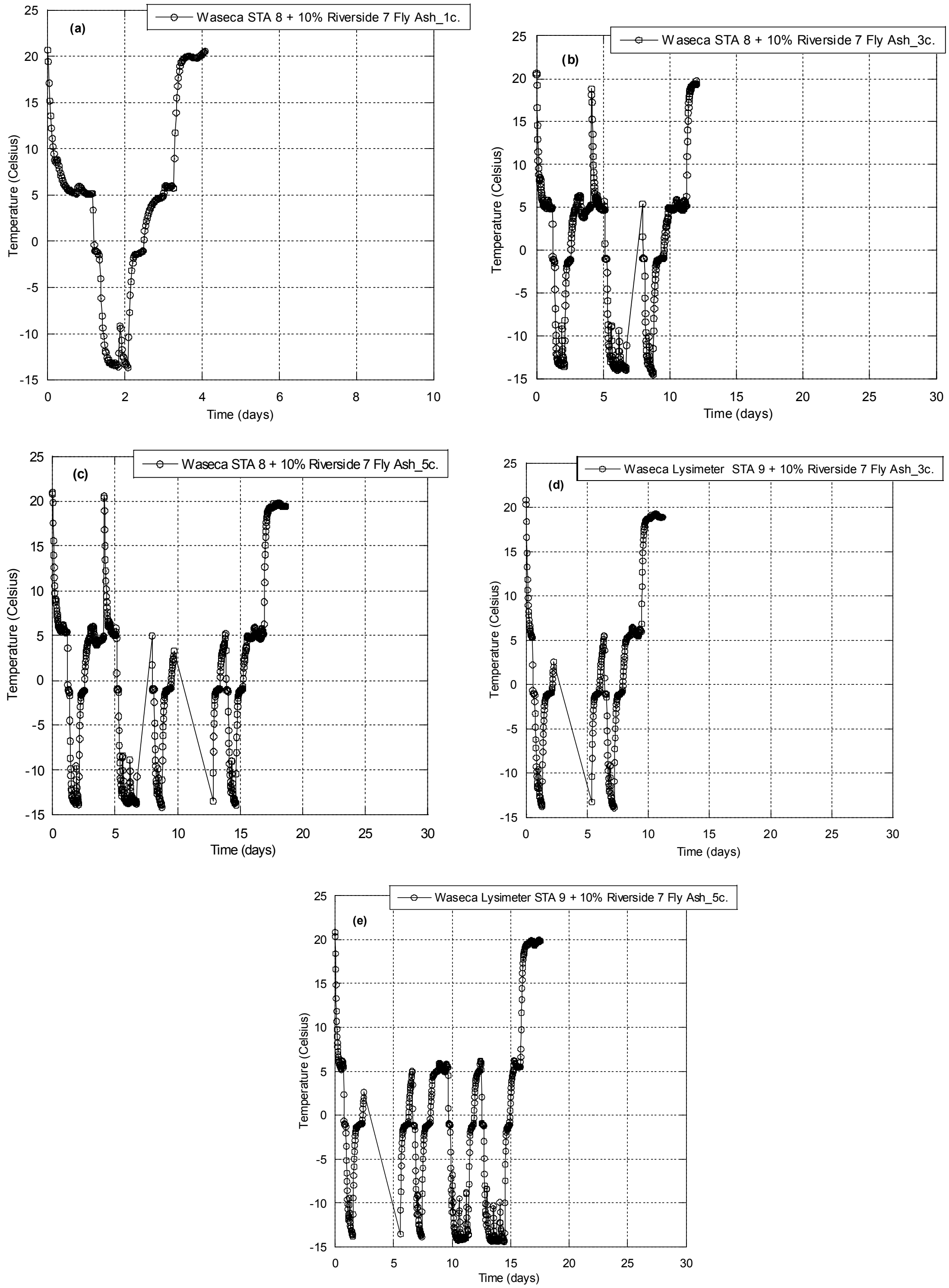
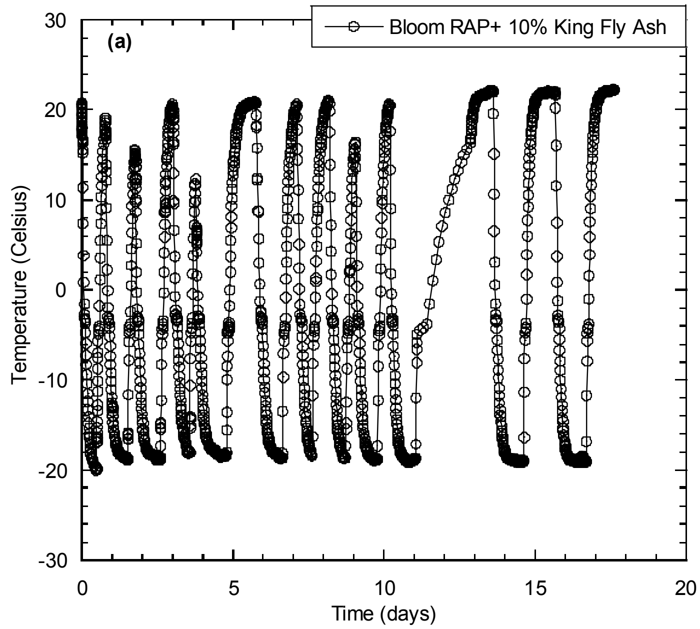


Figure E.5. Waseca freeze-thaw cycles (a), (b) and, (c) STA 8 + 10% Riverside 7 fly ash (1, 3, and, 5 cycles respectively); (d) and (e) Lysimeter STA 9 + 10% Riverside 7 fly ash (1 and 5 cycles respectively)



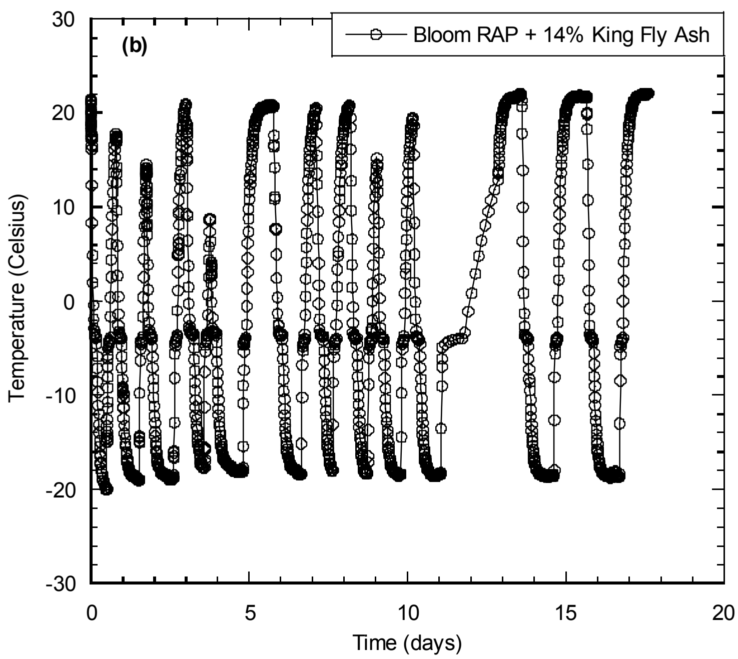


Figure E.6. Bloom RAP freeze-thaw cycles (a) 10% King fly ash after 12 freeze-thaw cycles and (b) 14% King fly ash after 12 freeze-thaw cycles

APPENDIX F

RESILIENT MODULUS MACHINE CALIBRATIONS

F. CALIBRATION OF GEO-ENGINEERING RESILIENT MODULUS MACHINE

F.1 CELL PRESSURE CALIBRATION

F.1.1 Procedure

1. Go to ***Measurement and Automation*** on the desktop (see Figure F.1.1.).

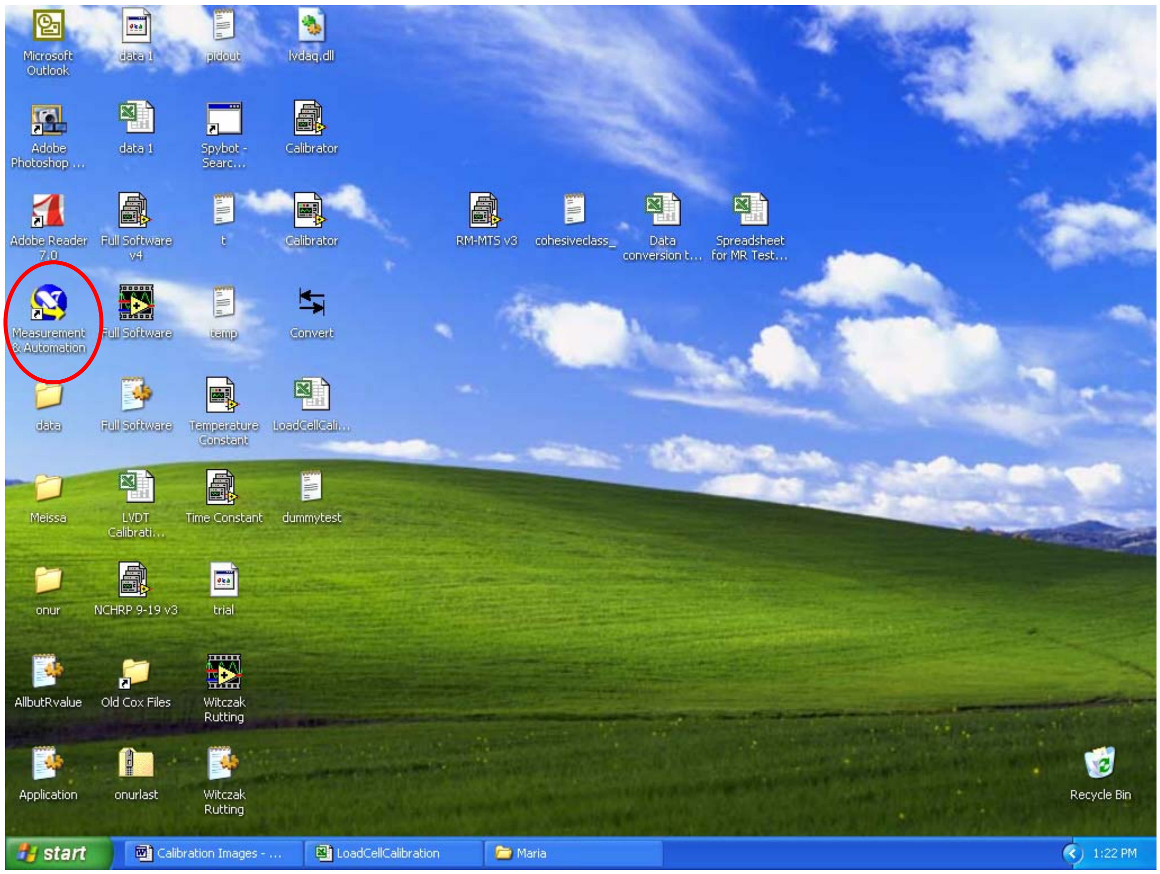


FIGURE F.1.1. Measurement and Automation Program

2. Under ***Configuration*** go to ***My System*** ⟶ ***Devices and Interfaces*** ⟶ ***NI-DAQmx Devices*** ⟶ ***PCI-6221:"Dev1"*** (see Figure F.1.2.). From this panel the operator can control all the measurement devices used in the resilient modulus test.
3. Under ***Test Panels*** go to ***Analog Output*** ⟶ ***Channel Name: Dev1/a$_o$1*** (e.g. analog output channel 1). The ***Output Mode*** should be in DC Voltage

and the ***Max. and Min. Outputs Limits*** equal to ± 10 Volts (see Figure F.1.2.).

4. In the ***Output Voltage,*** presented in Figure F.1.2, start with 0 volts, click ***Update*** and take reading from the pressure gauge adjusted to the Geo-engineering resilient modulus machine. Important: Pressure gauge units are in pounds per square inches (psi). Record the voltage and pressure gauge reading.

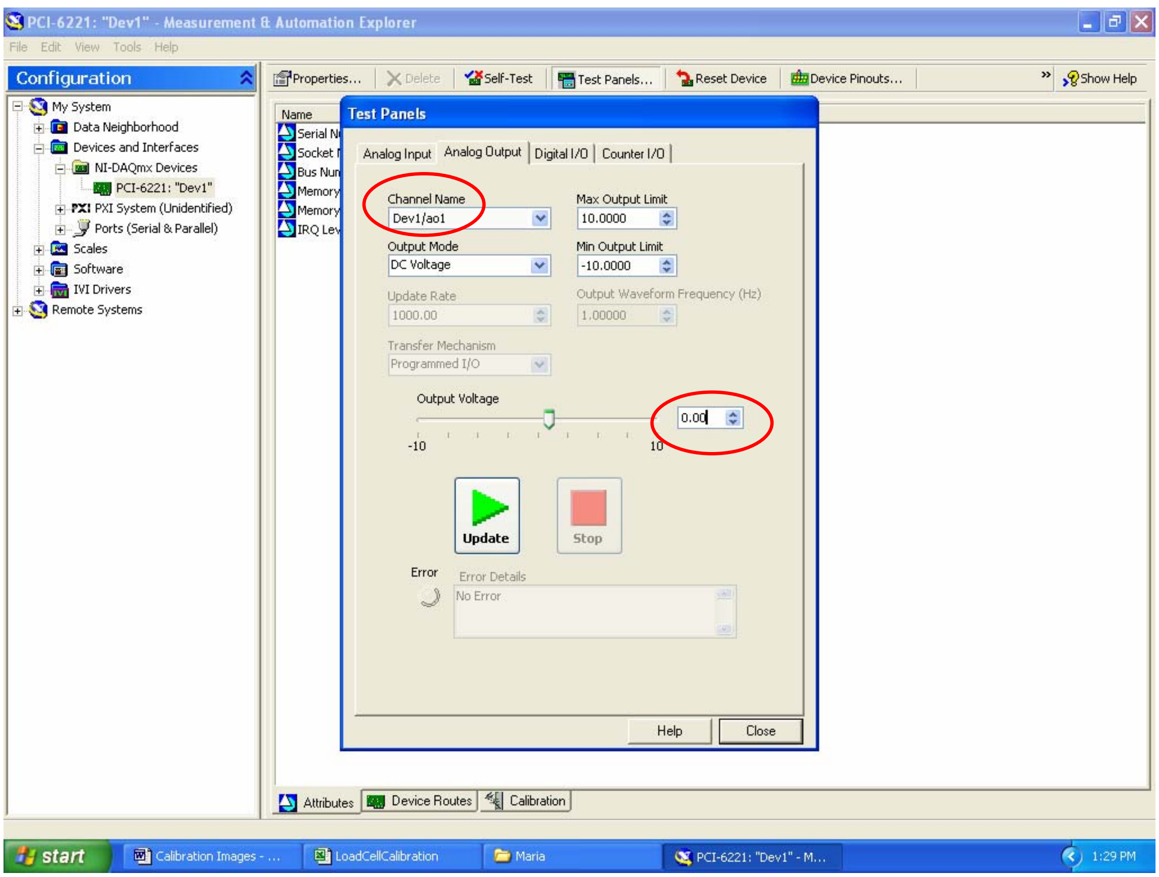


FIGURE F.1.2. Reference screens for steps 2, 3, and 4

5. Plot the entered ***Output Voltage*** (x-axis) against the pressure readings from the pressure gauge (y-axis) converted from psi to kPa. Find the slope of the linear regression line.

6. An example of the actual cell pressure calibration is shown in Table F.1.1. and Figure F.1.3.

TABLE F.1.1. Voltage entered and pressure values from the gauge

Voltage[1] (V)	Pressure (psi)	Pressure[2] (kPa)
0	0.2	1.378
1	9.4	64.766
2	19.8	136.422
3	30	206.7
4	40.2	276.978
5	50.4	347.256
6	60.8	418.912

[1] Analog Output [2] 1psi=6.89kPa

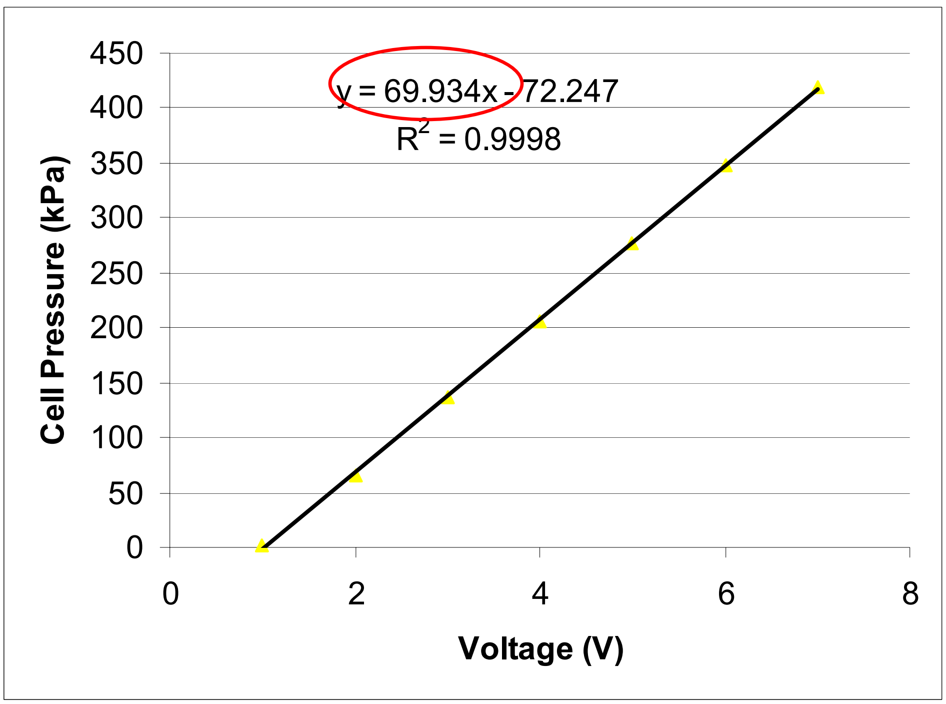


FIGURE F.1.3. Cell pressure calibration recommended slope

7. The recommended pressure cell calibration slope value is 69.934. The slope needs to be change directly in LabView. For this is necessary to consult with Professor Peter Bosscher.

8. After changing the slope. The cell pressure calibration is ready. Run a resilient modulus test to verify if the pressure gauge readings are equivalent or close enough to the target cell pressures.

F.2. LOAD CALIBRATION

F.2.2. Procedure

1. For this calibration a proof ring is necessary. Place the proof ring in the Geo-engineering Resilient Modulus machine like in Figure E.2.1. A proof ring can be found in the Asphalt Lab on the 3rd Floor of Engineering Hall.

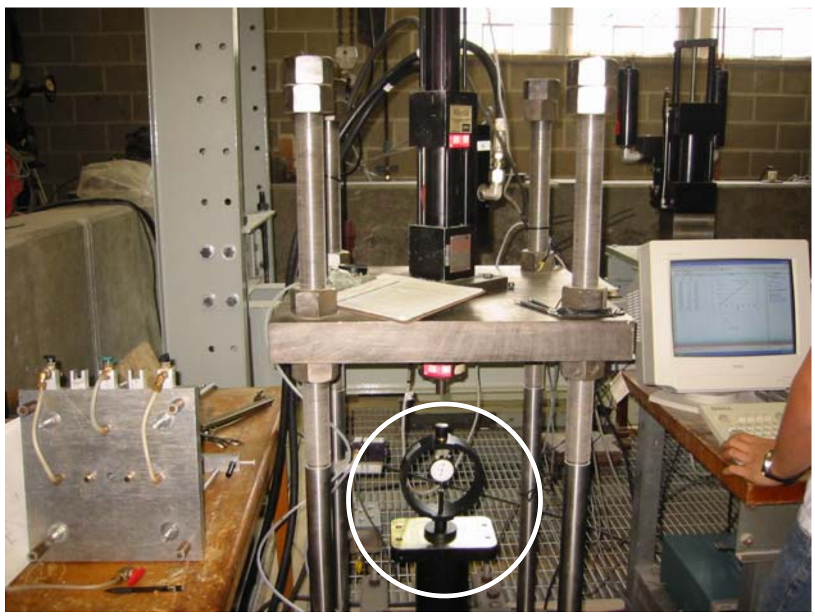


FIGURE F.2.1. Proof Ring

2. Go to ***Measurement and Automation*** on the desktop (see Figure F.1.1.)
3. Under ***Configuration*** go to ***My System*** → ***Devices and Interfaces*** → ***NI-DAQmx Devices*** → ***PCI-6221:"Dev1"*** (see Figure F.2.3.). From this panel the operator can control all the measurement devices used in the Resilient Modulus Test.
4. Under ***Test Panels*** go to ***Analog Output*** → ***Channel Name: Dev1/a$_o$0*** (e.g. Analog output channel 0). The ***Output Mode*** should be in DC Voltage

and the ***Max. and Min. Outputs Limits*** equal to ± 10 Volts (see Figure F.2.3.).

5. In the ***Output Voltage*** presented in Figure F.2.3. start with 0 volts, clicks ***Update*** and takes reading from the proof ring. Record the voltage and proof ring reading.

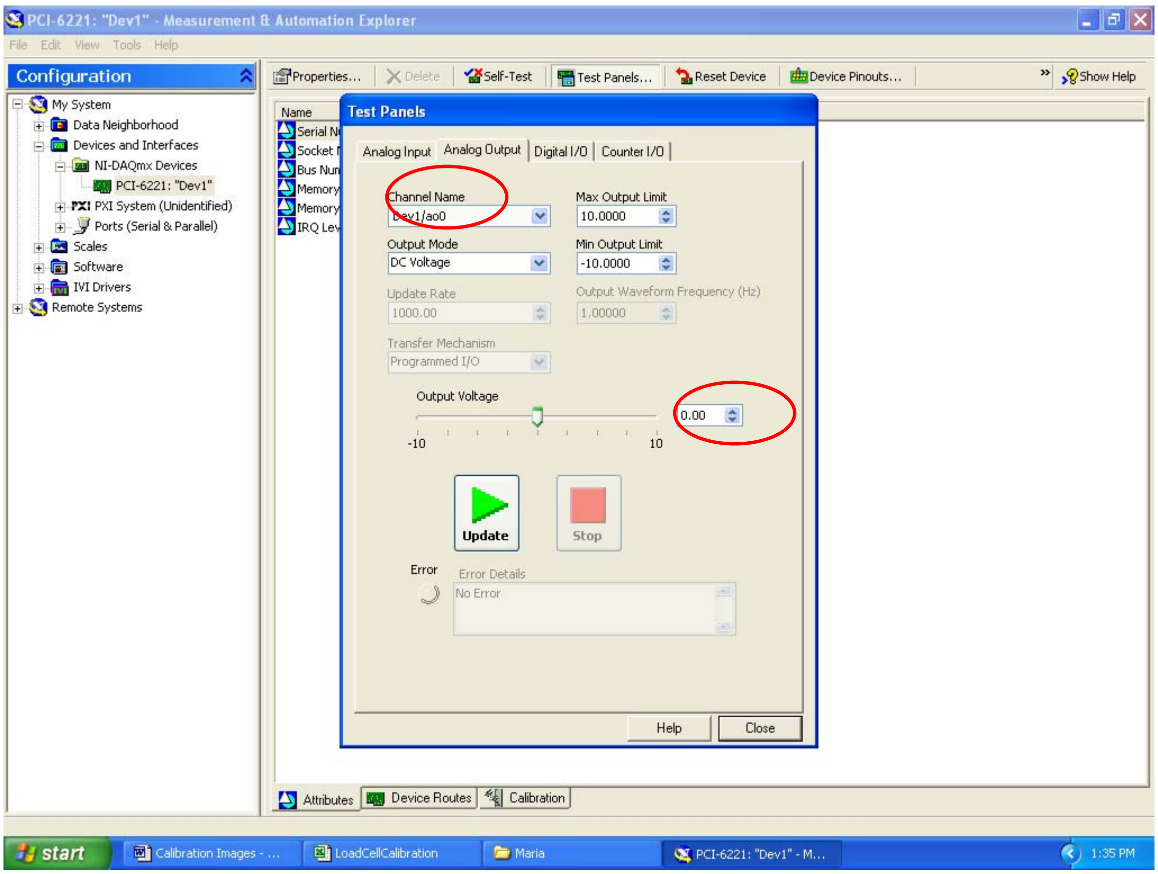


FIGURE F.2.3. Reference screens for steps 3, 4, and 5

6. Repeat Step 5 decreasing the ***Output Voltages***. The ***Output Voltages*** should be negative for the piston, which is the device that applies the load, goes down. Bring back the voltage to zero each time a different ***Output Voltage*** is entered (e.g. 0V, -0.5V, 0V, -1.0V, 0V, etc.).
7. Plot the **Output Voltage** (x-axis) in volts against the pressure readings from the proof ring (y-axis) converted to kN. Find the slope of the linear regression line.

8. Close the ***Test Panel*** screen. Under ***Configuration*** go to ***Scales*** → ***NI-DAQmx Devices Scales*** → ***Load***. In this screen, the ***Scale Units*** are in kN (y-axis) and ***Pre-Scale Units*** are in Volts (V) (x-axis). Under ***Pre-Scaled to Scaled*** change slope with the new calibrated slope value (e.g. the actual slope value of 2.224 kN/Volts is recommended) and change the Y-Intercepts with the new calibrated value (e.g. recommended and actual Y-Intercept value is 0.557) (see Figure F.2.4.)
9. After changes click "***Save Scale***" (see Figure F.2.4.) Note: The proof ring is not an accurate device that is why the manufacturer slope and y-intercept is recommended.

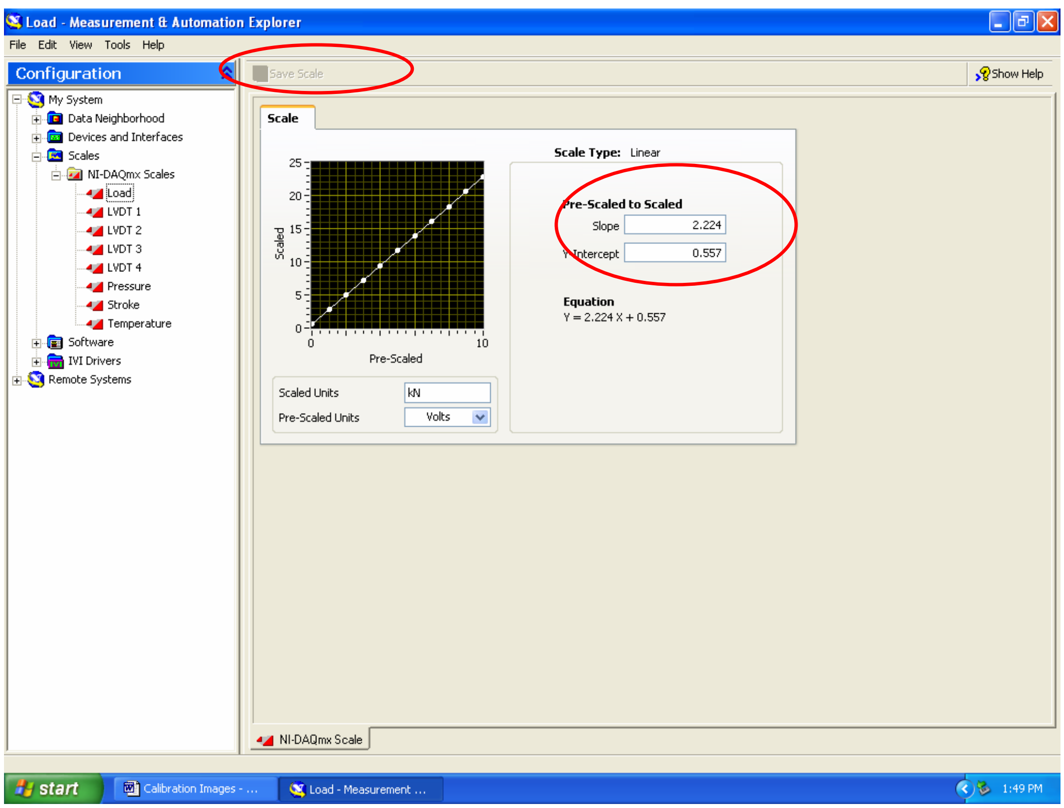


FIGURE F.2.4. Calibrated and actual slope and y-intercept value

10. Close and Exit the program. The load calibration is ready. Run a resilient modulus test to verify if the load values are equivalent or close enough to the target values.

F.3 LINEAR VARIABLE DISPLACEMENT TRANSDUCERS (LVDT'S) CALIBRATION

F.3.1 Procedure

1. For this calibration a micrometer is necessary. A micrometer can be found at Engineering Hall 2211 on the Geotechnical Lab.
2. Turn on the LVDT's Power Supply. The Power Supply should be at 21V on both sides.
3. Open the ***Voltmeter*** at the desktop (see Figure F.3.1.).

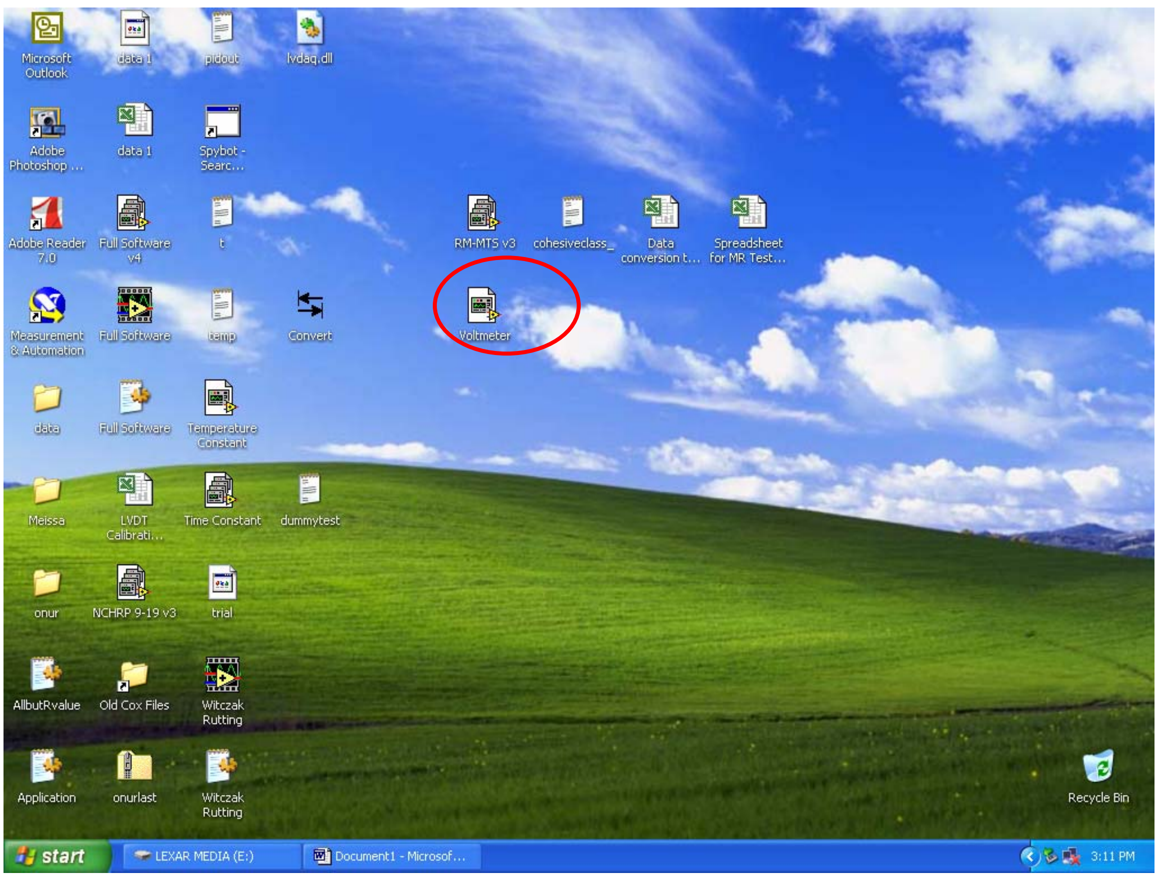


FIGURE F.3.1. Location of the voltmeter on the desktop

4. LVDT 1 model is M 781820-09 and LVDT 2 model is M781820-18.
5. In the voltmeter on the ***Channel Parameters*** the ***Physical Channel*** of LVDT 1 is ***Dev 1/a2*** and LVDT 2 is ***Dev 1/ a3***. The ***Channel Parameters Minimum***

and Maximum Value should be in -10 and +10 respectively (see Figure F.3.2.).

6. For the calibration place the LVDT 1 inside the micrometer and take readings. Record the micrometer reading of each point and convert the readings to inches (in). Also record the voltmeter reading show in the screen.
7. Plot the micrometer readings in mm (y-axis) versus the obtained ***Voltage*** in Volts (x-axis). Find the slope of the linear regression line.
8. An example of LVDT 1 calibration is shown in Table F.3.1. and Figure F.3.3.
9. Go to ***Measurement and Automation*** on the desktop.
10. Under ***Configuration*** go to ***Scales*** → ***NI-DAQmx Devices*** → ***Scales LVDT 1***. ***Scale Units*** are in mm (y-axis) and ***Pre-Scale Units*** are in Volts (V) (x-axis). Under ***Pre-Scaled to Scaled*** change slope with the new calibrated slope value (e.g. actual slope value is 1.0405 kN/Volts) and change the ***Y-Intercepts*** with the new calibrated value (e.g. recommended to use the actual ***Y-Intercept*** value) (see Figure F.3.5.).
11. After change the slope click "***Save Scale***" (see Figure F.3.5.).
12. Repeat Steps 5 to 10 for LVDT 2. Actual slope value is 1.1078 (see Figure F.3.5.)
13. Example of ***LVDT 2*** Calibration is presented in Table F.3.2. and Figure F.3.4.
14. Close and Exit the program. The LVDT 1 and LVDT 2 calibration is ready.

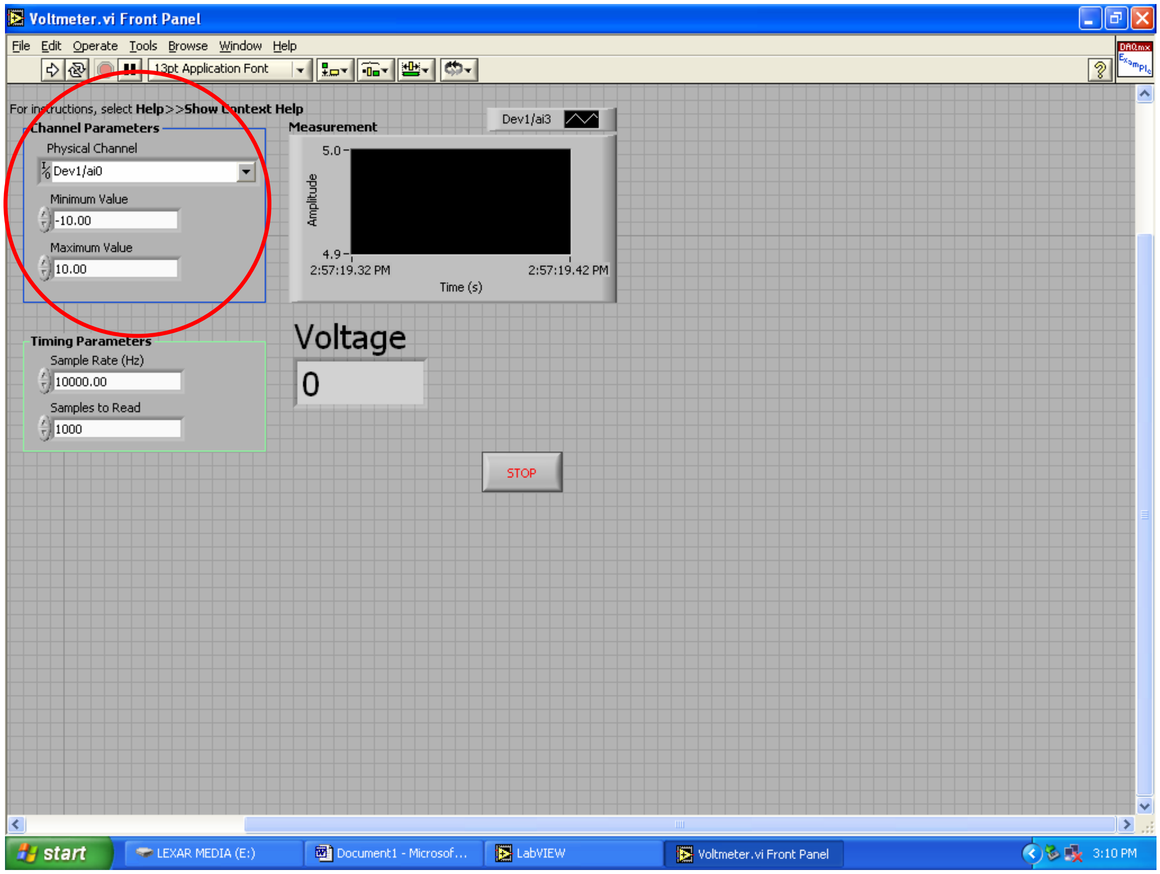


FIGURE F.3.2. Voltmeter screen

Table F.3.1. LVDT 1 Calibration

Micrometer Readings (mm)	Micrometer Readings (inches)	Voltage (V)
12.7	0.5	-0.1766
15.24	0.6	2.25
17.145	0.675	4.087
10.16	0.4	-2.605
7.62	0.3	-5.028

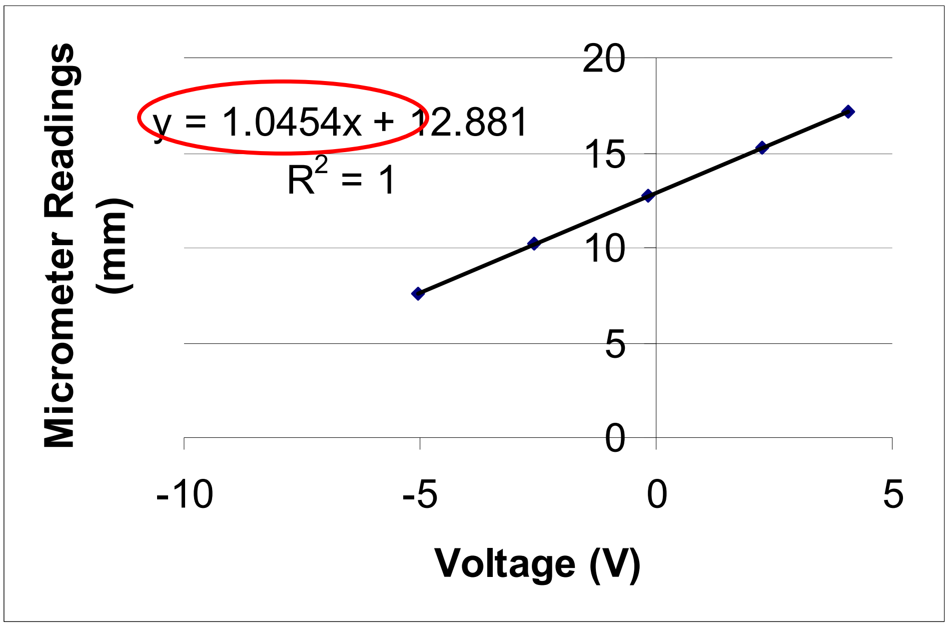


Figure F.3.3. LVDT 1 Calibration

Table F.3.2. LVDT 2 Calibration

Micrometer Readings (inches)	Micrometer Readings (mm)	Voltage (V)
0.5	12.7	-0.0593
0.6	15.24	2.234
0.7	17.78	4.539
0.4	10.16	-2.356
0.3	7.62	-4.63

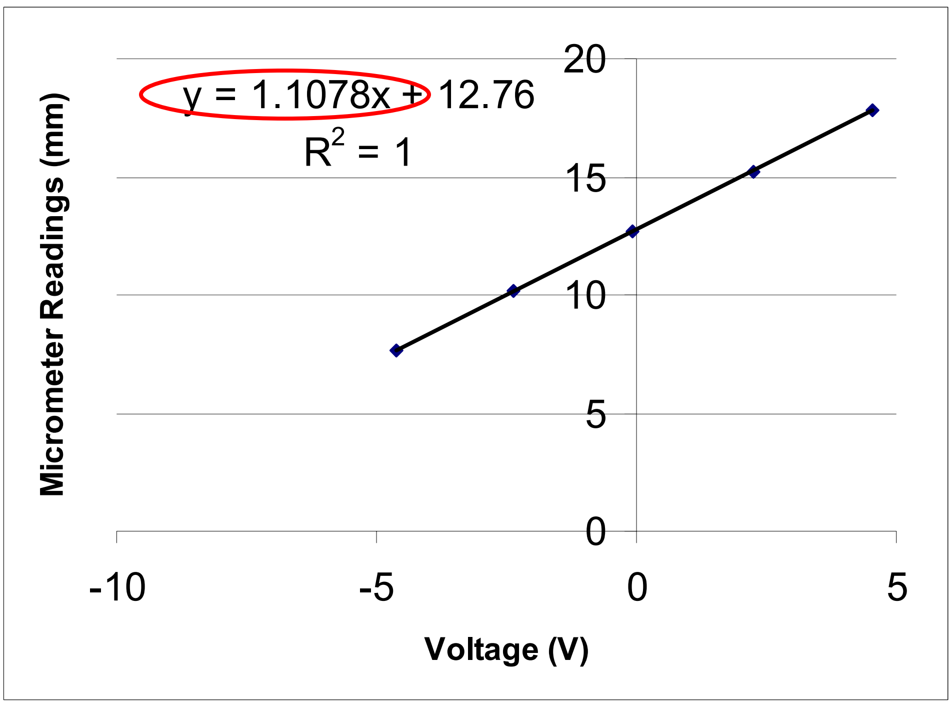


Figure F.3.4. LVDT 2 Calibration

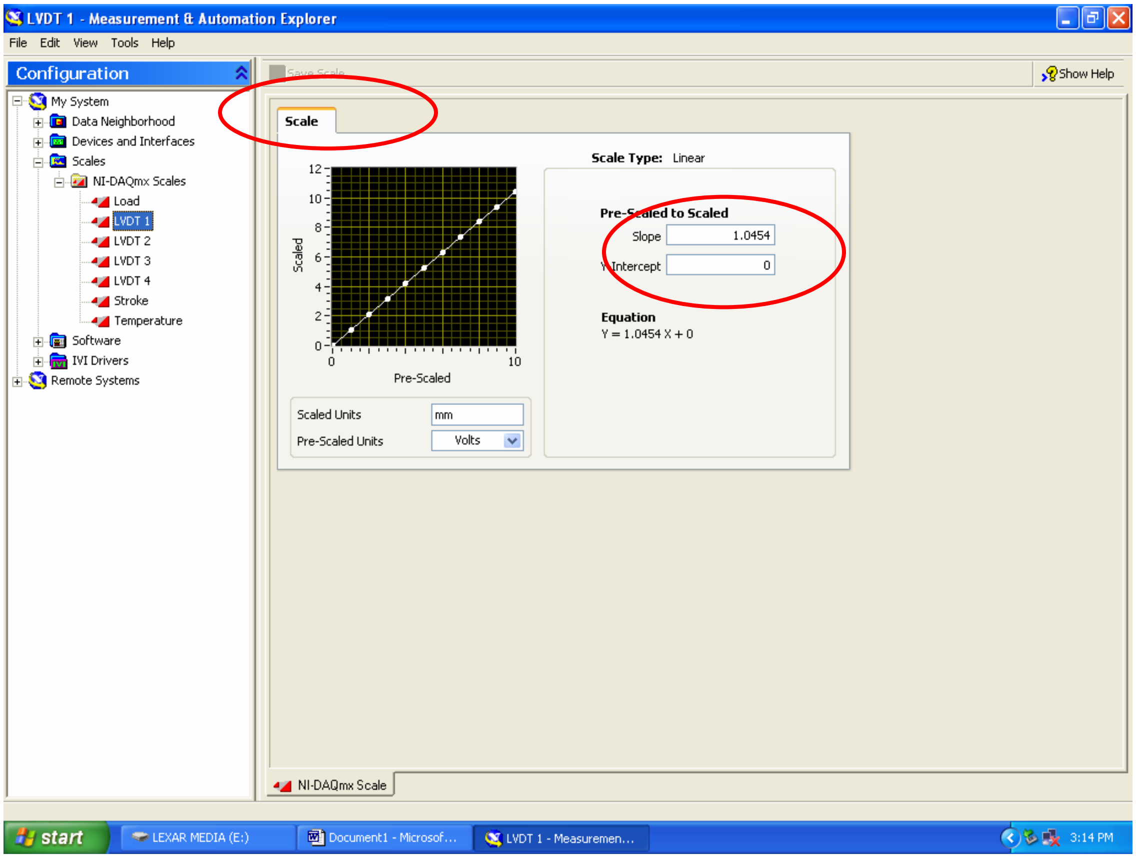


FIGURE F.3.5. Actual and recommended LVDT 1 Slope and Y-intercept

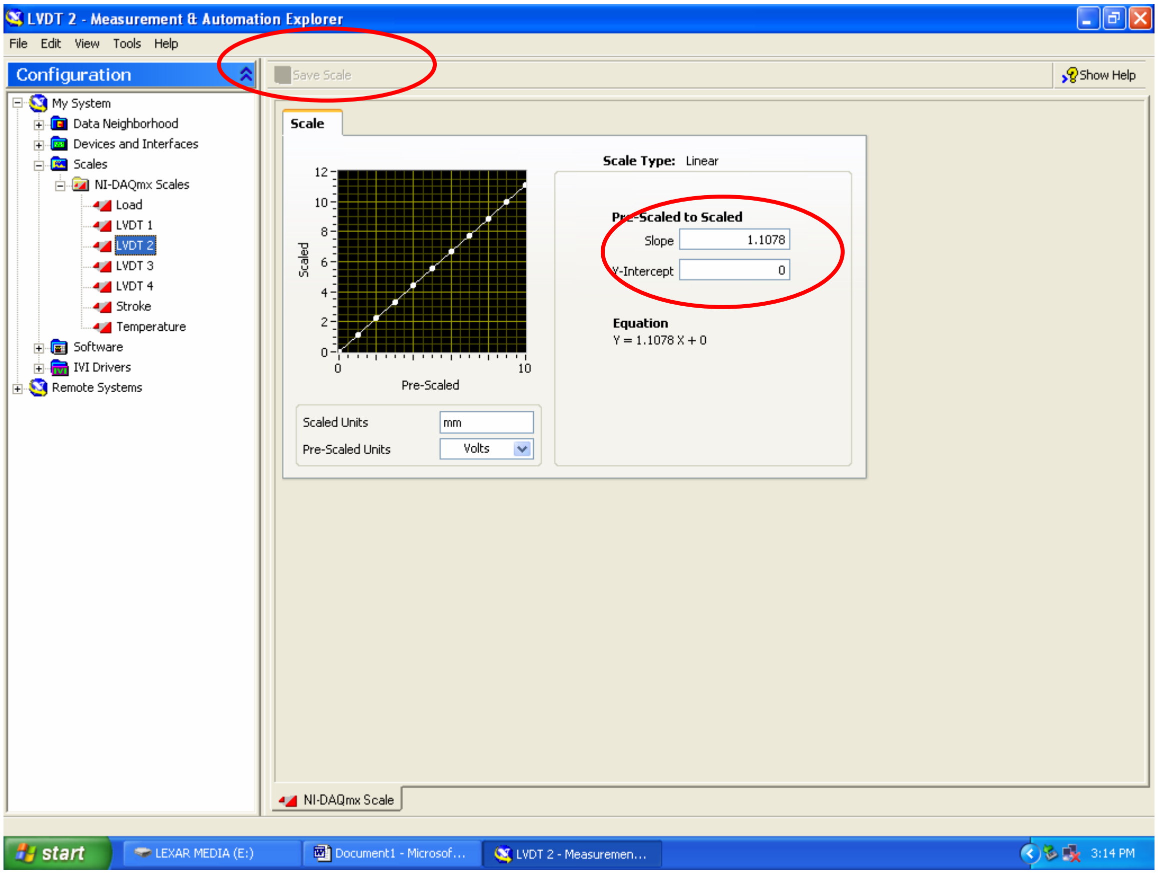


FIGURE F.3.6. Actual and recommended LVDT 2 Slope and Y-intercept

APPENDIX G

MEDIUM SAND RESILIENT MODULUS CALIBRATION TEST

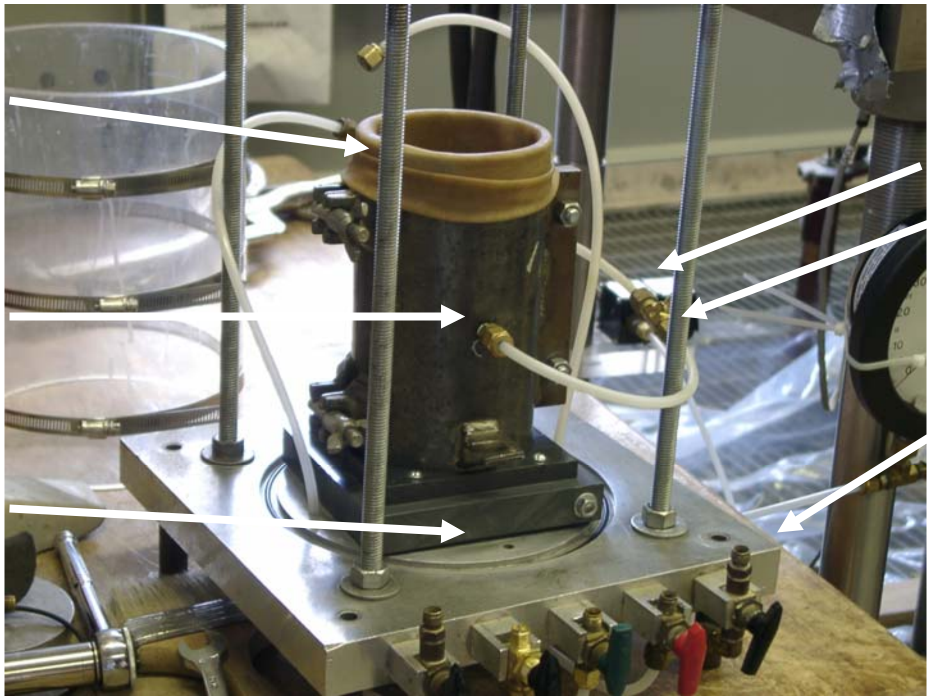
The loading sequence used to test the medium sand is summarized in Table G.1. Only 50% of the deviator stresses in the testing sequence for Type 1 soils were applied as recommended by Sawangsuriya et al. 2003. The sand samples used for the reproducibility tests were prepared at dry units weights of 15.36 and 15.53 kN/m^3. Equipment set up pictures for the medium sand resilient modulus test is shown in Fig. G.1.

Repeatability among the resilient modulus calibration tests with medium sand is shown in Fig. G.2. For specimens prepared at a dry unit weight of 15.36 kN/m^3, the resilient modulus range between 47-138 MPa (bulk stress range of 62-537 kPa) and 43-155 MPa (bulk stress range of 53-480 kPa). For the sample prepared at a dry density of 15.53 kN/m^3, the resilient modulus range was 44-146 MPa (bulk stress range of 52-486 kPa).

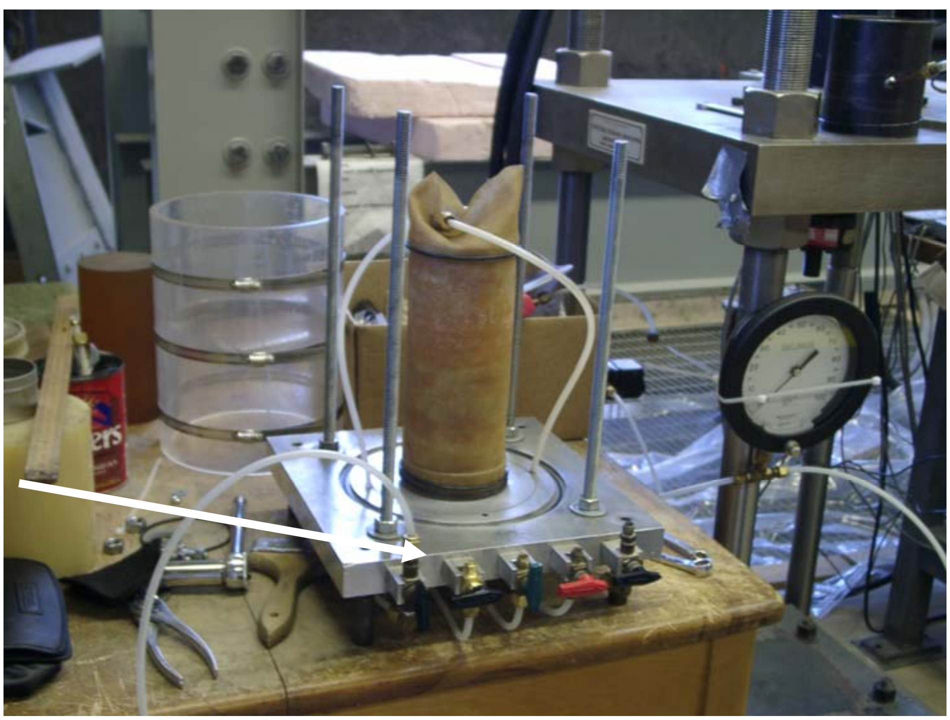
Table G.1. AASHTO T 294-94 testing sequence for Type 1 soils.

Conditioning	Sequence Number	Deviator Stress (kPa)	50% Deviator Stress (kPa)[a]	Confining Pressure (kPa)	Number of Repetitions
Specimen Conditioning	1	103	51.5	103	1000
Testing	2	21	10.5	21	100
	3	41	20.5	21	100
	4	62	31.0	21	100
	5	34	17.0	34	100
	6	69	34.5	34	100
	7	103	51.5	34	100
	8	69	34.5	69	100
	9	138	69.0	69	100
	10	207	103.5	69	100
	11	69	34.5	103	100
	12	103	51.5	103	100
	13	207	103.5	103	100
	14	103	51.5	138	100
	15	138	69.0	138	100
	16	276	138	138	100

Note: A seating load of 13.8 kPa was used. [a] 50% deviator stress used for medium-sand test.



(a)



(b)

Figure G.1. (a) Mold set-up for medium sand resilient modulus test (a) and Medium sand sample ready for testing (b).

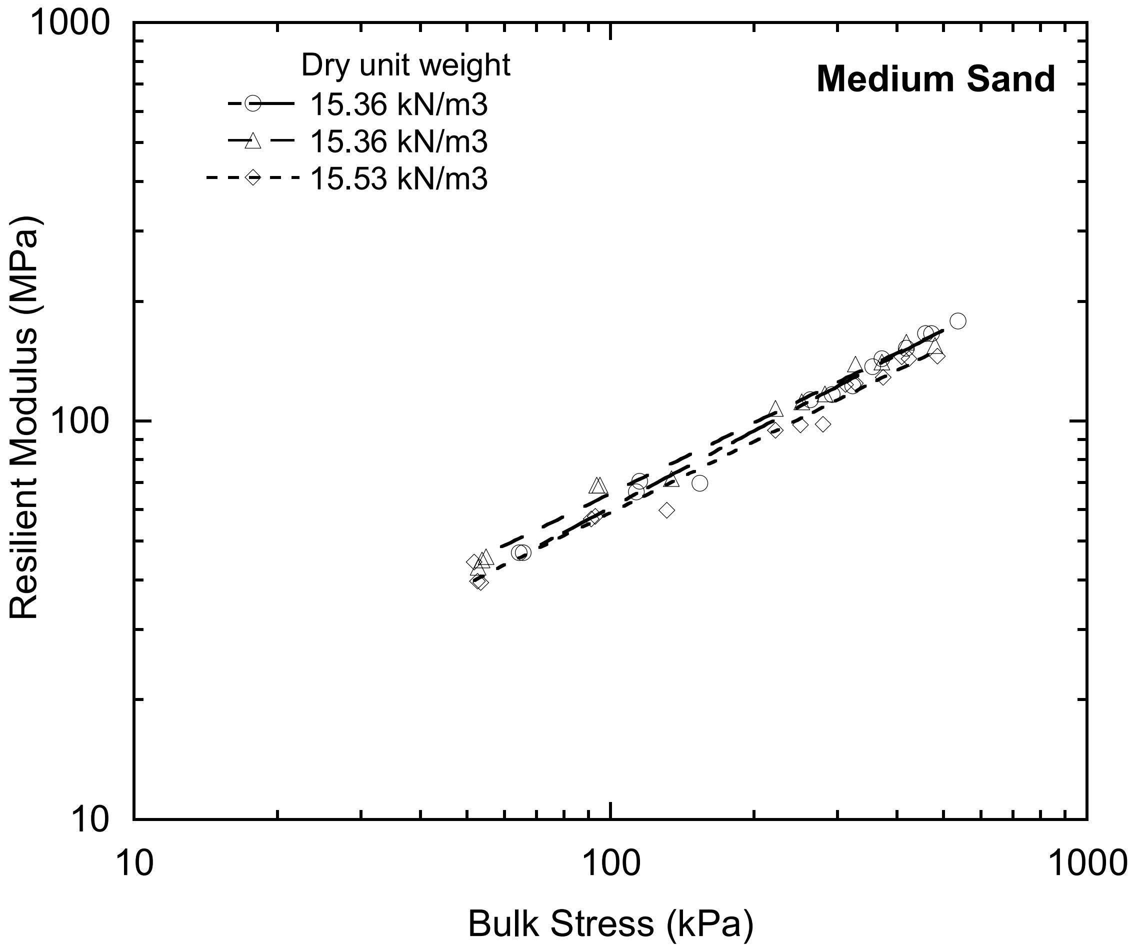


Figure G.2. Resilient modulus vs. bulk stress for medium sand.

APPENDIX H

SOIL-FLY ASH MIXTURES VOLUME CHANGE AFTER FREEZE THAW CYCLING

Table H.1. USH 12 STA 614 + 12% Columbia Fly Ash mixture samples change in volume after freeze-thaw cycles

Soil-Fly Ash Mixture	Freeze-Thaw Cycles	Sample	Diameter (cm)	Height (cm)	V after F-T cycles (cm^3)	V before F-T cycles (cm^3)	$\frac{\Delta V}{V_0}$	$\frac{\Delta V}{V_0}$ average	$\frac{\Delta V}{V_0}$ average * 100%
USH 12 STA 614 + 12% Columbia Fly Ash	1	A	10.18	20.025	1629.9	1647.4	-0.011	-0.012	-1.2
		B	10.19	19.92	1624.5	1647.4	-0.014		
	3	A	10.23	20.12	1653.7	1647.4	0.004	0.003	0.3
		B	10.226	20.11	1651.6	1647.4	0.003		
	5	A	10.2	20.59	1682.5	1647.4	0.021	0.010	1.0
		B	10.09	20.57	1644.8	1647.4	-0.002		

Table H.2. Lawson-fly ash mixtures samples volume change after freeze-thaw cycles

Soil-Fly Ash Mixture	Freeze-Thaw Cycles	Sample	Diameter (cm)	Height (cm)	V after F-T cycles (cm^3)	V before F-T cycles (cm^3)	$\frac{\Delta V}{V_0}$	$\frac{\Delta V}{V_0}$ average	$\frac{\Delta V}{V_0}$ average * 100%
Lawson + 20% Columbia Fly Ash	1	A	10.16	19.9	1613.4	1647.4	-0.021	-0.020	-2.0
		B	10.11	20.13	1616.0	1647.4	-0.019		
	3	A	10.1	20.12	1612.0	1647.4	-0.021	-0.012	-1.2
		B	10.19	20.17	1644.9	1647.4	-0.002		
	5	A	10.15	20.14	1629.6	1647.4	-0.011	-0.012	-1.2
		B	10.19	19.93	1625.3	1647.4	-0.013		
	10	A	10.116	20.475	1645.6	1647.4	-0.001	0.006	0.6
		B	10.18	20.495	1668.1	1647.4	0.013		
Lawson + 20% Dewey Fly Ash	1	A	10.19	20.19	1646.5	1647.4	-0.001	-0.001	-0.1
		B	10.2	20.13	1644.9	1647.4	-0.002		
	3	A	10.17	20.12	1634.4	1647.4	-0.008	-0.014	-1.4
		B	10.12	20.07	1614.4	1647.4	-0.020		
	5	A	10.08	19.95	1592.0	1647.4	-0.034	-0.027	-2.7
		B	10.09	20.2	1615.2	1647.4	-0.020		
Lawson + 20% King Fly Ash	1	A	10.1	20.1	1610.4	1647.4	-0.022	-0.021	-2.1
		B	10.14	20.01	1615.9	1647.4	-0.019		
	5	A	10.14	20.22	1632.9	1647.4	-0.009	-0.013	-1.3
		B	10.17	19.95	1620.6	1647.4	-0.016		

Table H.3. County Road 53 + 10% Riverside 8 Fly Ash samples change in volume after freeze-thaw cycles

Soil-Fly Ash Mixture	**Freeze-Thaw Cycles**	**Sample**	**Diameter (cm)**	**Height (cm)**	**V after F-T cycles (cm^3)**	**V before F-T cycles (cm^3)**	$\frac{\Delta V}{V_0}$	$\frac{\Delta V}{V_0}$ **average**	$\frac{\Delta V}{V_0}$ **average * 100%**
CR 53 + 10% Riverside 8 Fly Ash	1	A	10.17	20.435	1660.0	1647.4	0.008	0.020	2.0
		B	10.267	20.54	1700.5	1647.4	0.032		
	3	A	10.233	20.425	1679.8	1647.4	0.020	0.016	1.6
		B	10.25	20.215	1668.1	1647.4	0.013		
	5	A	10.257	20.45	1689.8	1647.4	0.026	0.024	2.4
		B	10.27	20.35	1685.8	1647.4	0.023		

Table H.4. Waseca STA 8 and STA 9 Lysimeter + 10% Riverside 7 Fly Ash volume change after freeze-thaw cycles

Soil- Fly Ash Mixture	Freeze-Thaw Cycles	Sample	Diameter (cm)	Height (cm)	V after F-T cycles (cm^3)	V before F-T cycles (cm^3)	$\frac{\Delta V}{V_0}$	$\frac{\Delta V}{V_0}$ * 100%
Waseca STA 8 RPM + 10% Riverside 7 Fly Ash	1	A	10.202	20.505	1676.2	1647.4	0.017	1.7
	3	A	10.183	20.55	1673.6	1647.4	0.016	1.6
	5	A	10.253	20.475	1690.5	1647.4	0.026	2.6
Waseca STA 9 RPM + 10% Riverside 7 Fly Ash	1	---	---	---	---	---	---	---
	3	A	10.183	20.61	1678.5	1647.4	0.019	1.9
	5	A	10.23	20.585	1692.0	1647.4	0.027	2.7

APPENDIX I

SOIL AND SOIL-FLY ASH MIXTURE WATER CONTENT CHANGE

Table I.1 Soil and soil-fly ash mixture moisture content change

Soil or soil-fly ash mixtures samples	Soaked or Unsoaked	Moisture State	Target wc (%)	F-T Cycles	wc (%) before F-T cycles or after sample preparation	wc (%) after F-T cycles or after sample was tested	Change in wc (%)	Average wc (%) change
USH 12 STA 614	Unsoaked	7% wet of optimum	22.5	0	21.7	20.9	-0.8	-1.3
				0	22.5	20.7	-1.8	
USH 12 STA 614 + 12% Columbia Fly Ash	Unsoaked	7% wet of optimum	22.5	0	18.3	17.1	-1.2	-1.6
				0	18.5	16.6	-1.9	
				1	19.0	17.5	-1.5	-1.4
				1	18.5	17.2	-1.3	
				3	18.2	16.4	-1.8	-1.5
				3	18.7	17.5	-1.2	
				5	15.8	15.4	-0.4	-1.4
				5	16.9	14.4	-2.5	

ABSTRACT

Effect of Freeze and Thaw Cycling on Soils Stabilized using Fly Ash

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A considerable amount of research has been devoted to the stabilization of soils, aggregate, and recycled pavement materials using fly ash in highway applications, which demonstrated improvement in shear strength, compressibility and stiffness. However, how there is limited amount of research regarding how these materials stabilized with fly ash behave after exposed to winter conditions in the field.

Aging pavements, increasing wheel loads, and traffic frequency, combined with the effects of seasonal frost action are the main factors responsible for the rapid degradation of the highways in the northern regions of the United States. Pavements subjected to seasonal frost, experience freezing in the winter and thawing in the spring. During winter, an increase in strength and stiffness of the base and subgrade is observed. When spring comes, the base and subgrade become nearly saturated as the soils thaws and the snow and ice melt; which produce a reduction in strength and stiffness, often to values lower than prefreezing conditions. The recovery of the soil takes a long time and is partial. As a result, the weakened pavement cannot support the load for which it was originally designed, and damage occurs.

The objective of this research was to study how the resilient modulus and unconfined compressive strength of soils stabilized with fly ash change after freeze-thaw cycling. To reach this objective, resilient modulus and unconfined compression tests were conducted on a range of fly ash stabilized materials after freeze-thaw cycling (0, 1, 3, 5, 10, and 12 cycles). The stabilized materials tested included fine-grained soil, coarse-grained soil, and recycled pavement material. Five different fly ashes were used [Columbia and Riverside 7 (classified as Class C); Dewey, King and Riverside 8 (classified as off-specification)] at different percentages (10%, 12%, 14% and 20%) and at three different water contents (7% wet of optimum, optimum, and at field water content). Tests were also conducted on soil alone (0% fly ash) without freeze-thaw cycling to define the reference condition.

For all the mixtures, with an exception of USH 12 STA 614 + 12% Columbia fly ash, the resilient modulus (M_r) decreases in response to freeze-thaw and then appears to level off in approximately 1 to 5 cycles. The drop in modulus ranges between 7 and 50%; with an average of 28.5%. From these results can be concluded that for highway design, the safest way to represent the freeze-thaw cycling on the resilient modulus of the soil-fly ash or granular-material fly ash mixtures is dividing the value by 2.

Recycled pavement materials (RPMs)-fly ash mixtures show a M_r reduction after freeze-thaw cycling as the percentage of fines increased. Lower M_r reduction after freeze-thaw cycling were obtained when soil-fly ash mixtures and coarse-grained material – fly ash mixtures are stabilized with high CaO content fly ashes.

A general trend of higher resilient modulus (156% to 56% higher) when soils are stabilized with fly ash even after freeze-thaw cycles is clearly observed.

In general, a reduction in unconfined compressive strength (qu) after freeze-thaw cycles up to 70% was obtained. Different qu behavior trends were observed:

- unaffected with freeze-thaw cycling
- unaffected up to 3 to 5 freeze-thaw cycles then strength starts dropping
- drop in strength up to 3 freeze-thaw cycles and then levels off (fine-grained soils only)
- continue decrease in strength since the first freeze-thaw cycle (RPM only)

Higher qu reduction after freeze-thaw cycles were experienced by RPMs.

Fine-grained soils showed increase or less reduction in qu with freeze-thaw cycles when are stabilized with high CaO content fly ashes. Coarse-grained soil and RPMs showed increase or less reduction in qu after freeze-thaw cycles when are stabilized with $CaO/(SiO_2+Al_2O_3)$ content fly ashes.

A general trend of higher unconfined compressive strength (between 157% and 9.3%) when fine-grained soils are stabilized with high CaO content fly ashes even after freeze-thaw cycles is clearly observed. Qu test could not be performed on coarse- grained soil and RPMs because are loose material.

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