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Resilient Modulus Characterization of Alaskan Granular Base Materials

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When spring comes to cold regions, the active layer (the top few feet of soil that freezes and thaws seasonally) thaws quickly, while deeper soil remains frozen. The active layer becomes saturated with water from snowmelt that collects atop the frozen layer. In these circumstances, roads across Alaska almost “float” on a soft foundation. Too often, poorly supported pavement buckles and sags under the weight of heavy tractor trailers and other vehicles, and it remains deformed once the soils drain and re-stabilize. One way to reduce this damage is to control the amount of fines (essentially rock dust) in a pavement mixture. This project investigated base course materials commonly used in Alaska’s roads. Liu’s team observed changes in the stiffness of the materials, as well as how their soil-water characteristics change under freeze-thaw cycles, and how different percentages of fines and moisture influence material properties. Field tests in Alaska’s three DOT&PF regions and subsequent lab research have been completed. Data from this study will be used to produce better pavement designs, particularly in some rural areas, where project engineers might be forced to use locally available material with high fines content.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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EXECUTIVE SUMMARY

Resilient modulus (M_R) of base course material is an important material input for pavement design. In Alaska, due to distinctiveness of local climate, material source, fines content and groundwater level, resilient properties of D-1 granular base course materials are significantly affected by seasonal changes. The presence of fines (P_{200}) affects frost susceptibility of base materials and controls the aggregates' ability to support vehicular load, especially during the spring-thaw period. To systematically evaluate the impact of fines content on the resilient properties of D-1 base course materials with varied fines content, gradation, moisture content and temperature during thawing and provide regression coefficients k_i which are required for the flexible pavement design, a laboratory investigation was conducted on D-1 materials from Northern, Central, and Southeast Regions of Alaska Department of Transportation and Public Facilities (AKDOT&PF) at different temperatures, moisture and fines contents.

Tests were conducted first to determine physical properties such as gradation, optimum moisture content (OMC), maximum dry density, abrasion resistance, and flat or elongated particles of D-1 materials from three regions.

M_R data were determined by conducting repeated triaxial tests on D-1 materials with fines content ranging from 3.15% to 10% and moisture content from OMC-2% to OMC+0.7%, respectively. Permanent deformation data on aggregate specimens were collected from M_R tests as well. For M_R tests at subfreezing temperatures, a frost heave cell was designed and fabricated for specimen preparation. To simulate the natural frost heave in winter, aggregate specimens experienced a freezing process by using frost heave cell. The designed frost heave cell is an open system which allows free water intake during freezing process. Frost heave data and change of moisture contents after the freezing process were obtained from frost heave tests. M_R tests were also conducted on aggregate specimens after the freeze-thaw cycle. However, most of specimens collapsed during testing which indicated significant loss of M_R after the freeze-thaw cycle.

After the freezing process in the open system, aggregate specimens with low initial moisture and high fines content led to higher final moisture contents. Most of

frost heave values were less than 3 mm for D-1 materials with fines contents ranging from 3.15% to 10%. Frost heave increased with increases of fines content after the freezing process which could also be affected by initial moisture contents of aggregate specimens. M_R tests results showed that M_R decreased with an increase of moisture content. Within the scope of this study, impact of fines content varied which were affected by moisture content and material source. At subfreezing temperatures, there was a significant increase of M_R when compared with those at room temperature. Impacts of fines and initial moisture content on M_R values were weakened due to the change of moisture and aggregate structure after freezing process. Resilient behavior of D-1 materials were affected by temperature and deviator stress. At room temperature, M_R of D-1 materials from three regions increased with increase of confining pressure when moisture contents ranged from 3.3% to 6%. However, at low moisture content, M_R decreased as the applied deviator stress increased. When moisture content was at the OMC or higher, M_R increased with increase of deviator stress. However, effect of confining pressure became insignificant for D-1 materials at high moisture content. At subfreezing temperatures, the confining pressure did not provide significant effect on resilient modulus values. Temperature was found to be another important influencing factor on M_R of D-1 materials, especially when temperature ranged from -5 ^oC to 0 ^oC. As temperature decreased, M_R increased. However, when temperature was decreased to -5 ^oC, M_R values seemed to be stable and further change of temperature did not result in any significant change of M_R. Resilient properties of D-1 materials at room temperature after one freeze-thaw cycle were also investigated in this study. The reduction of M_R after the freeze-thaw cycle was inevitable and significant, especially for aggregate specimens with high fines contents. Permanent strain of D-1 materials was significantly affected by moisture content. The higher the moisture content, the higher the permanent strain. Effect of fines content on permanent strain of D-1 materials was insignificant which could be affected by other factor such as moisture content, aggregate shape, and material source. At subfreezing temperatures, permanent deformation increased with as increase of temperature, especially when temperature was close to 0 ^oC.

Regression equations were also developed to correlate M_R values with the physical properties (moisture and fines contents), stress states, and temperature conditions of D-1 materials. For D-1 materials tested in this study, at room temperature, M_R was found to be a function of stress state, moisture and fines contents. At subfreezing temperatures, M_R was a function of deviator stress, temperature, and aggregate type. These equations obtained can be used to predict M_R values of Alaskan D-1 materials for pavement design.

In this study, frozen D-1 material specimens were prepared using the one-dimensional frost heave cell for M_R tests at subfreezing temperatures. As the open system of water access represents the worst scenario that a pavement structure could possibly experience, it is necessary to investigate the behavior of base course materials under other conditions with limited water access during freezing in order to complement the existing research and have a better understanding of the combined effect of fines content and moisture content on the resilient behavior of D-1 materials under different conditions of water access. In this study, frost heave after freezing process for D-1 materials with high fines was not significant. One possible reason was that only one temperature gradient was used. Therefore, some other temperature gradients are suggested for specimen preparation in future studies to better understand the resilient behavior of D-1 materials at different temperature conditions. Also, physical properties of D-1 materials from three regions were similar. Therefore, more tests for D-1 materials from different sources are needed to characterize the effect of fines content on resilient behavior of D-1 materials.

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CHAPTER I

INTRODUCTION

Roadway pavement base course saturation and weakening because of partial thawing is the typical springtime condition in most northern regions and is normally reflected by reductions in the resilient properties of the affected materials. The percentage of fines usually controls the aggregate's ability to support vehicular load, especially during the springtime thaw period. However, the impact of fines content on the resilient behavior of granular base course materials during thawing has not been thoroughly investigated, especially with respect to the material types and climatic conditions typical of most northern regions.

D-1 materials (Table 1.1), defined according to Standard Specifications for Highway Construction (McHattie 2004), are commonly used as granular base course material in Alaska. Hence, this study systematically investigated the resilient behavior of D-1 base course materials commonly used in three regions of Alaska and effects of various factors on the materials performance, as presented in this thesis.

Table 1.1 Definition of D-1 Granular Base Course Material

Sieve size	1 in.	3/4 in.	3/8 in.	No.4	No.8	No.50	No.200
% Passing	100	70-100	50-80	35-65	20-50	8-30	0-6

Problem Statement

In 1980s, an extensive performance study of 120 Alaskan flexible pavement sections (McHattie 1980, McHattie 1982, Esch et al. 1981, Esch and McHattie 1983) indicated that the percentage of fines (weight-based percentage of particles finer than 0.075mm) usually controls the aggregate's ability to support vehicular load, especially during the springtime thaw period. Excess fines will cause springtime softening in layers supporting the pavement surface layer. In addition, the critical excess fines content (i.e. threshold fines content) varies with different aggregate sources, gradations, and moisture contents. To best evaluate the service life of pavements under cold-region climates, these factors become mandatory in pavement design. Yet these factors have not been thoroughly investigated, especially with respect to the material types and climatic conditions typical of most northern regions. Therefore, there is a need to investigate the impact of fines content on the resilient behavior of aggregate base course materials during thawing for D-1 materials with different material sources, gradations, and moisture conditions.

Research Objectives

In this study, the following objectives are addressed:

- To systematically evaluate the impact of fines content on the resilient modulus of base course materials during thawing,
- To investigate the associated performance of base course materials with varied fines content, gradation, and moisture content. The performance to be investigated includes permanent deformation potential, and soil-water characteristic curves (SWCC), and
- To evaluate the material properties, k_i which are required for the flexible pavement design.

Research Methodology

To meet the objectives of this study, the following major tasks are accomplished:

- Task 1: Literature Survey
- Task 2: Laboratory Study
- Task 3: Test Results and Analyses
- Task 4: Conclusions and Recommendations

Task 1: Literature Survey

A comprehensive literature review was conducted to gather information related to this study. Resilient properties of base course materials and M_R testing methods were reviewed. Literature review also included the impact of factors such as moisture content, fines content, temperature, and freeze-thaw cycle on resilient properties of granular materials.

Past literature on simulation of frost heave process was also gathered, as it related to sample preparation of M_R test at subfreezing temperatures. Literature review on previous regression models pertaining to resilient modulus under subfreezing and nonfreezing temperatures was also performed. The literature findings were summarized and documented in Chapter II.

Task 2:Laboratory Study

In this study, tests were performed mainly in the laboratory. Three D-1 base course materials commonly used in each of the three regions (Southeast, Northern, and Central Regions) of AKDOT&PF were used. Four fines contents, three moisture contents, and a series of temperatures were selected to investigate the effects of those factors on resilient behavior of D-1 materials from three regions. Prior to the mix design and specimen preparation, the aggregate fundamental physical properties were evaluated including aggregate gradation, OMC, maximum dry density (MDD), and abrasion resistance. Aggregate specimens were prepared using the impact compaction method with a soil compactor. For specimens to be tested at subfreezing temperatures, a frost heave cell was used to simulate the natural frost heave processes.

In this study, the laboratory tests were conducted according to test protocols described in American Society for Testing and Materials (ASTM) or American Association of State Highway and Transportation Officials (AASHTO) Standard Test Methods. Frost susceptibilities of the soil specimens were identified by evaluating frost heave values and change of moisture contents after the freezing process. Repeated load M_R tests were conducted to determine the M_R of aggregate specimens in a temperature-controlled environmental chamber. The experimental details were presented in Chapter III.

Task 3:Test Results and Analyses

In this task, laboratory test data were statistically analyzed. The frost heave data and resilient properties of D-1 base course materials were summarized. The effects of factors such as fines content, stress state, temperature, freeze-thaw cycle, and moisture contents on material performance were also evaluated. Also, permanent deformation data were collected for D-1 materials after M_R tests. Models were developed for predicting M_R of D-1 base courses materials with different fines and moisture contents at different temperatures. Chapter IV presents the detailed work under this task.

Task 4: Conclusions and Recommendations

Based on the tasks above, a summary of conclusions were presented in this task. Also, recommendations regarding future study of resilient properties of D-1 used as base course material were made and presented in Chapter V.

CHAPTER II

LITERATURE RIVIEW

A comprehensive literature review was performed to gather information related to resilient properties of granular base course materials under repeated loading. Topics covered begin with a brief introduction of M_R definition and testing methods. This is followed by a discussion of the factors that affect the resilient properties of base materials such as fines, temperature, moisture content, stress state, and freeze-thaw cycle. Also, review of previous regression models pertain to M_R under subfreezing and nonfreezing temperature was performed. The literature findings are summarized and documented as follows.

M_R and Its Testing Methods

Definition of M_R

The concept of M_R was firstly introduced by Seed et al. (1955). The resilient response of granular materials is usually characterized by M_R at a given stress state. As a vehicle passes over pavement structure, a stress pulse is applied to base course layer. Unbound granular base course material exhibits a combination of resilient strains, which are recovered after each load cycle, and permanent strains, which accumulate with every load cycle. The stress-strain relationship for unbound granular base course material is non-linear as illustrated in Figure 2.1.

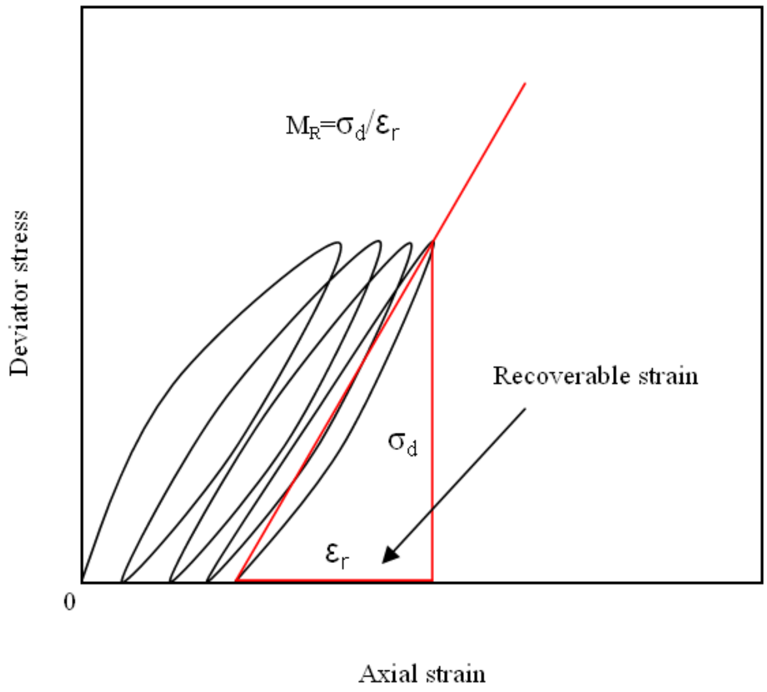


Figure 2.1 Definition of M_R

M_R is mathematically defined as the applied deviator stress divided by the "recoverable" strain that occurs when the applied load is removed from the test specimen (Equation 2.1).

$$M_R = \frac{\sigma_d}{\varepsilon_r} \tag{2.1}$$

where,

σ_d = deviator stress, and

ε_r = recoverable or resilient strain.

M_R was adopted by the AASHTO (1993). This procedure incorporated the M_R concept to properly describe behavior of pavement materials subjected to moving traffic. After that, M_R became an important required input parameter for pavement design.

Testing methods

Federal Highway Administration (FHWA) set a standard protocol for M_R testing which was known as Long Term Pavement Performance (LTPP) Protocol P46 (1996). For granular base course materials, LTPP Protocol P46 shares the same loading sequences with AASHTO T307 (2002). In subsequent years, a large amount of M_R data was produced and used in the late 1990s as the parameter became more heavily involved in pavement design processes.

M_R is determined by performing repeated load triaxial test on cylindrical specimens. In LTPP Protocol P46 or AASHTO T307, repeated dynamic haversine loading waveform with a loading duration of 0.1 sec is used. This load pulse is followed by a 0.9 s period in which only a seating load equal to 10% of the peak stress is applied to the specimen while the soil recovers from the loading to simulate the passing of one axle over a pavement followed by a period of rest before the passing of next axle in the field. Conditioning is utilized to reduce the disturbances due to specimen preparation procedures and minimize the effects of imperfect contact between end platens and specimen. This loading cycles for conditioning repeat at least 500 times at different confining pressure and deviator stress levels before M_R data collection. Then, every loading sequence repeats 100 times as long as the permanent strain was within a 5% limit for the entire loading history during testing. Loading sequences repeat 15 times at different confining pressure and deviator stress levels. For pavement design, it is anticipated that the resilient deformation increases more than the permanent deformation as more load cycles are applied, such that after a large number of cycles the deformation under each cycle is nearly recoverable. Hence, M_R values are calculated with the last five cycles in each loading sequence.

A new protocol known as the National Cooperative Highway Research Program (NCHRP) 1-28A (2002) was released to improve the old protocol. There were several differences between these procedures. One of the most significant differences is that the new protocol involves larger stresses on specimens. These stresses are large enough to cause the failure of some soil specimens. NCHRP 1-28A has a larger number of test sequence variations for different soil classifications and the load pulse that simulates traffic loading is lengthened from 0.1 s to 0.2 s. However, the primary difference for granular soils is the number of loading sequences. LTPP P46 requires cyclic testing at 15 different combinations of confining pressure and deviator stress levels, while NCHRP 1-28A requires 30. In addition, NCHRP 1-28A has not yet been widely implemented and the majority of existing M_R data was generated by using LTPP P46 or AASHTO T307. For these reasons, AASHTO T307 was selected as the testing method for determination of M_R in this study.

Influencing Factors of M_R

Since the introduction of the concept of M_R in 1955, a considerable amount of research has been devoted to evaluating the resilient properties of pavement materials. Experimental results on the M_R of granular materials with different aggregate sources, different testing procedures, and use of different numerical models have been reported.

Previous research efforts were directed towards the influencing factors on resilient properties of soil. Most of these studies were focused on the effects of moisture content, density, and stress condition (Mitry 1964, Smith and Nair 1973, Elliot and Thornton 1988, Elfino and Davidson 1989, Brown and Selig 1991, Kolisoja 1997). At room temperature, moisture content can affect resilient properties of granular materials significantly (Haynes and Yoder 1963, Hicks and Monismith 1971, Vuong 1992). Under undrained condition, with increase of moisture content, excess pore-water pressure develops due to repeated loading. As the development of pore-water pressure, effective stress decreases with a subsequent decrease in both strength and stiffness. Rada and Witczak (1981) evaluated the test results obtained from 10 research agencies and indicated that the primary variables that influence the M_R response of granular materials are the stress state, degree of saturation, and degree of compaction.

Principle stress is one of the most important factors affecting resilient properties of soil. Brown and Hyde (1975) reported that the permanent axial strain settled down to a constant related to the ratio of the deviator stress and confining pressure on crushed stone after M_R tests. Sweere (1990) stated that M_R increased considerably with an increase in confining pressure and sum of principal stresses.

Aggregate shape and texture is another influencing factor which should be considered for coarse materials. Heydinger et al. (1996) stated that gravel have a higher M_R than crushed limestone. Also, many other researchers (Hicks and Monismith 1971, Allen and Thompson 1974, Thom 1988, Thom and Brown 1989) showed that crushed aggregate, having angular to subangular shaped particles, provides better load spreading properties and a higher M_R than uncrushed gravel with subrounded or rounded particles. A rough particle surface is also shown to result in a higher M_R. Barksdale and Itani (1989) stated that the M_R of the rough, angular crushed materials was higher than that of the rounded gravel by a factor of about 50% at low mean normal stress and about 25% at high mean normal stress.

The variation of fines content in the range of 2-10% was reported by Hicks (1970) to have a minor influence on M_R. Limited studies revealed that when fines content in granular materials is relatively high, effect of fines content on resilient properties will display. Permanent deformation resistance in granular materials is reduced as the amount of fines increases (Barksdale 1991). Due to different gradations, effect of fines content on resilient properties of soil can also be different. An extensive performance study of 120 Alaskan flexible pavement sections (McHattie 1980, McHattie 1982, Esch et al. 1981, Esch and McHattie 1983) indicated that the percentage of fines (weight-based percentage of particles finer than the #200 sieve, also known as P_{200} content) usually controls the aggregate's ability to support vehicular load, especially during the springtime thaw period. The general relationship is low P_{200} content = good support and high P_{200} content = poor support. Excess P_{200} will cause springtime softening in subsurface base/subbase layers and subsequent deterioration of the surface layer. This is explained by a correlation between the amounts of frozen moisture to the P_{200} content in granular layers. Additional research led to the development of the excess fines method (McHattie et al. 1980 and 1982, Esch et al. 1981 and 1983) for Alaska flexible pavement design based on the establishment of an empirical relationship between the P_{200} content and the springtime pavement surface deflection corresponding to a thaw-weakened state. As a useful indicator of frost susceptibility for pavement design purposes, the critical excess fines content varies with different aggregate sources, gradations, moisture contents. Therefore, it is clearly important to investigate the impact of fines content on the resilient behavior of base course materials during thawing for different material sources, gradations, and moisture conditions.

Previous studies indicated that the resilient properties demonstrate extensive variation due to seasonal freezing and thawing actions. Any realistic pavement design or evaluation must account for these changes. Fredlund et al. (1975) found a significant reduction in matrix suction after freeze-thaw cycling, which was associated with a reduction in M_R. Bergan and Fredlund (1972) presented that similar reductions in matrix suction occurred in undisturbed subgrade samples during spring thaw. The resilient properties of granular materials from frozen to thawed conditions were investigated (Johnson et al. 1978, Cole et al. 1986, and Berg et al. 1996). Simonsen et al. (2002) summarized basic conclusions from these investigations as follows: (1) significant loss of strength upon thaw for most soils tested; (2) a gradual regain of strength as moisture drained from the soil during the recovery period; and (3) a two-to three-order magnitude increase in strength of all materials at subfreezing temperatures. Associated with a reduction in M_R was a significant reduction in matric suction after freeze-thaw cycling (Fredlund et al. 1975). The reduction in matric suction was substantial below the optimum moisture content, diminishing above optimum. Also, extensive M_R laboratory test during full freeze-thaw cycle on various coarse and fine-grained subgrade soils were carried. However, closed-system and omnidirectional freezing and thawing utilized in this study are a limitation compared to many naturally occurring systems. Water uptake was not allowed during freezing. Moreover, realistic freezing of triaxial soil samples requires uniaxial freezing and thawing, which is difficult to combine with M_R testing without disturbing the sample.

M_R Modeling

Resilient behavior of granular materials is affected by many factors. The complexity of the problem has made it difficult for researchers to model the relationship between stress states with material stiffness. After several decades of endeavor, in order to describe stress dependence of the M_R, several constitutive models have been developed using various stress variables. A great majority of the models found in the literature are based on simple curve-fitting procedures by using the data from laboratory triaxial testing. Some of the models are presented as below.

Dunlap (1963) and Monismith et al. (1967) showed that the M_R of granular materials increase with confining pressure and is sensibly unaffected by the magnitude of repeated deviator stress. Also, the following expressions (Equations 2.2 and 2.3) based on the effect of confining pressure were proposed:

$$M_R = k_1 \sigma_3^{k_2} \tag{2.2}$$

$$M_R = k_1 \left(\frac{\sigma_3}{p_a} \right)^{k_2} \tag{2.3}$$

where,

p_a = normalizing stress (atmospheric pressure, 14.5 psi),

k_1, k_2 = regression constants, and

σ_3 = minor principal stress/ confining pressure.

Actually, the main drawback of this model is that deviator stress was not included. May and Witczak (1981) stated that the magnitude of the shear strain induced mainly by deviator stress can also result in change of the in-situ M_R of a granular layer.

One of the most popular models in dealing with the effect of stress on material stiffness is the expression solely based on the sum of the principal stresses (bulk stress). Seed et al. (1967), Brown and Pell (1967), and Hicks (1970) developed the following relationship which was known as $K-\theta$ or bulk stress model shown in Equations 2.4 and 2.5.

$$M_R = k_1 \theta^{k_2} \tag{2.4}$$

$$M_R = k_1 \left(\frac{\theta}{p_a} \right)^{k_2} \tag{2.5}$$

where,

θ = bulk stress $= \sigma_1 + \sigma_2 + \sigma_3$ and

$\sigma_1, \sigma_2, \sigma_3$ = principal stresses.

Due to its simplicity, $K - \theta$ model is widely accepted by engineers for analysis of stress dependence of granular material stiffness. However, this model also has several drawbacks. The main drawback is that it does not account for shear stress and strain developed during loading and this model could not be able to properly handle volumetric strains or dilative behavior of soil. In addition, the effect of stress on M_R is accounted for solely by the sum of the principal stresses which means this model potentially provides the same M_R when similar bulk stresses are used. These bulk stresses can be obtained either from higher confining pressure and lower deviator stress or lower confining pressure and higher deviator stress. This indicates the model does not incorporate the realistic response of confining and deviator stresses on the resilient properties. Several studies have shown this to be insufficient and additional stress parameters are required.

Recognizing the drawbacks of confining pressure and $K - \theta$ models, several other models were developed by other researchers. Among them, one model known as octahedral stress state model developed by Witczak and Uzan (1988) appears to be a feasible and realistic one due to the introduction of deviator stress into $K - \theta$ model as shown in Equations 2.6 and 2.7.

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\sigma_d}{p_a}\right)^{k_3} \tag{2.6}$$

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a}\right)^{k_3} \tag{2.7}$$

where,

τ_{oct} = octahedral shear stress $= \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2}$ and

σ_d = deviator stress $= \sigma_1 - \sigma_3$.

This model not only accounts for dilation effects by incorporating shear stresses as one of the attributes, but it also accounts for the confining pressure effects. For triaxial conditions, since intermediate and minor principal stresses are equal ($\sigma_2 = \sigma_3$ and $\tau_{oct} = \frac{\sqrt{2}}{3}\sigma_d$), these parameters can be determined by the confining and deviator stress applied in a triaxial test. This model can be used for various soil types without altering two model attributes, octahedral normal and shear stresses. The resilient properties of the soils are dependent on confining pressure, normal stress, and deviator stress state. The octahedral normal and shear stresses provided a better explanation for the stress states of a material.

In Mechanical-Empirical Pavement Design Guide (MEPDG) (ARA, Inc. 2000), the regression model, modified based on Equation 2.7, was presented for M_R prediction as shown in Equation 2.8.

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \tag{2.8}$$

Pavement structures are always “stronger” below 0 °C than those above 0 °C. According to Simonsen et al.’ study (2002), a mathematical expression of the regression model shown in Equation 2.9, was used to represent the resilient properties of materials under subfreezing temperature. Temperature is the only independent parameter in this model.

$$M_R = e^{k_1 + k_2 / T} \tag{2.9}$$

where,

k_1, k_2 = regression constants and

T = temperature, degree Celsius.

Frost Heave Simulation

Topics on frost heave of soil have been studied for nearly 80 years. Casagrande (1932) proposed that under natural freezing conditions and with sufficient water supply one should expect considerable ice segregation in non-uniform soils containing more than 3% of grains smaller than 0.02 mm. Generally, three essential conditions are required for frost heave: (1) frost susceptibility of the soil or aggregate within the depth of frost penetration, (2) the availability of water, and (3) the magnitude and duration of freezing temperatures at the surface. Water residing within the soil or aggregate structure at the time of freezing can be a principal source of moisture in the formation of ice lenses even in closed systems (Hermansson 2000, Guthrie and Hermansson 2003). However, the rate of frost heave in such cases is ultimately limited by the decreasing availability of free water in the soil. In other words, when soil water is replenished from below as readily as it is redistributed to the freezing front above, frost heave may remain uninterrupted. Such sustained frost heave activity requires the continuous intake of new water by the freezing soil strata. In the field, the availability of free water is determined to a large degree by the relative proximity of the ground water table to the freezing front. A deep water table can reduce the supply of water to the freezing front, thereby limiting the occurrence of frost heave in upper layers. Conversely, a shallow water table can facilitate an increased flow of water to the freezing zone and lead to greater frost heave.

Konrad and Morgenstern (1980) performed frost heave test to determine unique frost heave characteristics and stated that at the beginning of freezing, the ice may strain without breaking. However, as the frozen front penetration rate decreases, temperature change rate across the sample is also reduced. Water is then able to accumulate at a given level for a longer period of time. The adjacent ice is then strained to a higher degree which will lead to failure of the ice grains. Further water accumulation and freezing result in the formation of a discrete ice lens and surface cracking and roughness of pavement will appear. This ice lens is now unstressed and grows onto a roughly planar substrate composed of soil particles, unfrozen water, and the pore ice at the top of the frozen fringe. However, in spring, frozen soil melts from top to inside. Continuous accumulation of water resulted in reduction of road bearing capacity during spring thaw in upper part of roadbed (Taber 1930).

CHAPTER III

LABORATORY STUDY

This chapter consists of a description of the experimental details in this study including the materials, specimen preparation, testing setup, and testing methods. General aggregate properties such as aggregate gradations, OMC and MDD, abrasion resistances, frost susceptibilities and other properties are presented in this chapter. Repeated load triaxial tests were conducted to evaluate the seasonal variation of the resilient properties and permanent deformation of D-1 granular base course materials with different fines and moisture contents, and temperatures.

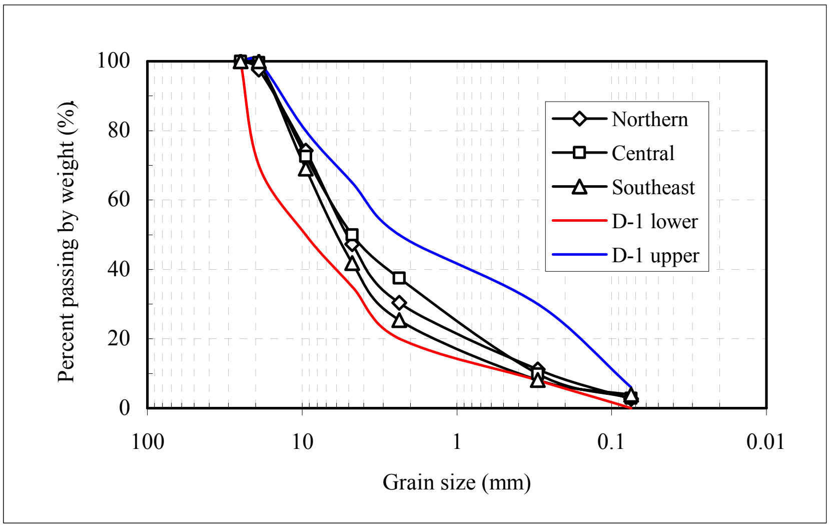
Materials

D-1 materials used in this study were collected from three regions: Northern (Fairbanks), Southeast (Juneau), and Central (Anchorage) regions of AKDOT&PF (Figure 3.1). Prior to the specimen preparation, aggregate properties were evaluated including aggregate gradation, abrasion resistance, OMC, and MDD.

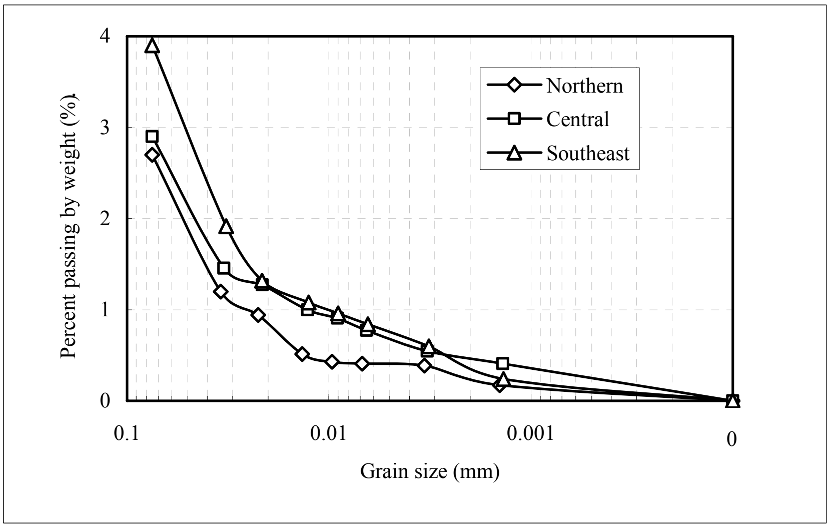
Figure 3.1 D-1 materials from three regions of AKDOT&PF

Gradation

Particle size distribution was analyzed by conducting sieve analysis and hydrometer tests for D-1 materials from three regions according to AASHTO T 27 (2002) and ASTM D422-63 (2007), respectively. Gradation curves with particle size greater and less than 0.075 mm (Sieve No. 200) are presented in Figures 3.2 (a) and (b).



(a) Particle size greater than 0.075 mm



(b) Particle size less than 0.075 mm

Figure 3.2 Gradation curves for D-1 materials from three regions

It can be seen from Figure 3.2 (a), D-1 materials from three regions had very close gradations. For particles with sizes greater than 10 mm, the gradations of three materials were close to the upper limit while for particles with sizes less than 2.36 mm

(Sieve No. 50), the gradations of three materials were close to the lower limit of the D-1 material. Since the gradations for three D-1 materials were very close, a single gradation, obtained by averaging the original gradations for three D-1 materials in Figure 3.2, was used to represent the D-1 materials from all three regions. The representative gradation, with a fines content of 3.15% as shown in Figure 3.3, was the reference gradation. Figure 3.2 (b) shows that contents of particles with size less than 0.02mm for D-1 materials from three regions are less than 3% which indicated they were non-frost susceptible soils according to Casagrande's criteria (1932). Appendix A tabulates the gradation results for D-1 materials from three regions.

As summarized in Chapter II, frost susceptibility is highly related to the fines content of the material, which could significantly affect the ability of aggregate to support the vehicle load on pavement surface. One of the objectives of this study was to investigate the influence of fines content on resilient properties of D-1 materials during freeze-thaw cycles. Fines content of 6% is the maximum fines content specified in Alaska Standard Specifications for Highway Construction (Table 1.1). In order to evaluate the variation of resilient properties of D-1 materials with high fines content, fines contents of 8%, and 10% were also selected. Gradation curves of D-1 materials with four different fines contents, 3.15% (reference), 6%, 8%, and 10% are shown in Figure 3.3, where FC represents fines content. The gradations for aggregates with 6%, 8%, and 10% fines contents were obtained by increasing the fines (<0.075 mm) contents in the reference gradation from 3.15% to the target percentages while maintaining proportions of particles with size greater than 0.075mm unchanged (Figure.3.3). Table 3.1 summarizes the contents of particles with size less than 0.02mm for D-1 materials from three regions with different fines content. It can be seen that at fines content of 8%, D-1 material from Central Region was frost susceptible. All D-1 materials from three regions with a fines content of 10% were frost susceptible. With the introduction of higher fines to D-1 materials, the susceptibility to frost heave increased. The gradations of D-1 materials used in this study are tabulated in Appendix B.

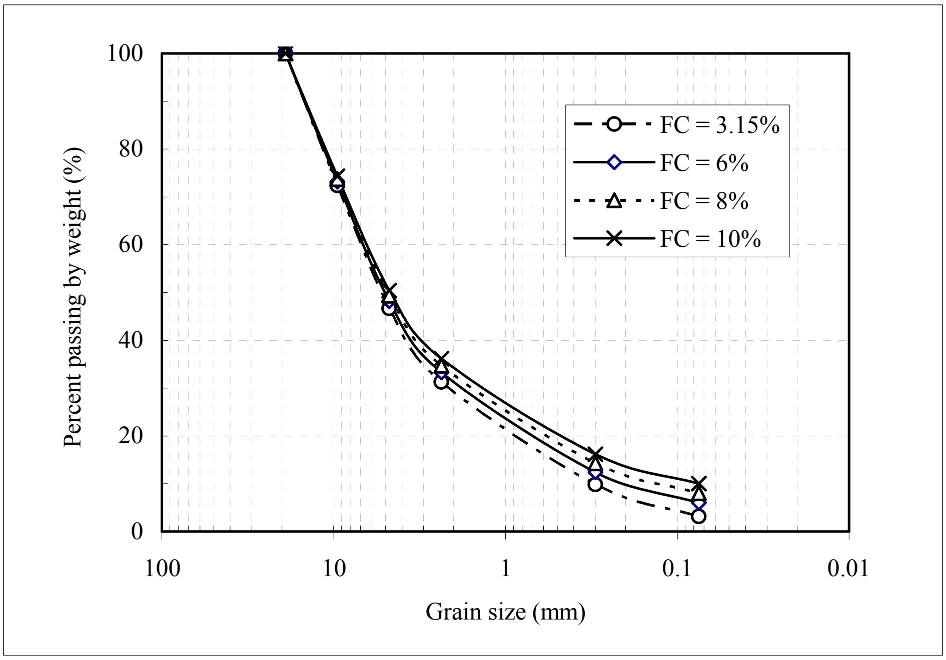


Figure 3.3 Gradation used for D-1 materials

Table 3.1 Contents of Particles with Size less than 0.02 mm in D-1 Materials

Fines(<0.075mm)	% less than 20 u (0.02 mm)			% less than 2 u (0.002 mm)		
	Northern	Central	Southeast	Northern	Central	Southeast
Original	0.83%	1.23%	1.28%	0.24%	0.45%	0.37%
3.15%	0.97%	1.34%	1.03%	0.28%	0.49%	0.30%
6%	1.84%	2.54%	1.97%	0.53%	0.93%	0.57%
8%	2.46%	3.39%	2.63%	0.71%	1.24%	0.76%
10%	3.07%	4.24%	3.28%	0.89%	1.55%	0.95%

OMC, MDD, Classification, and Soil -Water Characteristic Curve

In order to determine OMCs and MDDs of D-1 materials, compaction tests were conducted according to ASTM D 1557 Method C (2007). Aggregate specimens were compacted in five layers and each layer was subjected to 56 blows with a 10 lb and a hammer drop of 18 inches (457 mm). The specimens used in this study were 8 inches in height and 4 inches in diameter according to M_R test (AASHTO 2002). The mechanical compactor used in this study is shown in Figure 3.4. Table 3.2 summarizes the physical properties of the D-1 materials from three regions.

Figure 3.4 Automatic mechanical compactor

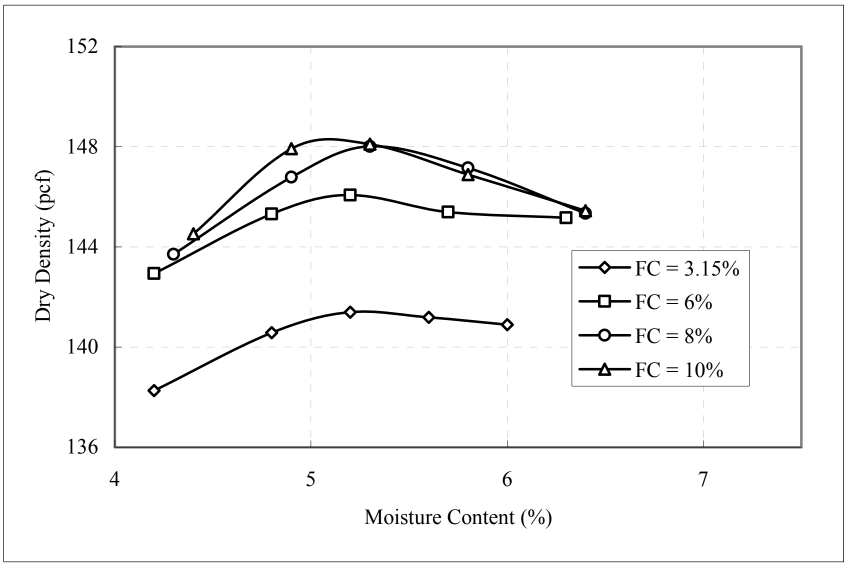
Table 3.2 Physical Properties of D-1 Materials

Fines content	Classification*		Central (%, pcf)		Northern (%, pcf)		Southeast (%, pcf)	
	AASHTO	USCS	OMC	MDD	OMC	MDD	OMC	MDD
3.15%	A-1-a	GW	5.8	146.5	5.2	141.4	5.3	156.0
6%	A-1-a	GW-GM	6.0	147.9	5.2	146.1	5.3	156.2
8%	A-1-a	GW-GM	5.4	150.2	5.3	148.0	5.4	156.7
10%	A-1-a	GP-GM	5.3	151.0	5.3	148.1	5.5	156.8

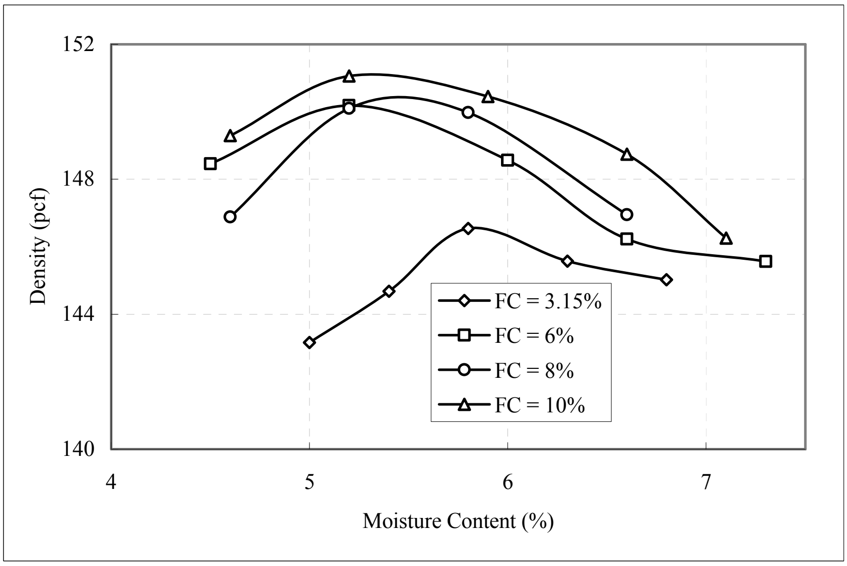
*GW — well-graded gravel with sand
GW-GM — well-graded gravel with silt and sand
GP-GM — poor-graded gravel with silt and sand

The results from compaction tests for D-1 materials from three regions with different fines contents are illustrated in Figure 3.5. Detailed compaction test results are shown in Appendix C. Among all these materials, D-1 materials from the Southeast Region had the highest MDDs. From Table 3.2 and Figure 3.5, one can find that MDDs slightly increased when fines content increased from 3.15% to 10%. However, when fines content was close to 10%, increase of fines content had little effect on aggregate density. Since all D-1 materials in this study were mostly composed of gravels and subsequently had low specific surface areas, all materials had low OMCs of approximately 5.3%. When moisture content was greater than

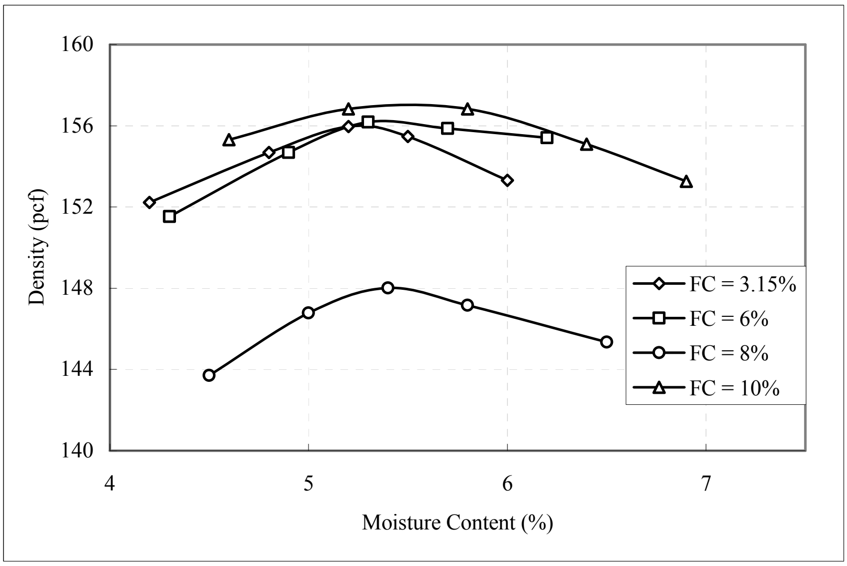
OMC, aggregate could easily be saturated and could not hold the original moisture content during compaction. Most aggregates started bleeding during compaction when moisture content was greater than 6%. Table 3.2 also presents soil classification results. According to the Unified Soil Classification System (USCS), the classifications of D-1 materials with four fines contents used in this study varied from well-graded gravel with sand (GW) to poor-graded gravel with silt and sand (GP-GM) as the fines contents increased from 3.15% to 10%. Based on AASHTO Soil Classification System, all D-1 materials are classified as A-1-a (granular material).



(a) D-1 materials from Northern Region



(b) D-1 materials from Central Region



(c) D-1 materials from Southeast Region

Figure 3.5 Density vs. moisture content curves for D-1 materials

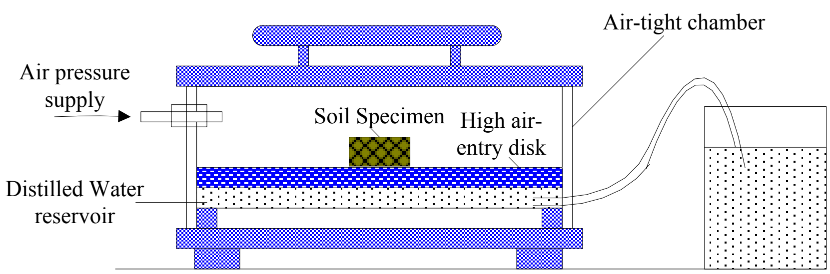
Soil-water characteristic curve (SWCC) is defined as the variation of water content or degree of saturation with suction. In this study, SWCCs of the three D-1 materials were obtained using a pressure plate extractors shown as in Figure 3.6a according to ASTM standars D2325-68. The matric suction was applied to a soil specimen by controlling the difference in the pore air pressure u_a and the pore water pressure u_w with both pressures being positive as shown in Figure 3.6b. The pore water pressure was controlled at an atmospheric pressure while the pore air pressure was changed to obtain the specific matric suction value. This procedure is referred to as the axis-translation technique (Hilf 1956). The main component of the pressure plate extractor is the high air entry disk as shown in Figure 3.6b that remains saturated for matric suction applications below the air entry value of the disk. The disk is always saturated and in contact with in a compartment below the compartment below the disk. The water pressure in the compartment is opened to the atmosphere to maintain at a positive pressure in the closed system. During the test soil specimen is placed on the high air entry disk. A good contact between the specimen and the disk results in the pore water pressure in the soil being controlled at the same pressure as the water pressure in the compartment. The air pressure is then applied to the specimen in order to impose the desire matric suction.

The equipments for the pressure plate suction test are the 1Bar, 5 Bar and 15 Bar ceramic plate extractors (Soil moisture Equipment Corp.). Pressure gauge is the ASHCROFT Laboratory Test Gauge CAT. No. 1082A with 0-300psi range was used. The procedure of the pressure plate tests were:

(1). completely submerge the high air entry disk in the distilled water for three days;
(2). install the high air entry value disk into the pressure plate extractor;
(3). fill enough distilled water into the pressure plate extractor until the high air entry value disk is submerged completely, install the pressure plate extractor and wait for one day.
(4). applying a target air pressure to squeeze the water out.
(5). uninstall the pressure plate extractor, keep the high air-entry value disk intact, and add a small amount of water to submerge the disk surface.
(6). trim the soil sample to a shape fitting with the disk surface, the soil sample is the same size as that for consolidation test, put the tested soil sample on the high air entry disk, make sure the bottom of the soil is in good contact with the surface of the disk. The water should submerge part of the soil sample.
(7). install the pressure plate extractor and apply an air pressure.
(8). note that during the test, frequently observe the air pressure to avoid air leakage, and keep the air pressure constant. Keep the water level of the outlet to make sure that the whole pipe system is completely filled with water.
(9). after one week, record the matric suction and the soils were taken out to measure the water content. The water content tests were performed in a way as soon as possible to avoid the water evaporation.

After testing (Figure 3.7), samples were taken out to determine their water contents. This procedure was repeated for each applied suction. Figures 3.8 ~ 3.10 present SWCCs of D-1 materials from three regions with four fines contents in a semi-logarithmic scale. As can be seen in Figures 3.8 ~ 3.10, though the gradation curves of D-1 materials at the same fines content were the same, SWCC of D-1 materials from three regions were slightly different, indicating that which surface tensions of materials from different regions were different. Under same applied matric suction, water contents of D-1 materials with high fines contents were higher than those with low fines contents. This is reasonable since fines have larger specific areas and subsequent larger suction at the same water content. However, with increase of applied pressure, the difference of water content between D-1 materials with different fines content became insignificant. For D-1 material from Northern Region, water content varied slightly with different fines content which was not identical to the results for D-1 materials from Southeast and Central Regions.

(a) Pictures of pressure plate extractors



(b) Schematic plot of the pressure plate extractor

Figure 3.6 Suction test using the pressure plate extractors

Figure 3.7 Soil samples after pressure plate test

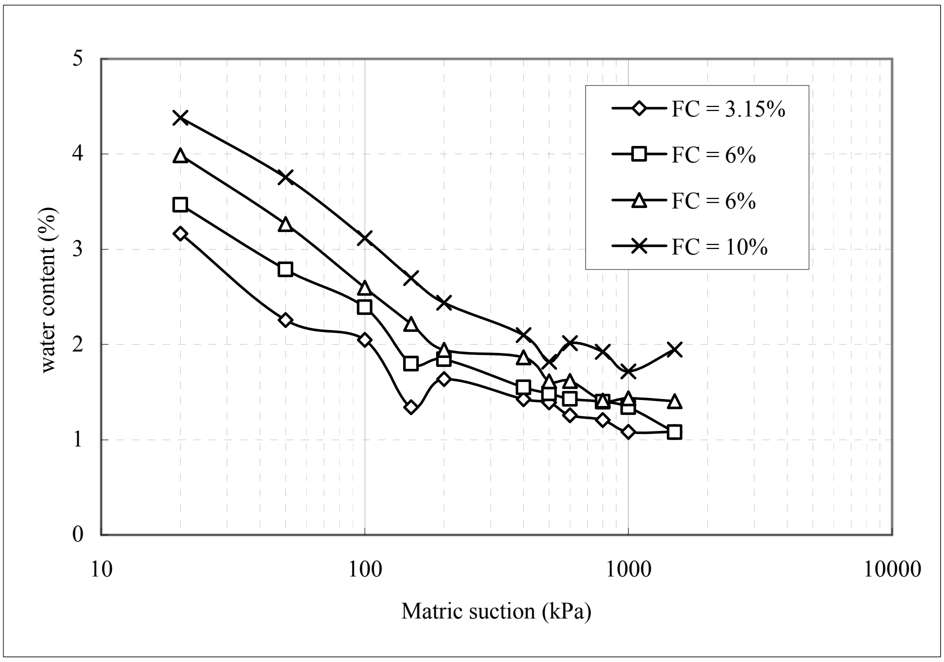


Figure 3.8 SWCC of D-1 material from Central Region

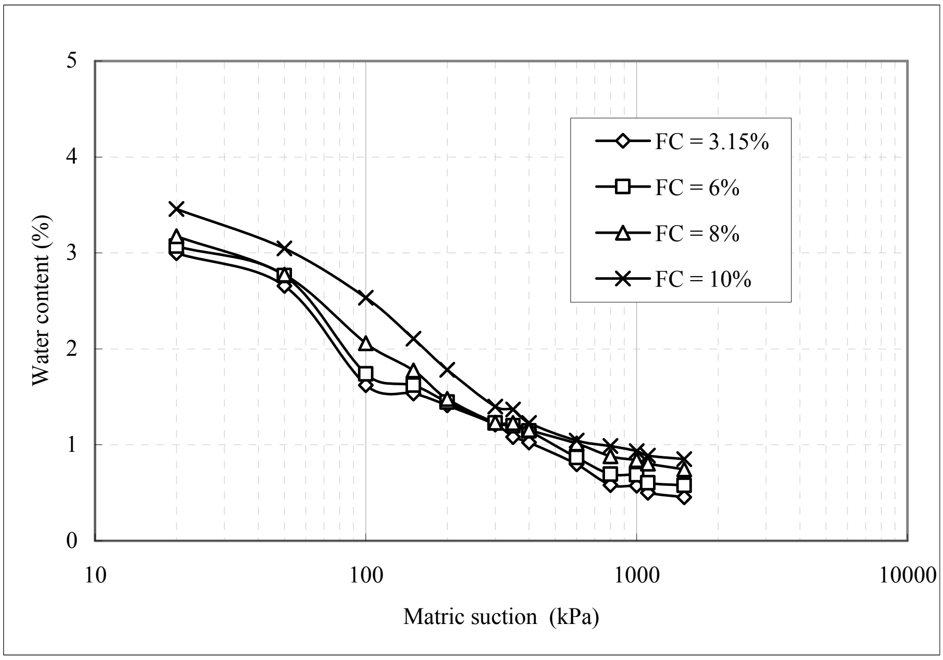


Figure 3.9 SWCC of D-1 material from Northern Region

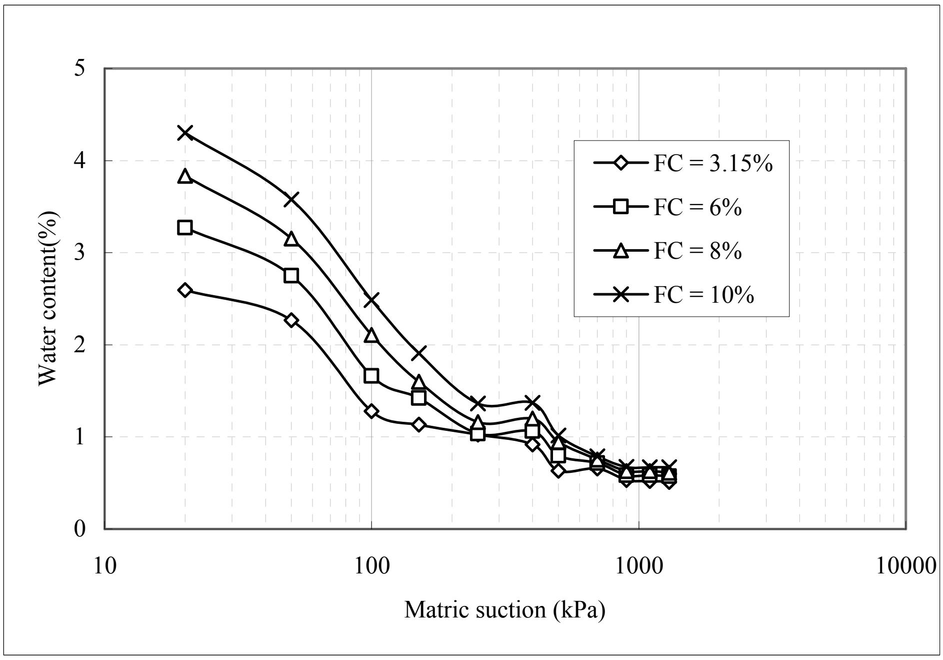


Figure 3.10 SWCC of D-1 material from Southeast Region

Abrasion Resistance and Shape of Aggregates

Micro-Deval tester was used to measure the abrasion resistance of aggregates in this study (ASTM D6928 2010) as shown in Figure 3.11. To determine the abrasion resistance of coarse aggregates, aggregates were firstly oven dried. 1500 g representative testing aggregate was used with approximately 750 g for each group of aggregates with particle sizes ranging from 3/4" to 1/2" and 1/2" to 3/8". The aggregates were then immersed into two liters of water for a minimum one hour in a container. This was followed by placing the soaked aggregates with water into the Micro-Deval abrasion container with 5000 g steel balls. Micro-Deval container was then placed on the machine shown in Figure 3.11. The machine was left running at 100 rpm for 12000 revolutions. The aggregates were then carefully poured over a No. 4 Sieve superimposed on a No. 16 Sieve. Steel balls were removed by using the magnetic bar shown in Figure 3.11. Particles passed the No. 16 Sieve were discarded. The material left was oven-dried to a constant mass. Loss of mass after testing divided by the original mass is defined as Micro-Deval abrasion loss.

Figure 3.11 Micro-Deval tester

Percent fractured face (one face) and flat or elongated particles based on ASTM D5821 (2006) and ASTM D4791 (2005), respectively, were evaluated for D-1 materials from three regions. Testing results and requirements of D-1 materials specified in Alaska Standard Specifications for Highway Construction (McHattie

2004) are summarized in Table 3.3. It can be seen that D-1 material from Northern Region had the best abrasion resistance performance when compared with D-1 material from Southeast and Central Regions. Percent fractured face is a measure of aggregate angularity. In flat or elongated particles test, if the ratio of length to width or width to thickness of an aggregate particle is 5:1 or over, it is defined as a flat or elongated particle. The test result is presented by the percentage of the number of flat or elongated particles over the number of entire aggregate particles. 100% particles of D-1 materials from Southeast Region had at least one fractured surface, while the percent of fractured surface for Central and Northern Regions were 91.7% and 84.5%, respectively. In these tests, D-1 materials from Southeast Region contained 3% flat and elongated particles and none of them were found in materials from Central and Northern Regions.

Table 3.3 Engineering Properties of D-1 Materials from Three Regions

Source	Micor-Deval (% Loss)	Percent Fractured Face (one face) (%)		Flat or Elongated Particles (5:1, %)	
		Test Results	Requirement	Test Results	Requirement
Southeast Region	9.7	100	≥80	3	≤8
Central Region	5.8	91.7		0	
Northern Region	2.7	84.5		0	

Specimen Preparation

Factors considered in this study to affect the M_R of D-1 materials in Alaskan pavements include fines content, temperature, and moisture content. The laboratory test factorials are summarized in Table 3.4. Four fines contents used were 3.15%, 6%, 8%, and 10%. Eight different temperatures were used, which were -10 ^oC, -7 ^oC, -5 ^oC, -4 ^oC, -3 ^oC, -2 ^oC, -1 ^oC, and 20 ^oC to simulate the natural freeze-thaw cycle. According to Simonsen (2002), M_R of aggregate will not change much for any temperature below -10 ^oC. Attempts were made to evaluate resilient properties for frozen D-1 materials at 0 ^oC. However, it was found that it was very difficult to maintain the environmental chamber to a constant temperature of 0 ^oC. Three moisture contents were used, which were 3.3% (OMC-2%), 5.3% (OMC), and 6%

(OMC+0.7%). The moisture content here was the design moisture content when aggregate blended with water. Moisture content of 6%, which is 0.7% above OMC, was found to be the maximum moisture content at which aggregate can hold during compaction.

Table 3.4 Design Factors

Factors	No. of levels	Levels
Fines content (FC)	4	3.15%, 6%, 8%, and 10%
Temperature	8	-10°C,-7 °C,-5 °C,-4 °C,-3 °C,-2 °C,-1°C, and 20°C
Moisture content (MC)	3	OMC, OMC-2% and OMC+0.7%

The aggregates with specified gradations according to Figure 3.3 were firstly mixed with water to specified moisture contents according to Table 3.4, and then compacted using the compactor shown in Figure 3.4. Following the same procedure as described in compaction test, one of the aggregate specimens was obtained and shown in Figure 3.12. For M_R test at room temperature, aggregate specimen was covered around by rubber membranes with help of a membrane stretcher (Figure 3.13).

Figure 3.12 Compacted aggregate base course specimen

Figure 3.13 Membrane and membrane stretcher

After covering with membrane, aggregate specimen was immediately used for the resilient tests under unfrozen conditions. For M_R tests at subfreezing temperatures, an open system frost heave cell was designed for the specimen preparation. Aggregate specimens were put into the frost heave cell to perform frost heave test first. A cylindrical plastic mold (Figure 3.14) was used to hold the specimen in the frost heave test. Part of the plastic mold bottom was cut off to ensure free water intake. A thin film of porous stone (Figure 3.15 (a)) was placed at the bottom of the mold. This was followed by covering a piece of filter paper (Figure 3.15 (b)) on the porous stone. Then, aggregate specimens were ready to be placed into the mold.

Figure 3.14 Cylindrical plastic mold

(a) (b)

Figure 3.15 Porous stone and filter paper

Aluminum plates were used to cover the top of specimen molds. The aggregate specimen after installation before putting into the frost heave cell is shown in Figure 3.16. Also, five holes were predrilled symmetrically along two sides of the mold to provide some space for installation of thermocouples. The thermocouples were fixed to the plastic mold using duct tape and used to monitor temperatures at different heights during the freezing process.

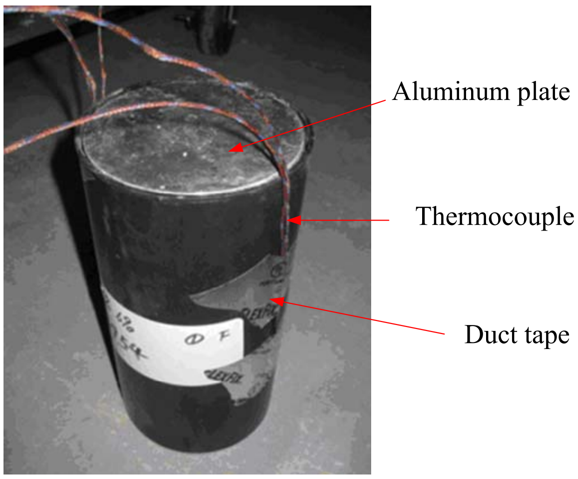


Figure 3.16 Specimen before frost heave test

After installation of thermocouples, the specimen was ready to be moved into the frost heave cell for one dimension frost heave test, which will be described later. When the frost heave test was accomplished, the frozen aggregate specimen was taken out, and covered by a rubber membrane for the M_R test.

Testing program

Frost Heave Test

A frost heave setup was designed and constructed to simulate the natural freezing process, which allowed performing one-dimensional frost heave tests for 8 aggregate specimens simultaneously. Bottoms of the aggregate specimens covered by porous stones were soaked in water bath to ensure the availability of free water. Due to the capillary force and suction gradient, free water can reach up to the freezing front which is an important condition needed for formation of ice lenses. The testing device allowed specimens to uniaxially freeze. Frost heave tests were performed to generate the frozen aggregate specimens needed for the M_R tests under subfreezing temperatures. Side-view of the frost heave setup is shown in Figure 3.17.

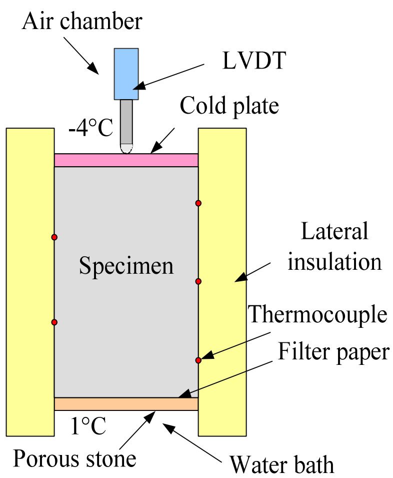


Figure 3.17 Schematic diagram of frost heave setup (side-view)

Temperatures of the air chamber and water bath were controlled at -4°C and 1°C which indicated a temperature gradient of 0.25°C/cm. This temperature gradient was obtained by analyzing Alaskan climate data using a geotechnical thermal model named Multilayer User-Friendly Thermal Model in 1 Dimension (MUT1D) (Braley and Zarling 1990). At side face of specimens, thick insulation was placed to block lateral heat transfer. As a result, aggregate specimens were frozen from top to bottom one dimensionally in the vertical direction. The porous stone shown in Figure 3.15 (a) was placed at the bottom of specimen to ensure free water uptake. Also, the filter paper shown in Figure 3.15 (b) was used to keep fines from flowing out of aggregate specimen. Linear variable displacement transducers (LVDTs) (Figure 3.17) were also placed at top of aggregate specimens to monitor vertical displacement during the freezing process. Frost heave and temperature data were collected electronically by installed LVDTs and thermocouples. Figure 3.18 schematically shows a top view of the frost heave setup. In order to maintain a constant temperature at the bottom of aggregate specimens, heat pipes were placed in the water bath with anti-freezing liquid circulation inside. Figures 3.19 (a) and (b) show the front- and inside- views of the frost heave machine, respectively.

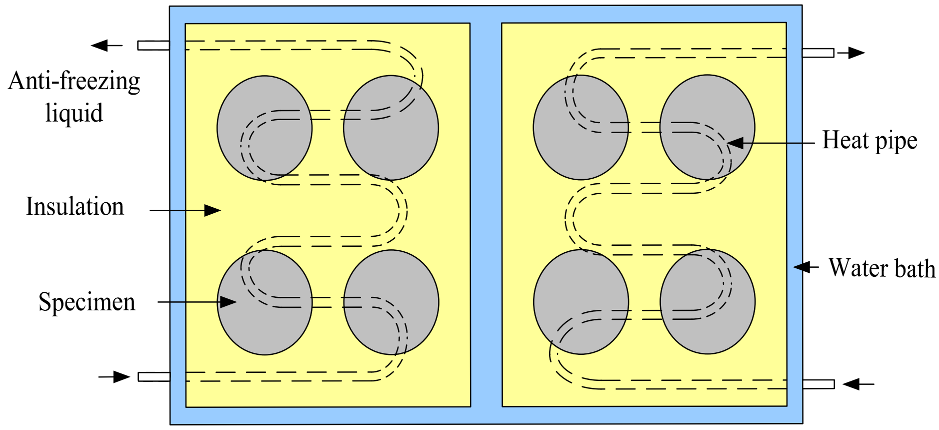


Figure 3.18 Schematic diagram of frost heave setup

(a) Front view of the frost heave cell **(b) Inside of the frost heave cell**

Figure 3.19 Frost heave test setup

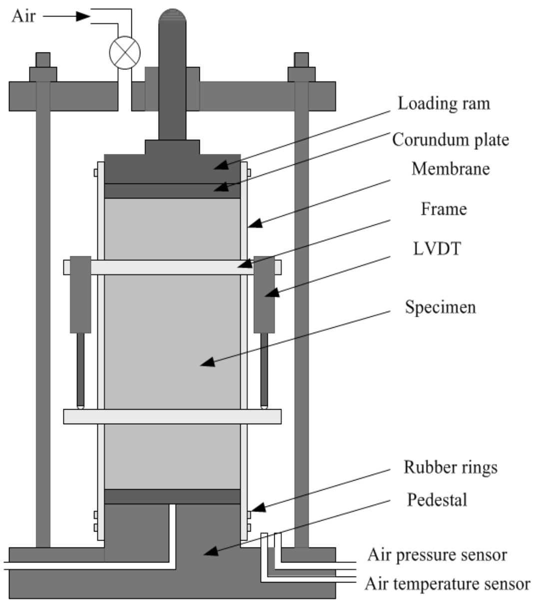
When temperatures and frost heave values of aggregate specimens stabilized, the frost heave process was considered to be complete. Usually it took 2 days for this process. Figure 3.20 shows two aggregate specimens after frost heave testing (Aluminum plates were removed).

Figure 3.20 Specimens after frost heave test

Due to the temperature of water bath (1 ^{o}C), after frost heave test, the bottom of specimens in the plastic molds were still unfrozen and specimen could break easily. Therefore, aggregate specimens were then moved up to the air chamber in the frost heave machine. After a certain time, when totally frozen, specimens were taken out and then the molds were removed.

M_R Test

Resilient properties of granular materials are significantly affected by freeze-thaw cycles during seasonal changes. However, there were limited data regarding M_R at subfreezing temperatures. Much less data were available for variations of M_R after a freeze-thaw cycle for frozen aggregate specimens generated in an open system. In order to determine seasonal variations of resilient properties, this experimental study was conducted to investigate resilient properties of D-1 materials at different temperatures, moisture and fines content conditions. Resilient properties were evaluated by repeated loading triaxial test. Figure 3.21 (a) schematically shows the setup for the M_R tests used in this study and Figure 3.21 (b) shows a photo of the test setup.



(a) Schematic plot

(b) Test setup

Figure 3.21 Repeated loading triaxial test setup

In order to determine resilient moduli at different subfreezing temperatures, aggregate specimens after frost heave test were conditioned in the environmental chamber to the lowest testing temperature of -10 ^{o}C. Usually, 8 hours were spent on this conditioning process to ensure sufficient time for aggregate specimens to reach thermal equilibrium everywhere. After the temperature of aggregate specimens stabilized, specimens were subjected to repeated loading sequences according to AASHTO T307 (2002).

A repeated dynamic haversine loading waveform with a loading duration of 0.1 sec and a rest period of 0.9 sec was used in M_R test to simulate the passing of one axle over a pavement followed by a period of rest before the next axle in the field (Figure 3.22). The maximum load is 10 times as much as the constant contact stress and the difference between them is the deviator stress.

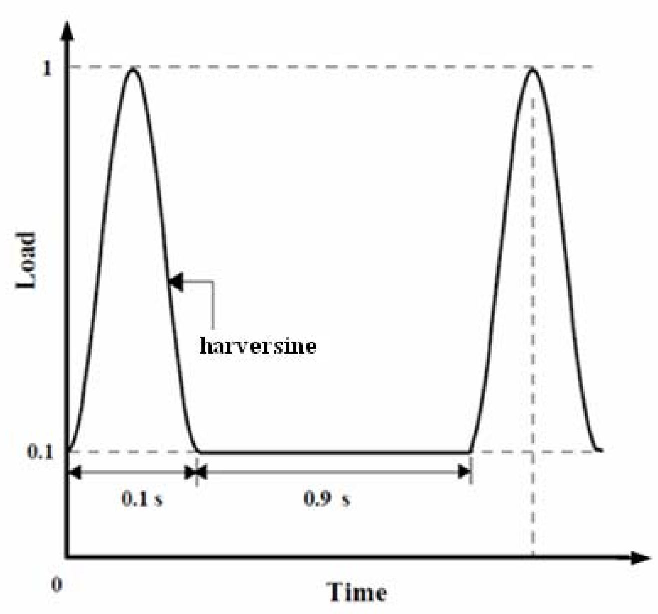


Figure 3.22 Applied load form in M_R tests

Preloading was utilized to reduce the disturbances due to specimen preparation procedure and minimize the effects of imperfect contact between end platens and specimen (conditioning sequence in Table 3.5). After conditioning, every loading sequence was repeated 100 times as long as the permanent strain was within a 5% limit (AASHTO 2002). The criterion was used for the entire loading history with confining pressure ranging from 3 psi to 20 psi and deviator stress ranging from 2.7 psi to 36 psi. Table 3.5 shows the loading sequences used in this study, where CP and DS represents confining pressure and deviator stress, respectively.

Table 3.5 Loading Sequences Used in M_R Tests

Sequence number	CP (psi)	DS (psi)	Bulk stress (psi)	Contact stress (psi)	Loading applications
Conditioning	15	13.5	58.5	1.5	500
1	3	2.7	11.7	0.3	100
2		5.4	14.4	0.6	100
3		8.1	17.1	0.9	100
4	5	4.5	19.5	0.5	100
5		9	24	1	100
6		13.5	28.5	1.5	100
7	10	9	39	1	100
8		18	48	2	100
9		27	57	3	100
10	15	9	54	1	100
11		13.5	58.5	1.5	100
12		27	72	3	100
13	20	13.5	73.5	1.5	100
14		18	78	2	100
15		36	96	4	100

In order to avoid possible leakage and disturbances of the specimen, air temperature was monitored by a thermometer located in the environmental chamber, as well as a thermocouple located inside the triaxial cell. The readings from these devices were then used to determine if the temperature had stabilized at the target temperature. Vertical deformation was monitored by using two LVDTs which were mounted on circumferential rings clamped on the specimen. The load was monitored with a miniature load cell located on the loading ram outside the triaxial cell.

When M_R tests at the temperature of -10 ^oC were completed, the temperature in the environmental chamber was changed to the next testing temperature, which was -7 ^oC. The aggregate specimens were kept in the chamber for another 8 hours to reach thermal equilibrium before performing M_R tests at the next testing temperature. The same procedures were followed for the M_R tests at the sequential temperatures of -5 ^oC, -4 ^oC, -3 ^oC, -2 ^oC, and -1^oC. When the M_R test at the temperature of -1^oC was completed, the same aggregate specimen was further used for M_R test at the room temperature of 20 ^oC. In order to determine the time needed for thawing, the aggregate specimen with the internal thermocouple was exposed to ambient room temperature. At the center of the aggregate specimen, change of temperature with increasing time was plotted as shown in Figure 3.23.

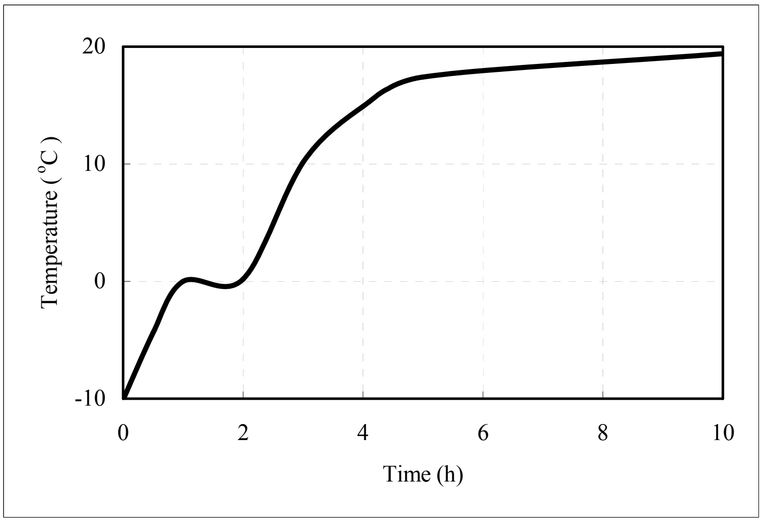


Figure 3.23 Temperature vs. time

Based on Figure 3.23, it was found that two hours are needed for completely thawing the aggregate specimen. For M_R tests after a freeze-thaw cycle, aggregate specimens were thawed inside the triaxial cell to eliminating external disturbances due to handling and no outflow of water was allowed during thawing.

CHAPTER IV

TEST RESULTS AND ANALYSIS

In this chapter, frost heave, M_R, and permanent deformation tests results on D-1 aggregate base course materials from three regions of AKDOT&PF are presented. The effects of factors such as temperature, stress state, and moisture and fines contents on the M_R of D-1 materials from three regions were analyzed. Regression models were then developed to correlate the results of M_R tests with these influencing factors.

Frost Heave Behavior

In order to determine the testing time needed for the freezing process, a pilot frost heave test was conducted first. Frost heave value was obtained by placing a LVDT at the top surface of an aggregate specimen. Therefore, the vertical displacement at the top surface was considered to be the frost heave. Figure 4.1 illustrates a typical curve to show the accumulation of frost heave with an increase of time.

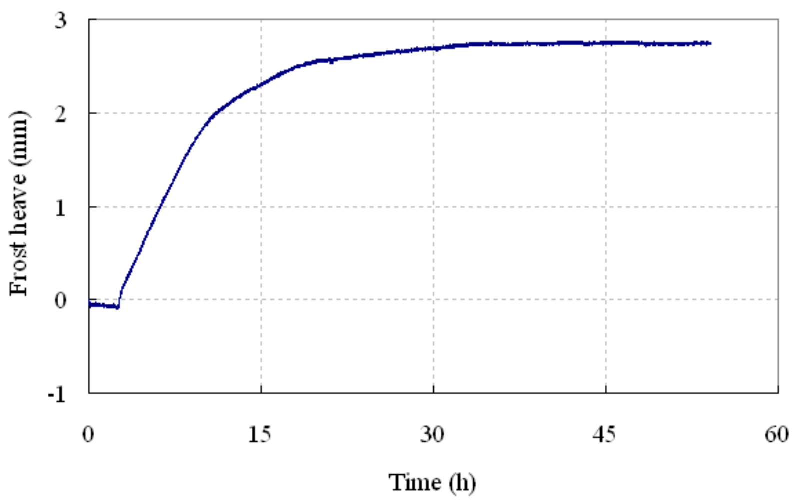


Figure 4.1 Frost heave vs. time

It can be seen from Figure 4.1, in the first 15 hours, frost heave increased quickly with time. However, after approximately another 15 hours, frost heave nearly reached a constant value. In this study, to ensure sufficient time for frost heave to reach a constant value, all D-1 materials from three regions were subjected to a two-day freezing process. The moisture and fines contents ranged from 3.3% to 6% and 3.15% to 10%, respectively. Temperature gradient applied was maintained at a constant of 0.25 °C/cm with -4 °C on the top and 1 °C at the bottom. Frost heave results of D-1 materials with different initial fines and moisture content levels after testing are summarized in Tables 4.1~ 4.3. When frost heave tests on D-1 material from Southeast Region were conducted, LVDTs had not been installed. Therefore, frost heave results for D-1 material from Southeast Region are not available in Table 4.1.

Table 4.1 Frost Heave Test Results on D-1 Material from Southeast Region

Initial condition		Change of MC	Final MC
MC	FC		
6.00%	10%	1.20%	7.20%
5.30%	10%	2.40%	7.70%
3.30%	10%	4.80%	8.10%
6.00%	8%	1.80%	7.80%
5.30%	8%	2.30%	7.60%
3.30%	8%	4.20%	7.50%
6.00%	6%	1.60%	7.60%
5.30%	6%	1.40%	6.70%
3.30%	6%	4.10%	7.40%
6.00%	3.15%	0.30%	6.30%
5.30%	3.15%	0.50%	5.80%
3.30%	3.15%	2.20%	5.50%

Table 4.2 Frost Heave Test Results on D-1 Material from Northern Region

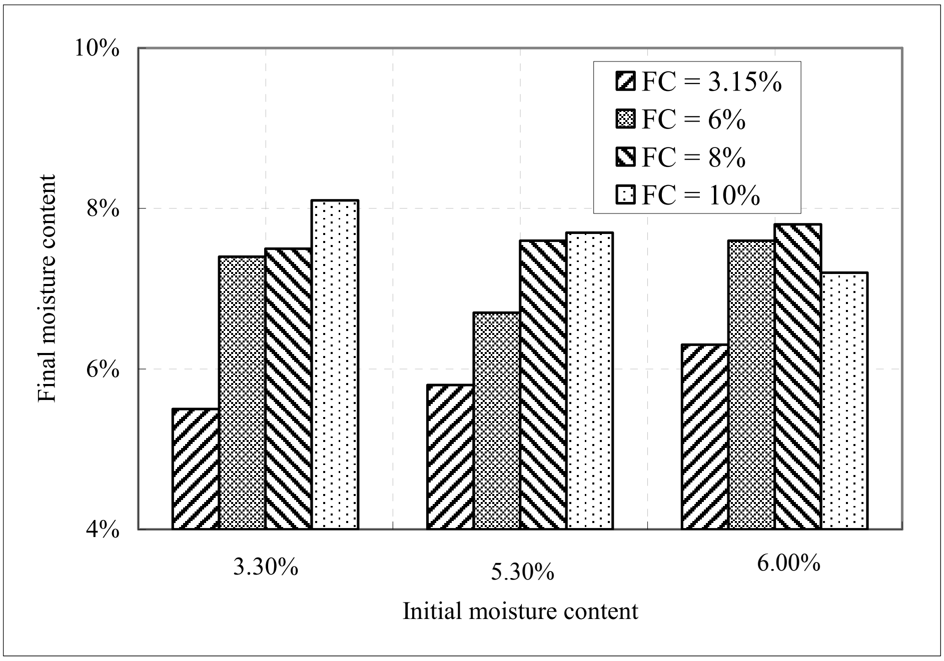
Initial condition MC	Initial condition FC	Change of MC	Final MC	Frost heave (mm)
6.00%	10%	1.53%	7.53%	2.1
5.30%	10%	1.82%	7.12%	3.3
3.30%	10%	4.41%	7.71%	5.1
6.00%	8%	1.45%	7.45%	-
5.30%	8%	1.74%	7.04%	2.55
3.30%	8%	4.63%	7.93%	3.42
6.00%	6%	1.15%	7.15%	1.69
5.30%	6%	1.69%	6.99%	1.5
3.30%	6%	4.26%	7.56%	2.68
6.00%	3.15%	1.13%	7.13%	1.24
5.30%	3.15%	1.33%	6.63%	0.36
3.30%	3.15%	3.66%	6.96%	0.34

Table 4.3 Frost Heave Test Results on D-1 Material from Central Region

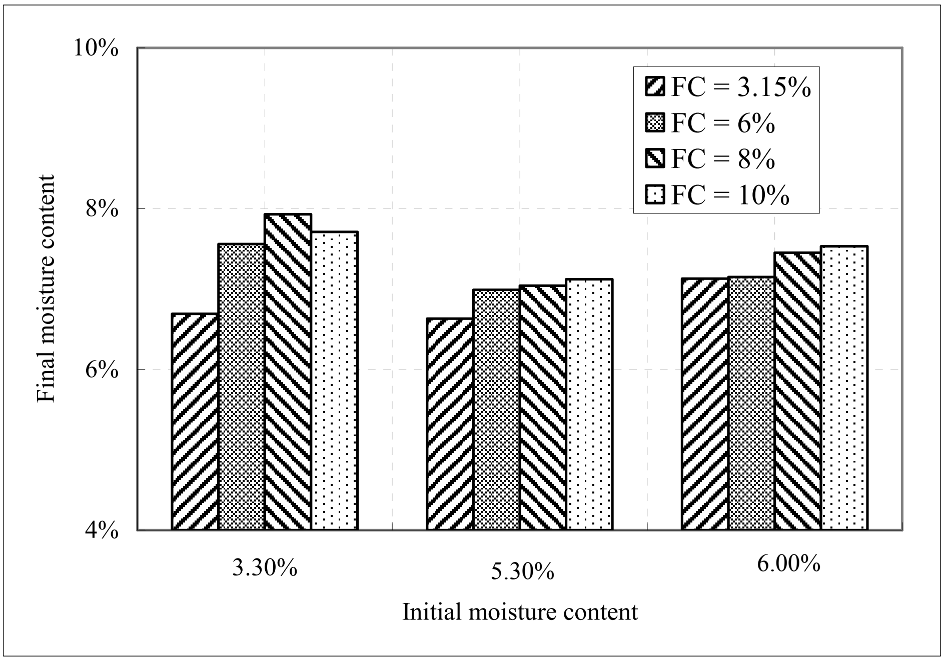
Initial condition MC	Initial condition FC	Change of MC	Final MC	Frost heave (mm)
6.00%	10%	0.38%	6.38%	2.84
5.30%	10%	1.12%	6.42%	4.83
3.30%	10%	4.72%	8.02%	5.24
6.00%	8%	1.46%	7.46%	1.00
5.30%	8%	1.91%	7.21%	2.08
3.30%	8%	4.79%	8.09%	2.16
6.00%	6%	0.53%	6.53%	-
5.30%	6%	1.81%	7.11%	2.5
3.30%	6%	4.28%	7.58%	1.19
6.00%	3.15%	1.97%	7.97%	4.24
5.30%	3.15%	2.12%	7.42%	2.52
3.30%	3.15%	4.69%	7.99%	0.6

Figure 4.2 illustrates the relationships between final and initial moisture contents at different fines contents on D-1 materials from three regions. Final moisture contents for D-1 materials after the freeze-thaw cycle were significantly affected by initial moisture and fines contents. According to Figure 4.2, aggregate specimens with low initial moisture and high fines content led to higher final moisture contents,

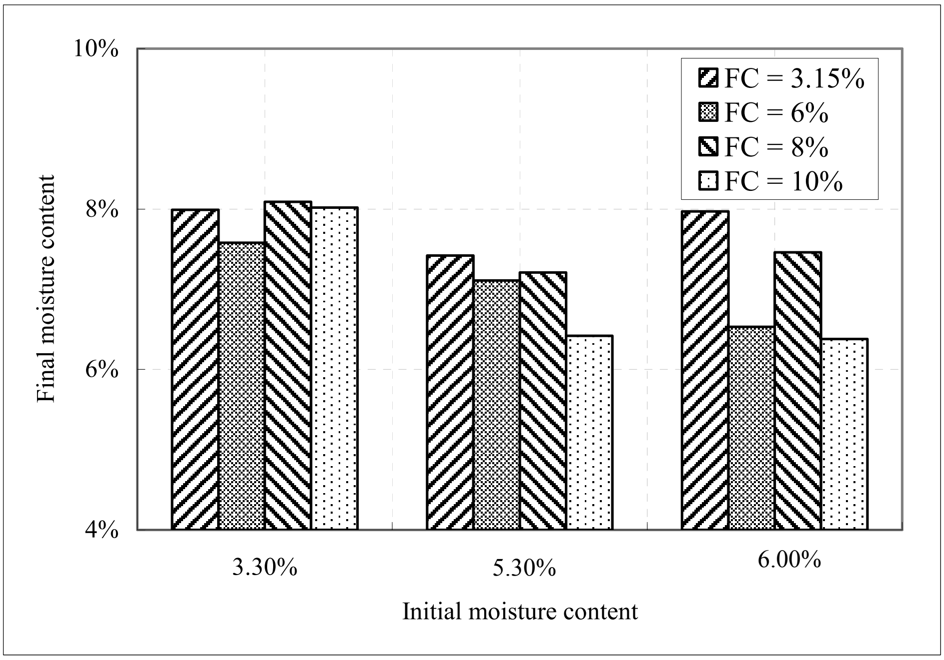
especially for D-1 materials from Southeast and Northern Regions. Final moisture content was highly related to degree of compaction of the aggregate which was somewhat controlled by the initial moisture content. At OMC, aggregate reached its maximum degree of compaction. Theoretically, aggregate also reached the minimum void ratio at OMC. This minimum void ratio provided limited room for water intake which could result in the lowest final moisture content.



(a) Southeast Region



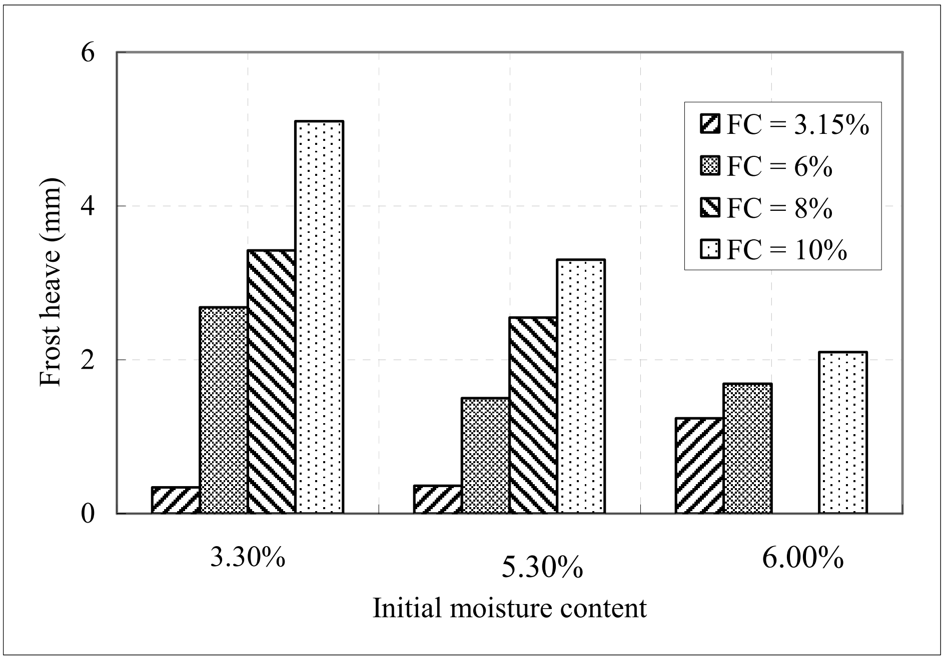
(b) Northern Region



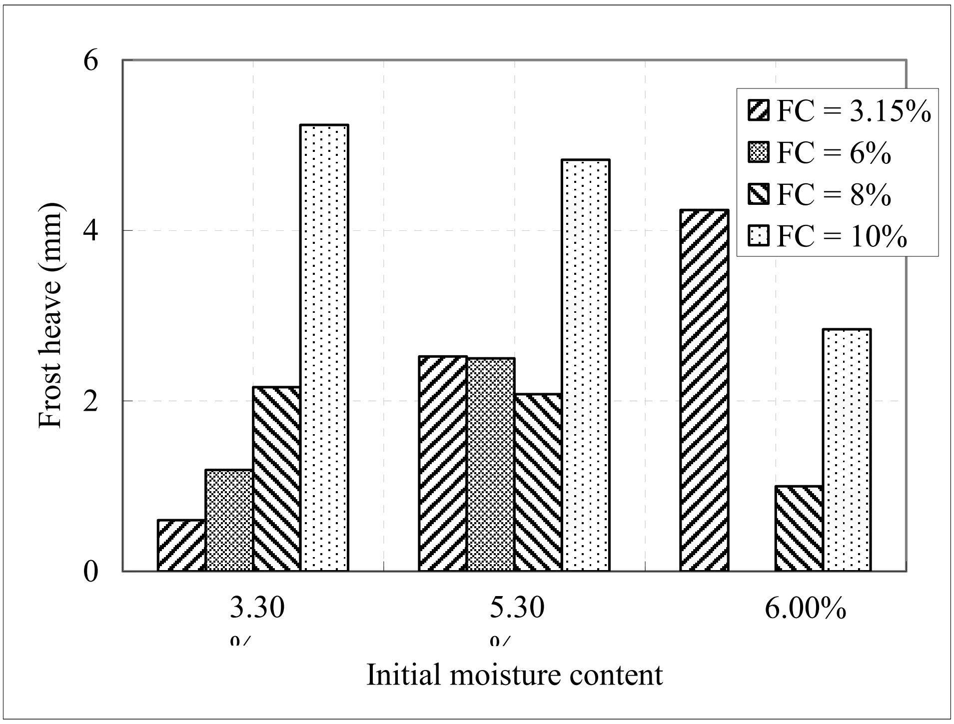
(c) Central Region

Figure 4.2 Final moisture contents after the freeze-thaw cycle at varying initial moisture contents

In addition, when fines contents were relatively high, aggregate specimens with lower initial moisture contents produced high frost heave after testing. Figure 4.3 illustrates the relationships between frost heave and initial moisture contents at different fines contents for D-1 materials from Northern and Central Regions. According to Figure 4.3, frost heave of D-1 materials increased with increases of fines content. Frost heave data could also be affected by initial moisture contents of D-1 materials. Low initial moisture and high fines contents produced highest frost heave value which correlated with final moisture content results very well. Table 3.1 shows that for D-1 materials from three regions with fines content of 10%, percentages of grains smaller than 0.02 mm of are greater than 3%. According to Casagrande's (1932) opinion, D-1 materials at fines content of 10% are considered to be frost susceptible. However, in Tables 4.2 and 4.3, most of the frost heave values were less than 2.5% increase of original specimen height which indicated insignificant frost heave. Also, based on visual observation after the freezing process, no visible ice lenses were found. In this study, only one temperature gradient (0.25 ^{o}C/cm) was used. However, temperature gradient could be an important factor which affects frost heave of frost susceptible soil (Konrad 1980). Significant frost heave may be expected under different temperature gradients.



(a) Northern Region



(b) Central Region

Figure 4.3 Frost heave after the freeze-thaw cycle at varying initial moisture contents

Resilient Behavior

M_R test results of D-1 materials from Southeast, Northern, and Central Regions are summarized in Tables 4.4~ 4.6, respectively. Some specimens collapsed during testing at high confining pressure and deviator stress levels. Hence, the corresponding testing results were not available in the tables.

Table 4.4 M_R Test Results on D-1 Material from Southeast Region

M_R(ksi)		10% FC			8% FC			6% FC			3.15% FC		
CP*	DS	6%	5.3%	3.3%	6%	5.3%	3.3%	6%	5.3%	3.3%	6%	5.3%	3.3%
3	2.7	6.2	11.5	46.7	5.9	12.7	23.6	5.0	7.8	22.4	5.4	12.2	24.8
	5.4	9.0	13.0	30.8	9.3	14.2	33.5	7.7	10.8	20.6	8.2	14.9	19.4
	8.1	11.8	14.5	29.5	12.3	16.2	28.8	10.3	12.0	21.2	11.0	16.7	21.7
5	4.5	8.0	11.3	45.7	7.9	19.0	39.8	7.3	8.8	25.3	7.5	12.1	28.3
	9.0	12.4	16.8	39.0	12.9	17.8	35.4	11.2	12.4	26.0	11.8	19.4	28.5
	13.5	15.9	18.3	39.5	16.9	19.9	36.4	14.8	14.5	28.1	15.5	20.9	29.1
10	9	13.9	19.9	60.0	15.5	22.7	53.5	15.4	20.0	38.9	15.5	27.3	41.1
	18	20.2	23.7	57.4	21.7	26.2	52.1	19.6	19.2	40.5	20.8	28.8	43.4
	27	25.0	26.5	56.6	27.1	27.0	51.8	23.6	21.0	42.7	24.8	31.0	44.6
15	9	16.8	22.8	84.0	17.8	22.4	73.8	19.4	11.5	50.1	17.9	34.9	53.0
	13.5	18.2	22.6	69.4	19.7	23.4	61.9	19.2	22.4	48.0	17.9	29.1	47.7
	27	26.3	28.3	68.3	28.4	22.1	60.8	25.2	27.9	51.4	25.6	36.2	52.0
20	13.5	20.7	23.7	85.1	21.9	30.5	78.4	23.5	18.1	58.0	21.5	35.8	61.3
	18	22.5	25.8	79.2	24.5	30.4	71.4	23.8	21.9	57.2	22.5	36.5	58.9
	36	26.3	28.0	68.4	28.2	31.3	61.9	26.0	28.6	58.7	25.9	39.3	52.9

*CP-confining pressure = $\sigma_2 = \sigma_3$,

DS-deviator stress = $\sigma_1 - \sigma_3$,

Table 4.5 M_R Test Results on D-1 Material from Northern Region

M_R(ksi)		10% FC			8% FC			6% FC			3.15% FC		
CP	DS	6%	5.3%	3.3%	6%	5.3%	3.3%	6%	5.3%	3.3%	6%	5.3%	3.3%
3	2.7	7.5	15.1	19.2	7.2	7.2	22.7	5.5	13.5	16.2	7.1	7.1	19.5
	5.4	9.3	14.0	18.8	8.9	9.6	20.8	8.1	13.5	17.2	8.8	7.4	19.6
	8.1	11.0	16.0	21.7	9.5	10.6	22.1	8.8	14.5	19.1	10.7	10.5	20.3
5	4.5	11.1	21.4	21.2	8.2	7.8	27.9	8.1	16.1	24.1	7.6	13.3	26.7
	9	13.0	18.7	24.5	10.8	10.3	28.2	9.8	16.4	25.3	10.9	13.4	26.8
	13.5	15.0	19.8	27.4	11.5	11.1	30.1	11.6	16.9	26.2	11.6	13.9	26.9
10	9	18.8	29.3	37.8	15.7	8.7	44.4	19.7	17.5	40.0	10.0	19.3	41.3
	18	21.4	28.8	34.2	17.3	12.0	44.9	21.3	19.1	39.4	12.5	22.1	40.4
	27	25.2	30.4	34.2	19.1	15.5	45.5	24.1	18.0	40.5	-	21.3	39.1
15	9	26.3	38.5	41.1	20.8	-	54.7	23.5	17.9	53.5	-	13.1	53.8
	13.5	26.9	33.6	-	21.1	-	51.3	23.3	16.9	45.2	-	18.8	45.6
	27	31.7	33.5	-	24.4	-	53.8	27.6	21.4	47.7	-	19.6	46.5
20	13.5	35.3	31.8	-	27.1	-	62.1	29.0	21.8	58.8	-	20.7	55.9
	18	36.1	28.8	-	28.0	-	61.2	29.3	22.1	56.2	-	22.7	51.9
	36	34.3	29.5	-	26.4	-	56.8	28.4	23.1	55.1	-	23.1	45.6

Table 4.6 M_R Test Results on D-1 Material from Central Region

M_R(ksi)		10% FC			8% FC			6% FC			3.15% FC		
CP	DS	6%	5.3%	3.3%	6%	5.3%	3.3%	6%	5.3%	3.3%	6%	5.3%	3.3%
3	2.7	4.8	10.1	19.8	5.7	11.9	19.4	3.9	14.3	24.3	8.6	14.2	14.5
	5.4	8.5	14.0	20.8	10.3	13.8	18.8	7.1	15.2	22.4	10.8	14.6	17.0
	8.1	11.3	16.2	23.5	12.2	16.5	22.3	9.9	17.5	24.2	12.1	16.3	19.5
5	4.5	6.6	11.8	25.8	8.0	15.3	21.2	5.5	17.6	26.3	9.6	18.8	24.2
	9	11.5	16.6	28.5	12.4	17.4	25.6	10.4	20.3	28.9	12.8	19.5	25.4
	13.5	14.1	20.1	31.3	14.7	21.8	28.9	13.7	20.1	30.6	15.5	19.9	27.9
10	9	9.2	18.9	42.6	10.0	-	38.7	9.9	24.3	41.4	16.6	25.4	36.1
	18	15.2	23.4	46.8	16.4	-	40.9	16.1	26.5	45.2	20.1	28.4	36.6
	27	19.2	26.6	49.6	20.1	-	45.9	20.0	25.9	46.4	22.4	27.0	37.7
15	9	7.3	20.6	56.3	8.7	-	52.4	12.1	22.6	57.1	14.1	24.8	44.8
	13.5	10.8	21.1	54.1	11.5	-	45.5	12.2	24.0	50.5	14.9	26.4	42.6
	27	19.3	26.4	59.9	20.1	-	50.1	20.0	29.2	55.3	21.8	31.4	44.7
20	13.5	10.2	21.8	67.0	10.6	-	55.4	13.0	30.5	70.3	13.4	33.5	54.2
	18	14.5	21.8	67.1	14.5	-	53.7	14.5	31.1	65.1	16.2	34.0	52.4
	36	21.8	25.7	63.3	20.0	-	53.3	20.0	30.3	56.2	23.8	32.1	46.4

Influencing Factors

Moisture content is an important factor which can affect resilient properties of granular materials significantly (Vuong 1992, Haynes and Yoder 1963, Hicks and Monismith 1971). The results obtained from this study reflected this as well (Tables 4.4 ~ 4.6). Figures 4.3 and 4.4 illustrates typical results of effect of moisture content on resilient properties of D-1 materials with different fines content levels using materials from Southeast Region as examples. Effect of moisture content on resilient properties of D-1 materials from Central and Northern Regions are shown in Appendix D.

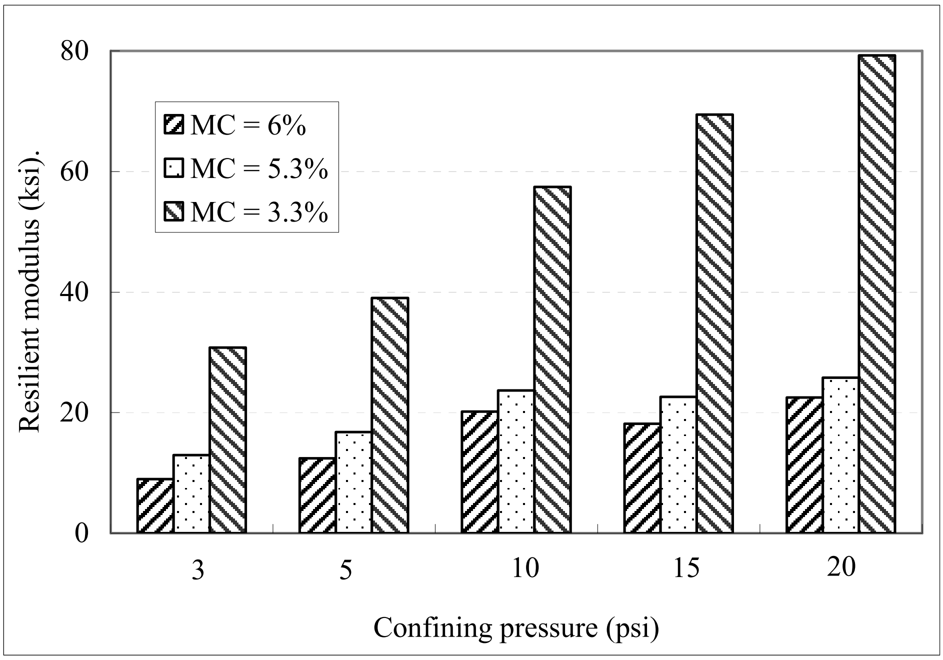


Figure 4.4 M_R at varying moisture contents (FC=10%, Southeast Region)

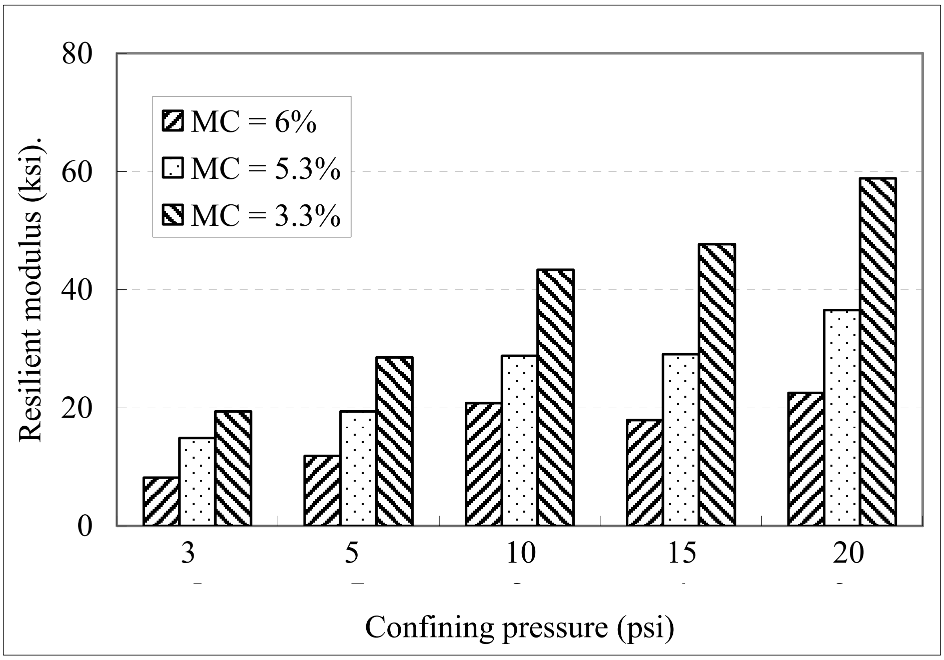


Figure 4.5 M_R at varying moisture contents (FC=3.15%, Southeast Region)

It can be seen from Figures 4.4 and 4.5, under different confining pressure levels, M_R decreased with increase of moisture content. The presence of water had lubricating effect on particles. Hence, when moisture content decreased from 5.3% to 3.3%, M_R increased 3 times which was consistent with the conclusions drawn by others (Heydinger et al. 1996). Raad et al. (1992) showed that the effect of moisture on the resilient behavior of unbound aggregates is perhaps most significant in well-graded materials with a high proportion of fines. The loading is really transitory and positive pore-water pressure developed by rapidly applied load, which reduced the effective stress and resulted in low stiffness and deformation resistance of aggregate. The reason for this is water can be more readily held in the pores between aggregate particles and undrained condition kept water from flowing out. In this study, three moisture content levels used were 3.3%, 5.3%, and 6%. At maximum moisture content of 6%, aggregate was not fully saturated. However, significant change of M_R still existed with an increase of moisture content from 3.3% to 6%. With increase of moisture content, under undrained condition, excess pore-water pressure will develop due to repeated load. As pore-water pressure develops, effective stress decreases with a subsequent decrease of aggregate stiffness. It can be argued that it is not the degree of saturation which affected the material behavior but rather that the pore water-pressure response controlled deformational behavior of aggregate.

Under the same deviator stress level, Figures 4.6 and 4.7 illustrates typical effects of fines content on resilient properties of D-1 materials from Southeast Region with different moisture contents at both lower and higher moisture levels as examples. Effect of fines content on resilient properties of D-1 materials from Central and Northern Regions are shown in Appendix E.

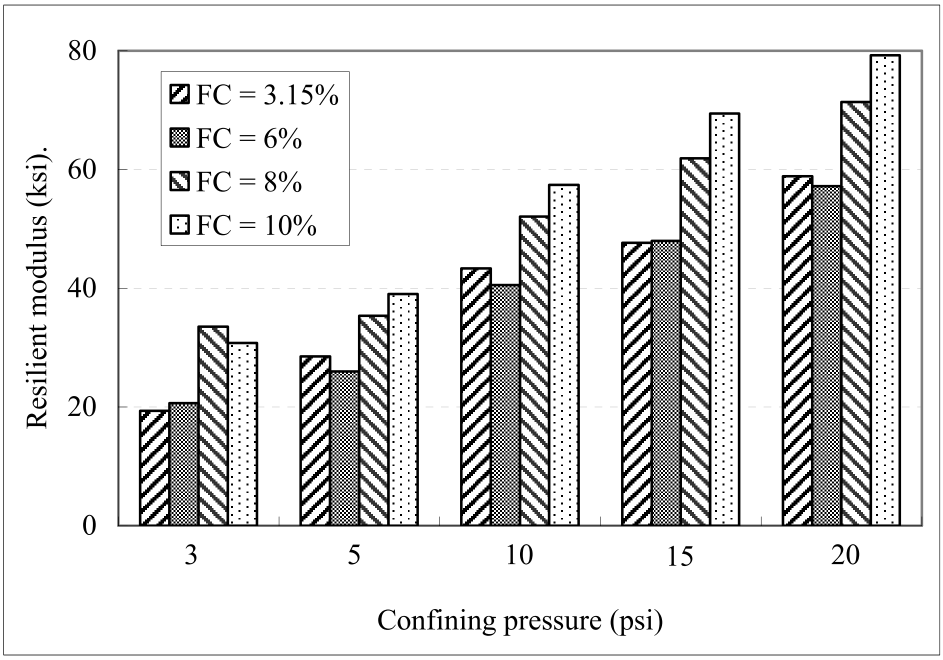


Figure 4.6 M_R at varying fines content (MC=3.3%, Southeast Region)

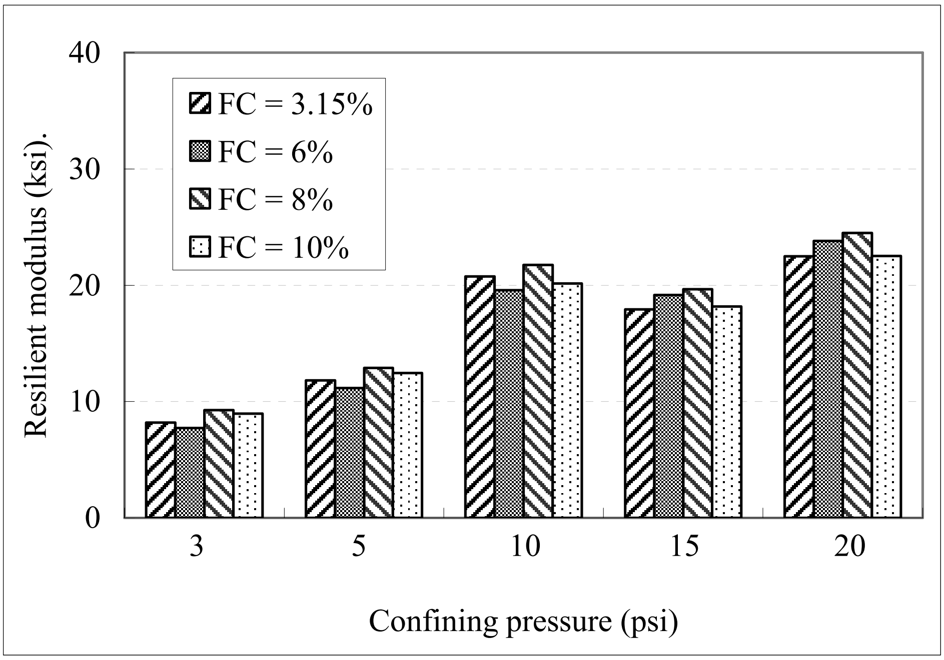


Figure 4.7 M_R at varying fines content (MC=6%, Southeast Region)

Impact of fines content on M_R of D-1 materials varied with the moisture content (Figures 4.6 and 4.7). At lower moisture content level (3.3% in Figure 4.6), most of D-1 materials from Southeast Region with high fines (8 or 10%) contents produced higher M_R values when compared to those with low fines content (3.15 or 6%). As shown in Figure 4.6, highest M_R could be found from aggregate specimen with high fines content. However, as shown in Figure 4.7, when moisture content was higher (6%), impact of fines content was not significant. Impact of fines content for D-1 materials from Southeast Region is not consistent with those for D-1 materials from Central and Northern Region. Therefore, impact of fines content on resilient properties of D-1 materials could also be affected by material source. The variation of fines content ranging from 2% to 10% was reported by Hicks (1970) to have a minor influence on M_R of aggregate which was consistent with this study results for D-1 materials at high moisture content level. Possible reason for this is fines were not enough to fill up the voids between gravels and sands, and particle interlock was not significantly influenced by the presence of fines. When the load was transmitted via coarser particles, smaller number of particle contacts resulted in less total deformation and consequently higher stiffness. Hence, there is no drastic change of soil structure when fines content is relatively low. With increase of fines content, interlocks between particles were partially eliminated and M_R reduced subsequently. However, with continuous increase of fines, large particles became suspended in fines and small aggregate particles, and interlocks between particles were reduced. Excess fines replaced the coarse particles so that the mechanical performance of aggregate depended only on the fines. Thus, aggregate stiffness decreased and aggregates behaved like pure clay or silt.

Figures 4.8 ~ 4.11 illustrates typical results of effect aggregate source on resilient properties using D-1 materials from three regions with moisture content of 5.3% as examples. Effect of aggregate source on resilient properties of D-1 materials at fines contents of 6% and 3.3% are shown in Appendix F.

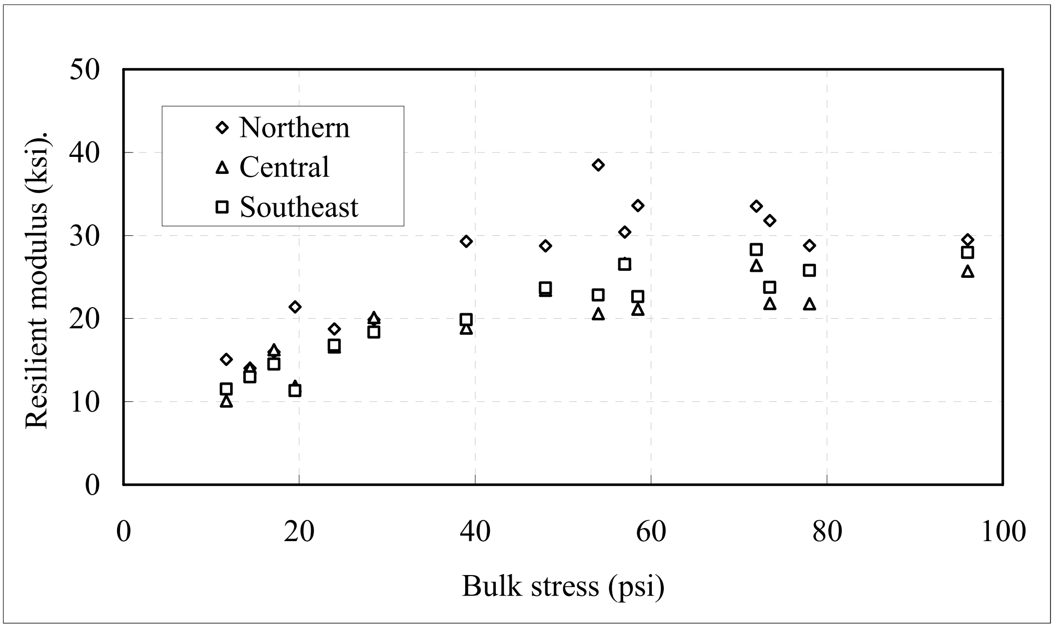


Figure 4.8 Comparison of M_R of D-1 materials (FC=10%, MC=5.3%)

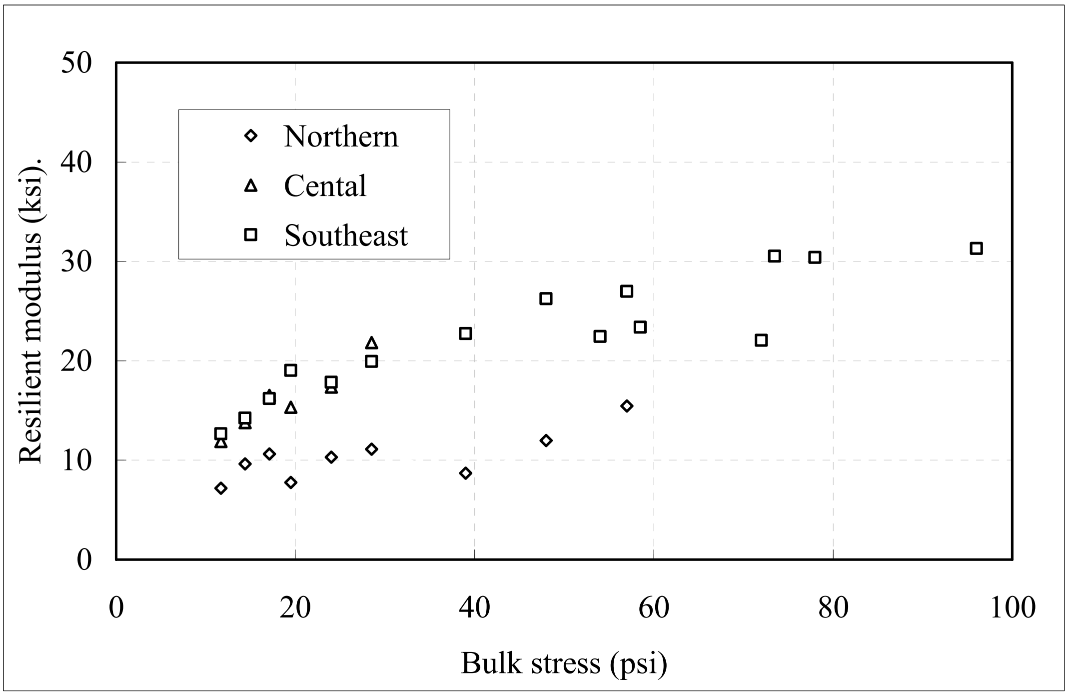


Figure 4.9 Comparison of M_R of D-1 materials (FC=8%, MC=5.3%)

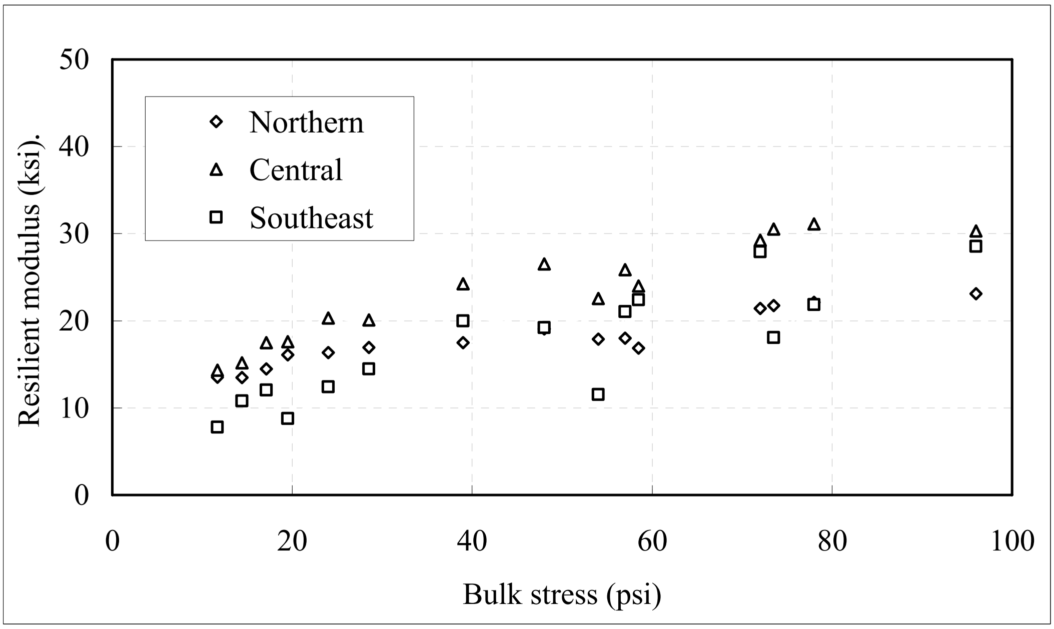


Figure 4.10 Comparison of M_R of D-1 materials (FC=6%, MC=5.3%)

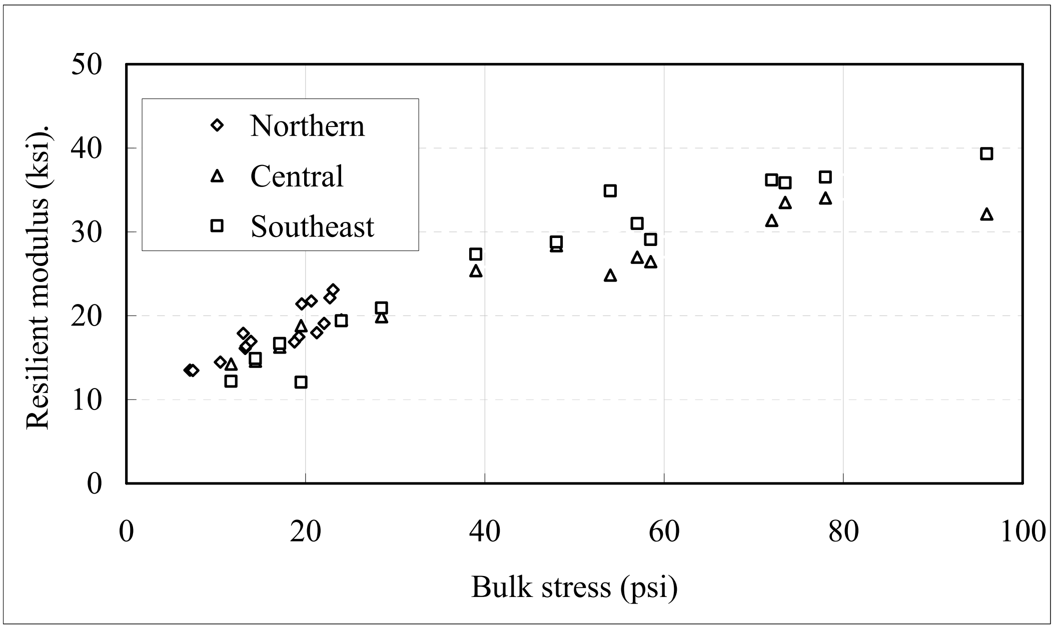


Figure 4.11 Comparison of M_R between D-1 materials (FC=3.15%, MC=5.3%)

Figures 4.8 ~ 4.11 show that most of the resilient moduli of D-1 materials from three regions varied from 10 to 40 ksi with bulk stress ranging from 11.7 to 96 psi. However, no clear differences could be found between resilient moduli of D-1 materials from three regions at different fines content levels. According to previous studies (Hicks 1970, Hicks and Monismith 1971, Allen 1973, Allen and Thompson 1974, Thom 1988, Barksdale and Itani 1989, Thom and Brown 1989), crushed aggregates, with angular to subangular shaped particles, provided higher M_R than that of uncrushed gravels with subrounded or rounded particles. Rougher particle surface also contributed to higher M_R. D-1 material from Southeast was composed of crushed aggregate with 3% percent of flat or elongated particles which is, in some degree, different from D-1 materials from Northern and Central Regions of Alaska which are mostly composed of subrounded or rounded gravels with no flat or elongated particles. Further investigation on the effect of shape and texture of Alaskan aggregates on resilient behavior should be considered.

As described in Chapter II, stress state is another important factor which could also affect resilient properties of coarse grained materials. Figures 4.12 ~ 4.15 use D-1 material from Southeast Region as an example to illustrate the effect of stress state on resilient properties.

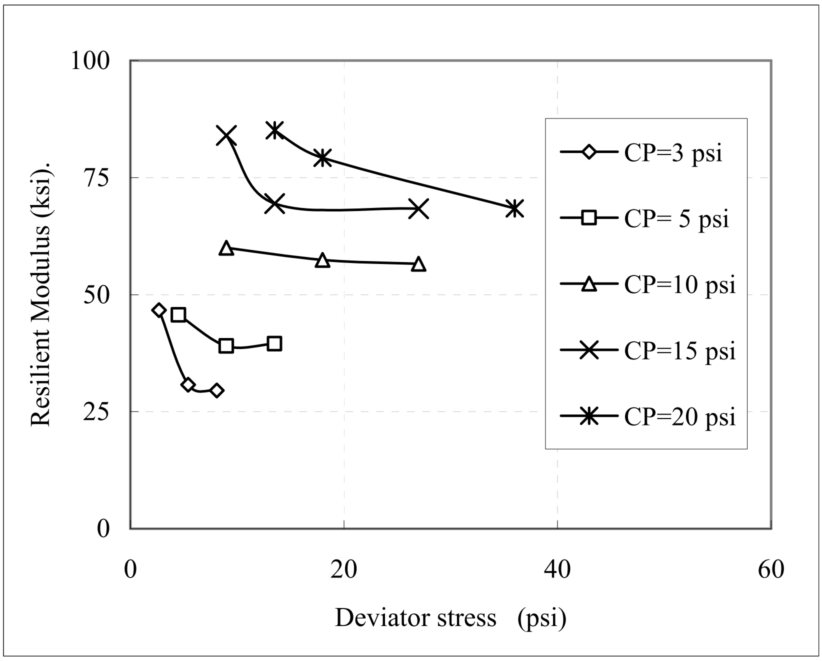


Figure 4.12 Variation of M_R at 20 °C (FC=10%, MC=3.3%)

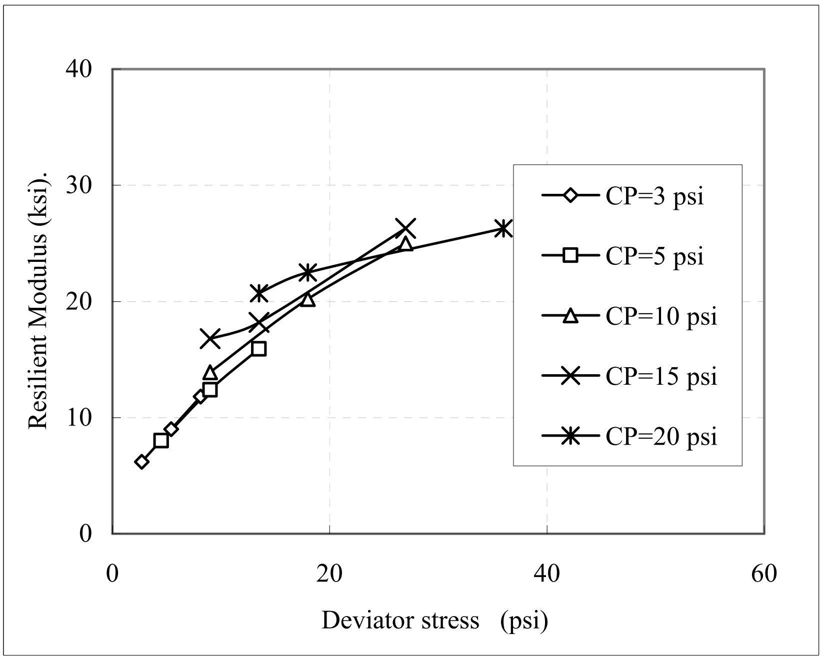


Figure 4.13 Variation of M_R at 20 °C (FC=10%, MC=5.3%)

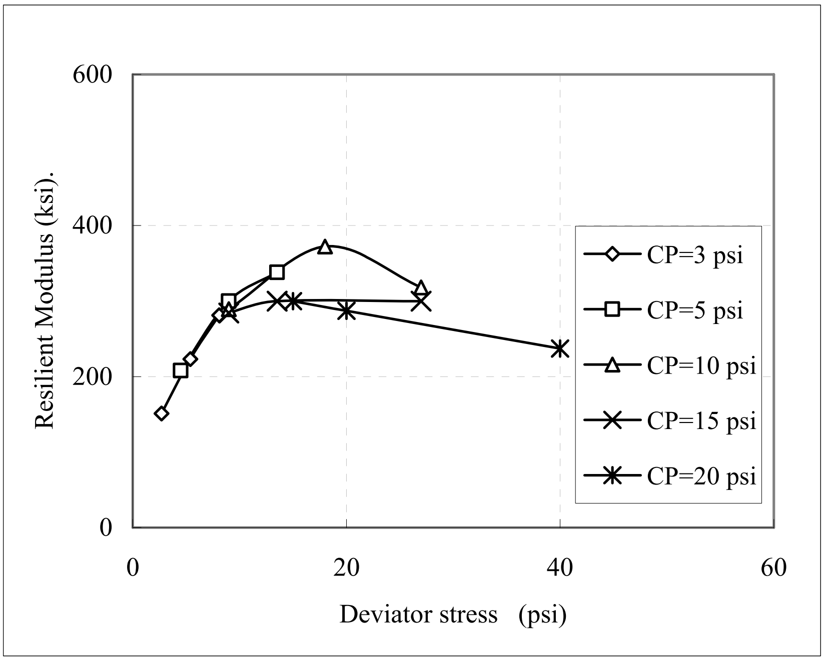


Figure 4.14 Variation of M_R at -1 ^oC (FC=10%, MC=3.3%)

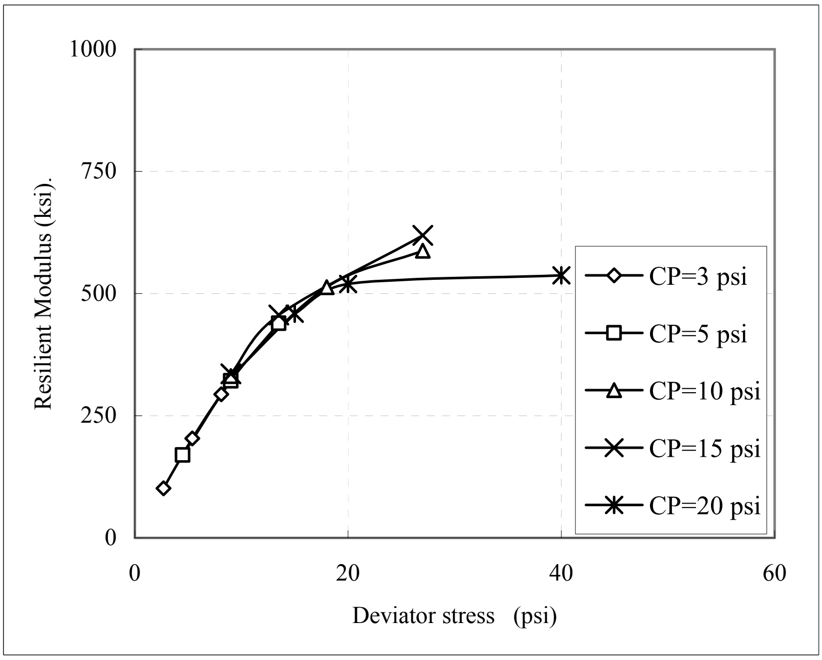


Figure 4.15 Variation of M_R at -4 ^oC (FC=10%, MC=3.3%)

It can be seen from Figures 4.12 ~ 4.15, effect of stress state on resilient properties of D-1 materials with temperature ranging from -10 to 20 °C was different. The effects of confining pressure and deviator stress varied with change of moisture content and temperature conditions. At temperature of 20 °C (Figure 4.12), resilient moduli increased with increase of confining pressure when moisture content was relatively low. This was consistent with the conclusions from the previous study (Lashine et al. 1971). At higher moisture level (Figure 4.13), impact of confining pressure was not as significant as that at low moisture content. Effects of deviator stress varied which was highly dependent on moisture content conditions. When moisture content was relatively low, M_R decreased slightly as the applied deviator stress increased (Figure 4.12). However, when moisture content was raised up to the OMC or higher, M_R increased with increase of deviator stress (Figure 4.13).

At subfreezing temperatures, effects of confining pressure and deviator stress were found to be different when compared to those at temperature of 20 °C. From Figure 4.14, at -1 °C, there is an increase of M_R due to the repeated load. However, with the increase of deviator stress and repeated loading cycles, resilient moduli decreased (Figure 4.14). This may be due to the unfrozen water in frozen aggregate. It was stated that as the temperature increases, unfrozen water content increases (Civan 2000). In addition, repeated load can also raise the unfrozen water content up which could result in decrease of the aggregate stiffness. When applied load was not high enough to generate significant strain on frozen specimen, with continuous decrease of temperature and increase of deviator stress and confining pressure, M_R increased linearly without significant change of strain as shown in Figure 4.15.

In order to determine effect of temperature on resilient properties of D-1 materials, specimens were tested under different temperatures which were -10, -7, -5, -4, -3, -2, -1, and 20 °C in one full freeze-thaw cycle. In this study, aggregate specimens after frost heave test were firstly frozen to -10 °C. Then, M_R tests were performed on these specimens from the lowest testing temperature (-10 °C) to the warmest testing temperature (20 °C) to simulate the spring thawing process in the field during spring thaw. Figures 4.16 ~ 4.18 uses D-1 materials under confining pressure of 20 psi and deviator stress of 36 psi as an example to show the effect of temperature on M_R.

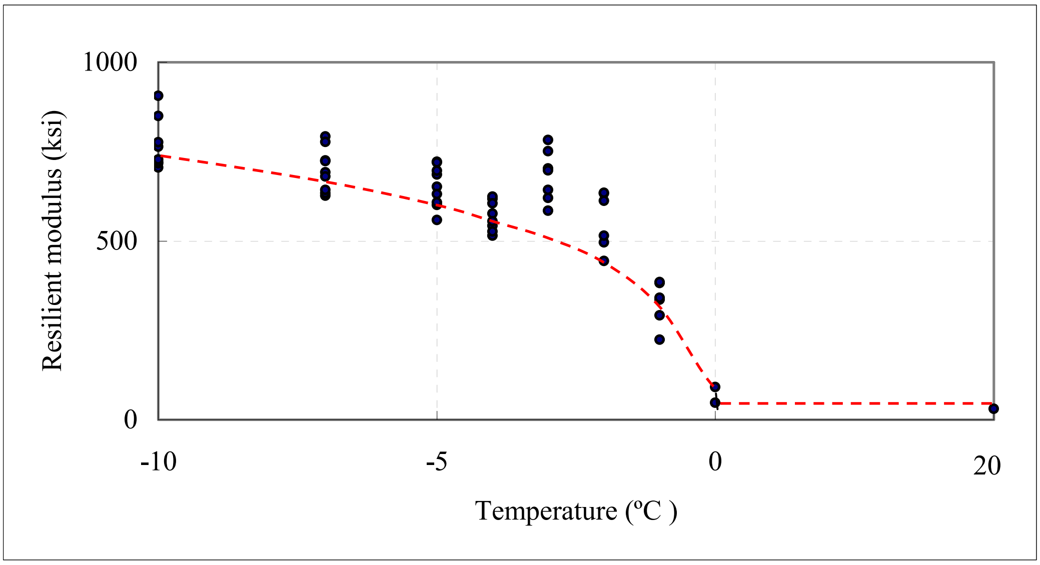


Figure 4.16 M_R vs. temperature (Southeast Region)

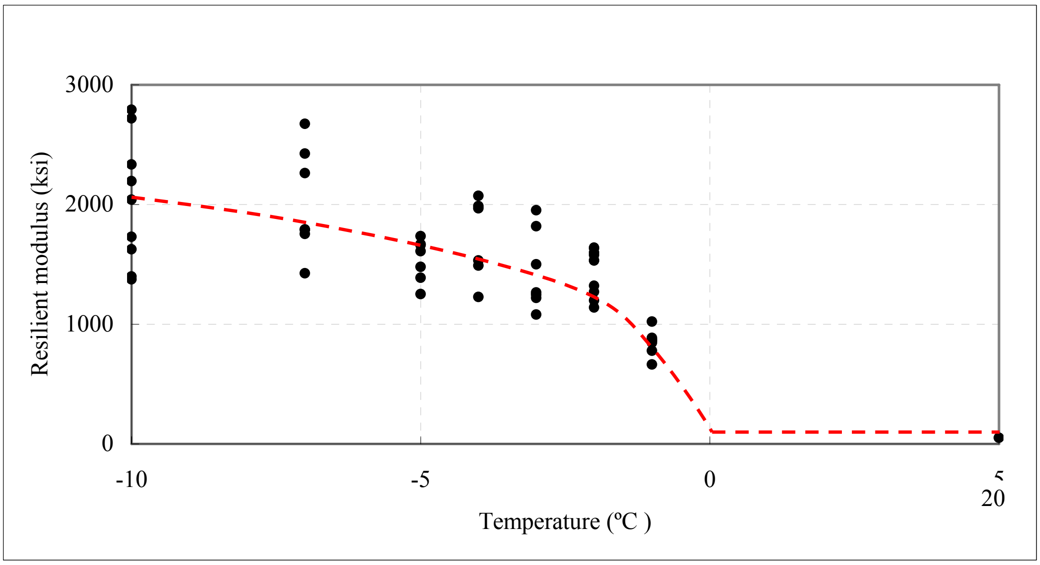


Figure 4.17 M_R vs. temperature (Northern Region)

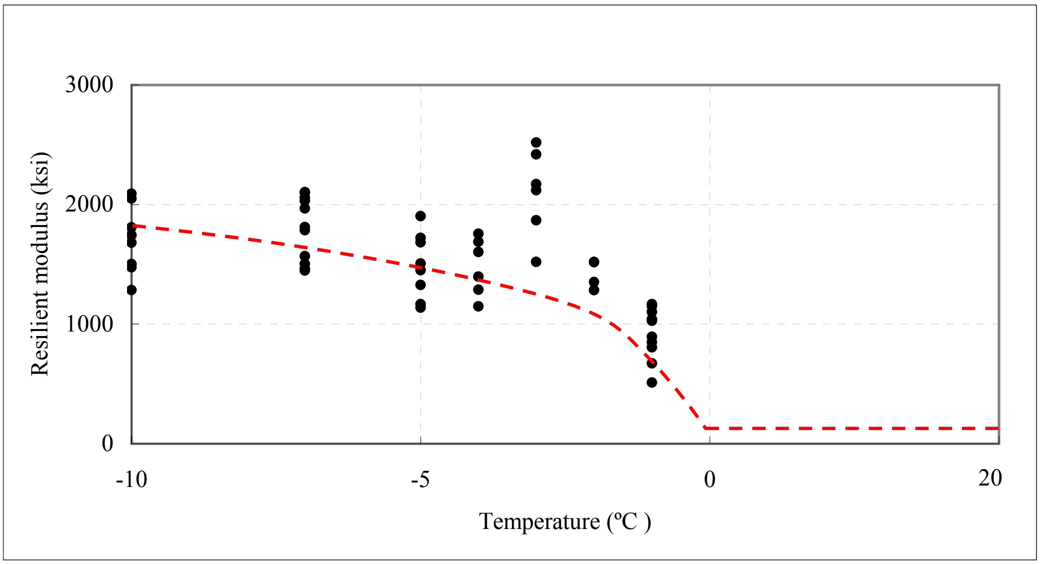


Figure 4.18 M_R vs. temperature (Central Region)

All aggregate specimens showed a high dependence on temperatures. At subfreezing temperatures, M_R decreased with an increase of temperature. Variation of aggregate resilient moduli due to different initial fines and moisture contents were inconspicuous at subfreezing temperatures. In another word, effects of initial fines and moisture contents were somewhat weakened or even eliminated when aggregate specimens were totally frozen. Generally, stiffness of frozen granular materials is basically controlled by three components: (1) ice stiffness; (2) aggregate stiffness; and (3) aggregate skeleton matrix interaction. Unfrozen moisture content is generally expected to be very low and stiffness of aggregate was basically dependent on ice and aggregate skeleton stiffness. Results for D-1 materials from Central and Northern Regions are pretty close which were nearly 3 times greater than those from the Southeast Region. Under frozen conditions, temperature was an important factor on resilient properties of aggregate. When the temperature increased from -10 to -5 °C, M_R of aggregate samples decreased at a considerably low rate. Based on Figures 4.16 ~ 4.18, aggregate M_R was considered to remain constant at this temperature range. Therefore, ice stiffness, which was significantly affected by temperature, was responsible for the change of M_R and significant difference of M_R between D-1 materials from three regions. When temperature increased from -5 to 0 °C, the unfrozen moisture content was considerably higher. In consequence, influence of unfrozen moisture content to the overall stiffness became more important. With continuous increasing of temperature, aggregate stiffness remained stable and aggregate skeleton matrix interaction was enhanced due to the repeated load. Generally, ice stiffness decreased due to increase of the subfreezing temperature which dominated the aggregate M_R. Consequently, resilient moduli of the frozen aggregate specimens significantly decreased due to the increased temperature. There were large drops of M_R ranging from -3 °C to 0 °C (Figures 4.16 ~ 4.18) on D-1 materials from all three regions. However, based on testing results, one can find that an unexpected increase of M_R at around -3 °C. Two possible reasons are considered to be responsible for this phenomenal: (1) due to repeated loading at each temperature, creep came up which densified aggregate specimen (Figure 4.19); (2) some of released water refreezed during conditioning. Thus, bonding effect was strengthened by increase of contact area after conditioning and repeated load. Also, released water due to the repeated load resulted in high moisture content. Hence, aggregate strength decreased at much higher rate. M_R values for granular materials were nearly unchanged at nonfreezing temperatures (Simonsen et al. 2002). That was also the reason why only one temperature (20 °C) was selected for M_R test at nonfreezing temperatures.

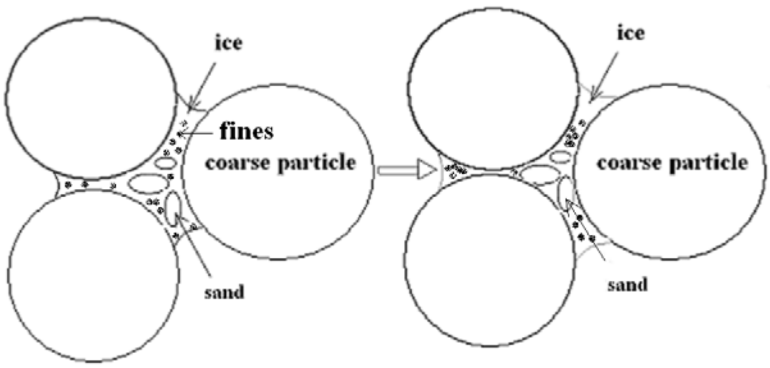


Figure 4.19 Aggregate structures before and after repeated loading

During seasonal changes, resilient properties of aggregate are significantly affected by the freeze-thaw action. It was suggested that the volume of dense aggregate might increase due to the freeze-thaw cycle, which would lead to slightly looser aggregate structure than before freezing. Thus, a freezing-thaw cycle may result in significant reduction of the M_R. Freeze-thaw cycle could result in a substantial reduction in M_R even without change in moisture condition (Fredlund et al. 1975).

For D-1 materials in Alaska, spring thaw weakening is one of the most common distresses for roadways. When warm weather induced thawing from the upper part of pavement structure, the frozen aggregate beneath the thawing front can obstruct vertical drainage and develop supersaturated conditions. Aggregate strength is dramatically reduced owing to the water concentrated above the thawing front. Also, vulnerability of the pavement structure increases because of the traffic loads.

To determine the effect of freeze-thaw cycle on resilient properties of D-1 materials form the three regions, M_R tests at room temperature for aggregate specimens experienced a freeze-thaw cycle were performed. For M_R tests at room temperature on D-1 materials from Southeast and Northern Regions after the freeze-thaw cycle, most of specimens collapsed during preloading process. M_R test results on D-1 material from Central Region after a freeze-thaw cycle are summarized in Table 4.7. It can be seen from Table 4.7, M_R data were significantly reduced when compared with those in Table 4.6. Figures 4.20 and 4.21 compare M_R results on D-1 material from Central Region tested at room temperature before and after a freeze-thaw cycle with fines and moisture contents varying from 3.15% to 10% and 3.3% to 6%.

Table 4.7 M_R Test Results after a Freeze-Thaw Cycle (Central Region)

Confining Pressure (psi)		3.0			5.0		
Deviator Stress (psi)		2.7	5.4	8.1	4.5	9.0	13.5
Fines	Initial Moisture	M_R (ksi)					
10%	6.0%	-	-	-	-	-	-
	5.3%	-	-	-	-	-	-
	3.3%	-	-	-	-	-	-
8%	6.0%	5.3	8.9	9.5	-	-	-
	5.3%	3.7	7.0	8.3	4.9	8.2	11.6
	3.3%	-	-	-	-	-	-
6%	6.0%	4.0	6.9	8.0	4.8	8.2	-
	5.3%	4.1	8.4	10.3	5.6	8.8	-
	3.3%	5.1	5.2	-	-	-	-
3.15%	6.0%	-	-	-	-	-	-
	5.3%	3.7	6.6	6.0	4.4	7.0	-
	3.3%	8.8	10.9	12.8	11.3	13.5	15.5

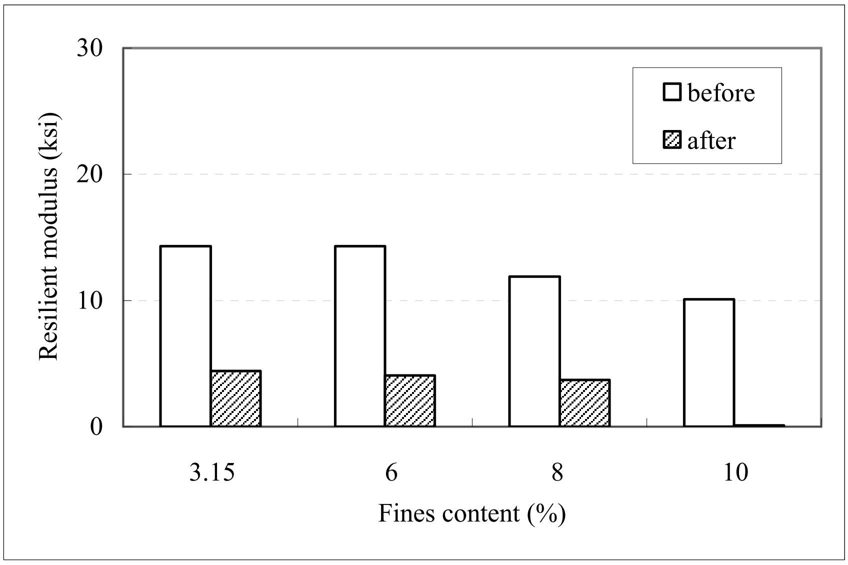


Figure 4.20 M_R results before and after the freeze-thaw cycle (Central, MC= 5.3%, CP = 3.0 psi, and DS = 2.7 psi)

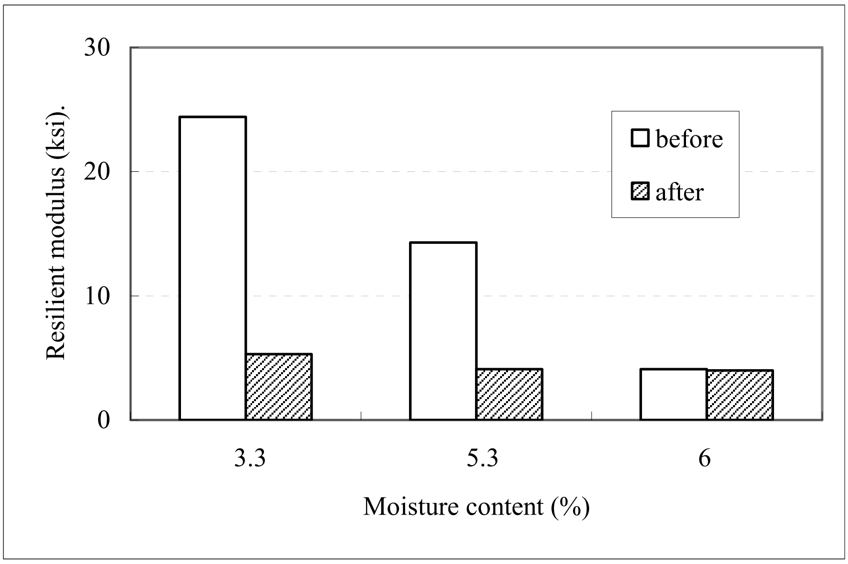
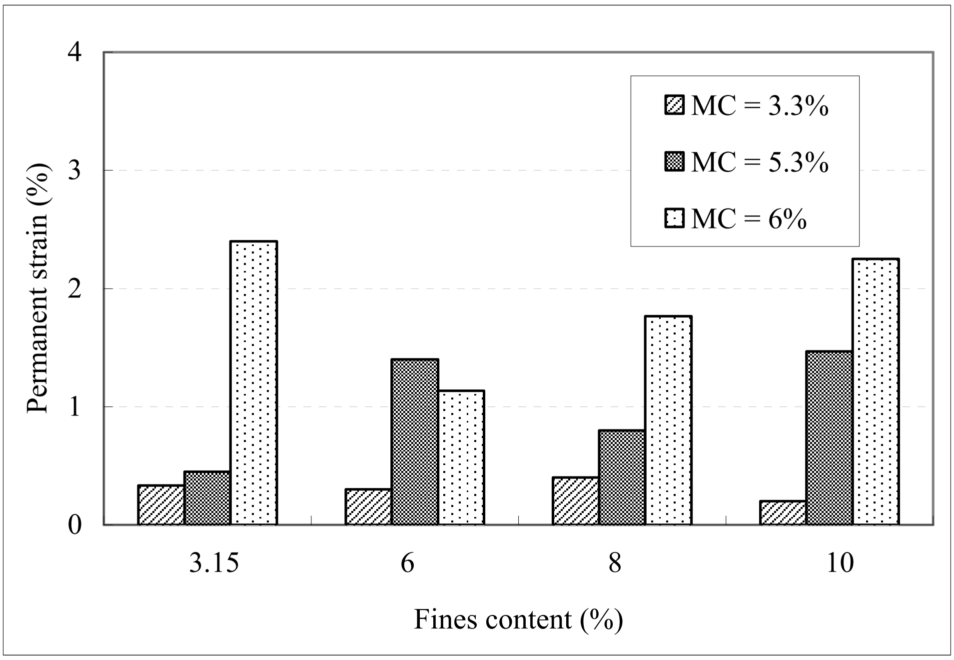


Figure 4.21 M_R results before and after the freeze-thaw cycle (Central, FC = 6%, CP = 3.0 psi, and DS = 2.7 psi)

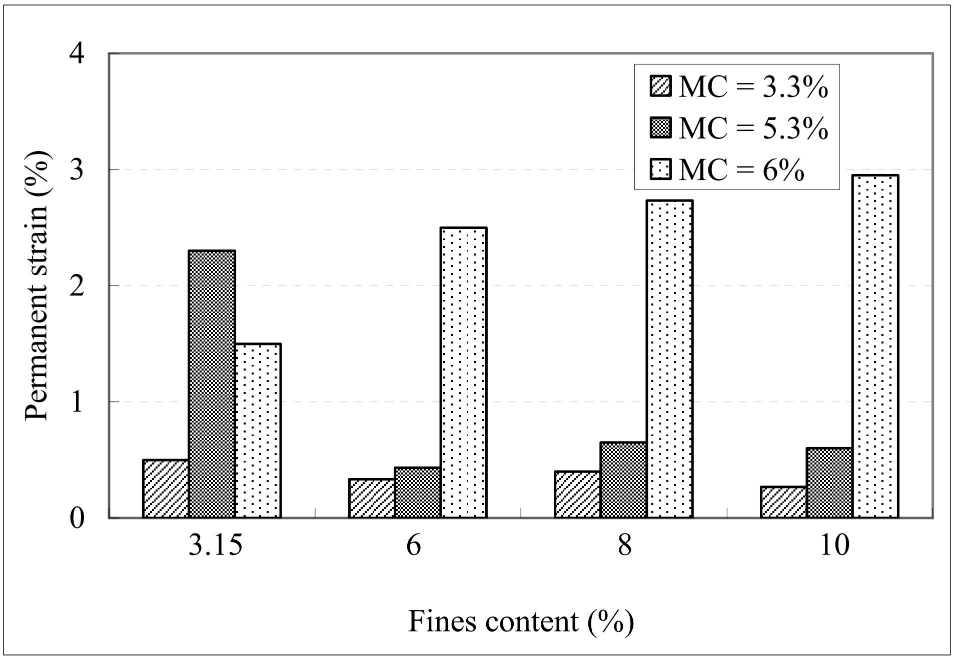
It can be seen from Figures 4.20 and 4.21, M_R results were reduced significantly after one freeze-thaw cycle. In Figure 4.20, with the increase of fines content, the M_R of aggregate after a freeze-thaw cycle decreased due to the high moisture content especially when fines content reached 10%. In Figure 4.21, aggregate specimens with low initial moisture contents demonstrated high M_R reduction after the freeze-thaw cycle. Theoretically, for granular materials, with the increase of fines content, the capability of holding water was enhanced. Because an open system was used for frost heave tests, water intake was allowed during testing. Hence, high fines content resulted in high final moisture content after freezing process. Two possible reasons may be responsible for the significant reduction of M_R after the freeze-thaw cycle. Firstly, moisture content of aggregate specimens significantly increased after testing due to the free water intake during freezing process. Secondly, freezing of moisture enlarged the space between aggregate particles which increased the volume of aggregate specimen. Thus, thawing of ice resulted in an increase of void ratio which could lead to change of the degree of compaction which is an important factor controlling aggregate resilient properties.

Permanent Deformation

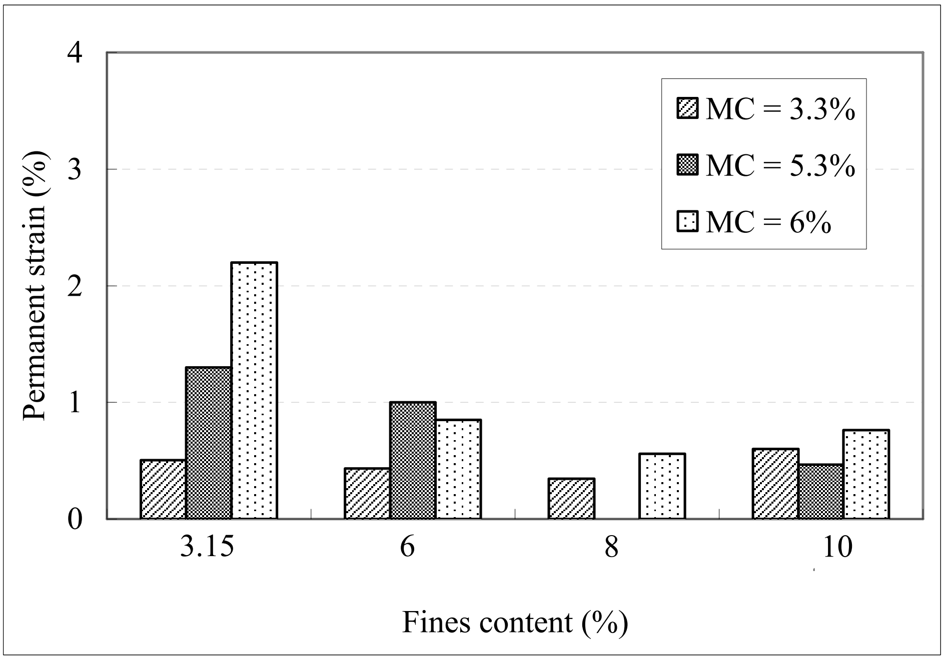
In this study, by conducting M_R tests, permanent deformation data after 15 loading sequences were collected on D-1 base course materials from three regions with different moisture contents, fines contents, temperatures, and sources. All aggregate specimens of D-1 materials from three regions after the freeze-thaw cycle collapsed during M_R testing. Therefore, permanent deformation is greater than 5% of specimen height. Appendix G tabulates the permanent strain data of D-1 materials from three regions at different moisture and fines contents. Figures 4.22 illustrate permanent deformation of D-1 materials from three regions at different moisture and fines contents.



(a) Central Region



(b) Southeast Region



(c) Northern Region

Figure 4.22 Permanent strains at different fines and moisture contents

As can be seen in Figure 4.22, permanent strain was highly dependent on moisture contents. At each fines content level, higher moisture content produced higher permanent strain compared to those at low moisture contents. This is reasonable since suction essentially works as a neutral confining stress. The lower the moisture content, the higher suction is, and subsequently the higher equivalent confining effect and lower permanent deformation. Variation of permanent deformation due to different fines content is insignificant in Figure 4.22 for D-1 materials from three Regions. One possible reason is fines ranging from 3.15% to 10% were not enough to fill up the voids between gravels and sands, and particle interlock was not significantly influenced by the presence of fines. In other words, aggregate strength was still controlled by the interlock between particles. With increase of fines content, specific surface area of D-1 materials increased and some water was absorbed by increased fines as well. Huurman and Molenaar (2006) stated that permanent deformation of unbound materials at a given stress state was highly affected by physical parameters such as gradation, mixture composition, angularity of the particles, and the degree of compaction. In this study, effect of fines content on permanent deformation was also affected by moisture content and angularity of D-1 materials. Effect of fines (ranging from 3.15% to 10%) can not be easily separated from the other factors. According to Figure 4.22, permanent strain on D-1 materials did not vary very much at different fines contents with moisture content of 3.3%. Reason for this is the angularities of D-1 materials from three regions were similar.

Permanent deformation of soil is somewhat related to its resilient behavior. Zlender (2008) stated that permanent deformation was a function of M_R and stress states. At subfreezing temperatures, due to the bonding effect of ice, D-1 material has very high stiffness and correspondingly smaller permanent deformation after the certain loading sequences used in triaxial test. Effect of temperature on permanent deformation was shown in Figure 4.23 using D-1 materials from three regions at fines content of 6% and moisture content of 5.3% as an example. As can be seen in Figure 4.23, permanent deformation increased with an increase of temperature, especially when temperature was close to 0 ^oC. All permanent deformations at subfreezing temperatures were insignificant (less than 0.5%).

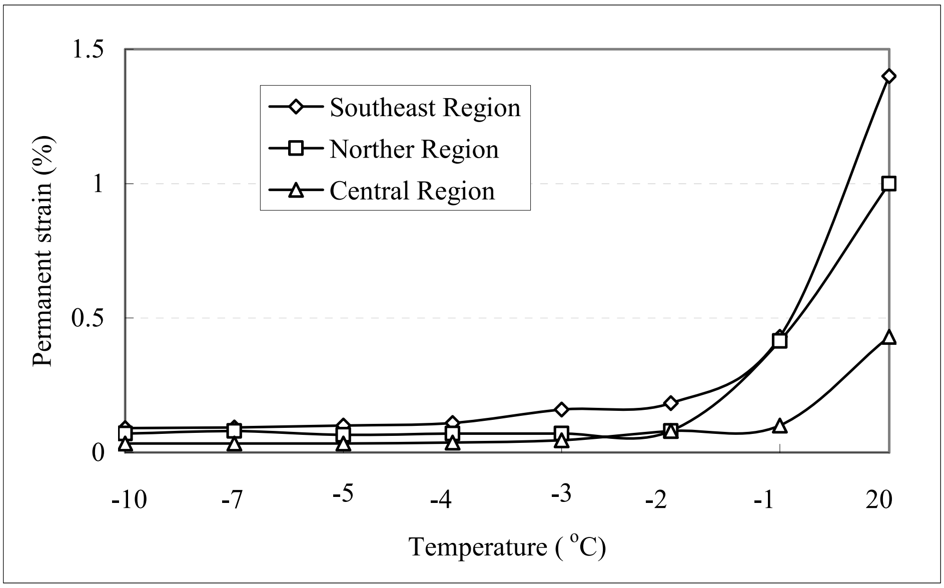


Figure 4.23 Permanent strains at different temperatures

M_R Modeling

Based on discussions above, it was found that moisture content, fines content, temperature, stress state, and freeze-thaw cycle could affect resilient properties of D-1 materials. Initially, an attempt was made to use all M_R values from three regions to develop a model valid at all temperatures. However, this was proved unsuccessful owing to the change of moisture content after freezing process and the significant change of M_R around 0 °C. As a consequence, resilient responses of aggregates above and below 0 °C were analyzed separately.

In this study, Equation 2.8 from the MEPDG (ARA, Inc. 2000), was chosen as a base model for regression analysis on M_R data of D-1 materials tested at temperature of 20 °C. The nonlinear elastic coefficients and exponents of the constitutive model are determined by using nonlinear regression analyses to fit laboratory generated M_R test data. In this base model, M_R was a function of bulk stress and octahedral shear stress. To determine k_i, regression analysis on M_R data of D-1 materials tested at temperature of 20 °C was performed. The obtained k_i are required input parameters for pavement design software of MEPDG. For D-1 materials in Alaska, stress state, temperature, freeze-thaw cycle, and moisture content were found to be the most important influencing factors which can affect resilient behavior of D-1 materials. Effect of freeze-thaw cycle on material behavior was substantial for D-1 materials. However, M_R data for D-1 materials after the freeze-thaw cycle were very limited. Therefore, regression analysis of M_R at 20 °C was performed only on D-1 materials without the freeze-thaw cycle. Since the particle size distribution curves of three D-1 materials were very close and effect of aggregate source was not significant, gradation and aggregate source were not introduced into the relationship between k_i and soil physical properties. Within the scope of this study (fines content ranging from 3.15% to 10%), fines content was a very important parameter which affected soil frost heave susceptibility. Also, fines content controlled the aggregate ability to support vehicle load, especially during spring thaw. Therefore, fines content was introduced into regression. Based on data analysis, effect of fines content on resilient properties of D-1 materials could be affected by moisture content. Therefore, interaction between fines and moisture content was also introduced into the regression. Since M_R results of aggregate specimens from three regions are very close, M_R results of D-1 materials from different regions were combined together to develop a comprehensive model for M_R prediction. The constitutive model was modified which is shown in Equation 4.1.

$$M_R = (c_1 + c_2 * f_c + c_3 * W_s + c_4 * f_c * W_s) * P_a * \left(\frac{\theta}{P_a}\right)^{(c_5 + c_6 * f_c + c_7 * W_s + c_8 * f_c * W_s)} * \left(\frac{\tau_{oct}}{P_a} + 1\right)^{(c_9 + c_{10} * f_c + c_{11} * W_s + c_{12} * f_c * W_s)} \quad (4.1)$$

where,

f_c = fines content,

W_s = moisture content, and

c_i = regression constants.

Both fines and moisture content are in decimal in Equation 4.1. Multiple nonlinear regressions were performed between M_R data and basic aggregate properties as well as stress conditions. The Matlab software was used for this multiple nonlinear regression. Detailed regression procedure is shown in Appendix H. The dependent variable was M_R and four independent variables were fines content, moisture content, bulk stress, and deviator stress. Regression results are shown in Equations 4.2 ~ 4.4. If fines and moisture contents were given, k_i coefficients can be obtained according to Equations 4.2 ~ 4.4. Figure 4.24 compares the predicted M_R based on Equation 4.2 and measured M_R values from laboratory tests. A R^2 of 89.3% indicated a good correlation between predicted and measured results.

$$k_1 = 2.54 + 5.37 * f_c - 32.56 * W_s - 72.76 * W_s * f_c \quad (4.2)$$

$$k_2 = 1.04 + 3.54 * f_c - 10.70 * W_s - 71.19 * W_s * f_c \quad (4.3)$$

$$k_2 = -2.19 + 1.54 * f_c + 44.36 * W_s - 49.18 * W_s * f_c \quad (4.4)$$

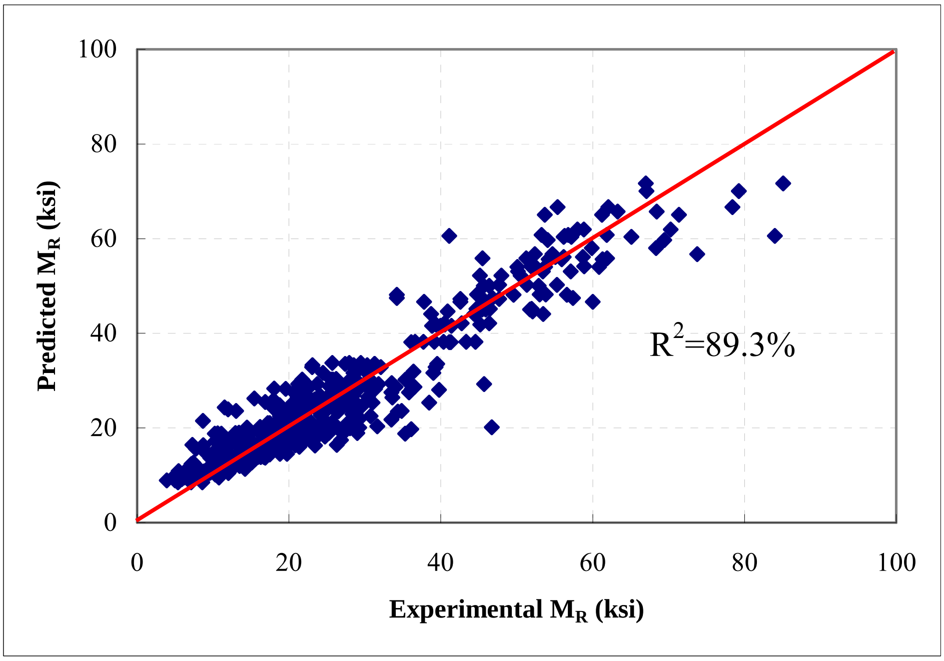


Figure 4.24 Predicted vs. measured M_R at 20 °C

Simonsen et al. (2002) proposed an equation to predict M_R of granular materials at subfreezing temperatures, as expressed by Equation 2.9. Temperature is the only independent parameter in that model. However, after analyzing the effects of confining pressure and deviator stress at subfreezing temperatures, it was found that deviator stress is an important influencing factor on resilient properties of D-1 materials. Consequently, to provide better prediction, deviator stress should be incorporated into Equation 2.9. As discussed before, ice stiffness, aggregate stiffness, and aggregate skeleton matrix interaction are the three components responsible for the resilient properties of granular material. Furthermore, for D-1 materials, variations of final moisture contents after the freeze-thaw cycle are not significant. However, abrasion resistance, which was related to aggregate roughness and stiffness, was considered to be a proper parameter to represent the aggregate type at subfreezing temperatures. Thus, on the basis of Simonsen et al's (2002) model, the factor of abrasion resistance was also introduced to this modified model as shown in Equation 4.5.

$$M_R = A_r^{k_1} e^{k_2 + k_3/T} \sigma_d^{k_4} \tag{4.5}$$

where,

A_r = abrasion resistance (percentage loss) obtained by using Micro-Deval tester.

Multiple linear regressions on M_R test results of D-1 materials from three regions were performed. Equation 4.6 expresses the M_R prediction of D-1 materials under subfreezing temperatures.

$$M_R = A_r^{-0.0371} e^{4.1014 + 0.7054/T} \sigma_d^{0.7346} \; (-1\ {}^oC < T < -10\ {}^oC) \tag{4.6}$$

Predictive models for M_R of D-1 granular materials from three regions of Alaska can be summarized as follows.

when T > 0 °C,

$$M_R = (2.54 + 5.37 * f_c - 32.56 * W_s - 72.76 * W_s * f_c) * P_a * \left(\frac{\theta}{P_a}\right)^{(1.04 + 3.54 * f_c - 10.70 * W_s - 71.19 * f_c * W_s)} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{(-2.19 + 1.54 * f_c + 44.36 * W_s - 49.18 * f_c * W_s)}$$

when -1°C > T > -10 °C,

$$M_R = A_r^{-0.0371} e^{4.1014+0.7054/T} \sigma_d^{0.7346} \quad (0.027 < A_r < 0.097, 2.7\ psi < \sigma_d < 36\ psi).$$

where,

M_R	=	resilient modulus, ksi,
f_c	=	fines content (decimal),
W_s	=	moisture content (decimal),
θ	=	$\sigma_1 + \sigma_2 + \sigma_3$, psi,
$\sigma_1, \sigma_2, \sigma_3$	=	principle stresses, psi,
p_a	=	normalizing stress (atmospheric pressure, 14.5 psi),
τ_{oct}	=	octahedral shear stress $= \frac{1}{3}\sqrt{(\sigma_1-\sigma_2)^2 + (\sigma_1-\sigma_3)^2 + (\sigma_2-\sigma_3)^2}$, psi,
A_r	=	abrasion resistance (decimal),
T	=	temperature, degree Celsius, and
σ_d	=	deviator stress, psi.

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

To determine the impact of fines content on resilient modulus reduction of base courses during thawing, frost heave and repeated-load triaxial tests were performed on D-1 granular base course materials from three regions (Northern, Southeast, and Central Regions) of AKDOT&PF with different fines and moisture contents, under different temperature and freeze-thaw conditions. After testing, frost heave, M_R and permanent deformation test results were obtained and analyzed. To incorporate into MEPDG, regression analysis was also performed on M_R data of D-1 materials tested at 20 ^oC without the freeze-thaw cycle. The k_i coefficients obtained from regression are required for pavement design by using MEPDG software. For regression analysis on M_R data of D-1 materials tested at subfreezing temperatures, a modified model was developed for M_R prediction.

Conclusions

The conclusions are listed as follows:

1. After the freezing process in an open system, aggregate specimens with low initial moisture and high fines content led to higher final moisture contents. Final moisture content was highly related to the initial moisture content of aggregate specimen. Frost heave of D-1 materials increased with increases of fines content after the freezing process. Frost heave data could also be affected by the initial moisture contents of D-1 materials. Low initial moisture and high fines contents produced highest frost heave value which correlated with change of moisture content after the freezing process very well. In this study, frost heave data of all D-1 materials was not significant. Significant frost heave may be expected under different temperature gradients.
2. When moisture content increased from 3.3% to 6%, pore-water pressure developed during M_R test under undrained condition. The developed pore-water pressure resulted in a reduction of effective stress. Also, increased moisture

content enhanced the lubricating effect. Subsequently, M_R data for D-1 materials from three regions decreased due to increased moisture content.

3. Within the scope of this study (fines content ranging from 3.15% to 10%), impact of fines content on resilient properties of D-1 materials varied and was affected by moisture content and material source. For D-1 material from Southeast Region, at lower moisture content, most of with high fines contents produced higher M_R values when compared to those with low fines content. With the increase of moisture content, impact of fines content became insignificant. In addition, fines content had different influences on D-1 materials from different regions.
4. Resilient behavior of D-1 materials were affected by temperature and deviator stress. At room temperature, M_R of D-1 materials from three regions increased with increase of confining pressure when moisture contents ranged from 3.3% to 6%. However, when other conditions were the same, at low moisture content, M_R decreased as the applied deviator stress increased. When moisture content was at the optimal moisture content or higher, M_R increased with increase of deviator stress. However, effect of confining pressure became insignificant for D-1 materials at high moisture content. At subfreezing temperatures, within the scope of this study (confining pressure ranging from 3 psi to 20 psi), the confining pressure did not provide significant effect on resilient modulus values.
5. When temperature increased from -10 to 0 °C, the unfrozen moisture content increased as well. Due to increased unfrozen water content, M_R data of D-1 materials from three regions decreased with an increase of temperature, especially when temperature ranged from -5 to 0 ^{o}C.
6. After the freeze-thaw cycle, aggregate specimens were weakened and most of specimens collapsed during the M_R test. Therefore, M_R data of D-1 materials from three regions could not be obtained. From limited M_R data for D-1 materials from Central Region, it was found that freeze-thaw cycle resulted in significant reduction of M_R values when compared to those of aggregate specimens without the freeze-thaw cycle.
7. Permanent strain was significantly affected by moisture content. Higher moisture content produced higher permanent strain. Effect of fines content on permanent

strain of D-1 materials was affected by moisture content and material source. Also, at subfreezing temperatures, permanent deformation increased with as increase of temperature, especially when temperature was close to 0 ^{o}C.

8. Regressions were performed on M_R data of D-1 materials from three regions tested under frozen and unfrozen conditions for M_R prediction of D-1 materials at different temperatures, moisture contents, and fines contents. Fines content, moisture content, and the interaction between them were introduced into the model specified in MEPDG to correlate with k_i coefficients which are required for flexible pavement design by the MEPDG software. A modified model was also developed for M_R prediction of D-1 materials at subfreezing temperatures. In this model, M_R of D-1 material was a function of temperature, deviator stress, and abrasion resistance of aggregates.

Recommendations

In this study, frozen D-1 material specimens were prepared using the one-dimensional frost heave cell for M_R tests at subfreezing temperatures. An open system was used in which aggregate specimens had free access to water from the bottom during the freezing process. The frozen D-1 material specimens were then tested to measure the M_R under different subfreezing temperatures and after the freeze-thaw cycle. However, during the M_R testing process, most of D-1 material specimens failed after a freeze-thaw cycle before any further data were generated. As the open system of water access represents the worst scenario that a pavement structure could possibly experience, it is necessary to investigate the behavior of base course materials under other conditions with limited water access during freezing in order to complement the existing research and have a better understanding of the combined effect of fines content and moisture content on the resilient behavior of D-1 materials under different conditions of water access.

According to Casagrande's criteria (1932), D-1 materials from three regions with a fines content of 10% were all slightly frost susceptible (because % finer than 20 um varied from 3.1% to 4.2% which are greater than 3%). However, this study did not show significant frost heave for those materials with high fines. One of the possible reasons was that only one temperature gradient (0.25 ^{o}C/cm) was used. Previous studies indicated that temperature gradient was a very important factor which could affect frost heave behavior of soil. Therefore, different temperature gradients are suggested for specimen preparation in future studies to better correlate the effect of fines content, frost heave and resilient behavior of these granular materials.

Physical properties such as abrasion resistance, flat and elongated particles, and fractured surface of aggregates can greatly affect the resilient behavior of granular materials. However, in this study, variations of M_R data due to different aggregate sources were not significant. A possible reason for this was that the physical properties of D-1 materials used in this study were similar. Therefore, more tests for D-1 materials from different sources are needed to characterize the effect of fines content on resilient behavior of D-1 materials.

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APPENDICES

Data Details

Appendix A

Table A.1 Original Gradations of D-1 Materials from the Three Regions

Northern Region (Fairbanks)		Central Region (Anchorage)		Southeast Region (Juneau)	
LL*	PI	LL	PI	LL	PI
0	0	0	0	0	0
Size (mm)	% passing	Size (mm)	% passing	Size (mm)	% passing
25	100	25	100	25	100
19.05	97.6	19.05	99.6	19.05	100
9.5	74.2	9.5	72.6	9.5	69.1
4.75	47.3	4.75	50	4.75	41.8
2.36	30.4	2.36	37.5	2.36	25.4
0.3	11.2	0.3	9.9	0.3	8.1
0.075	2.7	0.075	2.9	0.075	3.9
0.0344	1.1993	0.0332	1.4536	0.0323	1.9169
0.0224	0.9423	0.0214	1.2719	0.0214	1.3179
0.0135	0.514	0.0127	0.9994	0.0126	1.0783
0.0097	0.4283	0.0091	0.9085	0.009	0.9585
0.0069	0.4069	0.0065	0.7722	0.0064	0.8386
0.0034	0.3855	0.0033	0.5451	0.0032	0.599
0.0014	0.1713	0.0014	0.4088	0.0014	0.2396

* LL - liquid limit
PI - plastic index

Appendix B

Table B.1 Gradations of D-1 Materials Used in This Study

Sieve size (mm)	% passing			
19.05	100	100	100	100
9.5	72.36	73.17	73.74	74.31
4.75	46.65	48.22	49.32	50.42
2.36	31.28	33.3	34.72	36.14
0.3	9.8	12.45	14.31	16.18
0.075	3.15	6	8	10

Appendix C

Table C.1 Detailed Compaction Test Results

Fines content	Central Region		Southeast Region		Northern Region	
	Moisture content (%)	Dry density (pcf)	Moisture content (%)	Dry density (pcf)	Moisture content (%)	Dry density (pcf)
3.15%	5.0	143.16	4.2	152.23	4.2	138.26
	5.4	144.68	4.8	154.68	4.8	140.58
	5.8	146.54	5.2	155.96	5.2	141.39
	6.3	145.57	5.5	155.47	5.6	141.19
	6.8	145.02	6.0	153.31	6.0	140.9
6%	4.5	144.87	4.3	151.53	4.2	142.94
	5.2	146.83	4.9	154.68	4.8	145.32
	6.0	147.86	5.3	156.18	5.2	146.07
	6.6	147.14	5.7	155.87	5.7	145.39
	7.3	145.49	6.2	155.41	6.3	145.16
8%	4.6	146.88	4.5	153.17	4.3	143.7
	5.2	150.1	5.0	155.43	4.9	146.78
	5.8	149.98	5.4	156.47	5.3	148.01
	6.6	146.95	5.8	156.67	5.8	147.16
	-	-	6.5	154.65	6.4	145.34
10%	4.6	149.29	4.6	155.31	4.4	144.52
	5.2	151.06	5.2	156.83	4.9	147.92
	5.9	150.45	5.8	156.83	5.3	148.10
	6.6	148.74	6.4	155.09	5.8	146.89
	7.1	146.26	6.9	153.26	6.4	145.44

Appendix D

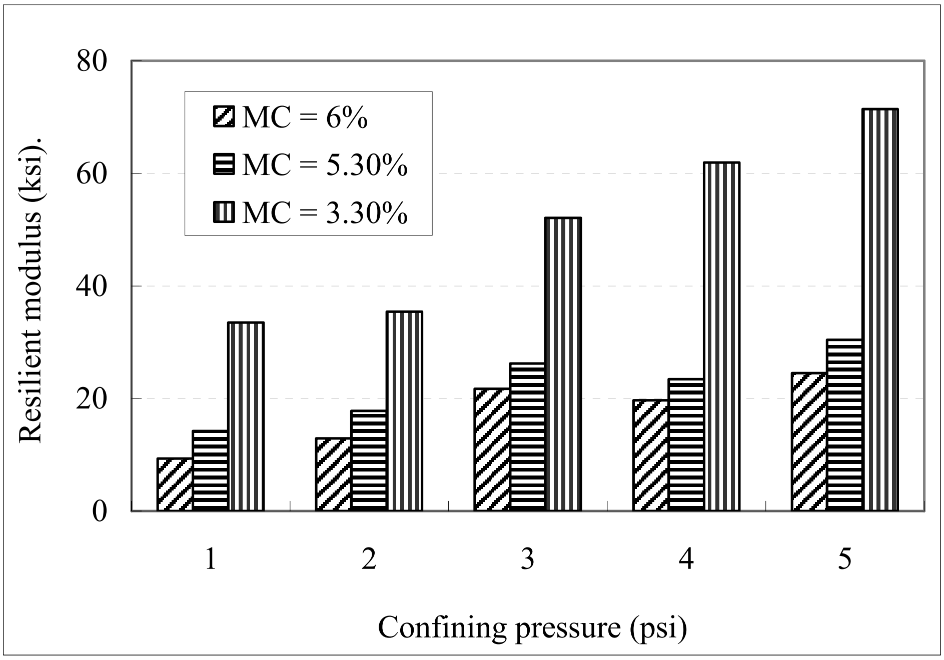

Figure D.1 M_R at varying moisture contents (FC=8%, Southeast Region)

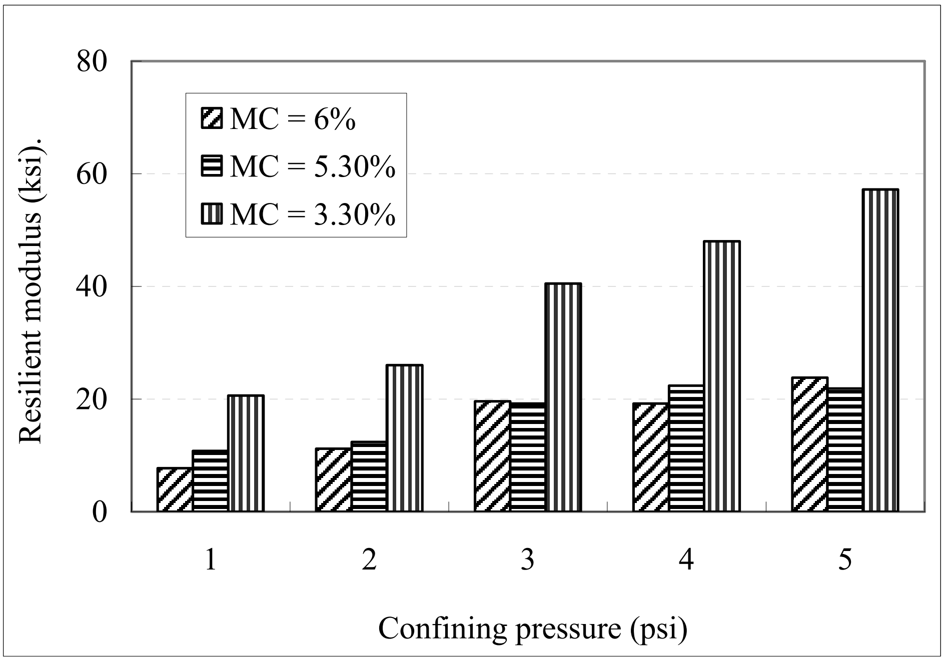

Figure D.2 M_R at varying moisture contents (FC=6%, Southeast Region)

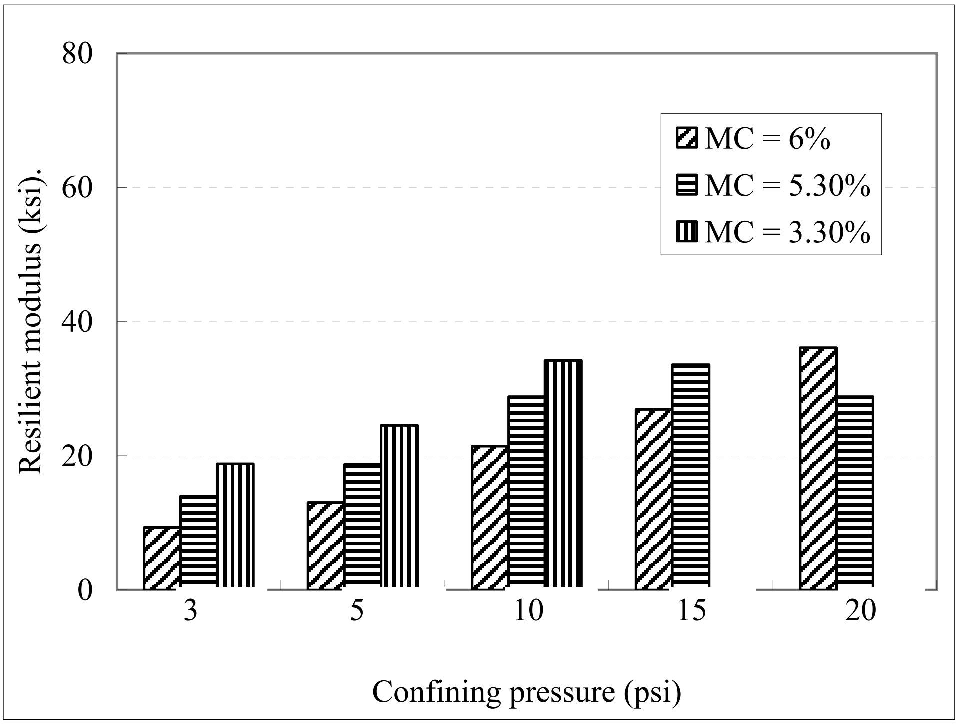

Figure D.3 M_R at varying moisture contents (FC=10%, Northern Region)

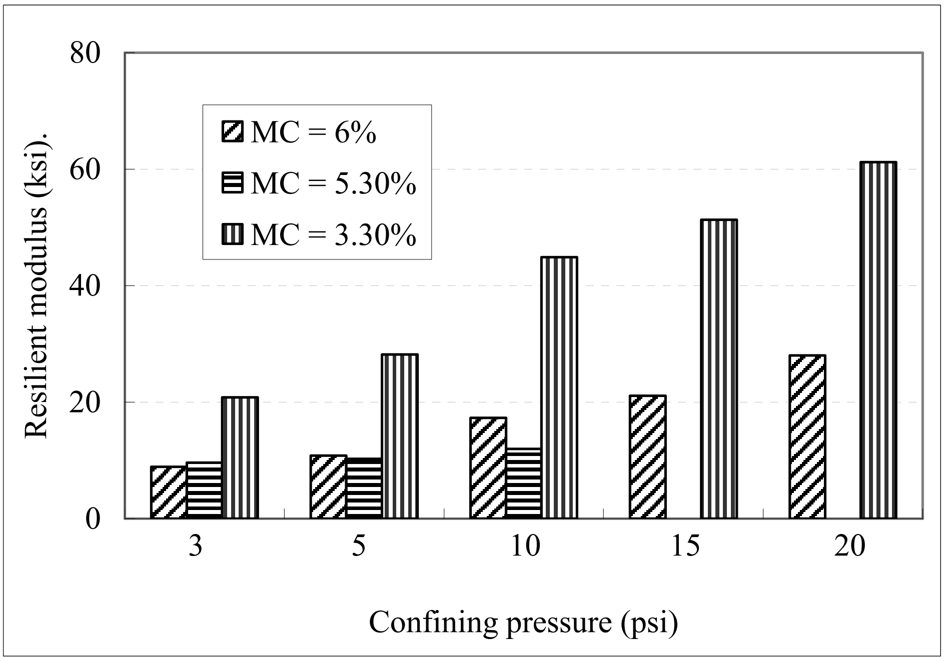

Figure D.4 M_R at varying moisture contents (FC=8%, Northern Region)

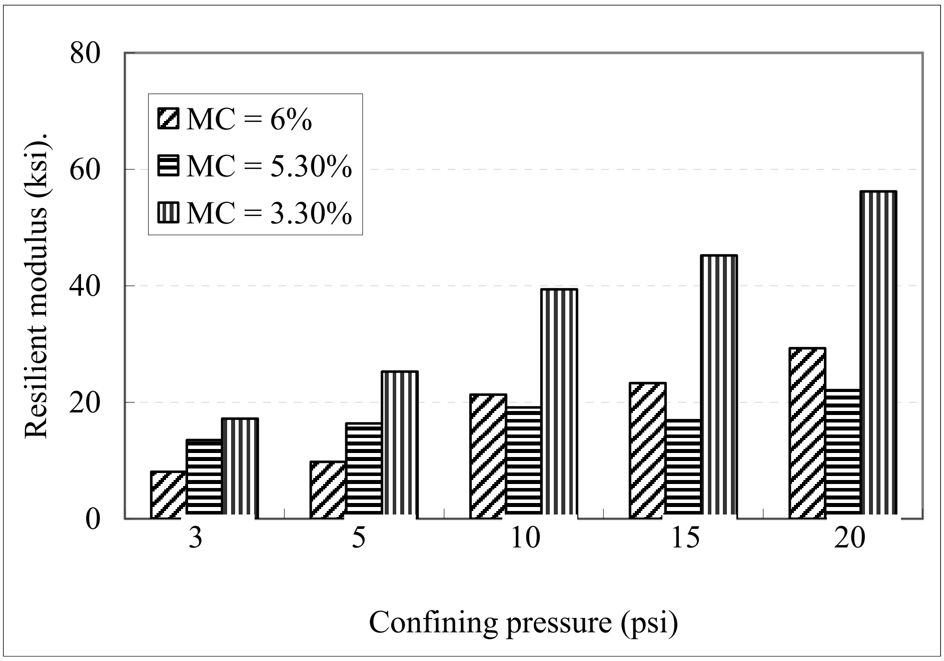

Figure D.5 M_R at varying moisture contents (FC=6%, Northern Region)

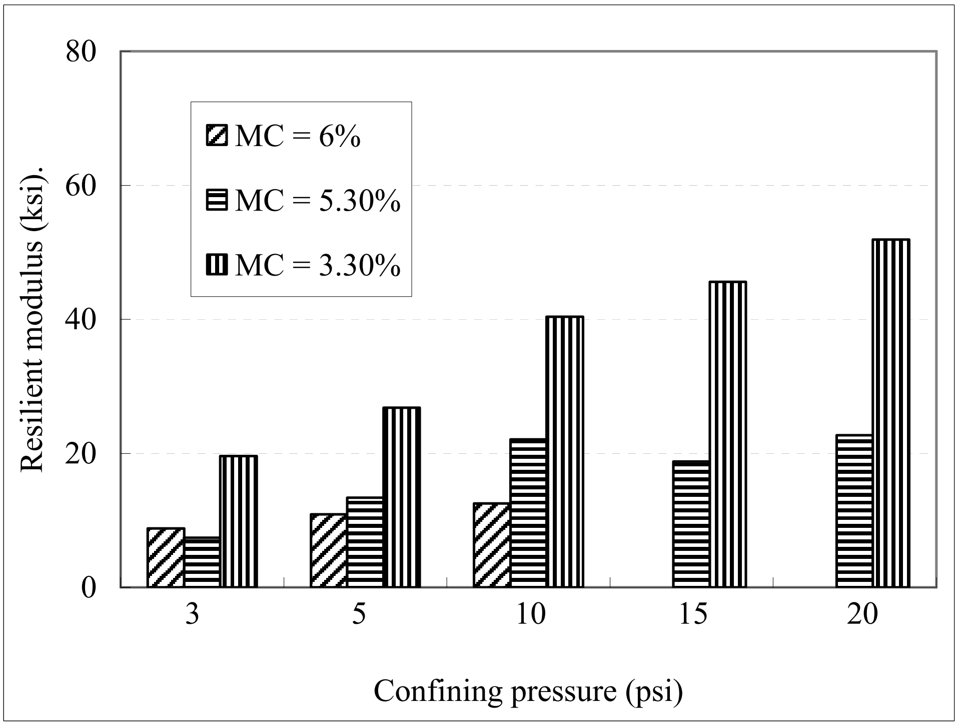

Figure D.6 M_R at varying moisture contents (FC=3.15%, Northern Region)

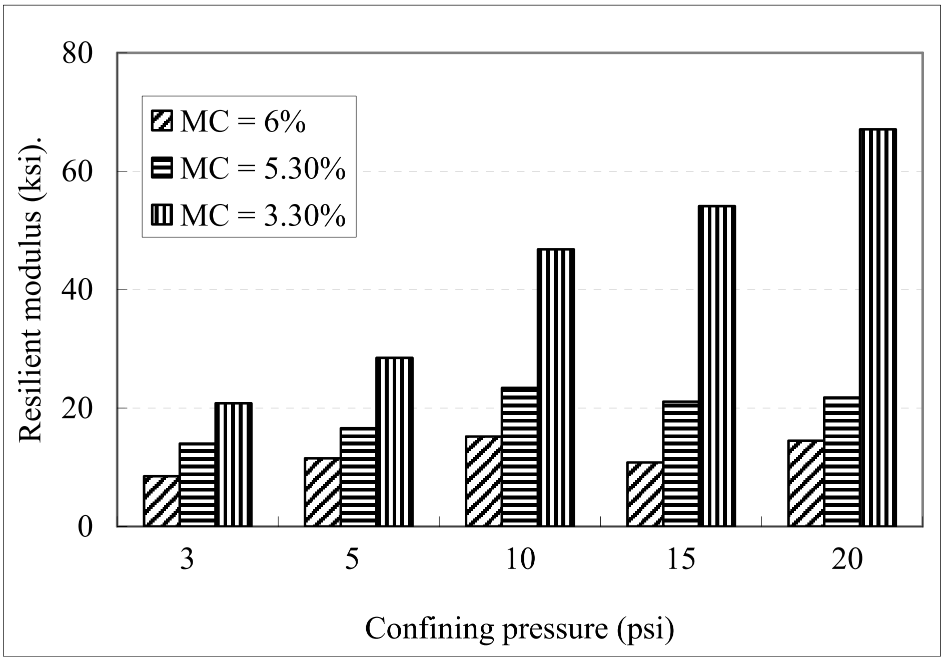

Figure D.7 M_R at varying moisture contents (FC=10%, Central Region)

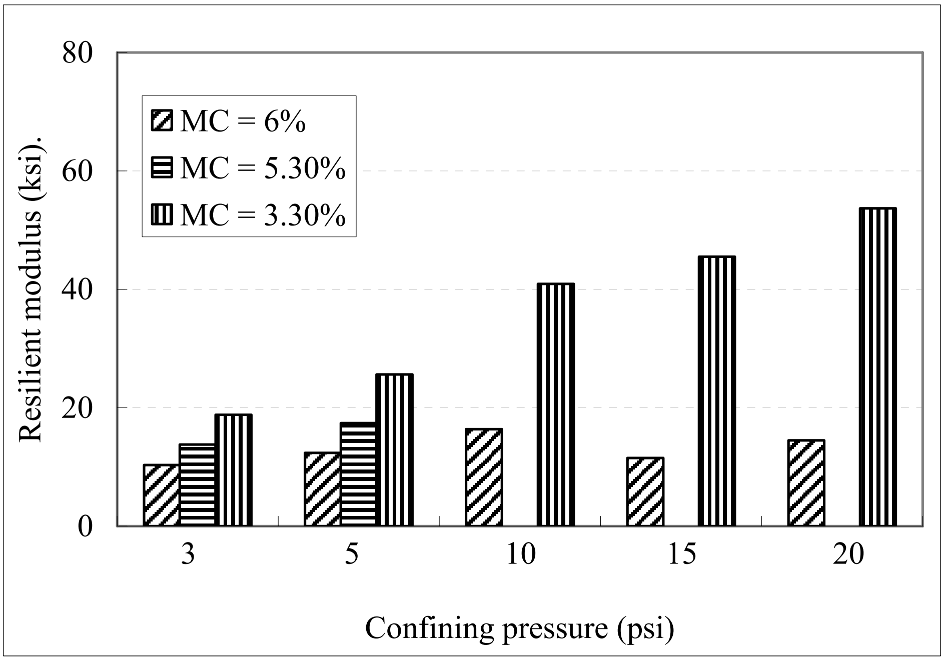

Figure D.8 M_R at varying moisture contents (FC=8%, Central Region)

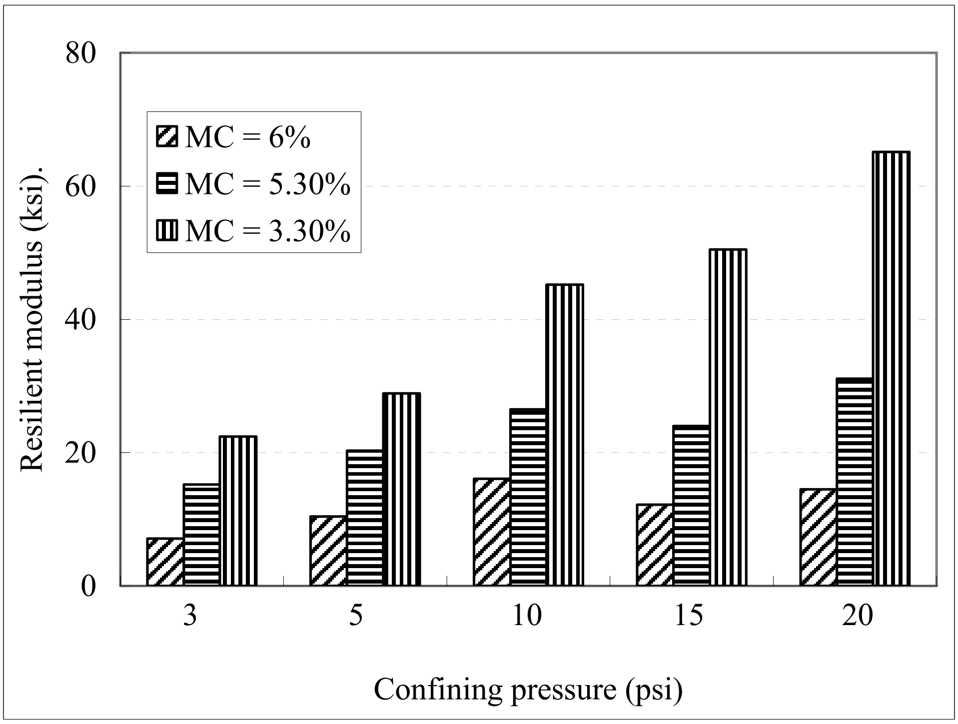

Figure D.9 M_R at varying moisture contents (FC=6%, Central Region)

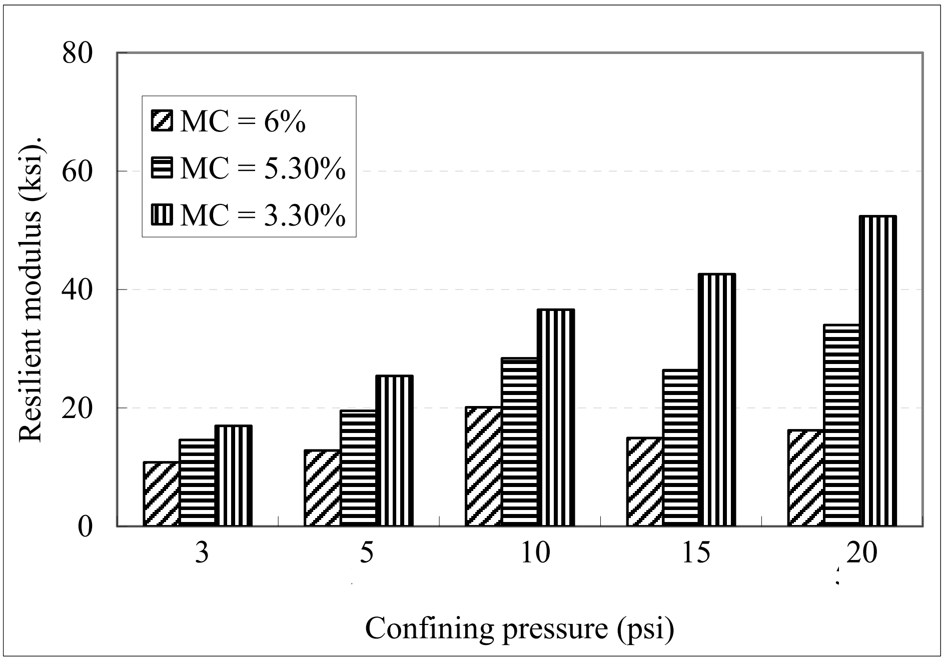

Figure D.10 M_R at varying moisture contents (FC=3.15%, Central Region)

Appendix E

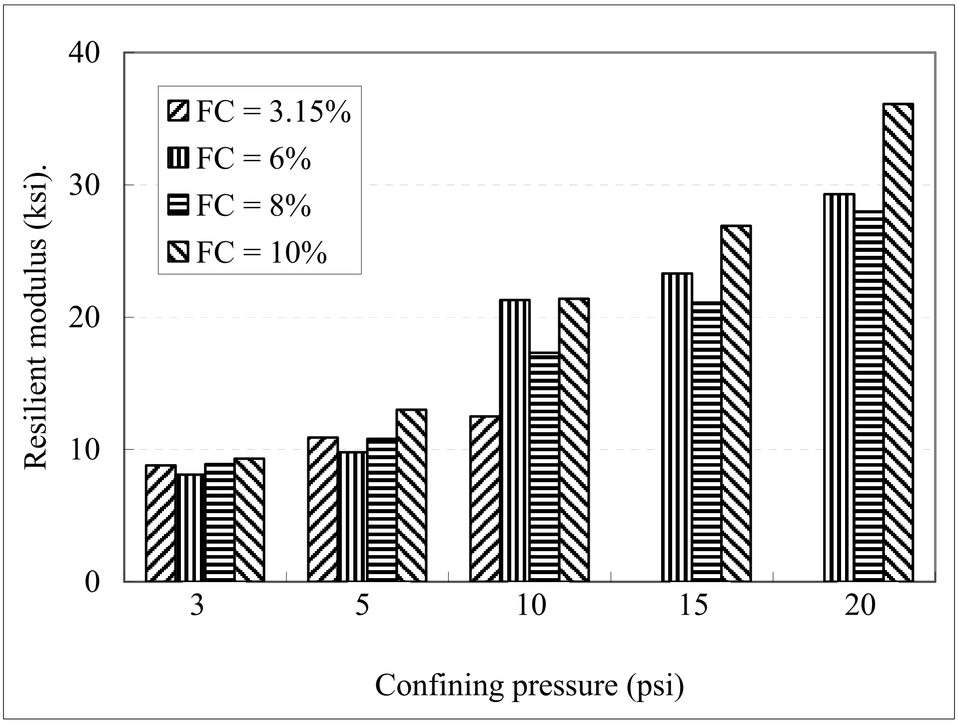

Figure E.1 M_R at varying fines contents (MC=6%, Northern Region)

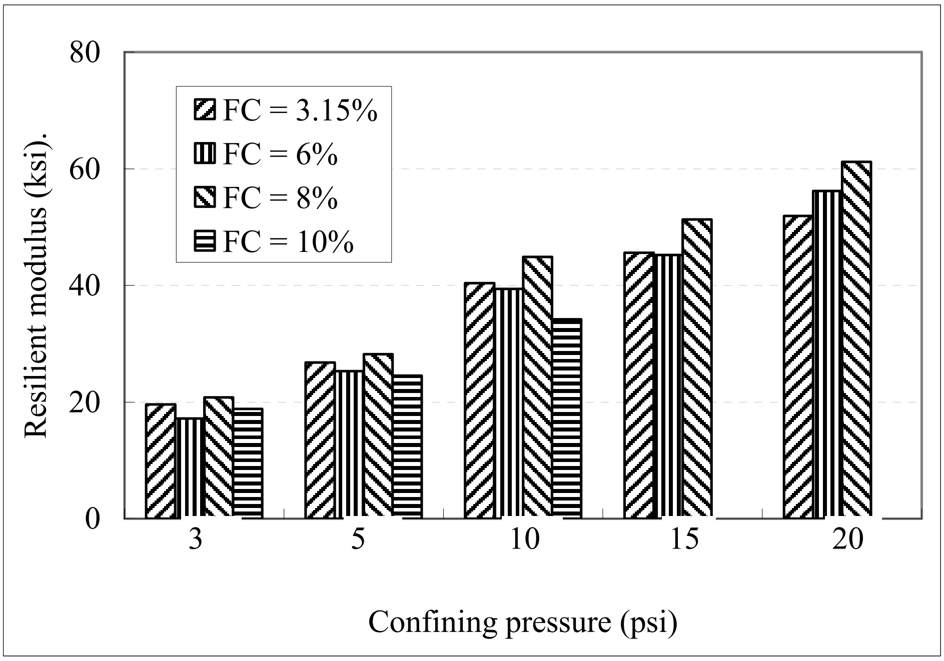

Figure E.2 M_R at varying fines contents (MC=3.3%, Northern Region)

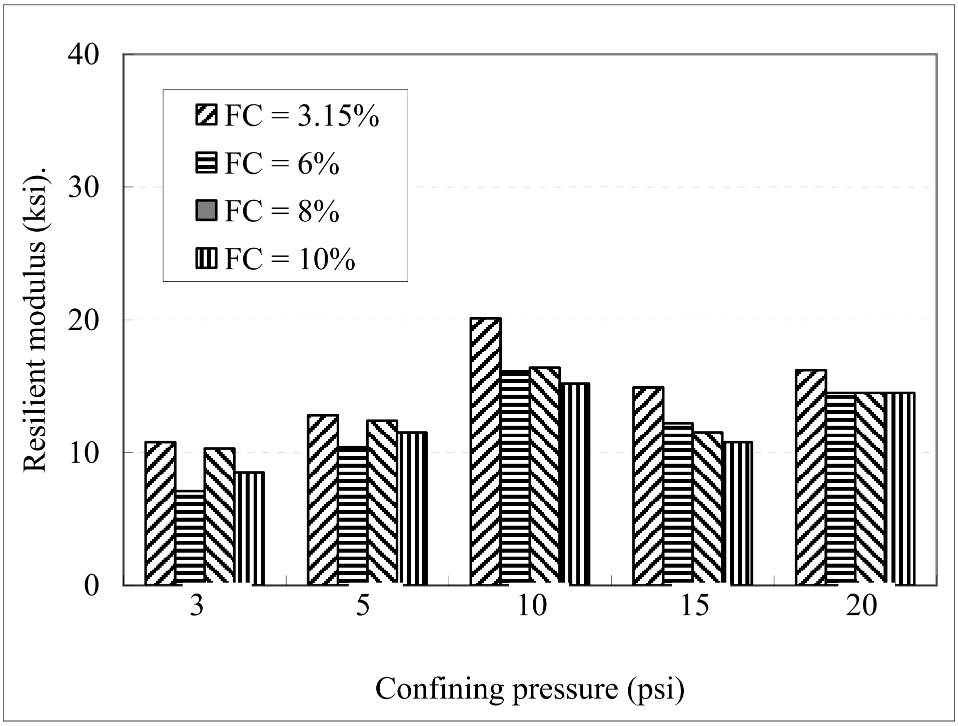

Figure E.3 M_R at varying fines contents (MC=6%, Central Region)

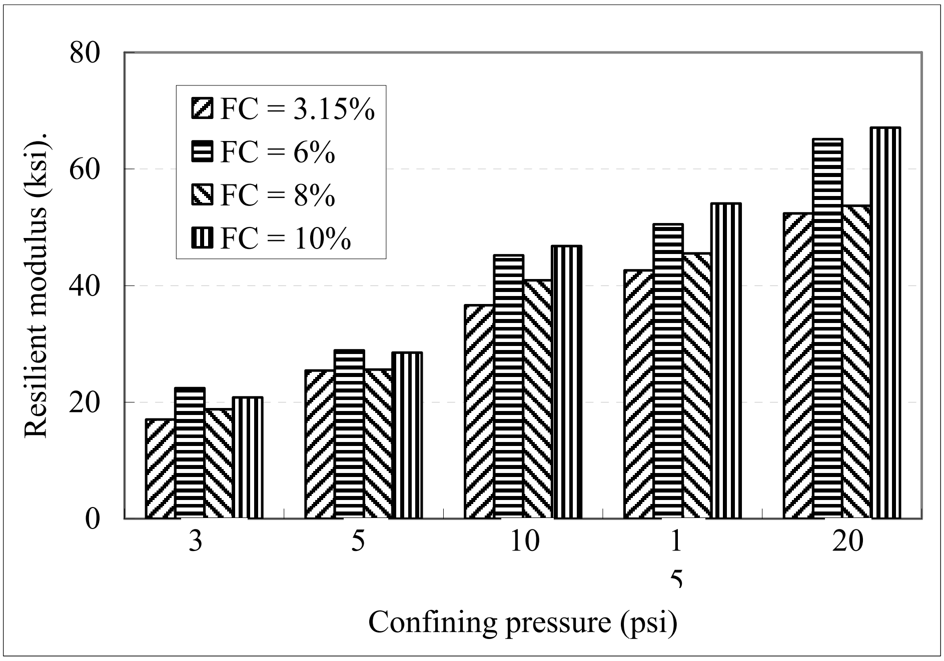

Figure E.4 M_R at varying fines contents (MC=3.3%, Central Region)

Appendix F

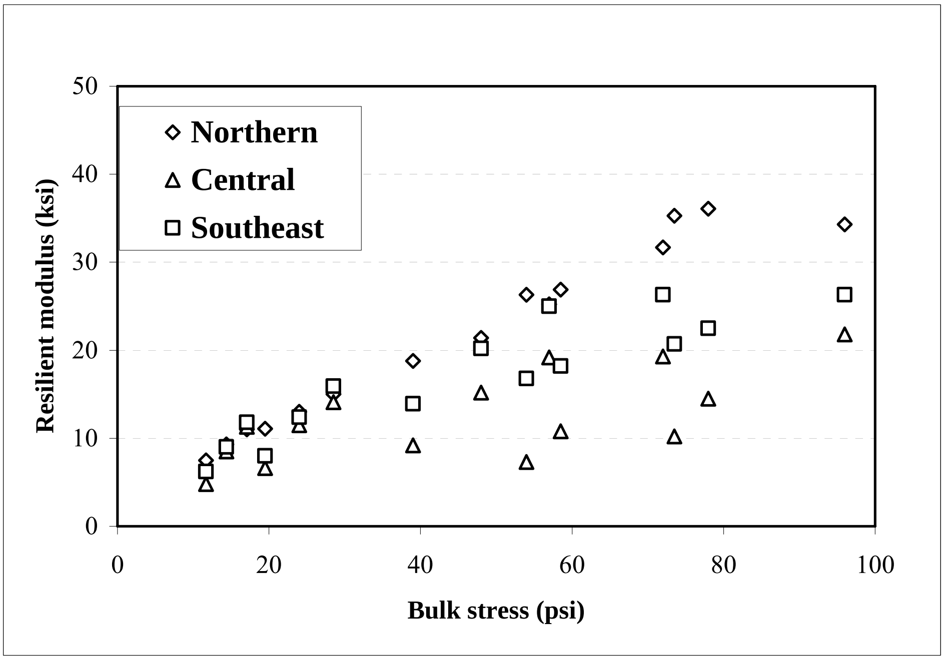

Figure F.1 Comparison of M_R for D-1 materials (FC=10%, MC=6%)

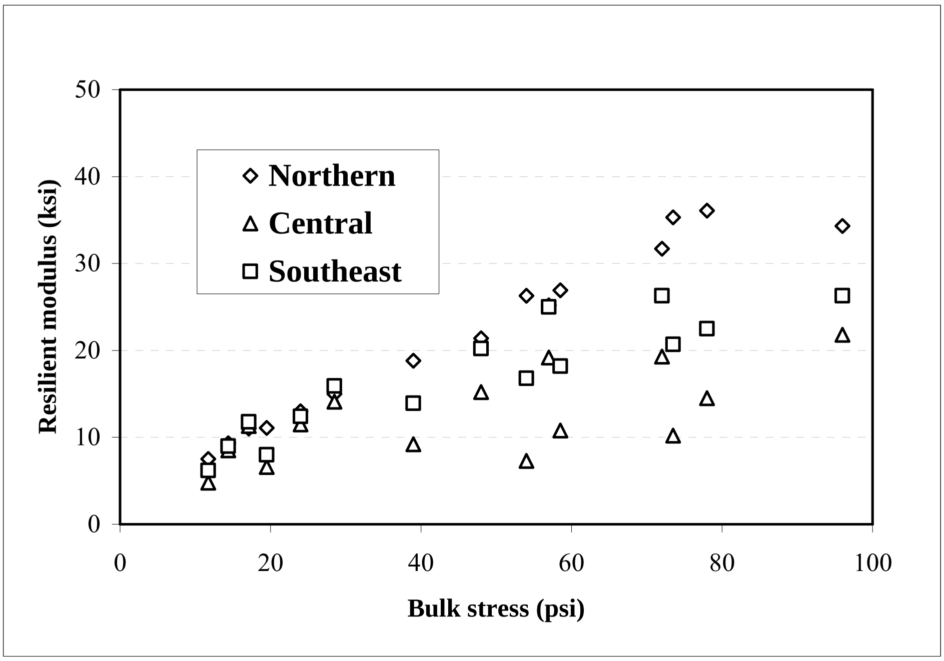

Figure F.2 Comparison of M_R for D-1 materials (FC=8%, MC=6%)

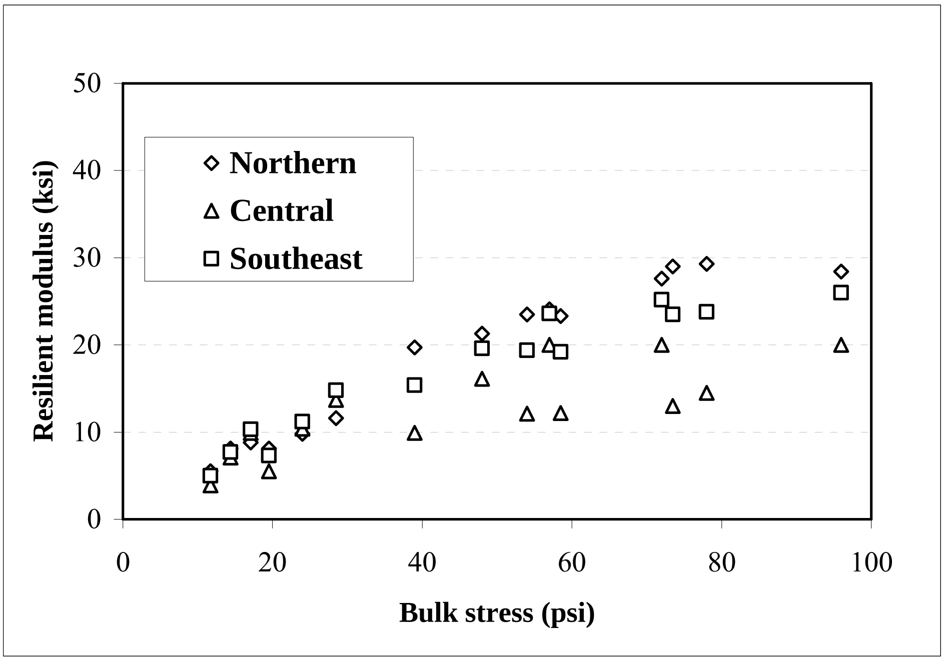

Figure F.3 Comparison of M_R for D-1 materials (FC=6%, MC=6%)

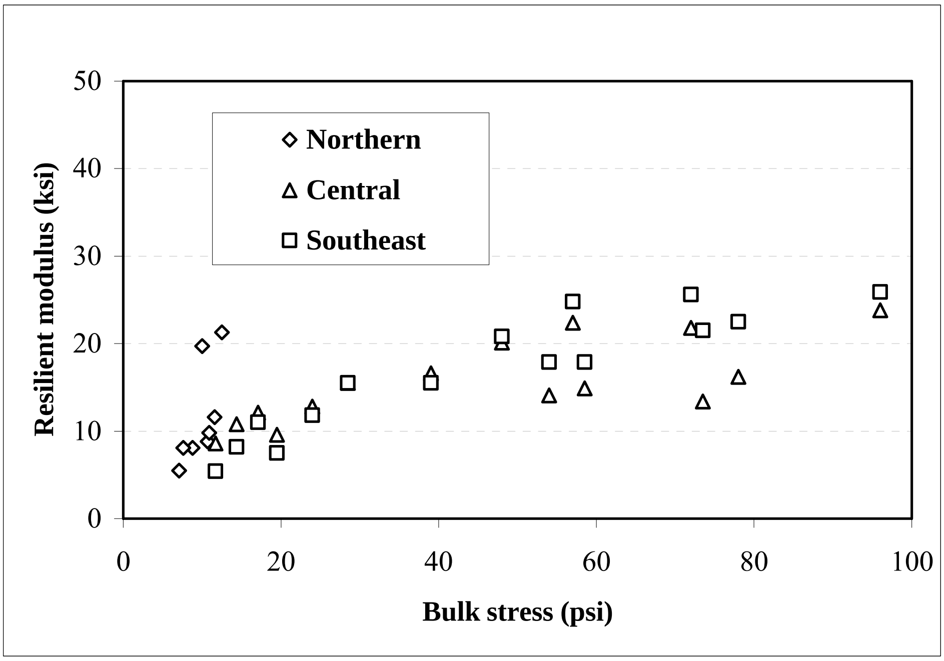

Figure F.4 Comparison of M_R for D-1 materials (FC=3.15%, MC=6%)

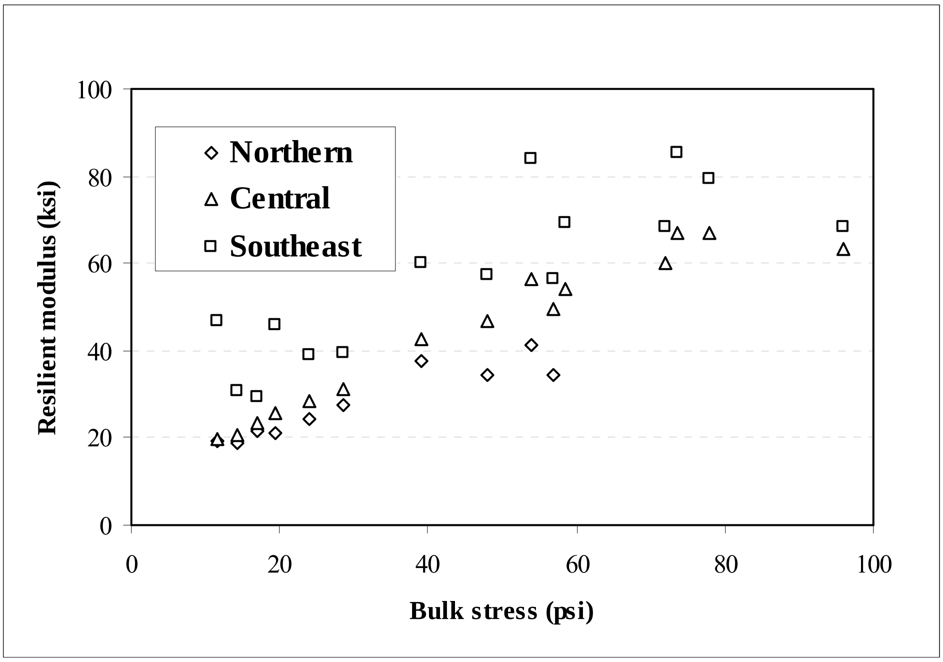

Figure F.5 Comparison of M_R for D-1 materials (FC=10%, MC=3.3%)

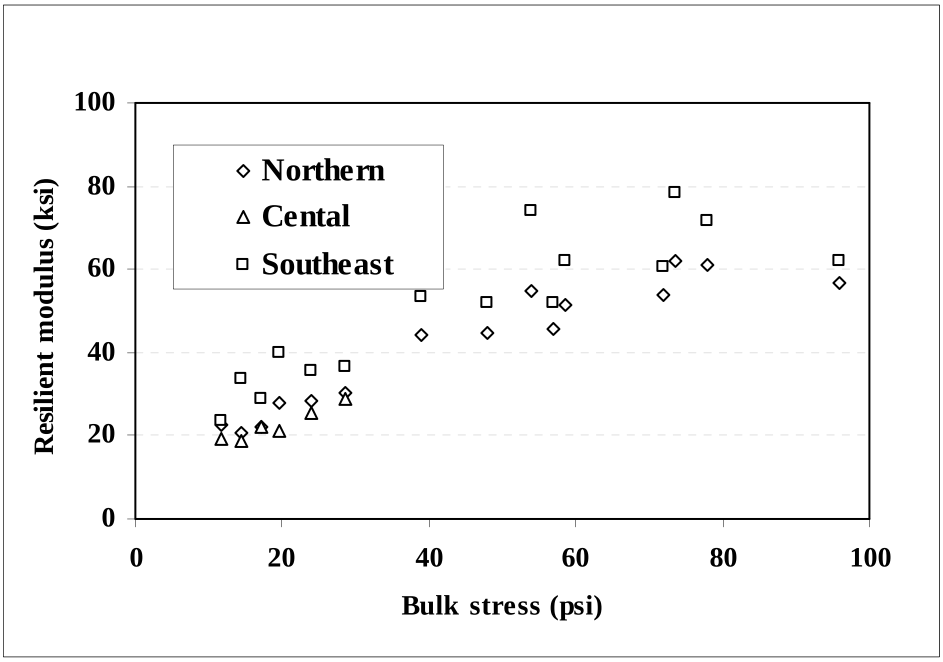

Figure F.6 Comparison of M_R for D-1 materials (FC=8%, MC=3.3%)

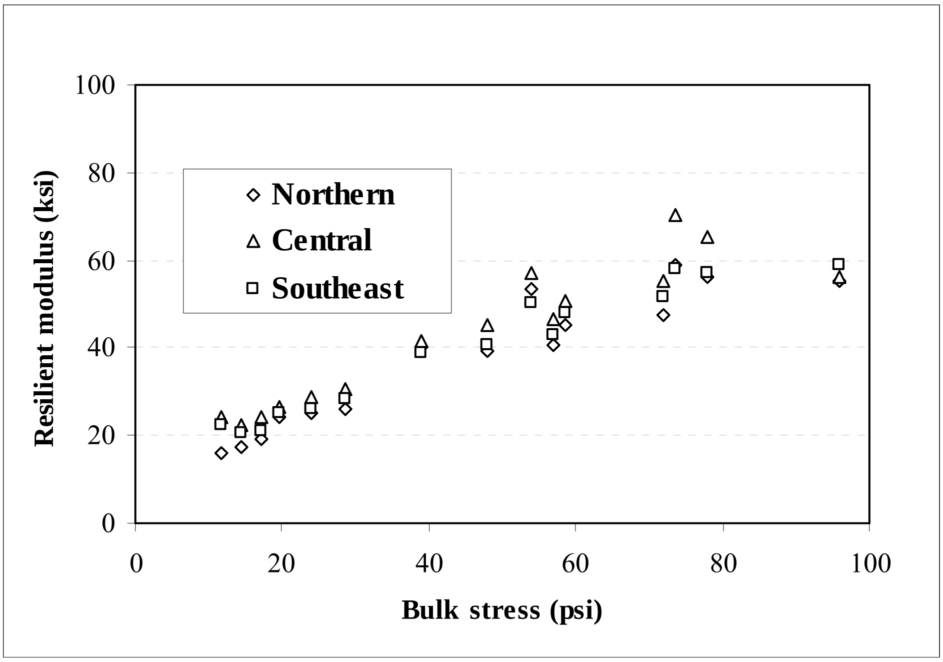

Figure F.7 Comparison of M_R for D-1 materials (FC=6%, MC=3.3%)

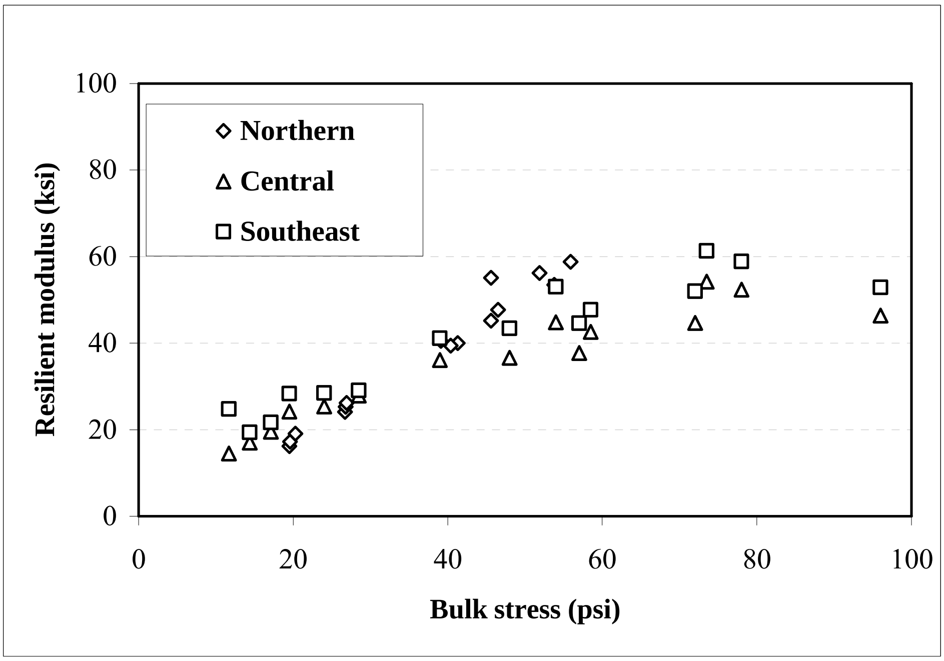

Figure F.8 Comparison of M_R for D-1 materials (FC=3.15%, MC=3.3%)

Appendix G

Table G.1 Permanent Strain of D-1 Materials from three regions

Material source	Permanent strain (%)											
	10% fines			8% fines			6% fines			3.15% fines		
	6%	5.3%	3.3%	6%	5.3%	3.3%	6%	5.3%	3.3%	6%	5.3%	3.3%
Central	2.95	0.60	0.27	2.73	0.65	0.40	2.50	0.43	0.33	1.50	2.30	0.50
Southeast	2.25	1.47	0.20	1.77	0.80	0.40	1.14	1.40	0.30	2.40	0.45	0.33
Northern	0.76	0.47	0.60	0.56	-	0.35	0.85	1.00	0.43	2.20	1.30	0.51

Appendix H

To perform the regression on M_R results of D-1 materials at room temperature by Matlab, the following procedures could be followed.

1) create a **.m file with codes as:

```
function yhat = **(beta,x)
yhat = myfun(beta, x);
```

2) define "x", "y", and "beta0"

3) call the nonlinear regression command as:

```
Beta = nlinfit(x,y,'**',beta0);
```

where,

** = file name,

beta = regression coefficients, which are k_i in this study,

x = matrix of independent variables, which are fines content, moisture content, bulk stress, and deviator stress in this study,

myfun = is a mathematical expression of regression equation, which is Equation 4.1 in this study,

y = column vector of dependent variable, which is M_R in this study, and

beta0 = column vector with initial guesses of beta.