PERFORMANCE BASED CHARACTERIZATION OF VIRGIN AND RECYCLED AGGREGATE BASE MATERIALS

by

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Abstract

Characterization of the effect of physical properties on the performance such as stiffness and drainage of unbound granular materials is necessary in order to incorporate them in pavement design. The stiffness, deformation and permeability behaviour of unbound granular materials are the essential design inputs for Mechanistic-Empirical Pavement Design Guide as well as empirical design methods. The performance based specifications are aimed to design, and construct a durable and cost effective material throughout the design life of a pavement. However, the specification varies among jurisdiction depending on the historical or current practice, locally available materials, landform, climate and drainage. A literature review on the current unbound granular materials virgin and recycled concrete aggregate base construction specification has been carried out in this study. Resilient modulus, permanent deformation and permeability tests have been carried out on seven gradations of materials from locally available sources. Resilient modulus stiffness of unbound granular material at two different conditioning stress level have been compared in the study. The long term deformation behaviour has also been characterized from results of the permanent deformation test using shakedown approach, dissipated energy approach and a simplified approach. The results show improvement in resilient modulus and permanent deformation for the proposed specification compared to the currently used materials as a results of reduced fines content, increased crush count and inclusion of larger maximum aggregate size into the gradation. A significant effect of particle packing on permeability of granular materials have also been found, in addition to the effect of fines.

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Notations and Abbreviations

AASHTO	Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
CBR	California bearing ratio
LL	Liquid limit
LTPP	Long Term Pavement Performance
MC	Moisture content
MEPDG	Mechanistic Empirical Pavement Design Guide
MR	Resilient modulus
N	Number of loading cycles
NCHRP	National Cooperative Highway Research Program
OMC	Optimum moisture content
PI	Plasticity index
p-value	The probability associated with the t-test
P_a	Atmospheric pressure at sea level
R^2	Coefficient of determination
RMSE	Root mean square error
UGM	Unbound Granular Material
τ_{oct}	Octahedral shear stress
σ_d	Deviator stress
ε_r	Resilient strain

ε_p	Permanent strain
θ	Bulk stress
σ_1	Total vertical stress
σ_2, σ_3	Horizontal principal stresses

Chapter 1 Introduction

1.1 BACKGROUND

The performance of unbound granular materials used as base layer in pavement has a significant role on the performance of the pavement throughout the design life. Base layer contributes to the pavement structural capacity to transfer the axle loads from traffic to the subgrade. The stresses in the layer is transmitted through aggregate to aggregate contact. Therefore, the gradation of the unbound granular materials has a significant impact on the overall performance of the pavement. Furthermore, the unbound granular materials are elastoplastic materials in behaviour. The material undergoes elastic and permanent deformation with the application of load. The damage in pavement layers is gradual and due to accumulation of the permanent deformations (Sharp & Booker, 1984). Resilient modulus and permanent deformation are important performance parameters and inputs in pavement design. The design values and the desirable properties of the materials depend on the type of materials. The stiffness and resistance to permanent deformation of the unbound granular materials depends on the aggregate source, maximum aggregate size, amount of fines, and shape of the coarse aggregates. Ideally, the materials gradation having a dense coarser gradation with crushed aggregates performs well regardless the type of materials. However, the drainability of the base layer also has a significant role in addition to having a dense stiff base layer. Resilient modulus and permanent deformation tests have been performed on unbound granular materials in several studies (Kolisoja, 1997; Lekarp & Dawson, 1997; Pan, Tutumluer, & Anochie-Boateng, 2006; Perez, Medina, & Romana, 2006; Rahman & Erlingsson, 2014a; Theyse, 1997; Sabine Werkmeister, Dawson, & Wellner, 2001). However, the results from the previously reported researches may not be directly comparable to materials from a different source because of the morphology, aggregate source, quality of aggregates, shape and texture of aggregates, and climatic effects (Lekarp, Isacsson, & Dawson, 2000a; Tutumluer, 2013). Level 1 input for the pavement design can be obtained from the laboratory test. Level 1 input is required for the most reliable design. Whenever the laboratory test data is not available, the resilient modulus and permanent deformation behaviour of a material can be estimated indirectly using mechanistic empirical pavement design guide empirical models using level 2 approach. The MEPDG empirical models can be calibrated from the test results of local laboratory test data. Using level 2 approach in design input reduces the reliability of the design. Previous studies showed that, a maximum performance in terms of resilient modulus and permanent deformation can be obtained from optimum amount of fines in the gradation (Haithem Soliman, 2015; Tutumluer, 2013). In local studies the permanent deformation resistance of the unbound granular materials have also been found to be improved with optimizing the amount of fines (Haithem Soliman, 2015). However, the resilient modulus performance have not been found to improve (Haithem Soliman, 2015). In order to obtain improved resilient modulus performance of unbound granular materials, other influencing factors such as increased maximum aggregate size, crush count at low fines content should be investigated.

1.2 OBJECTIVES OF THE RESEARCH

The aim of the research is to investigate the structural response, drainability and long term performance of locally available virgin and recycled unbound granular materials in order to characterize them more accurately. The specific objectives of the research are:

- Investigate the effect of aggregate physical and morphological properties such as gradation, maximum size and shape (crush count) on the resilient modulus performance.
- Examine the long term permanent deformation performance of several locally available virgin and recycled unbound granular materials in Manitoba and investigate the effect of physical and morphological properties on the long term performance of the base layer.
- Predict the long term performance of unbound granular materials from early age performance under repeated load using the shakedown approach.
- Predict the long term performance of unbound granular materials using simplified approach and compare the prediction with shakedown prediction.
- Evaluate the effect of different conditioning stress on the resilient modulus performance of unbound granular materials.
- Evaluate the permeability of the virgin and recycled unbound granular materials.
- Provide testing results to support updating of the base layer construction specification of Manitoba Infrastructure and City of Winnipeg in order to provide a durable base layer design.

1.3 SCOPE OF THE RESEARCH

Compared to neighbouring jurisdictions, the current specifications in Manitoba are based on a fine gradation below maximum density line, relatively smaller maximum aggregate size, a high percentage of fines of the available materials. A laboratory test program has been developed to investigate the resilient modulus, permanent deformation, permeability properties of virgin and recycled concrete aggregates. Effect of increased maximum aggregate size, increased crush count in virgin aggregates have been investigated in the laboratory testing. The feasibility of using recycled concrete aggregate as base course material and effect of variation of fines have also been investigated from the laboratory results of resilient modulus, permanent deformation and permeability tests.

1.4 SIGNIFICANCE OF THE RESEARCH

The findings from the research can be used as level 1 input parameters in pavement design. Level 1 design input will increase the reliability of the design which will help to optimize the cost of construction. The level 1 input will also facilitate more accurate prediction of the long term performance of the base layer which will help selecting the maintenance and rehabilitation time more accurately. The research can also be used to calibrate MEPDG empirical models. The calibrated models can be used to develop level 2 input data for future design whenever the laboratory test data is not available. In addition, the recommendations from the research will provide test results which can be used to develop performance based specifications.

1.5 THESIS ORGANIZATION

The organization of the thesis is as follows:

- Chapter 1 Introduction

  This chapter summarized the general background, problem statement, objective of the research, scope of the research, and the significance of the research.

- Chapter 2 Literature Review

  This chapter presents the review of the behaviour of the unbound granular materials under repeated load, the factors affecting the performance of the unbound granular materials and the test procedure in order to characterize the materials.

- Chapter 3 Review of Specifications of Unbound Granular materials

This chapter summarizes an environmental scan of the current specifications of Manitoba and neighbouring states and provinces. The chapter also summarizes some specifications of using recycled concrete aggregates in base layer construction.

- Chapter 4 Materials and Testing Program

  This chapter summarizes the material properties, sampling procedure, laboratory test procedures for characterization of the behaviours of unbound granular materials.

- Chapter 5 Results and Discussion

  This chapter discusses the test results from resilient modulus, permanent deformation and permeability. The test results have been interpreted in order to characterize the behaviour of the unbound granular materials. Shakedown behaviour, dissipated energy during permanent deformation and effect of different conditioning stress before resilient modulus test has also been discussed in this chapter.

- Chapter 6 Conclusions and Recommendations

  This chapter summarizes the results of study, the conclusions and recommendations for future works.

Chapter 2 Literature Review

2.1 INTRODUCTION

Unbound granular materials are used in the construction of base/subbase layers of pavements which provides structural capacity to pavements. It is also used as a working platform for the overlaying layers (Saeed, 2008). Unbound granular materials should have the stiffness to transfer the load coming from the overlaying layers to the subgrade. Therefore the structural capacity of an unbound granular layer is one of the most important considerations in pavement design. In addition, permeability of a pavement layer is also one of the important parameters in pavement structural design. Unbound granular materials layer acts as a drainable layer because of the permeable nature of the material. Better drainage removes excess moisture from base layers quickly which improves the structural stiffness reducing the generation of excess pore water pressure. Moreover, throughout the life cycle of a pavement, the layers experiences traffic load cycles. Therefore, the unbound granular materials must be able to sustain repeated action of traffic loading over its life cycle. The gradual accumulation in permanent deformation results in rutting of the pavement layers.

The study is focused on the stiffness performance, long term permanent deformation performance as well as the permeability of the unbound granular materials. Resilient modulus, permanent deformation, California bearing ratio and permeability performance has been investigated as the performance measuring parameters of various locally available unbound granular materials.

2.2 BEHAVIOUR OF UNBOUND GRANULAR MATERIAL UNDER REPEATED LOAD

2.2.1 Stresses in Base Layers

In pavement layers, stress pulses having varying magnitude of vertical, horizontal and shear stress components are induced due to movement of traffic. The magnitude of vertical, horizontal and shear stresses on base layer also depend on the design depth and material properties of the overlaying layers. The state of stress and their magnitude on any element in the base layer also changes with the motion of wheel load. Figure 2-1 shows the different state of stress and the principal stresses on a base layer element with the changes in wheel load position.

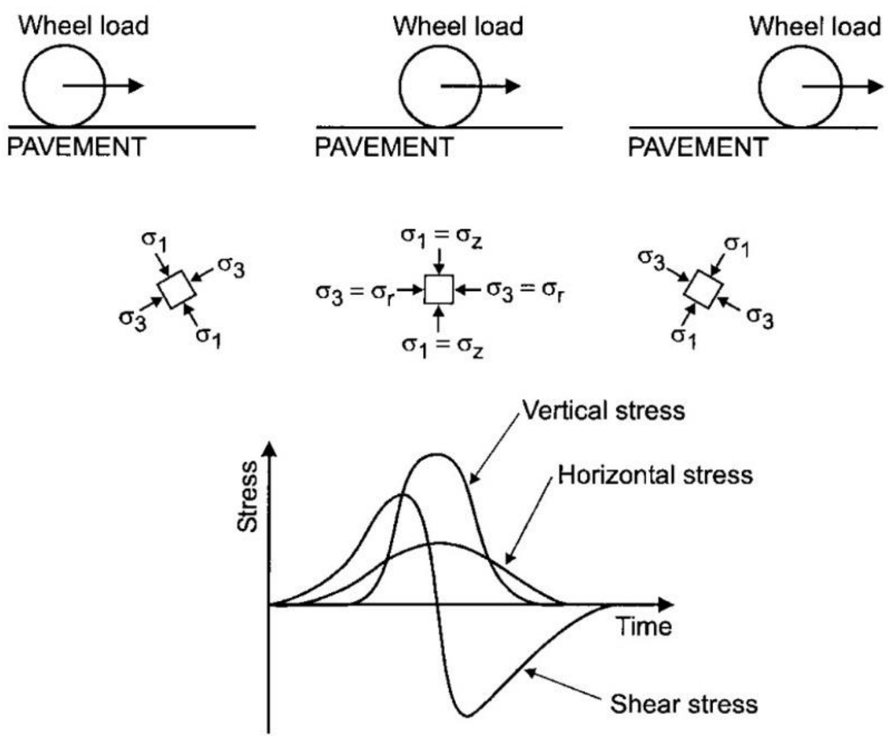


Figure 2-1 Various stress state of an element beneath wheel load (σ is the stresses on the soil element) (Lekarp et al., 2000a).

2.2.2 Deformation Response of Base Layers under Induced Stresses

Many researchers have conducted studies on the deformation behaviour of unbound granular materials subjected to repeated load (Arnold, 2004; Lekarp, Richardson, & Dawson, 1996; Thom & Brown, 1988; Sabine Werkmeister, Numrich, & Dawson, 2003). The deformation of the layer of the pavements can be characterized into two types, resilient deformation and permanent deformation. The unbound granular materials have been characterized as an elastoplastic material by several researchers (Chazallon, Koval, Hornych, Allou, & Mouhoubi, 2009; Habiballah & Chazallon, 2005). The granular materials undergo deformation under each loading cycles. A part of the deformation is recovered due to the elastic behaviour of the material. The remaining unrecovered part is called the permanent deformation (Figure 2-2). The stress strain behaviour of unbound granular materials can be plotted as a nonlinear hysteresis loop. The permanent deformation of unbound granular materials per load cycle reduces with the load application (Arnold, 2004; Sabine Werkmeister et al., 2003). The permanent deformation under repeated loading are mainly caused by three mechanisms: consolidation, distortion and attrition (Lekarp et al., 2000a; Luong, 1982). Rearranging and reorientation of the particle structure results in volumetric reduction in the consolidation stage (Kancherla, 2004). The distortion is governed by the microscopic interlocking of particles. Rolling and sliding occurs into this stage which is resisted by the inter particle friction (Kancherla, 2004). Attrition stage occurs when the applied load exceeds the strength of material, which results in crushing and breaking of particles (Kancherla, 2004). The breakage and crushing of particles are called degradation which affects the long term performance of the layer changing the particle size distribution (Kancherla, 2004).

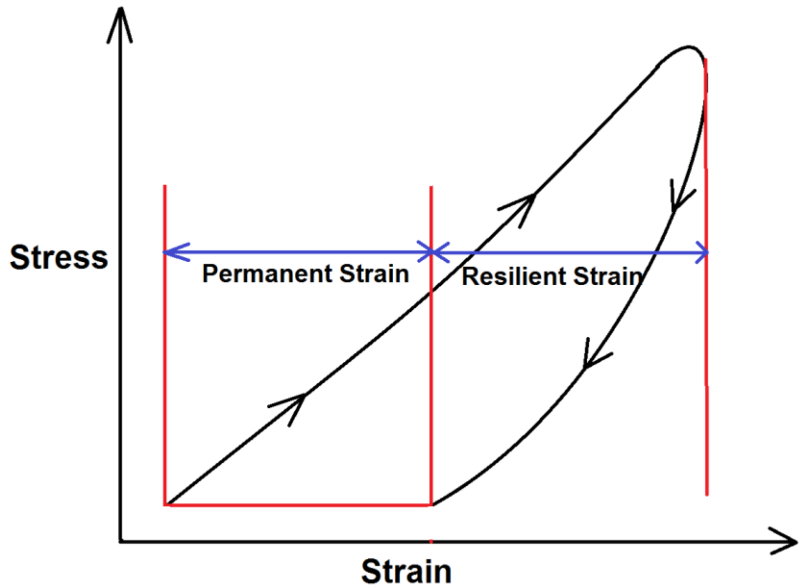


Figure 2-2 Stress strain hysteresis loop behaviour of Unbound Granular Materials under one cycle of load.

The deformation resistance of unbound granular materials have also been reported as a function of applied stress (Sabine Werkmeister et al., 2003). Under repeated loading, the unbound granular materials undergoes two major behaviours; Strain hardening and strain softening (Figure 2-3). The strain hardening happens at low stress level where the granular particles migrate to find stable and interlocked positions in a denser state caused by improved particle packing. At a higher stress level, increased volumetric strain causes strain softening (Sabine Werkmeister et al., 2003). The increased volumetric strain in densely packed material is caused by increased shear stress. The phenomenon happens because, the shear force squeezes and pushes the granular particles to climb onto other particle (Kancherla, 2004; Van Niekerk, 2002).

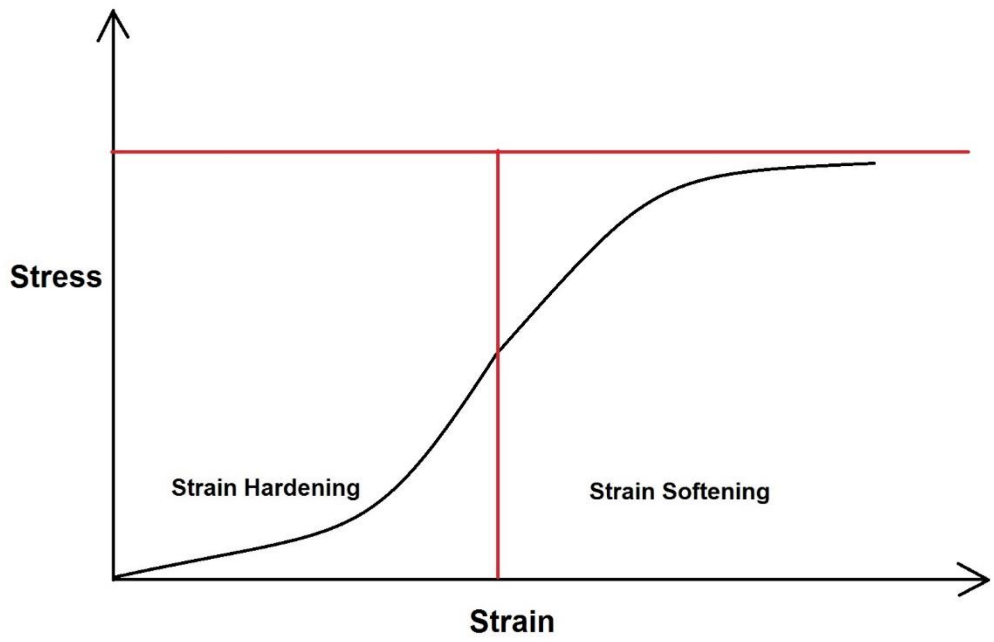


Figure 2-3 Stress Strain Behaviour of unbound granular materials (Sabine Werkmeister et al., 2003).

2.2.3 Shakedown Behaviour of Unbound Granular Materials

Shakedown theory has been widely used to characterize the behaviour of unbound granular materials under repeated loading (Austin, 2009; Cerni, Cardone, Virgili, & Camilli, 2012; Tao, Mohammad, Nazzal, Zhang, & Wu, 2010). Shakedown is an adaptation process where the accumulated permanent deformation stops increasing and the material responds elastically with the number of load repetitions (Kancherla, 2004). A pavement structure fails after an accumulation of permanent deformation or degradation of materials rather than a rapid failure (Sharp & Booker, 1984; Haithem Soliman, 2015). A pavement having a higher shakedown limit and subjected to less number of load repetitions would have a longer service life (Sharp & Booker, 1984). For a given material type, layer thickness and environmental condition, the shakedown limit and the critical load for a pavement can be determined (Sabine Werkmeister et al., 2003). The elastoplastic behaviour of unbound granular materials under repeated load can be categorized into three ranges as illustrated in Figure 2-4.

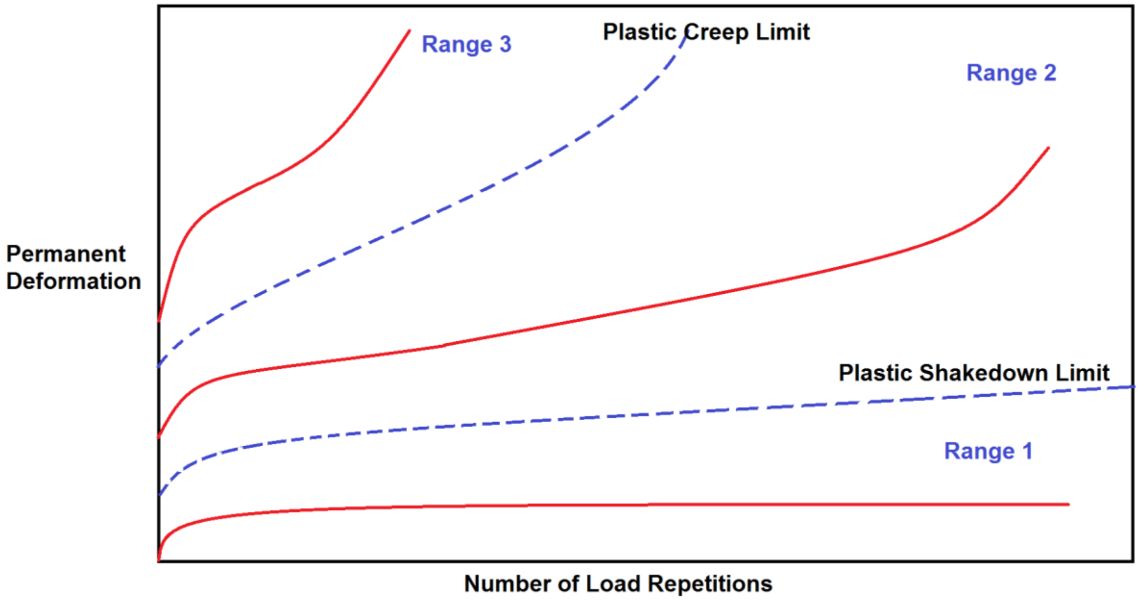


Figure 2-4 Shakedown ranges of permanent deformation behaviour of Unbound Granular Materials (Cerni et al., 2012).

- Range 1 (Plastic Shakedown): In this range, the unbound granular material response is plastic for some finite number of loading cycles at low stress level. However, the response becomes purely resilient followed by some initial plastic deformation.

- Range 2 (Plastic Creep): The materials response is initially similar to the response in range 1. However, the response in this range does not become purely resilient. Small rate of accumulation occurs with application additional of loading cycles over the plastic shakedown range. The material can experience incremental collapse after several millions of load application depending on the materials physical properties.

- Range 3 (Incremental Collapse): A high rate of permanent deformation with loading application is observed. The material undergoes an incremental collapse after some finite number of load application.

A desirable unbound granular layer should have a permanent deformation response in Range 1. Range 3 response is undesirable. The materials behaving in Range 2 should be designed and the permanent deformation estimation should be made with a rational precision (Sabine Werkmeister et al., 2003). The shakedown range can be calculated from repeated load triaxial test. The limit of shakedown ranges has been reported by (Sabine Werkmeister et al., 2003). The boundary of Ranges 1 and 2 has been found to be 4.5 x 10^{-5} mm/mm vertical permanent strain accumulated from 3,000 to 5,000 loading cycles and the boundary of Ranges 2 and 3 has been found to be 4.0 x 10^{-4} mm/mm vertical permanent strain accumulated from 3,000 to 5,000 loading cycles (Figure 2-4).

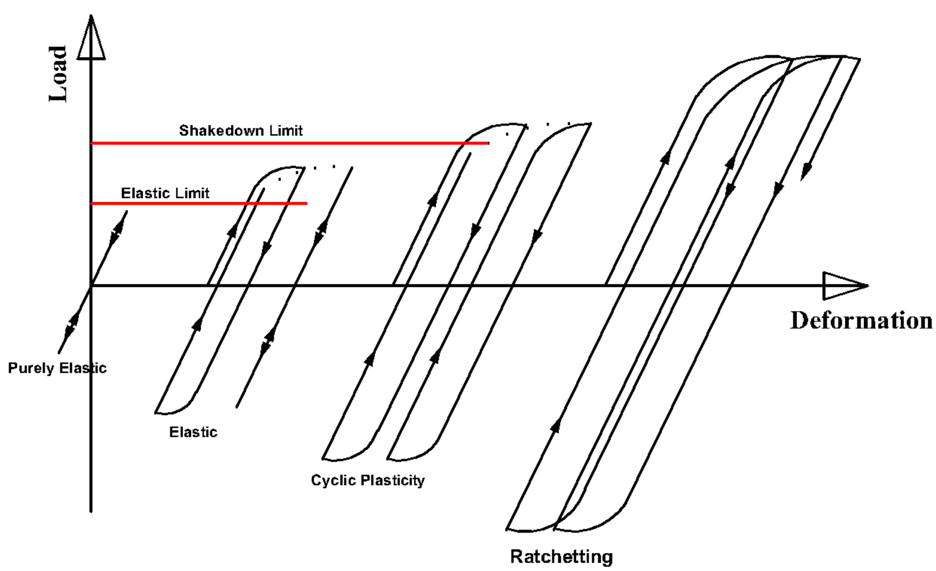


Figure 2-5 Shakedown behaviour of unbound granular materials under repeated loading (Johnson, 1987).

The shakedown process in unbound granular materials is also dependent on the stress level (Figure 2-5) (Johnson, 1987). The unbound granular materials does not have any plastic deformation with application of load if the applied load is within elastic range (Haithem Soliman, 2015). However, if the elastic limit is exceeded, initially the material experiences some plastic deformation. The material behaves purely resilient after some finite number of load applications. However, for a load above the shakedown limit, the material behaves either as cyclic plasticity or as ratchetting (Barber & Ciavarella, 2000). The failure of the material will be with fatigue in cyclic plasticity stage and with static plastic collapse in ratchetting stage (Barber & Ciavarella, 2000; Haithem Soliman, 2015).

2.3 FACTORS AFFECTING THE PERFORMANCE OF UNBOUND GRANULAR MATERIALS

2.3.1 Moisture Content

Pavement layers are often exposed to moisture. Presence of moisture is to facilitate the compaction at its maximum dry density. Deformation properties of unbound granular materials changes with the presence of moisture in between the voids of granular materials. The simulated laboratory performance and the in situ performance of unbound granular materials are affected by degree of saturation (Lekarp et al., 2000a). At complete saturation state, where all the air voids are filled up with water, the resilient response and permanent deformation resistance response are affected negatively (Smith & Nair, 1973; Vuong, 1992). Effect of moisture content increase on the resilient modulus of granular base materials have been reported in several studies (Barksdale & Itani, 1994; Haynes & Yoder, 1963; Heydinger, Xie, Randolph, & Gupta, 1996; Hickes, 1971). Excess pore water pressure is generated at the saturated condition during repeated loading. The generation of this excess pore water pressure decreases the stiffness of the material by reducing the effective stress (Lekarp et al., 2000a). The effect of pore water pressure controls the deformation behaviour and the degree of saturation controls the material behaviour. In effective stress analysis, the resilient material representing the stiffness of a material remains nearly unchanged whereas in total stress analysis, the modulus is affected because of increased excess pore water pressure (Seed, Mitry, Monismith, & Chan, 1965a). Furthermore, some argument has also been presented that, increasing degree of saturation will induce a lubricating effect among the larger particles which would increase the permanent deformation without affecting the excess pore water pressure generation at drained condition (Thom & Brown, 1987). In drained tests of crushed limestone at varying load frequencies 0.1-3 Hz, no noticeable chance up to the 85% saturation was found (Thom & Brown, 1987). Therefore, the only variable to reduce resilient modulus was the increasing lubricating effect among particles with increase in moisture content. Moreover, the higher moisture content decreases the localized suction thus reduces the inter particle contact force (Lekarp et al., 2000a). The effect of moisture on the resilient behaviour of a material has been found to be more significant in a well graded material having high proportion of fines. The materials gradation having less proportion of fines is more permeable. A permeable layer drains water quickly thus reduces the chance of generating excess pore water pressure (Lekarp et al., 2000a). At a moisture content below the optimum moisture content, the resilient modulus increases because of generation of suction in the material which increases the effective stress whereas, the effect becomes opposite beyond optimum moisture content (Lekarp et al., 2000a). The effect of moisture has been simulated by heavy vehicle simulator in situ which revealed that, the increase in moisture increases the rutting potential (Maree, Freeme, Van Zijl, & Savage, 1982). Therefore, a significant improvement of resilient modulus and the resistance to permanent deformation is possible by using freely drainable and stable material.

2.3.2 Gradation, Maximum Aggregate Size and Fine Content

The density of unbound granular materials mix is dependent on the packing of the different proportions of particles in a gradation. The maximum dry density of a gradation depends on the moisture content which might also be a driving factor to affect the response of the granular layer under repeated load. Therefore, there is an indirect effect of gradation on the performance of the granular aggregates due to change in the moisture content for different base material construction. This indirect effect can be controlled by designing a free draining unbound layer. Particle size distribution of granular materials is considered to have a minimal significant effect on the performance, although some researchers reported that, there is negligible effect of the type of gradation on the resilient performance of the material (Lekarp et al., 2000a). The uniformly graded aggregates have been reported to be stiffer than well graded aggregates (Thom & Brown, 1988). For aggregates having similar gradation and amount of fines, the resilient modulus and permanent deformation resistance increases with increase in particle maximum size (Gray, 1962). This increase in stiffness is due to the skeleton formation of larger particles, which transmits the major amount of stresses. Moreover, due to the presence of larger particles in the gradation, smaller number of particle contact results in less permanent deformation.

The fines fill the voids among the large particles. However, the increase in fines interfere the interaction among the larger aggregate by reducing the contact among coarse aggregate matrix. The resilient modulus and resistance to permanent deformation decrease with increase in fine content (Thom & Brown, 1987). However, it has also been observed that, for partially crushed coarse aggregates the resilient modulus reduced with increase in fines content whereas, when the aggregates are fully crushed, the behaviour has been reported to be opposite (Hickes, 1971). Minor influence of fine content on resilient modulus has been reported by a researcher within a range of

2-10% fine content (Hickes, 1971). On the other hand, about 60% drop in resilient modulus have been reported while increasing the fines into the gradation from 0 to 10% (Barksdale & Itani, 1994). The reduced performance is encountered because of the increase in fines in presence of moisture. Investigation on increasing clayey fines with crushed aggregate showed an initial increase and considerable decrease in resilient modulus afterwards (Jorenby & Hicks, 1986). The increase in stiffness is due to the filling of the voids and the considerable decrease in stiffness is due to the displacement of larger particles. The coarse aggregate and fine aggregate blend should be balanced in order to obtain the optimum density with maximum interaction among the coarse portion of the blend.

2.3.3 Aggregate Shape and Aggregate Surface Texture

Crush count is defined as the percent of coarse aggregates having at least one or two crushed faces. Crushed aggregates have a higher load spreading capacity and higher resilient modulus and less permanent deformation in comparison to rounded uncrushed particles because of having angular shaped particles (Allen & Thompson, 1974; Barksdale & Itani, 1994; Hickes, 1971; Thom & Brown, 1987). The rough textured and angular particles tends to interlock together by locking together with friction whereas the rounded particles slides past each other because of smooth surface (Kim, 2004). Crushed gravel base materials have also been found to have higher resilient modulus than crushed limestone (Heydinger et al., 1996). The natural rounded aggregates can be crushed to increase the aggregate interlock. It has also been found that, at low bulk stress the resilient modulus of crushed aggregated is 50% higher than uncrushed aggregates and at high bulk stress the resilient modulus is 25% higher (Barksdale & Itani, 1994). Flaky particles are subjected to greater amount of particle breakage, abrasion resulting in higher permanent deformation and lower resilient modulus (Selig & Roner, 1987). The increasing particle angularity and surface texture impacts the poison's ratio by increasing the resistance to lateral deformation as a result of better aggregate interlock (Hickes, 1971). Figure 2-6 shows the texture, form and angularity of an aggregate.

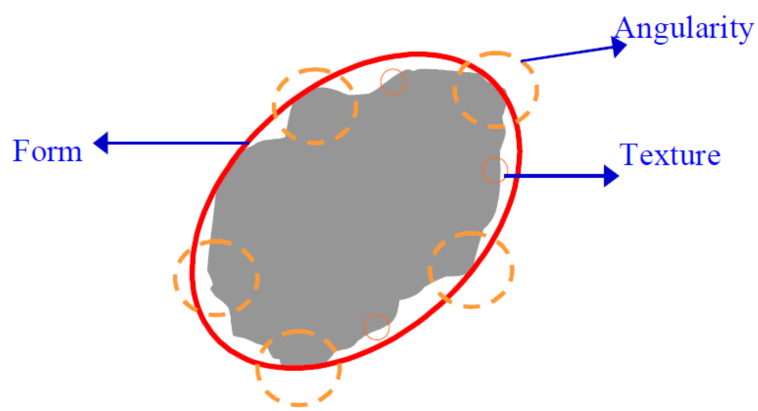


Figure 2-6 Aggregate Shape Properties (Kim, 2004; Masad, 2003).

2.3.4 Stress Level

Stress level is one of the most important factors which have significant impact on the performance of unbound granular materials. Resilient modulus performance of an unbound granular material is highly dependent on confining pressure and bulk stress (Seed et al., 1965a; Smith & Nair, 1973; Uzan, 1985). An increase in confining pressure of 10 to 200 kPa, the resilient modulus increased as great as 500% (Seed, Mitry, Monismith, & Chan, 1965b). For an increase in bulk stress from 70 to 140 kPa, resilient modulus increased about 50% (Smith & Nair, 1973). Compared to deviator stress, which is more dominating factor for subgrade stiffness, confining pressure and bulk stress have been found to have more influence on the stiffness of unbound granular base materials (Lekarp et al., 2000a). Resilient modulus under confining pressure has been found to decrease slightly with increase in repeated deviator stress (Morgan, 1966). On the other hand, at low deviator stress a slight softening and at higher deviator stress slight stiffening has been also observed (Hickes, 1971).

A direct dependency of cumulative permanent deformation has been found with number of load cycles and deviator stress level at a constant confining pressure (Morgan, 1966). However, the cumulative permanent deformation is inversely proportional to the confining stress level at a steady deviator stress level (Morgan, 1966). The permanent deformation increases with increase in deviator stress and with decrease in confining pressure (Barksdale, 1972a). The permanent deformation is dependent on the length of stress path and the stress ratio (Pappin, 1979). Shear strength behaviour of unbound granular materials have also been related with permanent deformation behaviour in some studies. However, the failure under repeated load has been described to be a gradual process by accumulation of permanent deformation which is unlike the static shear failure (Lekarp & Dawson, 1997).

2.3.5 Density

Increasing density of a material causes a material to become stiffer and stronger. The resilient modulus is generally increased with the increase in density (Hickes, 1971). With the increased degree of compaction, the number of particle contacts per particle increases resulting in decrease of average contact stress between particles (Lekarp et al., 2000a). As a result, the reformation of the particulate system decreases and the resilient modulus increases. However, there is relatively insignificant effect of density on deformation behaviour of granular materials as compared to other factors (Thom & Brown, 1988). The effect of density has been found to be higher in partially crushed aggregates compared to fully crushed aggregates (Hickes, 1971). The effect of change in density decreases with the increase in fine content into the granular materials. The effect of density on resilient modulus is more noticeable at lower stress level compared to high stress level

(Barksdale & Itani, 1994). The effect of compaction density also has effect on the long term performance of granular materials (Allen & Thompson, 1974; Lekarp et al., 2000a; Thom & Brown, 1988). Resistance to permanent deformation appears to be highly improved as a result of increase in density of granular materials. It is also observed that, there is an average 185% increase of permanent deformation for several materials while the materials are compacted at 95% of maximum dry density instead of 100% (Barksdale, 1972a). Furthermore, A 80% reduction in permanent deformation for limestone and 22% reduction in permanent deformation for gravel have been reported while the materials are compacted at modified proctor maximum dry density instead on the standard proctor maximum dry density (Allen & Thompson, 1974).

2.3.6 Stress History and Number of Load Cycles

The effect of stress history and number of load repetitions have a direct relationship to the permanent deformation (Lekarp, Isacsson, & Dawson, 2000b). The stress history also has some effect on resilient modulus (Lekarp et al., 2000a). Repeated effect of stress results in progressive densification of the granular materials and rearrangement of the particles. From the repeated load triaxial test on well graded limestone, it was observed that, there is effect of stress history on the material which can be eliminated by preloading the sample with a few cycles and avoiding the high stress ratio response during the resilient modulus test (J. R. Boyce, Brown, & Pell, 1976). The effect of stress history can be eliminated and a steady and stable resilient modulus can be achieved after application of approximately 100 cycles of same stress amplitude (Allen & Thompson, 1974; Hickes, 1971). It has also been suggested to condition the sample for approximately 1000 cycles of loads before the resilient modulus test (Allen & Thompson, 1974). On the contrary, other researchers stated that, the resilient characteristics of unbound granular materials is insensitive to the stress history (Mayhew, 1983). The resilient modulus increases with the increased number of load repetitions which is mainly because of the loss of moisture from the specimen during testing (Moore & Britton, 2004). Furthermore, it has also been observed that, the resilient properties of granular materials remains same after 50-100 load repetitions and after 25,000 load repetitions (Hickes, 1971).

The number of load repetitions is one of the most important factors for the analysis of the long term behaviour of a material. Many researchers reported a continuous increase in permanent deformation with the increased number of load repetitions (Barksdale, 1972b; Morgan, 1966; Sweere, 1990). It has also been reported that, the permanent deformation was still increasing with the number of load repetitions even after 2,000,000 load repetitions (Morgan, 1966). Permanent deformation accumulates linearly with the logarithmic increase in the number of load repetitions (Barksdale, 1972b). The equilibrium in permanent deformation is reached after approximately 1,000 load repetitions (Paute, Hornych, & Benaben, 1996). The rate of accumulation of permanent deformation is decreased constantly with increased number of application of load (Paute et al., 1996). Whereas, the permanent deformation rate has also been observed to be dependent on the stress level (Lekarp & Dawson, 1998). Even though, the stabilization of deformation rate is reached for a lower stress level, the material can still undergo progressive deformation and reach failure at higher stress level (Lekarp & Dawson, 1998). Since the material already reached to a stable condition may become unstable again under further loading, therefore material response is not simple.

2.4 RECYCLED CONCRETE AGGREGATES AS UNBOUND GRANULAR BASE LAYER

Recycled concrete aggregate is used as an alternative of virgin aggregate in construction of base layer of pavement. The source of the recycled aggregates are from demolition of concrete pavement and buildings (Gonzalez, 2002). During the crushing, sizing and blending phases of the production of recycled concrete aggregates, it is made sure that the material is clean and free from contamination like steel, wood, soil and organic substances (Gonzalez, 2002). However, the recycled concrete aggregates can be produced from a diversity of concrete debris which can affect the properties of recycled concrete aggregates. Therefore, before application, the physical, chemical and mechanical properties of recycled concrete aggregate should be controlled. The description of physical, mechanical and chemical properties described by the U.S. Department of Transportation has been summarized as follows (Chesner, Collins, & MacKay, 1997).

2.4.1 Physical Properties

Recycled concrete aggregate has similar physical characteristics as virgin aggregate. The recycled concrete aggregates are combination of natural aggregate and mortar. The presence of mortar makes the aggregate surface rougher with low specific gravity and higher absorption compared to virgin aggregates (Chesner et al., 1997). Typical physical properties of recycled concrete aggregates and virgin aggregates have been shown in Table 2.1 (Chesner et al., 1997; Tavakoli, Heidari, & Karimian, 2013).

Table 2.1 Typical physical properties of recycled concrete aggregates and virgin aggregates.

Properties	**Recycled Concrete Aggregate** (Chesner et al., 1997)	**Virgin Aggregates** (Tavakoli et al., 2013)
Specific Gravity		
Coarse Aggregates	2.2 to 2.5	2.4 to 2.9
Fine Aggregates	2.0 to 2.3	
Absorption, %		
Coarse Aggregates	2 to 6	0.2 to 4
Fine Aggregates	4 to 8	0.2 to 2

2.4.2 Chemical Properties

Chemical properties of recycled concrete aggregate depend on the properties of cement paste and properties of aggregates. The pH value of recycled concrete aggregate with water mixture exceeds 11 (Gonzalez, 2002). The alkalinity comes from the cement paste which has calcium-aluminum-silicate compounds and calcium hydroxide which are highly alkaline in nature. In addition, chloride and sulfate ions may be present in recycled concrete aggregate. The presence of sulfate may cause disintegration in the paste due to the expansion. The recycled concrete aggregate is also susceptible to alkali-silica reactivity (ASR) which causes cracking and expansion of mortar paste (Gonzalez, 2002).

2.4.3 Mechanical Properties

Coarse recycled concrete aggregate has been found to have favorable mechanical properties for use as aggregates. Los Angeles abrasion resistance of recycled concrete aggregates have been reported to be higher than that of high quality virgin aggregates (Gonzalez, 2002). California bearing ratio and magnesium sulfate soundness have also been found to be comparable to virgin aggregates. Typical values of mechanical properties of recycled concrete aggregate and virgin aggregate have been shown in Table 2.2 (Chesner et al., 1997; Sabnis, Moo-Young, & Sharma, 2001).

Table 2.2 Typical mechanical properties of recycled concrete aggregates and virgin aggregates.

Properties	**Recycled Concrete Aggregate** (Chesner et al., 1997)	**Virgin Aggregates** (Sabnis et al., 2001)
Los Angeles Abrasion Loss (ASTM C131)		
Coarse Aggregates	20 to 45	20 to 25
Magnesium Sulfate Soundness Loss (ASTM C88)		
Coarse Aggregates	**4 or less**	**3 or less**
Fine Aggregates	Less than 9	6 to 8

California Bearing Ratio (CBR)	94 to 148	130 to 180

2.5 RESILIENT MODULUS TEST

In order to characterize the resilient modulus behaviour of unbound granular materials, several test protocols have been studied. The test procedure highlights and the main differences among long-term Pavement Performance (LTPP) Protocol P46, National Cooperative Highway Research Program (NCHRP) Project 1-28A and AASHTO T 307-99 are discussed into the following sections.

2.5.1 Test Procedure LTTP P46

The Long-Term Pavement Performance (LTPP) program P46 protocol has been developed for determination of resilient modulus of unbound granular base, subbase and subgrade soils under representative state of stress under flexible and rigid pavement. The protocol is partially based on the AASHTO T292-91I test standard. The test procedure is applicable to undisturbed sample of natural and subgrade soil as well as disturbed sample of unbound base and subgrade soils reconstituted into the laboratory. The stress level applied according to this protocol is based on the location of sample in the pavement.

According to LTTP P46 developed under the Long Term Pavement Performance Program (LTPP) test program, the test sample for unbound granular materials have been divided into two types. The test samples are classified as Type 1 (152 mm – 6 inch diameter specimen) and Type 2 (152 mm – 6 inch diameter specimen).

The test specimen has to be conditioned before resilient modulus testing. A minimum 500 to a maximum 1000 repetitions of a load equivalent to a maximum axial stress of 103.4 kPa (15 psi) and a cyclic axial stress of 93.1 kPa (13.5 psi) with a confining pressure of 103.4 kPa (15 psi). The cyclic stress should be a haversine shaped load pulse shown in Figure 2-7, consisting of 0.1 second of load followed by 0.9 second of rest period.

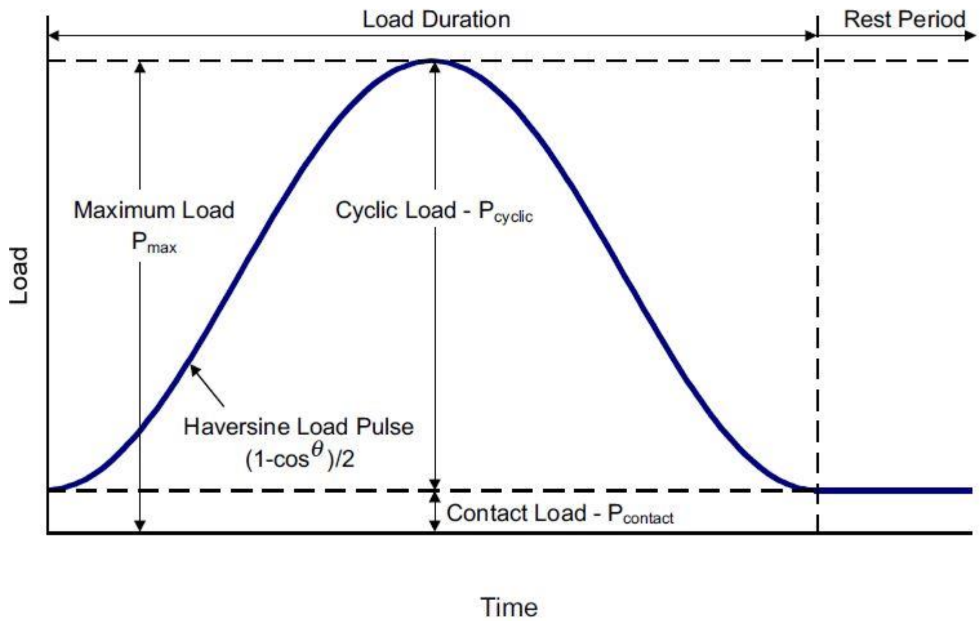


Figure 2-7 Haversine load pulse (National Cooperative Highway Research Program (NCHRP), 2004).

The resilient modulus test is a drained test, therefore the drainage valve should be kept open throughout the test length. The conditioning is applied in order to minimize the effect of imperfect contact between test specimen and loading plate, and test sample and the base plate. According to the LTTP P46 test protocol, the resilient modulus test should be carried out with 15 loading sequences followed by the conditioning. The test sequences have been shown in Table 2.3. The resilient modulus test should be terminated if the deformation of the sample exceeds 5 percent. However, if after the test, the sample does not deform more than 5 percent, a quick shear test should be carried out with a confining pressure of 34.5 kPa (5 psi) with an axial strain rate of 1 percent per minute until the test specimen fails.

Table 2.3 Testing sequence for resilient modulus test of base materials (LTTP P46) (U.S. Department of Transportation, 1996)

Sequence No.	Confining Pressure, S_3	Max. Axial Stress, S_{max}	Cyclic Stress, S_{cyclic}	Contact Stress, $0.1S_{max}$	No. of Load Applications
	kPa	kPa	kPa	kPa	
0	103.4	103.4	93.1	10.3	500-1000
1	20.7	20.7	18.6	2.1	100
2	20.7	41.4	37.3	4.1	100
3	20.7	62.1	55.9	5.2	100
4	34.5	34.5	31.0	3.5	100
5	34.5	68.9	62.0	6.9	100
6	34.5	103.4	93.1	10.3	100
7	68.9	68.9	62.0	5.9	100
8	68.9	137.9	124.1	13.8	100
9	68.9	206.8	186.1	20.7	100
10	103.4	68.9	62.0	6.9	100
11	103.4	103.4	93.1	10.3	100
12	103.4	206.8	186.1	20.7	100
13	137.9	103.4	93.1	10.3	100
14	137.9	137.9	124.1	13.8	100
15	137.9	275.8	248.2	27.6	100

2.5.2 Test Procedure NCHRP Project 1-28A

The test method for determining the resilient modulus of unbound granular materials have been developed under NCHRP Project 1-28A: “Harmonized Test Methods for Laboratory Determination of Resilient Modulus for Flexible Pavement Design”. The harmonized protocol has been developed from the research project NCHRP 1-28A. One of the objectives of the project was to develop a test method for measurement of resilient modulus of unbound granular base, subbase and subgrade materials by harmonizing the existing AASHTO T 294-92, LTTP P46 and AASHTO T 292-91 procedures.

The test procedure is different compared to the LTTP P46 in terms of sample preparation. Unbound granular materials sample should be either 6 inch diameter and 12 inch high or 4 inch diameter and 8 inch high depending on the maximum aggregate size of the gradation. The sample size may be 4 inch if the maximum aggregate size is less than 3/4 inch. Moreover, the compaction during the sample preparation also depends on the maximum aggregate size. The unbound granular materials specimen having maximum aggregate size less than 3/8 inch should be compacted using vibratory hammer instead of using impact compaction. Test procedure Ia is followed for the resilient modulus test of unbound granular materials. Unlike the LTTP P46, the resilient modulus test according to NCHRP 1-28A has 30 sequences in addition to conditioning. The test sequences have been shown in Table 2.4.

Table 2.4 Testing sequence for resilient modulus test of base materials (NCHRP 1-28A) (National Cooperative Highway Research Program (NCHRP), 2004)

Sequence No.	**Confining Pressure**	**Contact Stress**	**Cyclic Stress**	**Total Stress**	**No. of Load Applications**
	kPa	**kPa**	**kPa**	**kPa**	
Conditioning	103.4	20.7	207.0	227.7	1000
1	20.7	4.1	10.4	14.5	100
2	41.4	8.3	20.7	29.0	100
3	69.0	13.8	34.5	48.3	100
4	103.5	20.7	51.8	72.5	100
5	138.0	27.6	69.0	96.6	100
6	20.7	4.1	20.7	24.8	100
7	41.4	8.3	41.4	49.7	100
8	69.0	13.8	69.0	82.8	100
9	103.5	20.7	103.5	124.2	100
10	138.0	27.6	138.0	165.6	100
11	20.7	4.1	41.4	45.5	100
12	41.4	8.3	82.8	91.1	100

13	69.0	13.8	138.0	151.8	100
14	103.5	20.7	207.0	227.7	100
15	138.0	27.6	276.0	303.6	100
16	20.7	4.1	62.1	66.2	100
17	41.4	8.3	124.1	132.4	100
18	69.0	13.8	207.0	220.8	100
19	103.5	20.7	310.5	331.2	100
20	138.0	27.6	414.0	441.6	100
21	20.7	4.1	103.5	107.6	100
22	41.4	8.3	207.0	215.3	100
23	69.0	13.8	145.0	358.8	100
24	103.5	20.7	517.5	538.2	100
25	138.0	27.6	690.0	717.6	100
26	20.7	4.1	144.9	149.0	100
27	41.4	8.3	289.8	298.1	100
28	69.0	13.8	483.0	496.8	100
29	103.5	20.7	724.5	745.2	100
30	138.0	27.6	966.0	993.6	100

2.5.3 Test Procedure AASHTO T 307

In this test procedure the materials are classified as either Type 1 or Type 2 depending on the percent passing the 2.00 mm (No. 10) sieve and 0.075 mm (No. 200) sieve and the plasticity index of the material. The test is carried out with a cyclic stress consisting of 0.1 second of load followed by 0.9 second to 3.0 seconds of rest period. The samples should be prepared using split mold and vibratory compactor. The minimum diameter of the split mold should be five times the maximum aggregate size. However, the larger particles should be scalped if the maximum aggregate size exceeds 25 percent of largest mold diameter. Type 1 materials specimen should be at least 150 mm in diameter whereas the minimum specimen diameter of Type 2 material should be 71 mm. Unlike the compaction procedure of NCHRP 1-28A, AASHTO T 307 type 1 material is compacted in 6 layers having 50 mm each. The resilient modulus test procedure of AASHTO T 307 consists of 15 loading sequences in addition to the conditioning. The test sequences have been shown in Table 2.5. A quick shear test procedure similar to LTTP P46 is specified into AASHTO T 307.

Table 2.5 Testing sequence for resilient modulus test of base materials (AASHTO T307) (AASHTO: T 307-99, 2003)

Sequence No.	**Confining Pressure, S_3**	**Max. Axial Stress, S_{max}**	**Cyclic Stress, S_{cyclic}**	**Contact Stress, $0.1S_{max}$**	**No. of Load Applications**
	kPa	**kPa**	**kPa**	**kPa**	
0	103.4	103.4	93.1	10.3	500-1000
1	20.7	20.7	18.6	2.1	100
2	20.7	41.4	37.3	4.1	100
3	20.7	62.1	55.9	5.2	100
4	34.5	34.5	31.0	3.5	100
5	34.5	68.9	62.0	6.9	100
6	34.5	103.4	93.1	10.3	100
7	68.9	68.9	62.0	5.9	100
8	68.9	137.9	124.1	13.8	100
9	68.9	206.8	186.1	20.7	100
10	103.4	68.9	62.0	6.9	100
11	103.4	103.4	93.1	10.3	100
12	103.4	206.8	186.1	20.7	100
13	137.9	103.4	93.1	10.3	100
14	137.9	137.9	124.1	13.8	100
15	137.9	275.8	248.2	27.6	100

2.5.4 Regression Models of Resilient Modulus

Among all the significant parameters affecting resilient modulus behaviour of granular materials, the effect of stress parameters are most significant (Lekarp et al., 2000a). Therefore, it’s important to model the stress strain relationship for better prediction of resilient modulus. The simplest approach to express the stress stiffness relationship is modulus as a function of the sum of principle stresses. A simple hyperbolic relationship between modulus and bulk stress is known as K-θ model suggested by (J. R. Boyce et al., 1976; Hickes, 1971; Seed et al., 1965a) as follows:

$$M_R = K_1 \left[\frac{\theta}{P_a}\right]^{K_2} \tag{2.1}$$

Where,

$M_R = Resilient\ Modulus, MPa$

$\theta = Bulk\ stress, kPa = \sigma_1 + \sigma_2 + \sigma_3$

The model is widely accepted because of its simplicity. However, a drawback of the model is that, the model assumes a constant Poison's ratio, which is used to calculate the radial strain (Lekarp et al., 2000a). Many studies have shown that, the Poison's ratio varies with the variation of applied stress (H. R. Boyce, 1980; Hickes, 1971; Huang, Tsai, Lin, Wang, & Road, 2010; Kolisoja, 1997; Sweere, 1990). Moreover, another drawback of the model is, it accounts only bulk stress as a function of resilient modulus. Many studies shown that, solely bulk stress is insufficient to predict the resilient modulus (Lekarp et al., 2000a). Studies shown that, the resilient modulus is also affected by the magnitude of shear stress or deviator stress induced due to repeated load application (May & Witczak, 1981). The equation (2.2) proposed by (Uzan, 1985) considers the effect of deviator stress. In three dimensional case, the parameter is called octahedral stress. Compared to the K-θ, the model proposed by (Uzan, 1985) have shown better performance in prediction of resilient modulus (Kolisoja, 1997; P V Lade & Nelson, 1987).

$$M_R = K_1 P_a \left[\frac{\theta}{P_a}\right]^{K_2} \left[\frac{\tau_{oct}}{P_a} + 1\right]^{K_3} \tag{2.2}$$

Where,

$M_R = Resilient\ Modulus, MPa$

$\theta = Bulk\ stress, kPa = \sigma_1 + \sigma_2 + \sigma_3$

$\tau_{oct} = Octahedral\ shear\ stress, kPa = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2}$

$\sigma_1, \sigma_2\ and\ \sigma_3 = Principal\ stresses, kPa$

$$K1, K2 \text{ and } K3 = \text{Regression constants}$$

$$Pa = \text{Atmospheric pressure} = 101.35 \ kPa.$$

Nonlinear regression is used to evaluate the values of the regression constants K_1, K_2, and K_3 for. Equation (2.2) and the evaluated regression constants (K_1, K_2, and K_3) can be used to calculate the resilient modulus at any stress state. The resilient modulus test protocol recommends the use of Equation (2.2) to calculate and report the resilient modulus of unbound base materials for confining pressure ($\sigma3$) = 35 kPa and cyclic stress ($\sigma1$- $\sigma3$) = 103 kPa.

2.6 PERMANENT DEFORMATION TEST

Permanent deformation is the irrecoverable deformation or strain which is accumulated through the service life of a pavement. The accumulated permanent deformation results in rutting in flexible pavement layers. Measuring the rutting is relatively simple compared to the accurate prediction of rutting. At a given number of load repetitions under a given stress level, the permanent strain can be calculated as the ratio of the change in height of the material specimen to the original height of the material specimen.

For predicting the performance of a pavement, it is important to understand the possible behaviour of the granular materials under the effect of repeated load. Many researchers have performed numerous efforts to predict the permanent deformation performance of various pavement layers (Deacon, Harvey, Guada, Popescu, & Monismith, 2002; Perez et al., 2006; Rahman & Erlingsson, 2014a; Seed et al., 1965a, 1965b). There are several factors which can affect the resistance to permanent deformation. While investigating the effect of fines into base gradation, several researchers investigated the permanent deformation behaviour of granular materials and concluded that, the resistance to permanent deformation increases with decreasing fine content (Thom & Brown, 1988),(Barksdale, 1972b).

Constitutive models for predicting the permanent deformation with respect to number of load repetitions, with respect to loading condition as well as with respect to shakedown approach have been developed. Among the constitutive models to predict the permanent deformation behaviour, the models as a function of number of load repetitions are more popular among researchers (Perez et al., 2006). Current Mechanistic Empirical Pavement Design Guide also recommends the pavement design model for granular base and subbase, which is also a function of number of load repetitions.

In order to predict the permanent deformation behaviour of base materials, repeated load triaxial test is performed on several base course materials and a lognormal relationship of permanent deformation with number of load cycles have been found (Barksdale, 1972b). Researchers also developed relationship of permanent strain rate and number of load repetitions and stated that, the rate decreases logarithmically with number of load repetitions (Khedr, 1985). Although, others also tried to build a relationship between number of load repetitions and plastic strain in volumetric-share approach, no other verification was found to establish that (Jouve, Martinez, Paute, & Ragneau, 1987). Furthermore, other researcher investigated the long-term permanent deformation response of granular material with repeated load triaxial test and reported a log-log approach to predict permanent deformation (Sweere, 1990).

2.6.1 Regression Models of Permanent Deformation

According to various literatures, Equation 2.3 provides the best fit of permanent deformation for granular base and subgrade soils (Schreuders, 1989). This is the same model used in Mechanistic

Empirical Pavement Design Guide for permanent deformation prediction. ε_o, β and ρ are the regression coefficients for each specimen and ε_{1p} is the percent permanent strain.

$$\varepsilon_{1p}\,(\%) = \varepsilon_o\, e^{-\left(\frac{\rho}{N}\right)^{\beta}} \tag{2.3}$$

Chapter 3 Review of Specifications of Unbound Granular Materials

3.1 INTRODUCTION

Transportation agencies developed and evolved their own construction specifications for base course materials and they vary depending on current practice, locally available materials, landform, climate and drainage. These specifications aim to produce better designed and well performing pavements. The construction specifications of the highlighted jurisdictions have been reviewed in order to compare some of the important performance influencing parameter specifications. The list of reviewed specifications of the North American regions are shown in Table 3.1.

3.2 REVIEW ON CONSTRUCTION SPECIFICATION OF VIRGIN AGGREGATE BASE

Table 3.1 Investigated regions and the name of specifications

State/Province	Name of Specification	Year
British Columbia	Standard Specifications for Highway Construction	2012
Alberta	Standard Specifications for Highway Construction	2010
Saskatchewan	Saskatchewan Highways and Transportation (3500, 310-03, 320-2)	2004
Manitoba	Standard Construction Specifications (No. 700, 900)	2015
Ontario	Ontario Provincial Standard Specification-1010	2003
Alaska	Standard Specifications for Highway Construction	2004
Washington	Standard Specifications for Road, Bridge and Municipal Construction	2014
Nebraska	Standard Specification for Highway Construction	2007

South Dakota	Standard Specifications for Roads and Bridges	2004
North Dakota	Standard Specifications for Road and Bridge Construction	2014
Kansas	Standard Specification for State Road and Bridge Construction	2007
Iowa	Standard Specifications with GS-12005 Revisions	2014
Minnesota	Standard Specification for Construction- Materials Lab Supplement	2014
Louisiana	Standard Specifications for Road and Bridge Construction	2006
Illinois	Standard Specifications for Road and Bridge Construction	2012
Wisconsin	Wisconsin Department of Transportation Standard Specifications	2014

The specification review highlights several factors influencing the performance of unbound granular layers. The maximum aggregate size of a gradation, the crush count of the coarse aggregates, amount of fines (particles passing through #200 or 0.075 mm sieve) in a gradation, and the plasticity of fines. In the province of Manitoba, the specifications require a relatively finer gradation for the unbound granular layer construction compared to neighbouring jurisdictions (H Soliman, Shalaby, & Kass, 2014). Figure 3-1 and Figure 3-2 show the maximum aggregate size of Manitoba Infrastructure and neighbouring jurisdictions. The maximum aggregate size for the A base in Manitoba is 19.0 mm, while the maximum aggregate size varies from 19.0 to 50.0 mm for many of the reviewed jurisdictions. The maximum aggregate size is an important factor influencing the resilient and permanent deformation response.

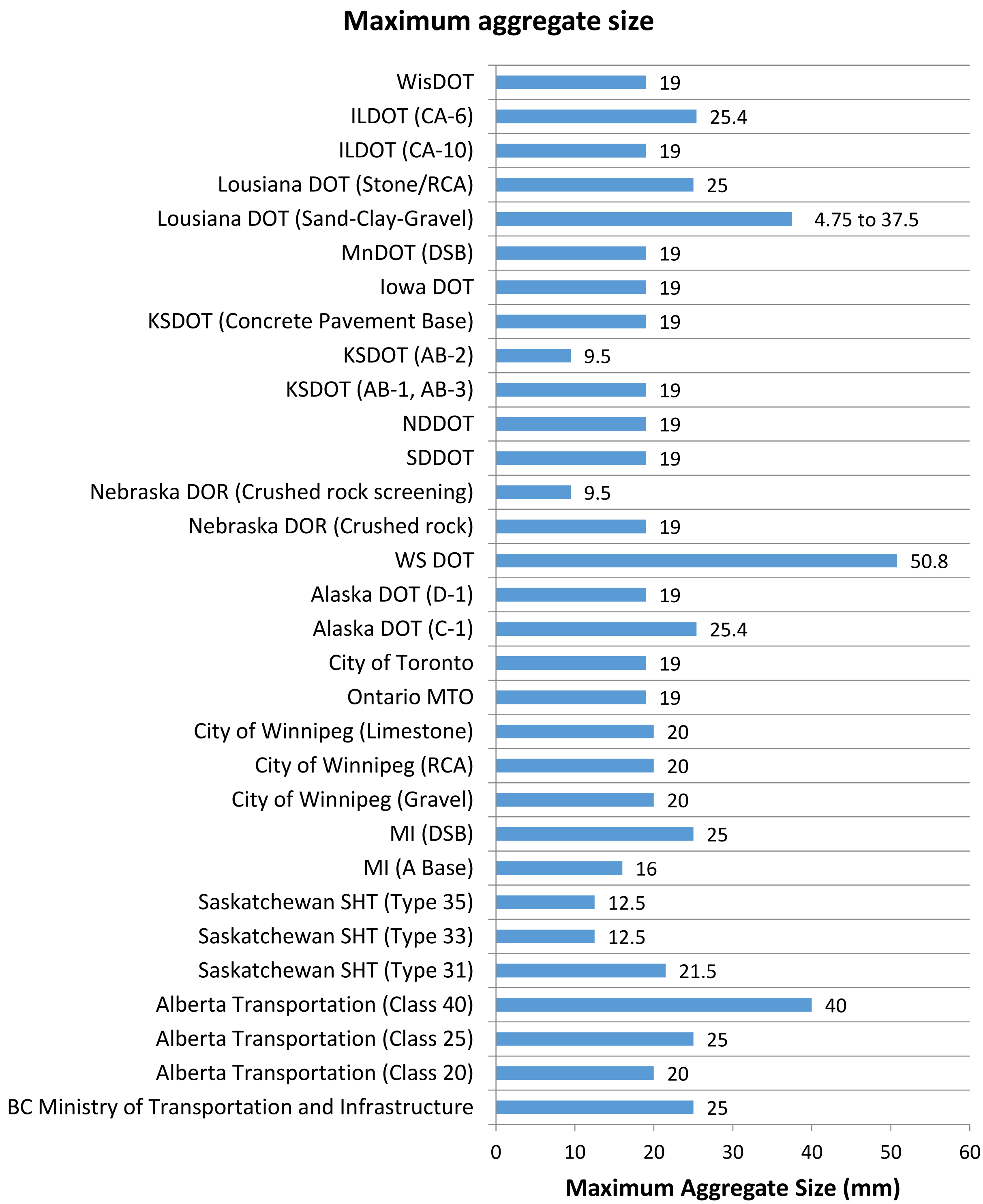


Figure 3-1 Maximum aggregate size of the reviewed specifications.

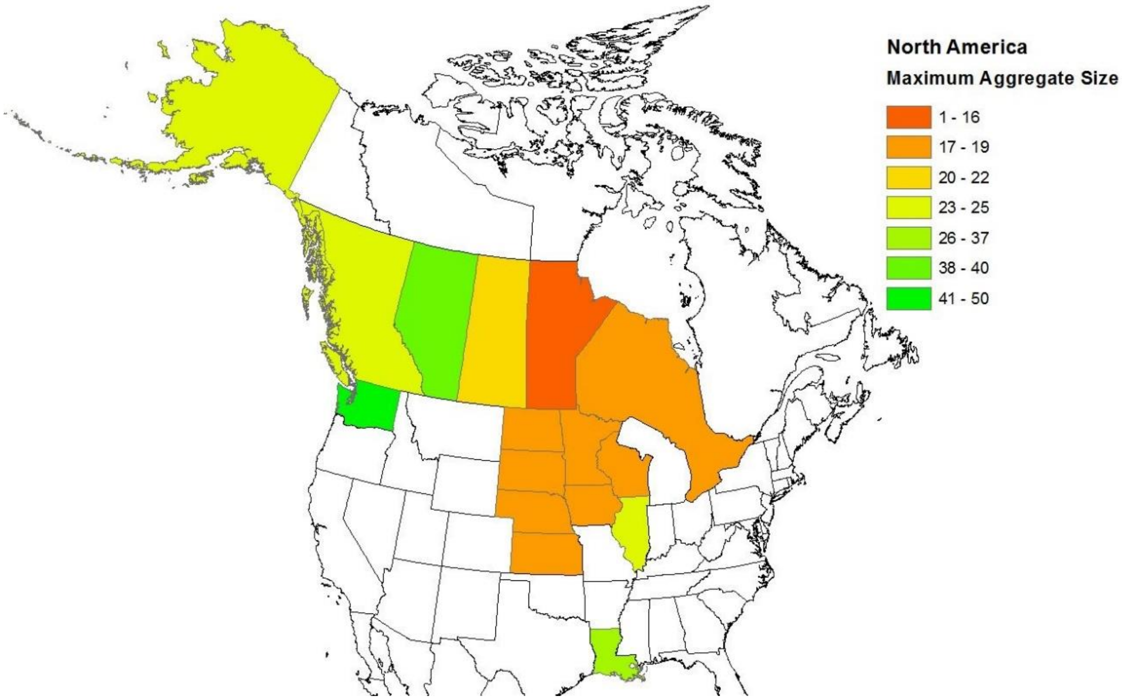


Figure 3-2 Maximum aggregate size of MI specification and several North American specifications

The stiffness of a granular layer is also influenced by the amount of fines permitted in the gradation. Fine particles fill the voids in between larger particles and supports the granular skeleton. Therefore, certain amount of fine is required to obtain higher resilient modulus and to control permanent deformations of unbound granular materials (Tutumluer, 2013). Figure 3-3 and Figure 3-4 shows the maximum fines content (%) of Manitoba Infrastructure base and neighbouring jurisdictions. Manitoba Infrastructure also specifies a relatively higher percentage of fines compared to neighbouring jurisdictions.

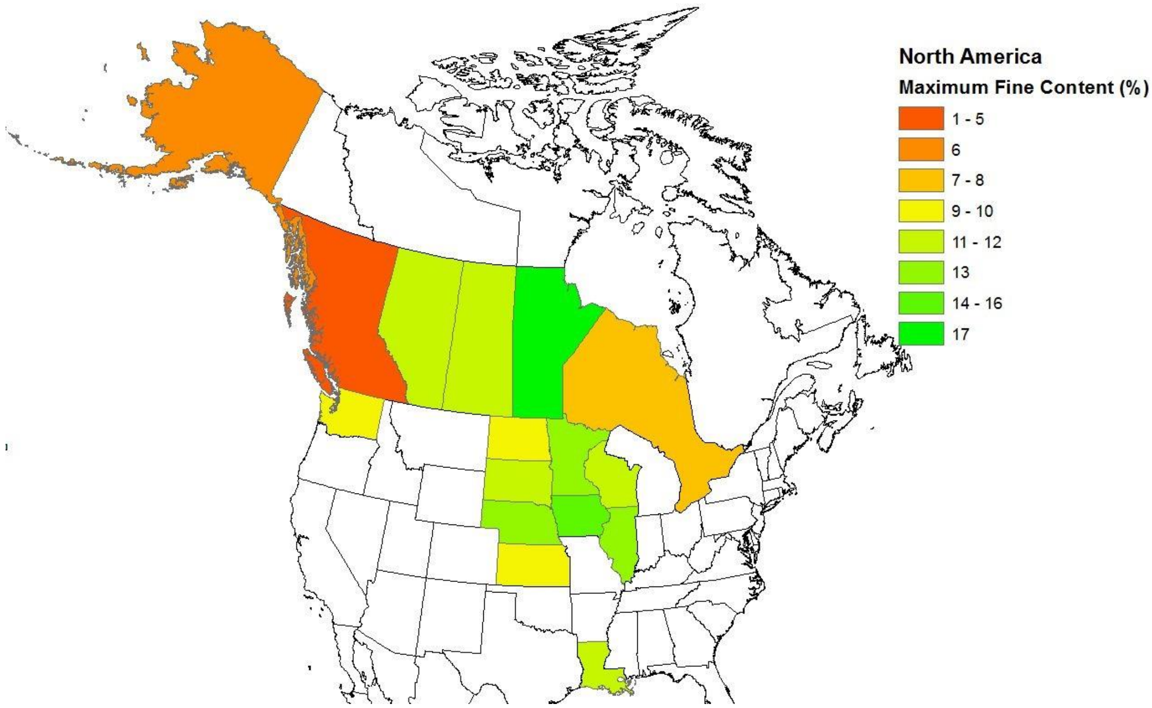


Figure 3-3 Maximum fine content (%) of MI specification and several North American specifications

An important morphological characteristic that influence the stiffness of unbound materials is the angularity of the coarse aggregates (Pan et al., 2006; Tutumluer & Pan, 2008). Angular shaped aggregates perform better compared to the rounded aggregates because of the interlocking effect (Barksdale & Itani, 1994; Poul V. Lade, Yamamuro, & Bopp, 1996; Tutumluer & Pan, 2008). The crushed particles can be at least one freshly fractured faces or two fractured faces depending on the specification. Figure 3-5 and Figure 3-6 show the specified percent crush count (%) of Manitoba Infrastructure base coarse aggregates and neighbouring jurisdictions. The province of Manitoba specifies at least 35 percent of fractured aggregates in the A base gradation. Other jurisdictions specified crush count varying from 10 to 100 percent. North Dakota state specifies a minimum 10 % crush count for their reclaimed aggregates as base. However, the state has alternative gradations having higher crush count requirement which is used as concrete aggregate but can be used as base materials if specified.

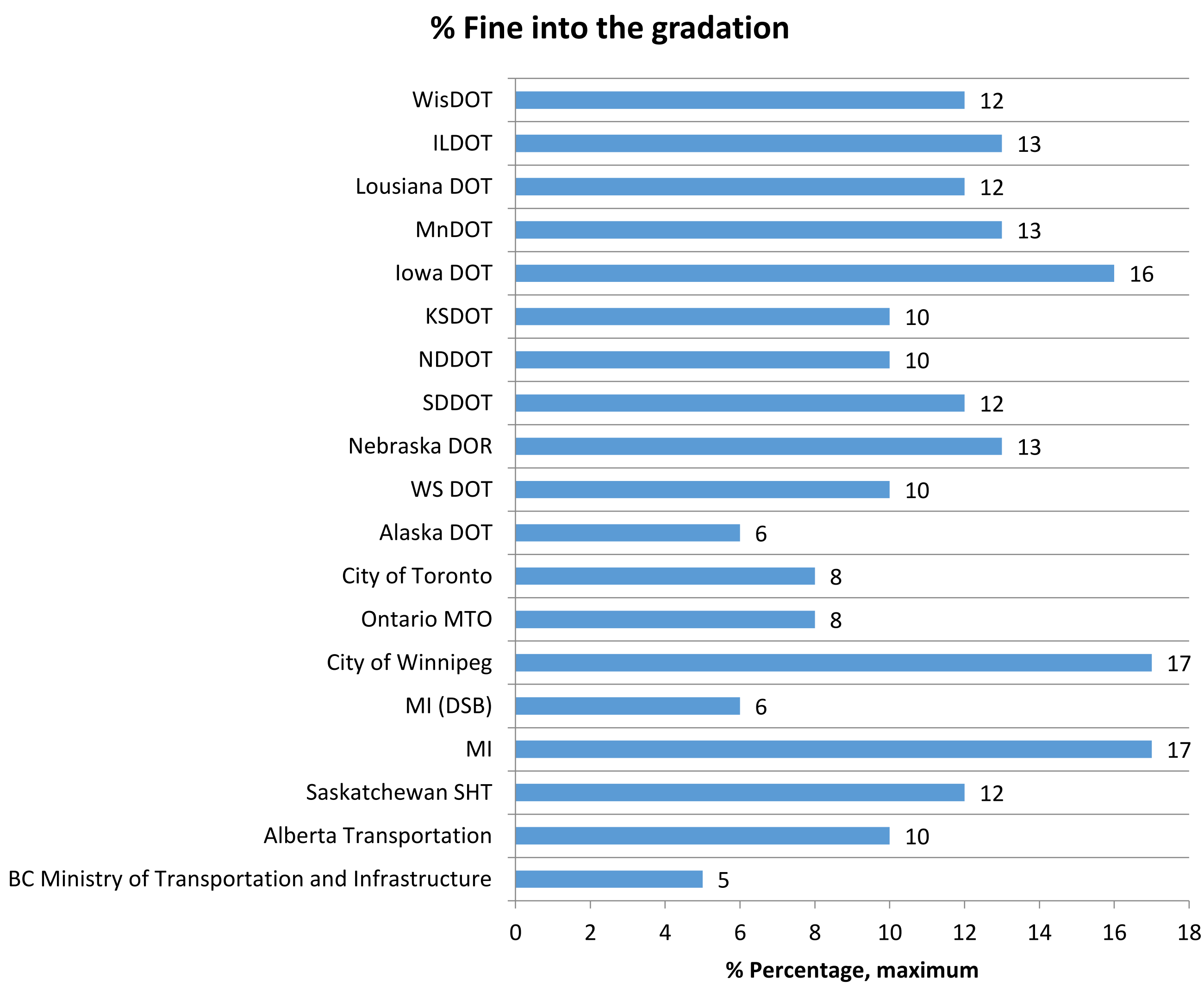


Figure 3-4 Maximum permitted percent of fines of the reviewed specifications

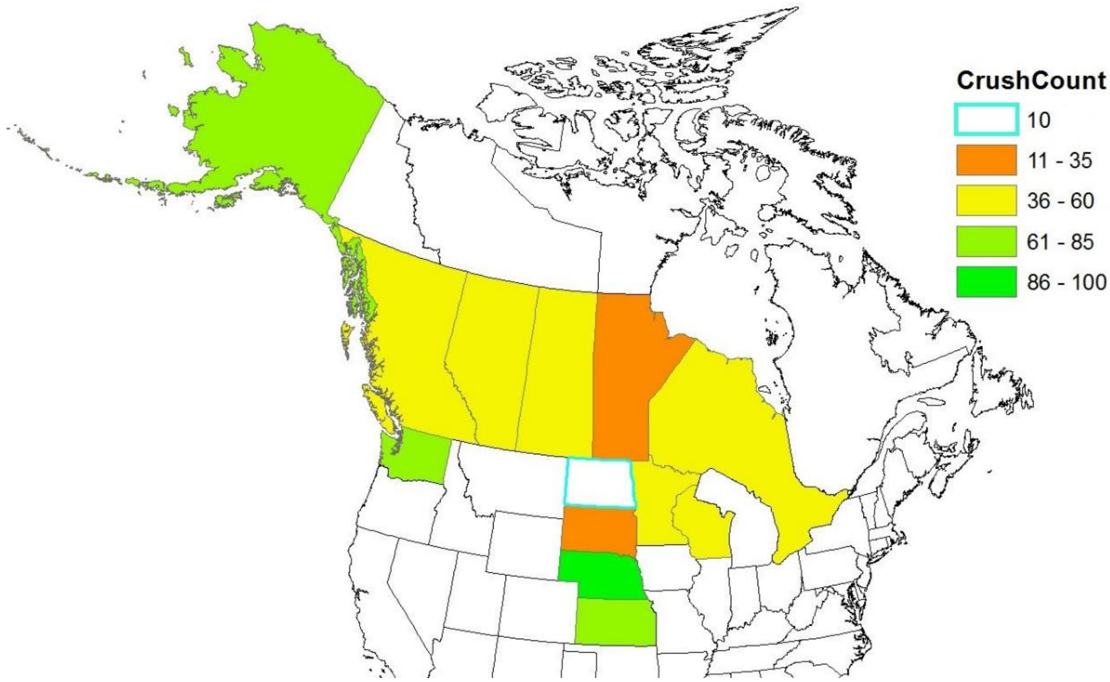


Figure 3-5 Minimum crush count of MI specification and several North American Specifications

In addition to the amount of fines, types of fine in a mix influences the permanent deformation behaviour of a unbound granular layer [23],[24]. Plasticity of fines affects the performance of unbound granular materials significantly (Tutumluer, 2013). Therefore, transportation agencies also specify Plasticity Index to control maximum allowable plasticity of fines. Figure 3-7 and Figure 3-8 show the specified maximum plasticity index of the studied jurisdictions. Manitoba Infrastructure specifies a minimum plasticity index of 10 for the clay binder to be mixed with gravel materials. This limit requires adding a clay binder to the relatively clean and non-plastic aggregate blend.

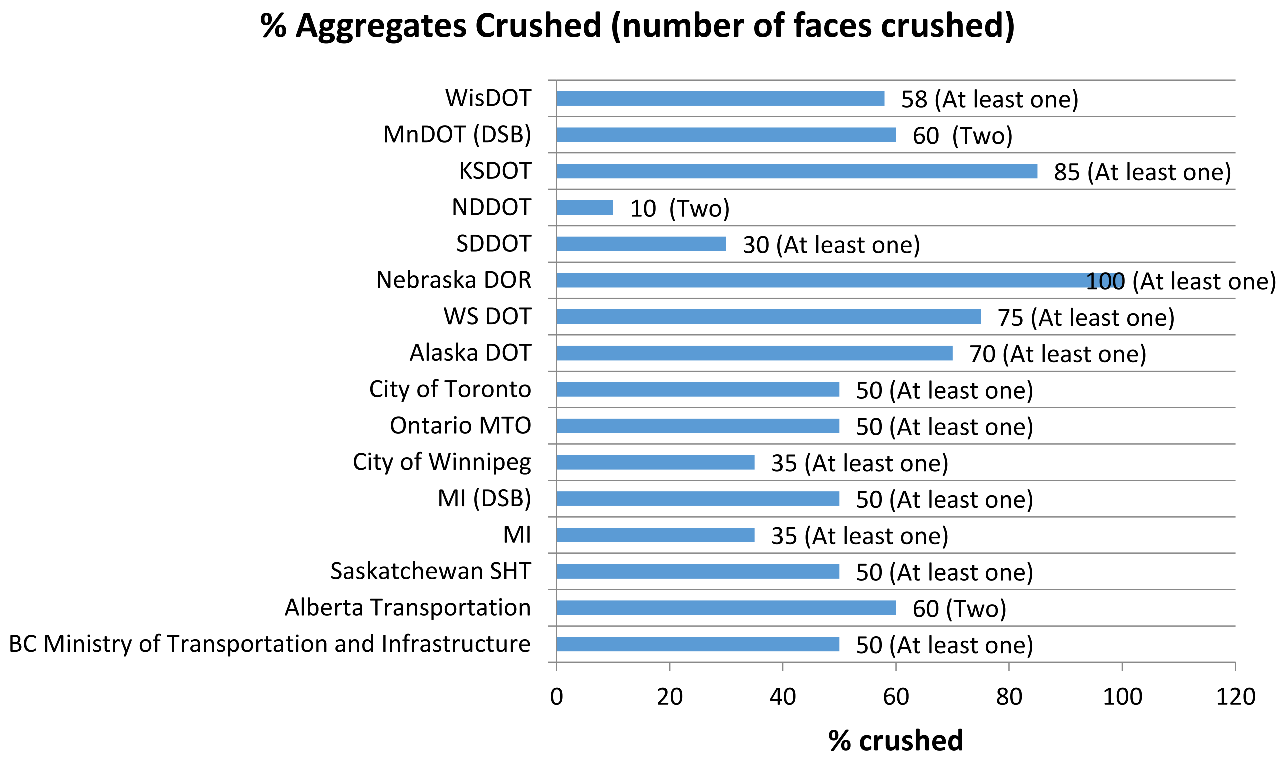


Figure 3-6 Crush count of the reviewed specifications.

Other than plasticity index, several transportation agencies also specifies maximum liquid limit of the fines. Maximum specified value of liquid limit for the Manitoba Infrastructure is 50 for clay binders. Moreover, Minnesota Department of Transportation and Louisiana Department of Transportation specify the maximum liquid limit value of 45 and 35, respectively. South Dakota, Kansas, Wisconsin Department of Transportation, and the City of Winnipeg specify a maximum liquid limit of 25.

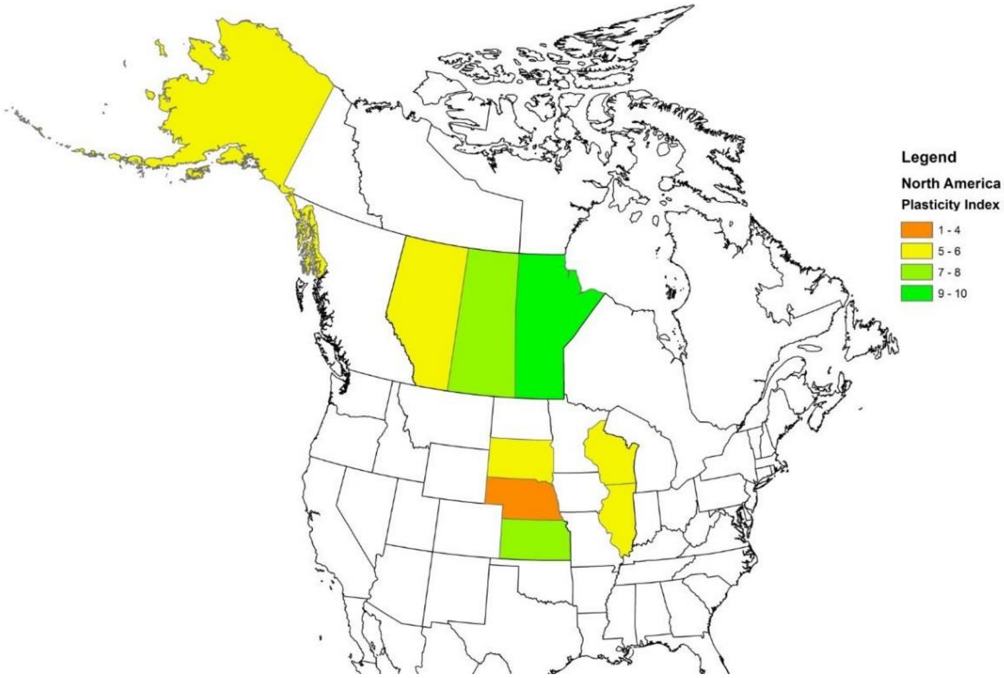


Figure 3-7 Plasticity Index (%) of MI specification and several North American specifications

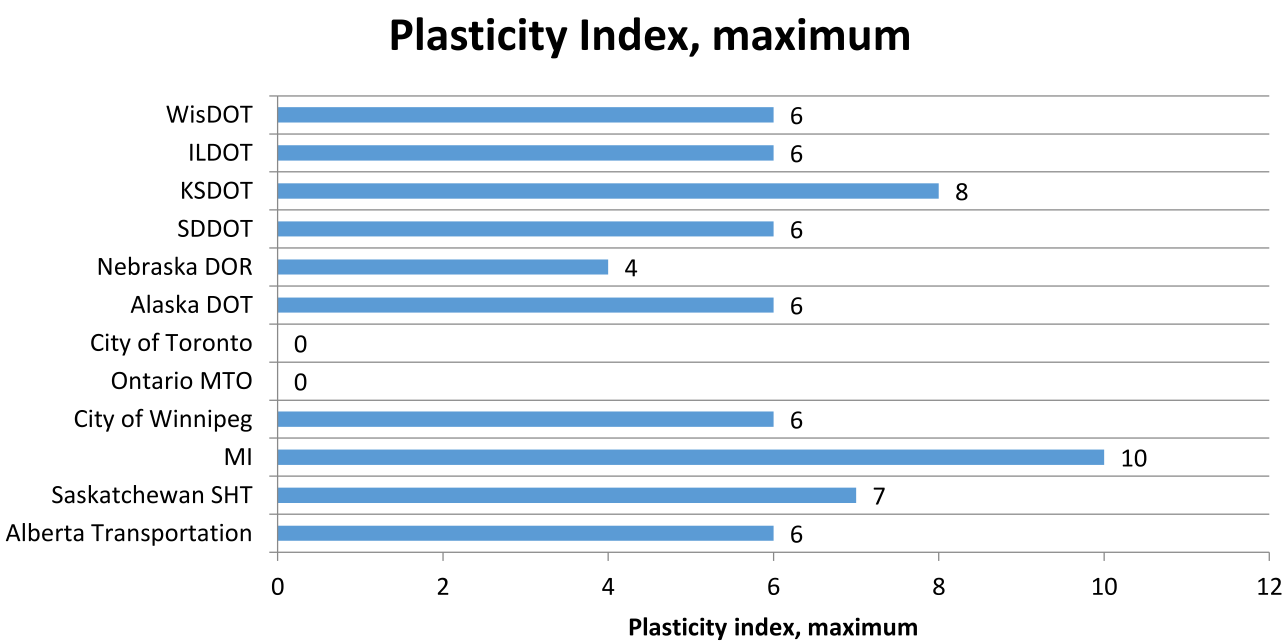


Figure 3-8 Plasticity Index limit of reviewed specifications

3.2.1 Deleterious Substances

Deleterious substances affect the performance of the unbound granular material layer. The deleterious materials include the lightweight particles, organic and degradable matters, flat or elongated particles, clay lumps or clay balls, shales, piece of glass. The deleterious substances mostly change their property or degrades with time and behaves differently than the required materials. Therefore, the transportation agencies specify limits for the acceptance of different types of deleterious substances into the mix. Table 3.2 shows the specified amount of deleterious substances into the gradation for the jurisdictions. Manitoba Infrastructure also specifies relatively higher amount of clay balls and shales compared to other jurisdictions.

Table 3.2 Comparison among transportation agency's specification of deleterious substances

State/Province	Deleterious Substance
British Columbia Ministry of Transportation	The aggregate particles should be uniform in quality and free from clay lumps, wood and free from an excess of flat or elongated pieces.
Alberta Transportation	5000 micron sieve shall not contain more than 3% detrimental matter based on the total mass of the combined.
Saskatchewan DHT	Lightweight pieces 5%, maximum Other deleterious material 2%, maximum
Manitoba Infrastructure	Maximum 12% shale, 10% clay balls.
Manitoba Infrastructure (DSB)	Maximum 3% shale, and free of clay balls, organics and other deleterious substances.
Ontario MTO	1% deleterious material.
City of Toronto	1% deleterious material. Granular A may contain up to 15% by mass of crushed glass and ceramic material combined.
City of Winnipeg	Crush rock and gravel should be free from soft material that will decay. Crushed concrete base allows up to 2% of deleterious material.
Alaska DOT	Free from clay balls, vegetable matter, or other deleterious matters.
Washington DOT	The portion of crushed surfacing retained on a No. 4 sieve shall not contain more than 0.15 percent wood waste. Gravel base material retained on a No. 4 sieve shall contain no more than 0.20 percent by weight of wood waste.

Nebraska DOR	2.5% (shale, clay lumps, or other deleterious substances). Crushed rock for base course shall be free from injurious quantities of dust, soft or flaky particles, loams, alkali, organic matter, paper, wood, or other deleterious material.
South Dakota DOT	Shall be free of sod, roots, vegetation, wood, paper, metal, glass, and other foreign objectionable material.
North Dakota DOT	AB- 12%, PBA- 8% (Gravel-Aggregate base and Permeable Base Aggregate)
Kansas DOT	Aggregates that is free from weeds, sticks, grass, roots and other undesirable foreign matter.
Iowa DOT	N/A
Minnesota DOT	Should not contain topsoil, organics or disintegrating rock. Shale: If No. 200<7%: Class 1-2: NA, Class 3-4: 10%, Class 5-5Q: 10%, Class 6: 7% and if No. 200>7%: Class 1-2: NA, Class 3-4: 7%, Class 5-5Q: 7%, Class 6: 7%. Should be free from clay balls.
Louisiana DOT	As determined by visual inspection, shall be reasonably free from foreign matter.
Illinois DOT	Class A- 5%, Class B- 6%, Class C- 10% (shale, clay lumps, coal, lignite, soft and unsound fragments, other deleterious)
Wisconsin DOT	By-product material: glass 12%, foundry slag 7%, steel mill slag 75%, bottom ash 8%, pottery cull 7%. Furnish by-product materials should be substantially free of deleterious substances.

3.3 REVIEW ON CONSTRUCTION SPECIFICATION OF RECYCLED CONCRETE AGGREGATE

Comparison of specification is not straight forward, since specifications are developed and evolved based on current practice, locally available materials, landform, climate and drainage. Also, the pavement design practice and the traffic over design life varies between departments of transportations. Controlling the specification of nonstandard aggregate such as recycled concrete aggregate is more important than that of the standard quarried aggregate. The specifications for using recycled concrete aggregates used in some North American, European and Australian department of transportation are briefly discussed and compared.

3.3.1 North American Specifications of Recycled Concrete Aggregate Use

The North American specifications reviewed are the specification of City of Winnipeg, District of Columbia DOT, Illinois DOT and South Carolina DOT. Table 3.3 presents a summary of specifications for the North American agencies.

Illinois DOT- The compaction in 4 inch lifts thickness of base course layer is required which should be compacted to 100% of proctor density (Illinois Department of Transportation, 2012). Illinois DOT specifies a maximum aggregate size of 25 mm and 37.5 mm respectively for CA 10 and CA 6 gradations for the base course application. In terms of abrasion loss, coarse aggregates must be less than 45% loss. In addition, the fines content (passing 0.075 mm) is limited to 4-12% and 5-13%for accepted in gradation CA 6 and CA 10, respectively. The California Bearing Ratio value of 80 is required for the base course layer.

South Carolina DOT- South Carolina DOT specifies a one lift compaction for base course layer thickness of less than 10-inch which should be compacted to 100% of maximum laboratory density. South Carolina DOT specifies a nominal maximum aggregate size of 2 inch for the base course application. The maximum Loss Angeles abrasion loss is 65% for coarse aggregates. In addition the fines content (passing 0.075 mm) is limited to 0-12%. The maximum specified liquid limit and plasticity index are 25 and 6, respectively. The aggregates should be free from lumps or balls of clay or other objectionable matter and should not contain metals, wood, brick, plastics, or other unacceptable debris.

City of Winnipeg- City of Winnipeg specifies a minimum 75 millimeter thickness of base course layer which should be compacted to 100% of standard proctor density (City of Winnipeg, 2015). Gradation and the materials property of the base course materials have been the main focus of the specification providing agency. The maximum aggregate size is 20 mm for the base course and 50 mm for sub-base course application. A maximum of 35% Los Angeles abrasion loss is accepted. Moreover, it is also specified that, the coarse aggregates should contain more than 35% crushed aggregates having at least one fractured face while being placed under asphaltic concrete pavement.

A higher percentage of fines passing Canadian metric sieve size 80 6-17%, is accepted in gradation. The specified liquid limit and plasticity index are maximum 25 and 6 respectively. A maximum 2 percent of the total dry weight of deleterious substances are allowed.

District of Columbia DOT- District of Columbia specifies a maximum compaction thickness of 6 inch for base course layer which should be compacted to 100% of proctor density under bituminous pavement and 95% of proctor density under PCC pavement (*Standard Specifications for Highways and Structures*, 2013). If the layer thickness exceeds 6 inches, the compaction should be done in two or more layers. It has also been specified in the specification that recycled concrete aggregates shall not be used where there is subsurface drainage problem in roadbed. District of Columbia specifies a maximum aggregate size of 63 mm for the base course application. In terms of abrasion loss, coarse aggregates having more than 50% loss is not accepted.

A 4-12% of fines (passing 0.075 mm) is accepted in gradation. The maximum specified liquid limit and plasticity index are 25 and 6 respectively. In terms of the deleterious substances into the gradation, the composite materials should be free from organic matter, asphalt, bricks, lumps and balls of clay and other non-concrete materials. Furthermore, District of Columbia specifies a minimum California Bearing Ratio value of 25 for the base course layer.

Table 3.3 Comparison and summary of North American specifications.

Parameters\Agencies	**City of Winnipeg**	**District of Columbia DOT**	**Illinois DOT**	**South Carolina DOT**

Maximum Aggregate Size (mm)	20.0	63.0	25 and 37.5[1]	2 inch/ 50 mm (Nominal maximum)
Fine Content (%)	6-17	4-12	4-12 and 5-13[1]	0-12
Crush Count (%)	35	-	-	-
LA/Micro Deval Abrasion, % (max)	35	50	45	65
Liquid Limit, % (max)	25	25	-	25
Plasticity index, % (max)	6	6	-	6
Stiffness and Strength Limits	-	Minimum CBR - 25	Minimum CBR- 80	-
Foreign Material Limit	Maximum 2% deleterious substances	Should be free from organic matter, asphalt, bricks, lumps and balls of clay and other non-concrete materials.	-	Aggregates should be free from lumps or balls of clay or other objectionable matter and does not contain metals, wood, brick, plastics, or other unacceptable debris.
[1]For Gradation CA 6 and CA 10 respectively.				

3.3.2 European Specifications of Recycled Concrete Aggregate use

The European specifications reviewed are the specification of The Netherlands and Finland. Table 3.4 also presents a summary of specifications for the European transportation agencies.

The Netherlands- The maximum aggregate size of recycled concrete aggregate is 20 mm (Gabr et al., 2011). In addition to that, the specification specifies 0-8% fines (passing 63 micron) into the gradation. Moreover, the crushing factor (crush count) of the aggregates have been specified as 0.65 (65%). The Netherlands specifies at least 80% crushed concrete into the recycled concrete aggregate gradation. The Netherlands specification allows a maximum of 5% asphalt, 10% other broken stone, 1% gypsum, metal and plastic and 0.1% organic materials. Furthermore, The Netherlands specifies a minimum California Bearing Ratio value of 50 for the base course layer.

Finland- Finland started using the recycled concrete aggregate in base construction in 1994. The aggregate is named as Betoroc™ crush (Gabr et al., 2011) (Forsman, Korjus, Kivekäs, & Määttänen, 2001). Four different gradations of varying quality has been reported by (Forsman et al., 2001). Maximum aggregate size of the Betoroc™ crush is 50 mm. The stiffness of the base materials varies from 200 to 700 MPa, which is based on either FWD test or plate load test. Two gradations of Betoroc™ crush among four have self-hardening property. The gradations may contain up to 30 percent of bricks. The specification also allows up to 1 percent of foreign materials such as, wood and plastic.

Table 3.4 Examples of European specifications (Gabr et al., 2011)(Forsman et al., 2001).

Parameters\Agencies	**The Netherlands**	**Finland**
Maximum Aggregate Size (mm)	22.4	50
Fine Content (%)	0-8	-
Crushing factor (Crush count, %)	0.65 (65%)	-
Stiffness and Strength Limits	CBR- 50	CBR- 90-140 (In general) Design E-modulus[1]- 200-700 MPa
Foreign Material Limit	Crushed concrete content should be at least 80%. Maximum Asphalt content- 5%. Maximum other broken crushed stone- 10%. Maximum organic substances- 0.1% Maximum Gypsum, metals and plastics- 1%	Maximum Brick content varies from 0 to 30 % Maximum other materials such as wood, plastics etc. should be 0.5% for type I Betoroc™ crush and 1% for other types of Betoroc™ crush

[1]E-modulus is the design strength of the Betoroc™ crush.

3.3.3 Australian Specifications of Recycled Concrete Aggregate Use

The specifications of State of Victoria, New South Wales and South Australia were reviewed. Table 3.5 presents a summary of specifications for these Australian transportation agencies.

Victoria (VicRoads) - VicRoads allows fine graded clayey sand or very fine clayey filler as additive into the crushed concrete material to improve the cohesion and permeability (VicRoads, 2009). The nominal maximum aggregate size and maximum aggregate size allowed are 26.5 mm and 19.0 mm respectively. Also the percentage of fines (passing 0.075 mm) should be from 5 to 9 percent. VicRoads also allows a liquid limit of 35 percent maximum. The maximum allowed plasticity index is 6. In terms of aggregate properties, maximum of 35 percent Los Angeles Abrasion loss and flakiness index of 35 are allowed.

In terms of the stiffness, VicRoads specifies a CBR value of minimum 100 at optimum moisture content and at 98 percent compaction density using modified compaction effort and 4 days of soaking. Moreover, VicRoads also allows maximum 2 percent of high density material such as metal, brick and glass, 0.5 percent of low density material such as plastic, rubber, plaster, clay lumps and other friable material and 0.1 percent of decomposable material.

New South Wales – The reviewed Specification of Roads and Traffic authority of New South Wales 3051 covers the specification for construction of unbound and modified base and subbase materials for surfaced road pavements ("Roads and Maritime Services QA Specification 3051 Granular Base and Subbase Materials for Surfaced Road Pavements," 2014). Similar to Victoria, New South Wales also allows nominal maximum aggregate size and maximum aggregate size 26.5 mm and 19.0 mm, respectively. The percentage of fine allowed into the gradation is 2 to 7 percent. New South Wales allows a maximum liquid limit of 27 for recycled materials base. New South Wales also specifies material properties based on traffic category of the road. Category A, B, C and D are defined as Freeways and major highways, rural highways, arterial and collector roads, and low trafficked rural or urban local roads respectively. New South Wales roads and traffic department doesn't specify any plasticity index for category A roads however, for B and C category, it is specified as maximum 2-6 and for category D, it is specified as maximum 8.

Unlike other agencies, the stiffness measuring parameter for unbound base is maximum dry compressive strength, unconfined compressive strength and wet strength. New South Wales roads and traffic department also limits the foreign material application. The maximum allowed foreign materials is reported in Table 3.5.

South Australia - The reviewed Specification of Government of South Australia Part R 15 covers the specification for construction of recycled unbound base materials for surfaced road pavements (Department of Planning Transport and Infrastructure, 2015). Similar to Victoria and New South Wales, South Australia also allows nominal maximum aggregate size and maximum aggregate size 26.5 mm and 19.0 mm respectively. The percentage of fine allowed into the gradation is 5 to 11 percent. Department for Transport, Energy and Infrastructure of South Australia specifies the maximum liquid limit of 25 percent and the plasticity index should range from 1 to 6 percent. In addition to that, the maximum linear shrinkage is specified as 3 percent.

Department for Transport, Energy and Infrastructure of South Australia specifies the stiffness and strength limits of the materials. The minimum value of resilient modulus, maximum permanent deformation rate, cohesion and minimum friction angle are shown in the Table 3.5.

Table 3.5 Comparison and summary of Australian specifications (Gabr et al., 2011)(VicRoads, 2009)(Department of Planning Transport and Infrastructure, 2015).

Parameters\Agencies	**Victoria**	**New South Wales**	**South Australia**
Maximum Aggregate Size (mm)	19.0	19.0	19.0
Fine Content (%)	7-14	5-11	3-11
LA/Micro Deval Abrasion, % (max)	35	30	40
Liquid Limit, % (max)	35	27	25
Plasticity index, % (max)	6	Category A: N/A Category B and C: 2-6 Category D: 8	1-6
Stiffness and Strength Limits	CBR- 100	MDCS[1] (min): 1.7 MPa UCS[2] (max): 1.0 MPa Wet Strength[1] (min): 70 kN	MR (min): 300 MPa PD (max): 10^{-8} C (max): 150 kPa F (min): 40 degree
Foreign Material Limit • High density (max) • Low density (max)	 2% 0.5%	 Category B: 3% Category C and D: 5% Category B: 0.2% Category C and D: 0.5%	 20% 1%

• Organic (max)	0.1%	Category B: 0.1% Category C and D: 0.2%	0.5% (1% bitumen in addition)
MDCS- Maximum Dry Compressive Strength UCS- Unconfined Compressive Strength MR- Resilient Modulus C- Cohesion		[1]For Category B, C and D [2]accelerated 7 days curing at 65 degree Celsius PD- Permanent deformation rate/load cycle F- Friction Angle	

The construction specifications are dependent on current practice, locally available materials, landform, climate and drainage. Also the specifications are dependent on the design practice of pavements. Most of the Australian specifications are focused on the higher stiffness performance of the base materials since the pavements are usually thin overlays over a strong and stiff base layer (Gabr et al., 2011). Whereas in City of Winnipeg, the recycled base layers are placed under either a rigid pavement or a composite of hot mix asphalt over concrete pavement.

3.4 SUMMARY OF THE FINDING FROM THE SPECIFICATION REVIEW

The base course construction specifications of different transportation agencies provide the detailed guidelines of selecting the quality of the aggregate in construction, construction method and quality control techniques. In this study, the specified material properties of virgin and recycled aggregates of different transportation agencies of North America, Europe and Australia have been reviewed. Following findings and conclusions can be drawn from the review of the specifications.

- The feasibility and quality of construction with a desirable quality of materials is dependent on the availability of the material in the vicinity of construction. The causes of adopting different specification are diversified morphology, landform, source of aggregates and climatic condition.

- The maximum aggregate size, fines content, crush count of aggregate, plasticity of fines, Los Angeles abrasion resistance have been found to be the major factors influencing the performance of unbound granular materials. The specifications of different transportation agencies also require these properties be met the requirements for their base course layer materials.
- The objective of the review of the specification was to collect available information of neighboring jurisdictions of Manitoba about their construction practice and to find out the differences in specification. The province of Manitoba allows relatively finer gradation having lower maximum aggregate size gradation and higher maximum fines content compared to neighboring jurisdictions. Furthermore, the specified crush count is also lower compared to neighboring jurisdictions.
- Another objective of the review of the specifications was to select different properties affecting the performance of the base materials. Based on the review of the specifications, the materials selection of laboratory investigation program has been setup. The identified different physical properties were maximum aggregate size, maximum fines content, crush count.
- The reviewed specification indicates the requirement of selecting materials with increased maximum aggregate size, reduced fines content, increased crush count of coarse aggregates, and selecting materials having not more than 35% Los Angeles abrasion loss.
- The recommended properties from the reviewed specification of recycled concrete aggregates can be used to develop specification for this material. Recycled concrete aggregate materials have to be tested at different fine content in order to determine the feasibility of the material for use as a base course layer.

Chapter 4 Materials and Testing Program

4.1 INTRODUCTION

This chapter includes a detailed description of the testing program used for the study including the tested material properties and the laboratory test procedures.

4.2 EXPERIMENTAL TESTING PROGRAM

The experimental testing program includes testing four types of unbound granular materials used in base course layer construction of pavement. The laboratory tested materials are Recycled Concrete Aggregates, Gravel, Limestone, and Granite. The selected materials are the locally available materials which are used in base layer construction. The selected materials are from the local recycled concrete suppliers, gravel pits used for the supply of base materials during construction, and local quarries. The gradation of the materials have been selected in a manner that, they are compatible with the base materials construction specification requirement of the local transportation departments such as Manitoba Infrastructure and City of Winnipeg.

The performed tests on unbound granular materials are Materials Sampling (ASTM C702/C702M)(ASTM International, 2011), composition analysis of coarse aggregates, sieve analysis (ASTM C136-06)(ASTM International, 2001), wet sieving (ASTM C117)(ASTM Committee C09, 2004), Standard Proctor (ASTM D698)(ASTM International, 2007), Micro-Deval (ASTM D 6928)(ASTM International, 2012), Resilient Modulus (NCHRP 1-28A)(National Cooperative Highway Research Program (NCHRP), 2004), Permanent Deformation, CBR (ASTM D1883)(ASTM International, 2014), and Permeability (ASTM D2434)(ASTM International, 2006). Table 4.1 shows the summary of the experimental testing program of the tested materials. At least 2 samples have been tested of each material gradation in each test. A third sample has been tested if high variability between two tested samples have been obtained.

Table 4.1 Summary of the Experimental Testing Program

Sample identification	**Resilient Modulus Test**		**Permanent Deformation Test**	**CBR Test**	**Permeability test**
	NCHRP 1-28A	**After Permanent Deformation**			
RCA 2.1%	2	2	2	2	2
RCA 6.0%	2	2	2	2	2
RCA 9.5%	2	2	2	2	2
PTH 75	2	2	2	-	2
Gravel	2	2	2	-	2
Limestone	2	2	2	-	2
Granite	2	2	2	-	2
Total Number of Tests	**14**	**14**	**14**	**6**	**14**

4.3 MATERIAL PROPERTIES

4.3.1 Recycled Concrete Aggregate (RCA)

4.3.1.1 Gradation of Recycled Concrete Aggregate

The recycled concrete aggregate is used by City of Winnipeg transportation department in base layer construction. Three gradations of recycled concrete aggregate have been tested in order to investigate the performance of recycled concrete aggregates at varying fine content. The target fine content of the gradations was 2.0%, 6.0% and 10.0%. However, actual percentage of fines in tested samples have been found to be 2.1%, 6.0% and 9.5% respectively. Figure 4-1 shows the tested RCA materials gradations. The 6.0% fines and 9.5% fines gradations were very similar in shape and density except the fine portion of the gradation. The 2.1% fines gradation had a less density compared to others. Also the gradation with low fine content is outside the specified boundary of the RCA gradation.

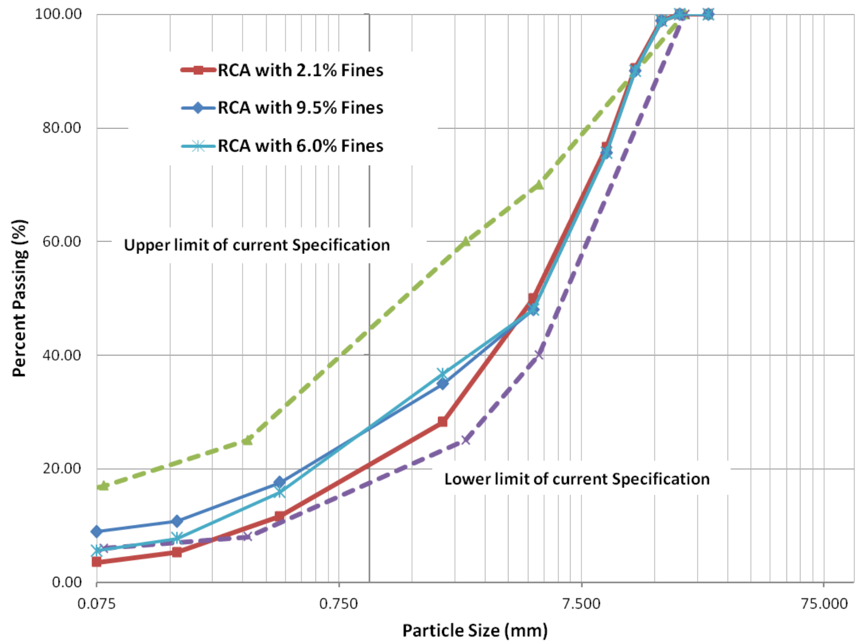


Figure 4-1 Gradations of RCA materials tested in the investigation.

4.3.1.2 Composition of Recycled Concrete Aggregate

An analysis of the composition of RCA materials (coarse blend) has been carried out to calculate the actual amount of RCA and other aggregates or impurities present into the blend. The analysis has been done from a representative sample of 8 kg. The compositions have been sampled by visual inspection and the percentages of constituents have been measured on a weight basis. Figure 4-2 shows the composition of the RCA coarse blend.

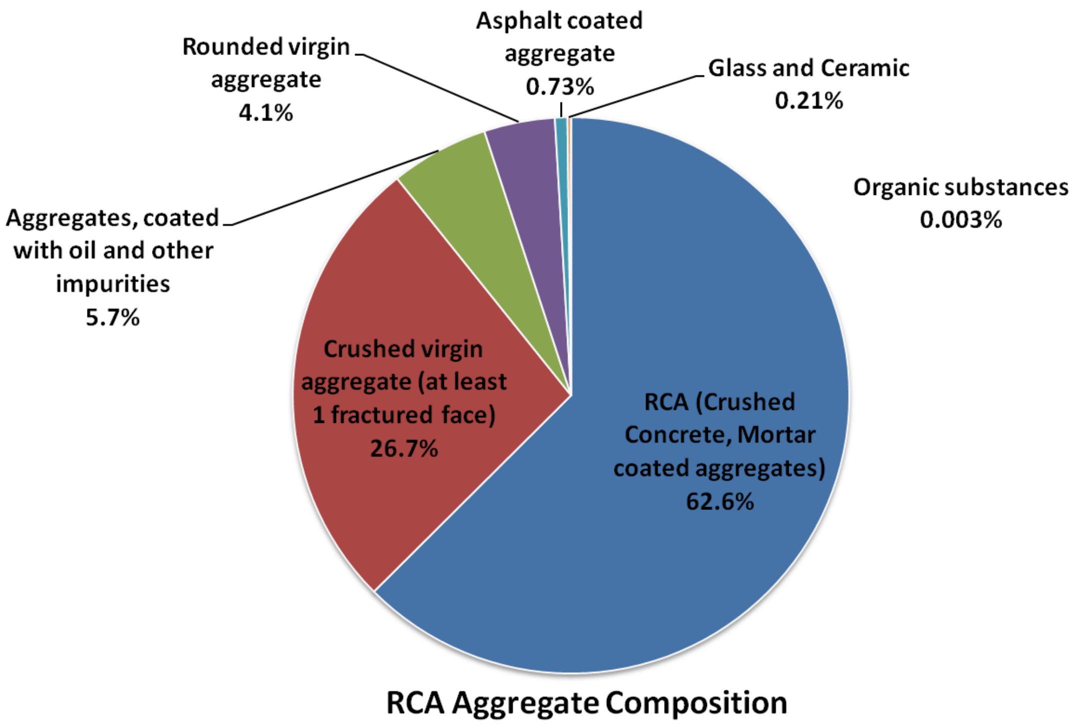


Figure 4-2 Composition of the RCA coarse aggregate blend.

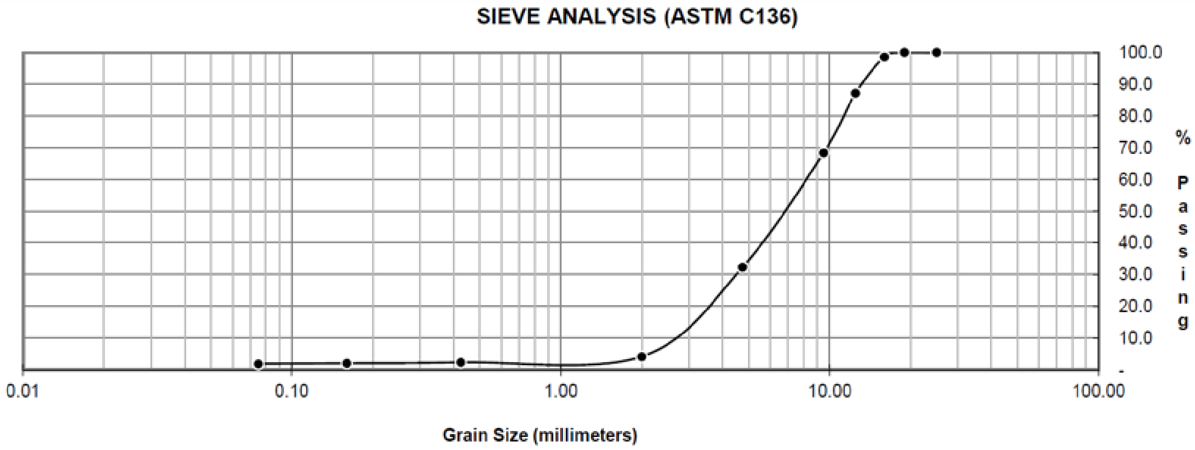
Actual crushed RCA was defined as the crushed concretes and the mortar coated aggregates. Approximately 62.5% of RCA have been found from the blend. Crushed and rounded virgin aggregates have also been found from the blend, and most of the virgin aggregate seemed to come from concrete by visual observation. Those aggregates were assumed to be either uncoated by mortars or lost the coating of the mortar. The pictures of the actual RCA, virgin crushed and uncrushed aggregates have been placed into the Figure 4-3. On the other hand, impurities have also been found from the blend. Aggregates coated with oil and other impurities, Asphalt coated aggregates, glass and ceramic and organic substances have been found. The pictures of the impurities found into the blend have been placed into the Figure 4-3.

Figure 4-3 From left: First row- RCA material, Crushed Aggregates, Uncrushed Aggregates, Aggregate with other impurity; Second Row- Asphalt Coated Aggregate, Organic Impurity, Glass and Ceramic.

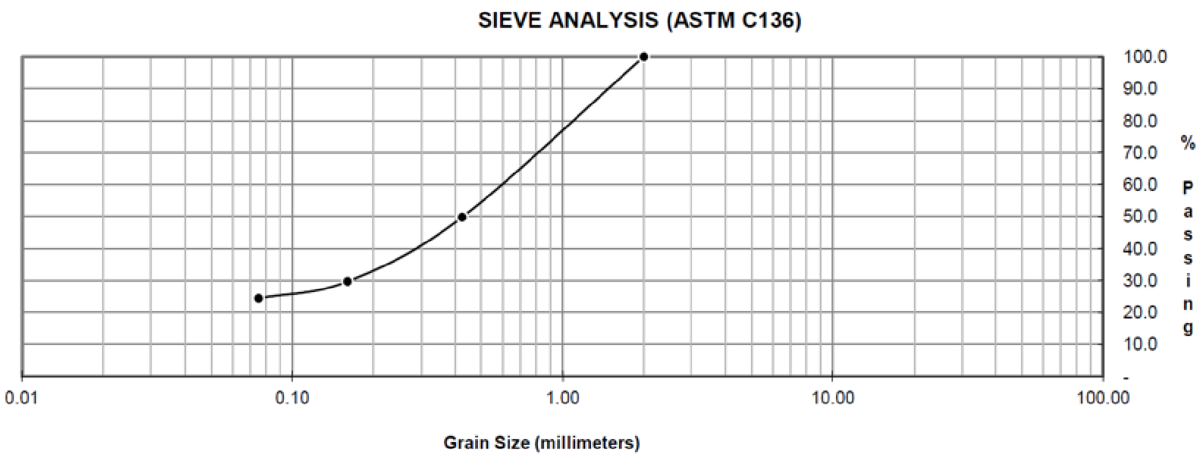
4.3.1.3 Sampling of Recycled Concrete Aggregate

In order to prepare representative samples of recycled concrete aggregate (RCA), base materials to meet three gradation having 2%, 6% and 10% fines (particles passing No. 200 sieve), following sampling procedures have been followed:

A large amount of recycled concrete aggregate (RCA) blend (120 kg approximately) has been sieved and splitted into three separate blends (following standard procedure ASTM C136). Those three blends are the particles passing 19 mm sieve and retaining on 4.75 mm sieve (coarse blend), particles passing 4.75 mm sieve and retaining on 2.0 mm sieve (intermediate blend) and particles passing 2.0 mm sieve (fine blend). Aggregate gradation of coarse-intermediate and fine aggregates have been shown in Figure 4-4.



(a)



(b)

Figure 4-4 Gradation of (a) coarse-intermediate blend and (b) fine blend.

The blends have further reduced to smaller quantity of approximately 5 kg (following standard procedure ASTM C702 / C702M) using splitter to facilitate the gradation adjustment and sample preparation. To calculate and adjust the actual amount of fines (particles passing No. 200 sieve) into the blends, wet sieving (following standard procedure ASTM C117) has been done. According to wet sieving, the coarse blend contains 1.1% fines, the intermediate blend contains

4.4% and the fine blend contains 23.6% fines. Based on the results of sieve analysis of blends and wet sieving of blends, the three blends have been mixed to different proportions to obtain target gradations having 2.0%, 6.0% and 10.0% fines. However, after the quality control check of the three gradation blends, actual percentage of fines tested have been found to be 2.1%, 6.0% and 9.5%, respectively. Standard proctor tests have been carried out for the 2.1%, 6.0% and 9.5% fines gradation to obtain the maximum dry density and optimum moisture content. Figure 4-5 shows the moisture density relationship of the three gradations. Figure 4-6 shows the proctor compaction of a sample into the 6-inch diameter proctor mould.

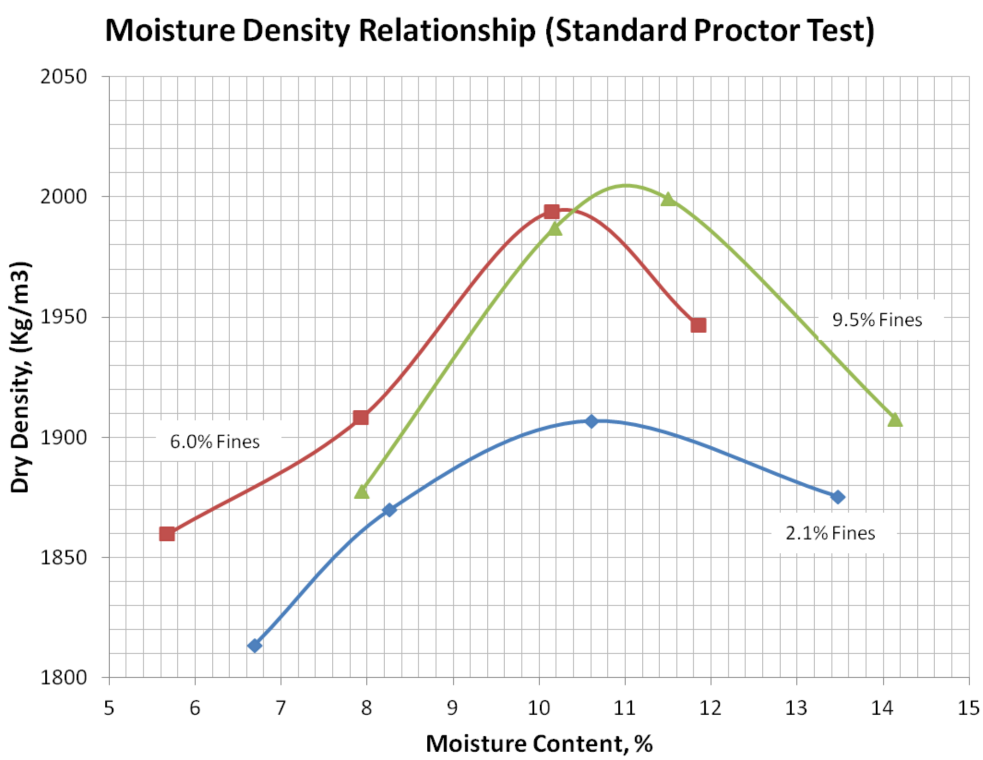


Figure 4-5 Moisture density relationships of the tested RCA gradations.

Figure 4-6 Extracted sample from the proctor mold after compaction.

The maximum dry density obtained from 2.1%, 6.0% and 9.5% fines gradations are 1,907 kg/m3, 1,995 kg/m3 and 2,005 kg/m3 respectively. Also, the optimum moisture content obtained from 2.1%, 6.0% and 9.5% fines gradations are 10.6%, 10.5% and 11.0% respectively.

4.3.2 Gravel, Limestone and Granite Aggregates

4.3.2.1 Gradation of Gravel, Limestone and Granite

Three similar gradations of gravel, limestone and granite have been selected for the investigation. Also a field sample from PTH75 highway construction project has been collected and tested for comparison of performance. The gradations have approximately 5.5-6.5% of fine content. The maximum aggregate size of the materials was 19.0 mm. Figure 4-7 shows the gravel, limestone, PTH75 highway project gradations selected for the investigation. The PTH75 highway project from 2014 to 2015 was the first attempt of Manitoba Infrastructure to implement a revised specification of A base increasing the maximum aggregate size, crush count and reduced fines content.

Table 4.2 Selected gradations of limestone, gravel, granite and PTH 75 gravel materials

Sieve Size, mm	PTH 75	Limestone	Gravel	Granite
	Percentage passing (%)			
25	100.00	100.00	100.00	100.00
19	88.90	87.27	86.97	87.16
16	81.11	81.61	79.27	81.50
12.5	73.72	74.42	71.95	74.27
9.5	66.37	64.14	61.54	63.99
4.75	50.73	45.70	43.78	45.57
2	37.02	30.33	28.79	30.71
0.425	11.49	15.58	15.32	16.10
0.18	7.23	9.11	9.22	9.37
0.075	6.08	5.50	6.11	5.50
0.01	0.00	0.00	0.00	0.00

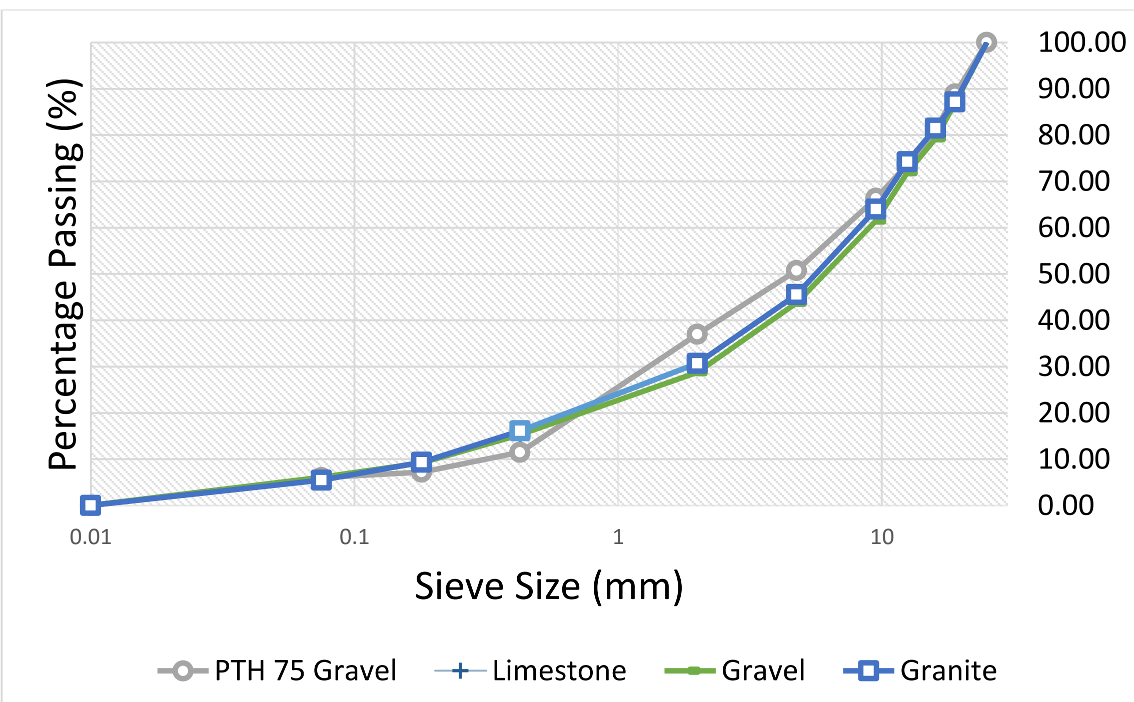


Figure 4-7 Gradations of gravel, limestone and granite materials tested in the investigation.

4.3.2.2 Sampling of Aggregates (PTH 75 Project)

The laboratory sampling procedure of the gravel sample blends (used in PTH 75 highway construction project) have been discussed.

- Approximately 35 kilograms of gravel blends have been oven dried for 24 hours.
- The materials have been reduced to testing size using standard procedure (ASTM C702).
- In order to obtain the actual fines content of the gradation of the reduced representative aggregates mix, wet sieving has been carried out using standard procedure (ASTM C117).
- The obtained gradation of the field sample from PTH 75 has been also shown in Figure 4.7.
- The compaction density and moisture content have also been obtained from the PTH 75 highway project quality control test results (Lab number: WAG150328 and Field Sample Number: # 47, 53, 67-69).

4.3.2.3 Sampling of Gravel, Limestone and Granite Aggregates

The gradations of the gravel, limestone and granite base course materials have been prepared in the laboratory by reconstituting the gradation from materials retained on individual sieves. In order to calculate the actual percentage of fine into the gradations, wet sieving has been carried out for each materials.

4.4 SUMMARY OF MATERIAL PROPERTIES

The material properties of the tested recycled concrete aggregate, gravel, limestone, granite, and PTH 75 project sample materials have been summarized in the Table 4.3.

Table 4.3 Summary of material properties.

Material	Maximum Aggregate size (mm)	Fine Content (%)	Maximum Dry Density (kg/m3)	Optimum Moisture Content (%)	Crush Count (%)	Plasticity	Abrasion (LA/Micro-deval) (Maximum)
Recycled Concrete Aggregate	16.0	2.1	1,907	10.6	90	NP	18.5*
	16.0	6.0	1,995	10.5		NP	
	16.0	9.5	2,005	11.0		NP	
PTH 75	19.0	6.0	2,221	8.5	60	NP	35
Gravel	19.0	6.1	2,329	6.0	60	NP	35
Limestone	19.0	5.5	2,313	6.15	100	NP	35
Granite	19.0	5.5	2,340	5.5	100	NP	35

NP = Non Plastic; *Micro-Deval test

4.5 LABORATORY TESTS

4.5.1 Resilient Modulus Test

Resilient modulus tests have been conducted according to the test protocol developed under NCHRP Project 1-28A. The test specimens were 101.6 mm in diameter and 203.2 mm in height. The test specimen was compacted using a vibratory compactor in eight layers, 25.4 mm each, to reach the target optimum moisture content and density. After applying 1,000 conditioning cycles, the test specimen was subjected to 30 loading sequences that represent different stress levels. The load pulse had a loading duration of 0.1 sec and a rest period of 0.9 sec. Pressurized air and computer-controlled regulator were used to apply the confining pressure. The air pressure inside the triaxial cell was monitored with pressure transducer. Four LVDTs were used for measuring the vertical deformation of the specimen. These LVDTs were mounted on the top and the bottom of a plate to measure the total vertical deformation of the specimen. Figure 4-8 and Figure 4-9 show the setup for resilient modulus test. At least two replicates were tested for each gradation.

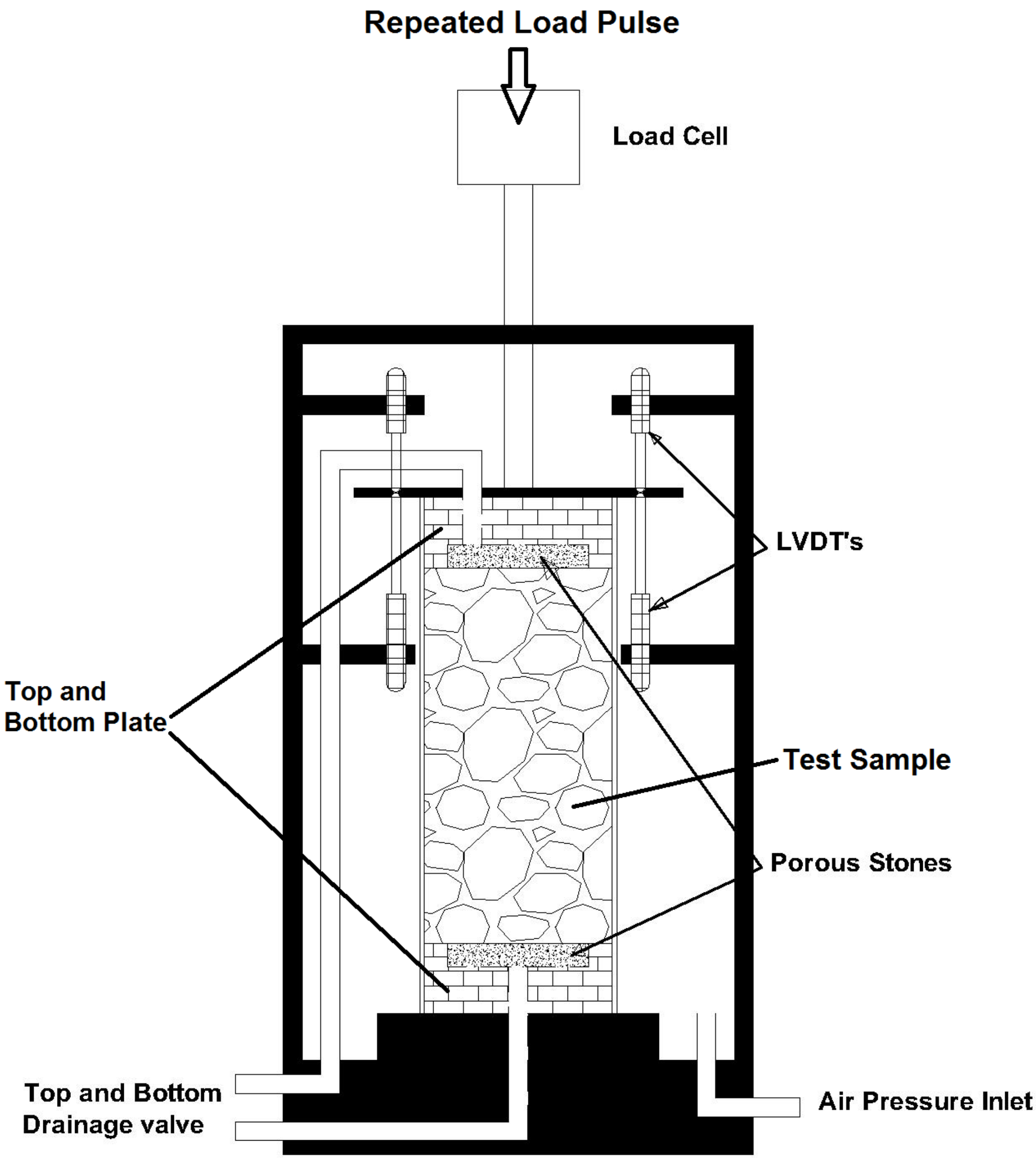


Figure 4-8 Typical resilient modulus test setup.

Figure 4-9 Resilient modulus test setup used for the research.

During each loading sequence, vertical deformations, cyclic load, and confining pressure were recorded for the last 5 cycles. The recorded data was processed with a computer code designed with MATLAB software to minimize the effect of any noise in the measured signals. The resilient modulus value was calculated for each cycle as the ratio of the applied cyclic stress to the measured recoverable strain. The resilient modulus values for the last 5 cycles were averaged.

Data from the 30 loading sequences were fitted to the following universal constitutive model:

$$M_R = K_1 P_a \left[\frac{\theta}{P_a}\right]^{K_2} \left[\frac{\tau_{oct}}{P_a} + 1\right]^{K_3} \tag{4.1}$$

Where,

$M_R = Resilient\ Modulus, MPa$

$\theta = Bulk\ stress, kPa = \sigma_1 + \sigma_2 + \sigma_3$

$\tau_{oct} = Octahedral\ shear\ stress, kPa = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2}$

$\sigma_1, \sigma_2\ and\ \sigma_3 = Principal\ stresses, kPa$

K_1, K_2 and K_3 = Regression constants

P_a = Atmospheric pressure = 101.35 kPa.

Nonlinear regression techniques were used to evaluate the values of the regression constants K_1, K_2, and K_3 for each specimen. The values of K_1, K_2, and K_3 of the two samples were averaged. Equation 4.1 and the evaluated regression constants (K_1, K_2, and K_3) can be used to calculate the resilient modulus at any stress state. The resilient modulus test protocol recommends the use of Equation 4.1 to calculate and report the resilient modulus of unbound base materials for confining pressure (σ3) = 35 kPa and cyclic stress (σ1- σ3) = 103 kPa.

4.5.2 Permanent Deformation Test

Repeated load triaxial test with a constant confining pressure and cyclic stress has been carried out with the test specimens. The test duration was 13,000 loading cycles having the cyclic load of haversine shape. Permanent deformation tests have been carried out for different duration to evaluate the long term performance as well as the early deformation performance under constant cyclic stress (Bilodeau, Doré, & Schwarz, 2011; Cerni et al., 2012; S Werkmeister, Dawson, & Wellner, 2004). Numerous studies have been carried out to evaluate the plastic deformation of the unbound granular materials ranging from 10,000 cycles to several million cycles (Barksdale, 1972b; Lekarp & Dawson, 1998; Perez et al., 2006; Rahman & Erlingsson, 2014b). The current study has been aimed to assess the early permanent deformation behaviour of the tested base materials. The haversine shaped cyclic loading has a loading duration of 0.1 second and a rest of 0.9 second. The permanent deformation test was conducted at a cyclic stress ranging from 106.5 kPa to 110.1 kPa and with a confining pressure ranging from 37.3 kPa to 40 kPa. The stress level was selected from previous literatures in which tests have been carried out at the same range. The range represents the average stress on the base course layers in pavement (Bilodeau et al., 2011; National Cooperative Highway Research Program (NCHRP), 2004). However, the stress level varies with the design of the pavement.

4.5.3 California Bearing Ratio (CBR) Test

California Bearing Ratio (CBR) test has been carried out according to the AASHTO T 193. CBR is a ratio of the bearing capacity of a material with respect to a well graded standard crushed stone. Thus, a high quality crushed stone should have a CBR of 100%. The compacted CBR sample has a dimension of 152.4 mm diameter and 177.8 mm height. CBR samples were compacted at its optimum moisture content to the 99 percent of the maximum dry density and kept into a water tank for 96 hours of soaking. After soaking, the penetration load was applied into the sample at a rate of 1.3 mm per minute. Figure 4-10 shows a diagram of CBR setup.

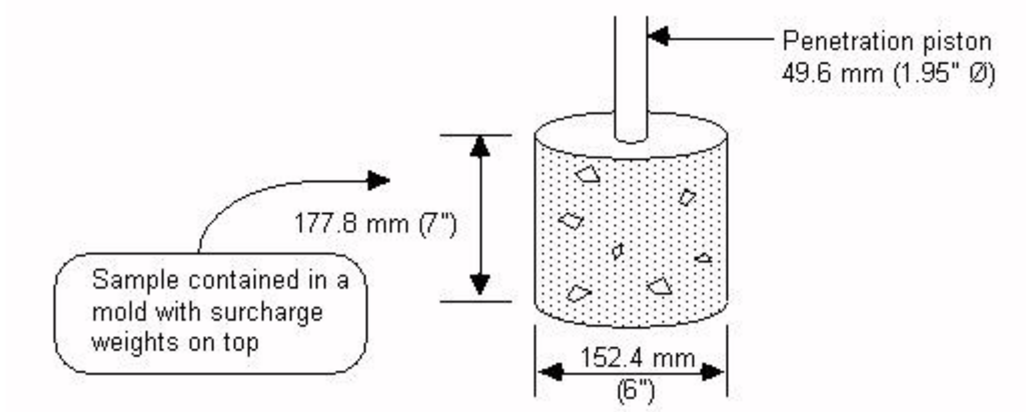


Figure 4-10 Diagram of CBR sample (Pavement Interactive, 2015).

Two annular plates of 2.27 kg each has been placed over the compacted sample during the soaking and test.

$$CBR\ (\%) = 100\ \left(\frac{x}{y}\right) \tag{4.2}$$

Where,

x = material resistance or the unit load on the piston for 2.54 mm or 5.08 mm of penetration.

y = standard unit load for well graded crushed stone (for 2.54 mm penetration, y = 6.9 MPa, for 5.08 mm penetration, y = 10.3 MPa).

4.5.4 Permeability Test

The permeability tests have been carried out with the unbound granular materials samples according to the procedure of ASTM D5865 “Standard Test Method for Measurement of Hydraulic Conductivity of Porous Material Using a Rigid-Wall, Compaction-Mold Permeameter”. The procedure includes the permeability test using single ring permeameter and double ring permeameter. A single ring permeameter having the dimension of 101.6 mm in diameter and 115.9

mm in height was used. Figure 4-11 shows the permeability measurement setup. The unbound granular materials have been compacted to 99 percent of the maximum dry density with optimum moisture content using a vibratory compactor. The compaction of the specimen, the permeameter was assembled according to the standard procedure. In order to saturate the compacted sample, a vacuum pump was connected with the effluent valve of the Permeameter to remove any trapped air inside sample. After the application of the vacuum, the effluent valve closed and influent valve connected to water tank with a constant head was opened to allow the created suction inside the mold to be replaced by water and saturate the specimen. Same procedure has been repeated until there is no air void coming from the effluent value after application of vacuum suction. After the saturation, the sample have been kept with a constant water head for 24 hours to become fully saturated.

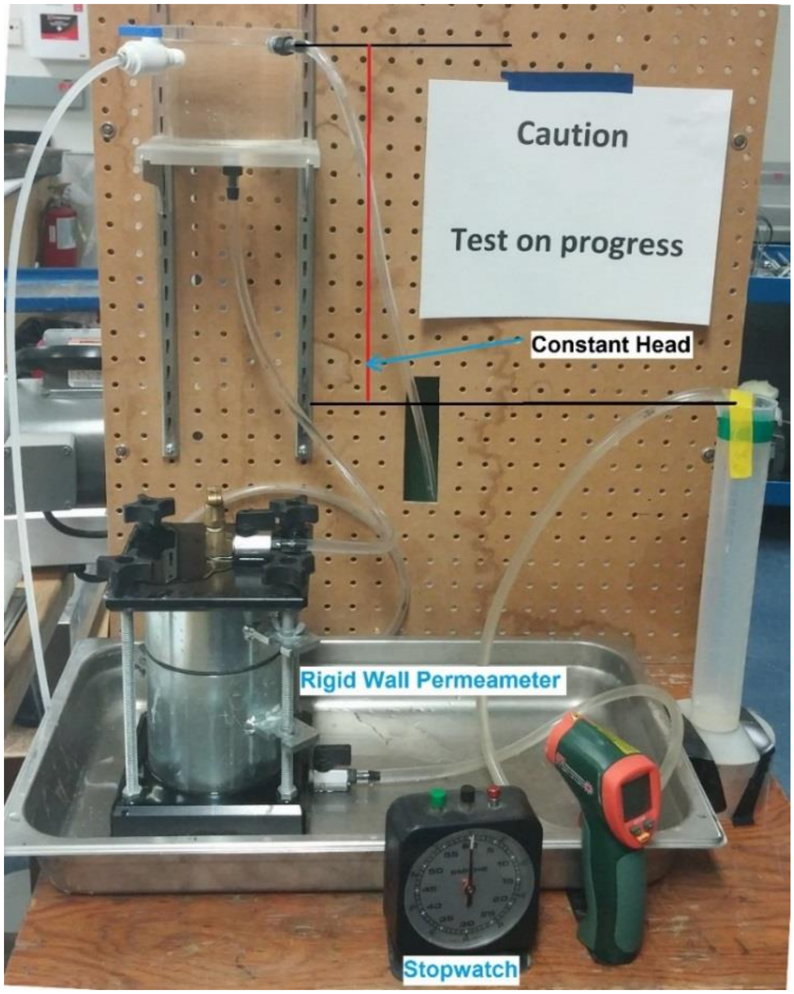


Figure 4-11 Permeability test setup.

Chapter 5 Characterized Behaviour of Unbound Granular Materials

5.1 STIFFNESS OF RECYCLED CONCRETE AGGREGATE

5.1.1 Resilient Modulus

Variation of resilient modulus within the fines contents range of of 2-10% was not very much significant as shown in Figure 5-1. However, the values obtained from the tests were higher than the design values anticipated by local transportation agencies. Also from the Figure 5-2 to Figure 5-4, it can be observed that the universal constitutive model predicted resilient modulus is also very similar for all the three fine content gradations. There is an increasing trend in resilient modulus with increase in confining pressure as well as deviator stress. This indicates that, within the range of percentage of fine there is insignificant amount of excess pore water pressure generation inside the tested specimens even in high bulk stresses. Table 5.1 shows the regression coefficients obtained from the analysis.

Moreover, the RCA samples have been tested at only optimum moisture contents. Resilient modulus test at different moisture contents should be tested in order to understand the moisture sensitivity behaviour of the materials. Specially, at a saturated condition, which simulates the spring thaw condition of a base layer, the resilient behaviour of the three gradations can be much different than that obtained at optimum moisture content.

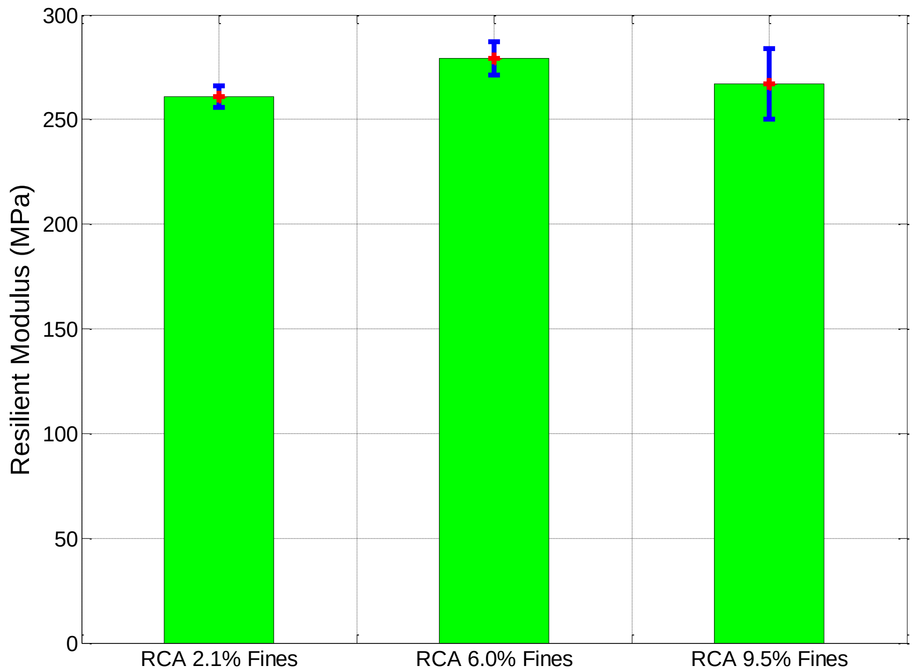


Figure 5-1 Resilient modulus results of the RCA gradations.

Table 5.1 Regression coefficients obtained from universal constitutive equation

RCA Fine Content	Moisture Content (%)	Resilient Modulus (MPa)	K1	K2	K3	R^2	p-value
9.5%	11.06	284	1.72	1.07	-0.73	0.929	<0.05
9.5%	11.43	250	1.51	0.95	-0.49	0.871	<0.05
6.0%	10.79	272	1.74	0.91	-0.56	0.884	<0.05
6.0%	10.62	288	1.92	0.75	-0.38	0.906	<0.05
2.1%	10.34	267	1.56	1.21	-0.89	0.830	<0.05
2.1%	10.68	257	1.49	0.92	-0.33	0.750	<0.05

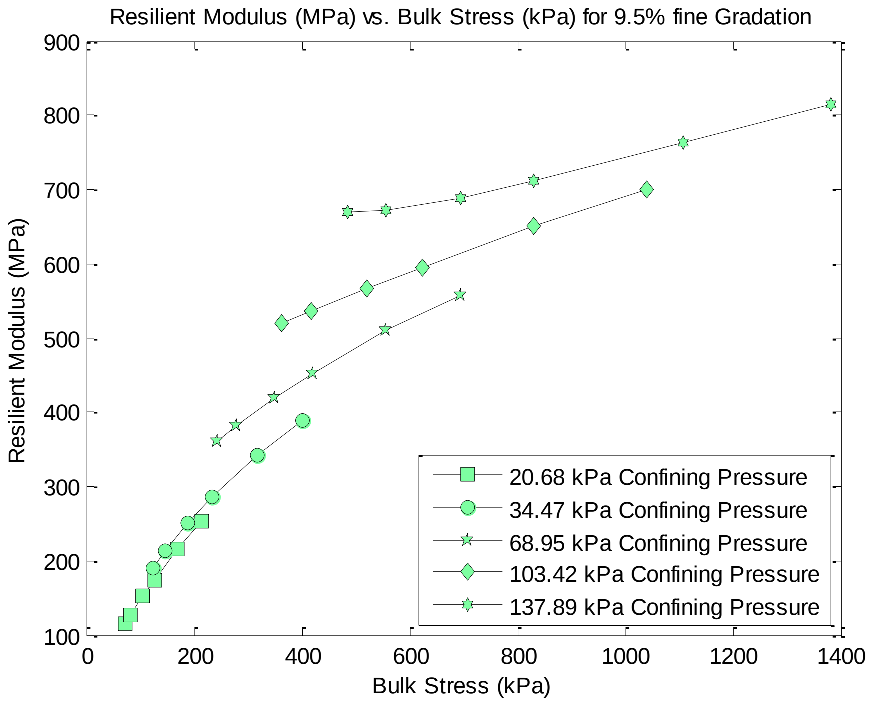


Figure 5-2 Predicted resilient modulus vs. bulk stress for 9.5% fine gradation

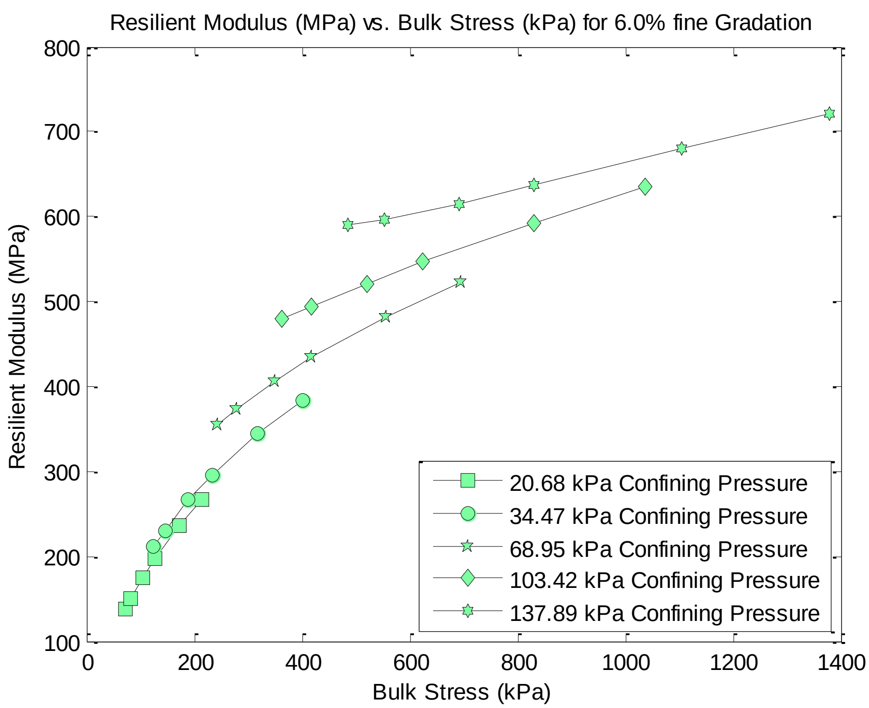


Figure 5-3 Predicted resilient modulus vs. bulk stress for 6.0% fine gradation

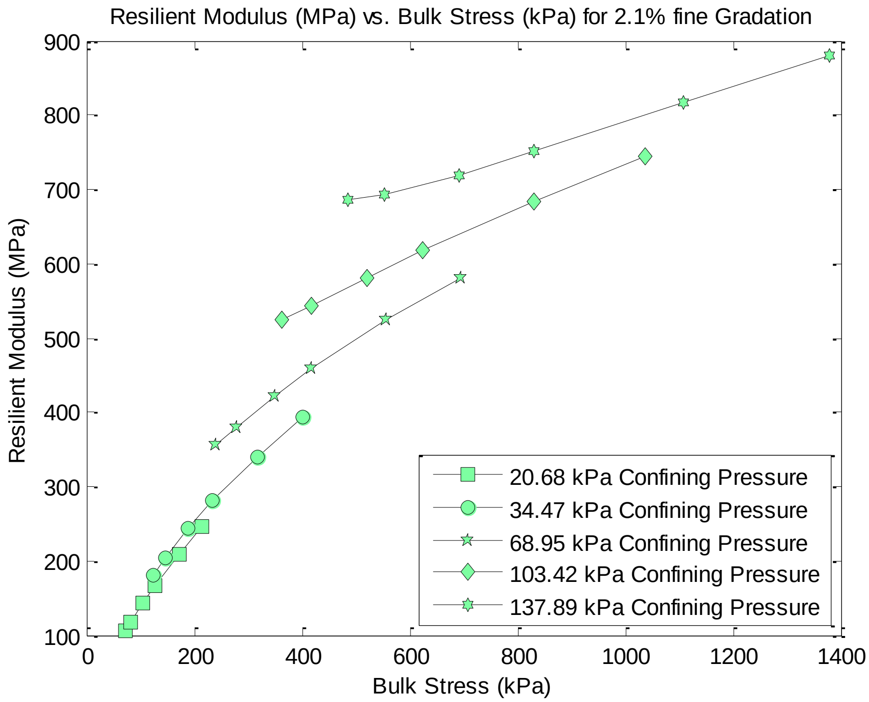


Figure 5-4 Predicted resilient modulus vs. bulk stress for 2.1% fine gradation

5.1.2 California Bearing Ratio (CBR)

Figure 5-5 shows the load-penetration curves of CBR test samples. At both penetration levels, the CBR value showed similar trend (Figure 5-6). However, compared to resilient modulus values, the sensitivity of CBR value with fine content is significant. The mechanism of CBR test is different compared to mechanism of resilient modulus test. The CBR test accounts only the penetration of a loading piston whereas the resilient modulus test accounts the material rebound response under cyclic stress at different stress ratio. A considerable drop in CBR value was observed for the percentage of fine increased from 6.0 to 9.5%. Moreover, the values obtained from the tests were higher than the previously anticipated design values. Besides the CBR value varied at 2.1% to 6.0% because of the density of the compacted gradation. The 2.1% fine gradation had a much lower density than that of 6.0% and 9.5%. Therefore, a lower CBR value has been obtained from the 2.1% fine gradation.

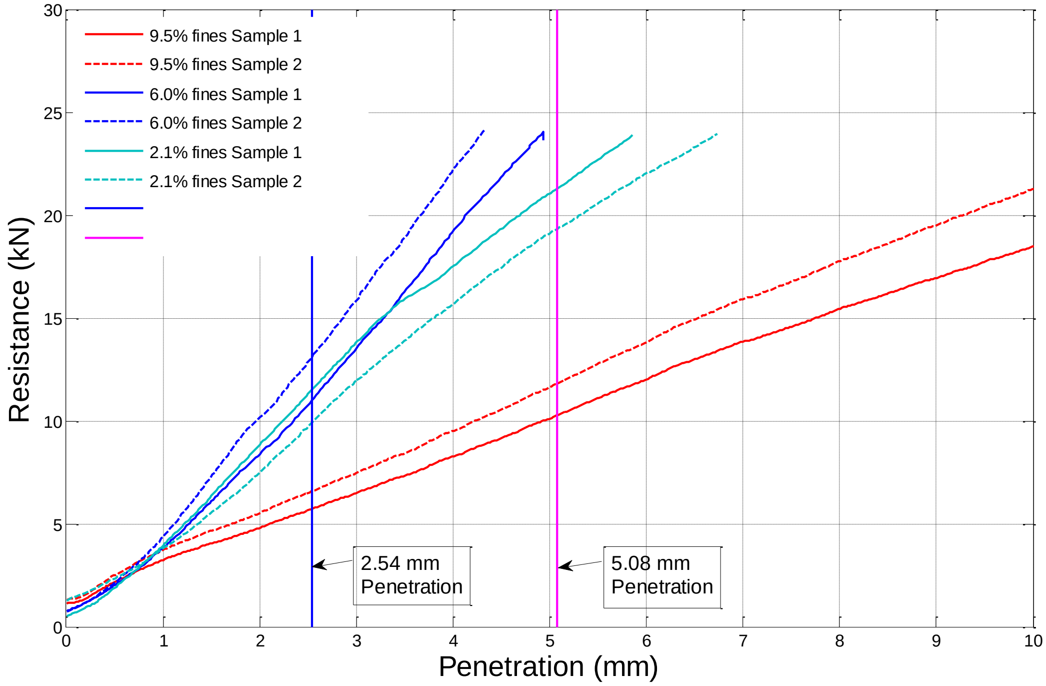


Figure 5-5 Load penetration curve for the CBR test samples of RCA gradations.

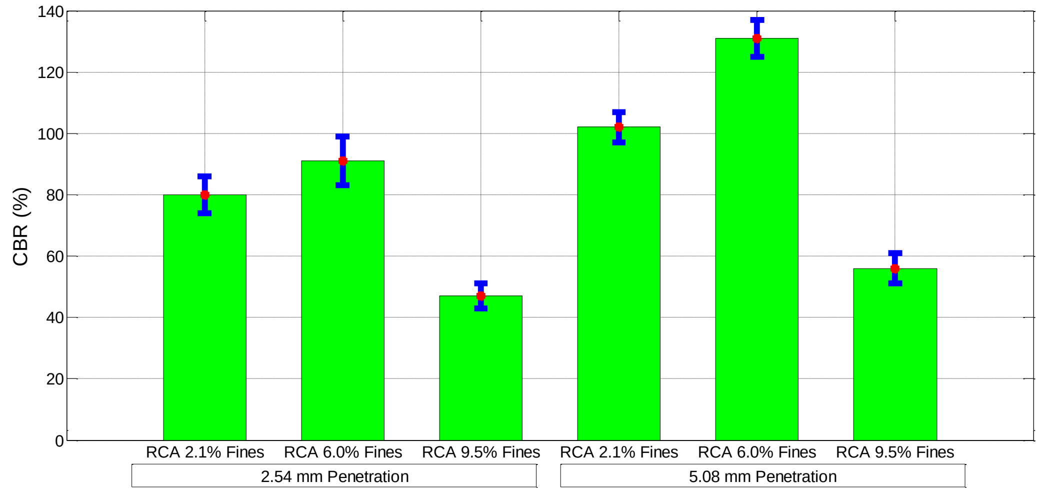


Figure 5-6 CBR values at 2.54 mm and 5.08 mm penetration levels.

5.2 STIFFNESS OF VIRGIN UNBOUND GRANULAR MATERIALS

5.2.1 Resilient Modulus

Resilient modulus test data for gravel, limestone, granite and gravel from PTH 75 highway project has been fitted with the universal constitutive model to predict the design resilient modulus. A significant increase of resilient performance have been observed as compared to the previously tested gravel, limestone and granite reported by other researcher in the same test facility (Haithem Soliman, 2015). The improved performance over the previously tested materials are mainly because of testing a coarser gradation and densely packed gradation. Furthermore, the test results have been compared with the uncrushed gravel materials tested by other researcher. The gravel materials tested in the current testing program has a crush count of at least 60 percent. Therefore, the material behaved more resilient because of the interlocking effect among the aggregates. Figure 5-7 shows the resilient modulus of gravel, limestone, granite and gravel from PTH 75 highway project. Figure 5-8 to Figure 5-11 also show the predicted resilient modulus vs. bulk stress relationship for the tested virgin base materials. Table 5.2 shows the predicted universal constitutive model parameters and predicted representative resilient modulus.

Table 5.2 Regression coefficients obtained from universal constitutive equation

Material	**Moisture Content (%)**	**Resilient Modulus (MPa)**	**K1**	**K2**	**K3**	**R^2**	**p-value**
Gravel	6.78	276	1.54	1.12	-0.59	0.925	<0.05
Gravel	6.84	290	2.06	0.6	-0.26	0.869	<0.05
Limestone	7.09	298	1.94	1.21	-1.16	0.902	<0.05
Limestone	7.39	278	1.70	0.98	-0.57	0.895	<0.05
Granite	6.59	205	1.26	0.84	-0.34	0.958	<0.05
Granite	6.34	233	1.43	0.94	-0.51	0.932	<0.05
Gravel PTH 75	8.95	185	1.02	1.13	-0.59	0.959	<0.05
Gravel PTH 75	8.39	203	1.22	0.89	-0.38	0.798	<0.05

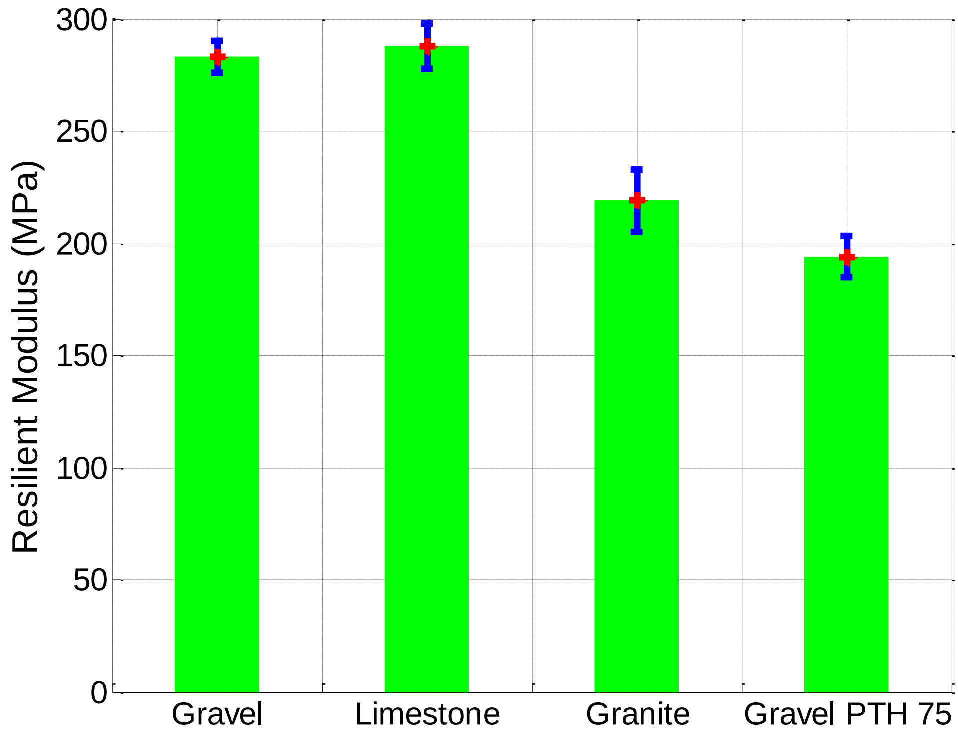


Figure 5-7 Predicted Resilient Modulus of the Gradations from Universal Constitutive Model

There is an increasing trend in resilient modulus with increase in confining pressure as well as deviator stress for all the virgin granular base materials. Therefore, it can be understood from the behaviour that, within the range of percentage of fines, there is insignificant amount of excess pore water pressure generation inside the tested specimens even in high bulk stresses. However, unlike gravel, granite and gravel from PTH 75, the limestone behaved slightly differently at high confining pressure. A decreasing trend in resilient modulus with increase in bulk stress was observed for some samples. The phenomenon can occur because of the generation of some excess pore water pressure in the small pores of a dense sample. Moreover, if the particles are densely packed, at a high confining pressure, with increase in deviator stress, the particles tend to ride on each other to move to a stable position resulting in reduction in resilient modulus value.

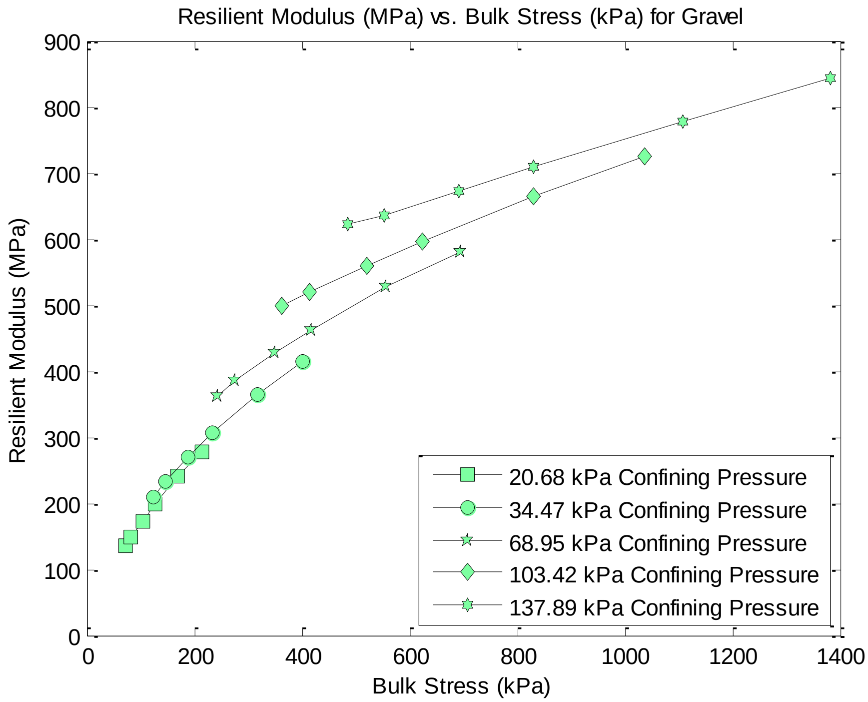


Figure 5-8 Predicted resilient modulus vs. bulk stress for gravel gradation

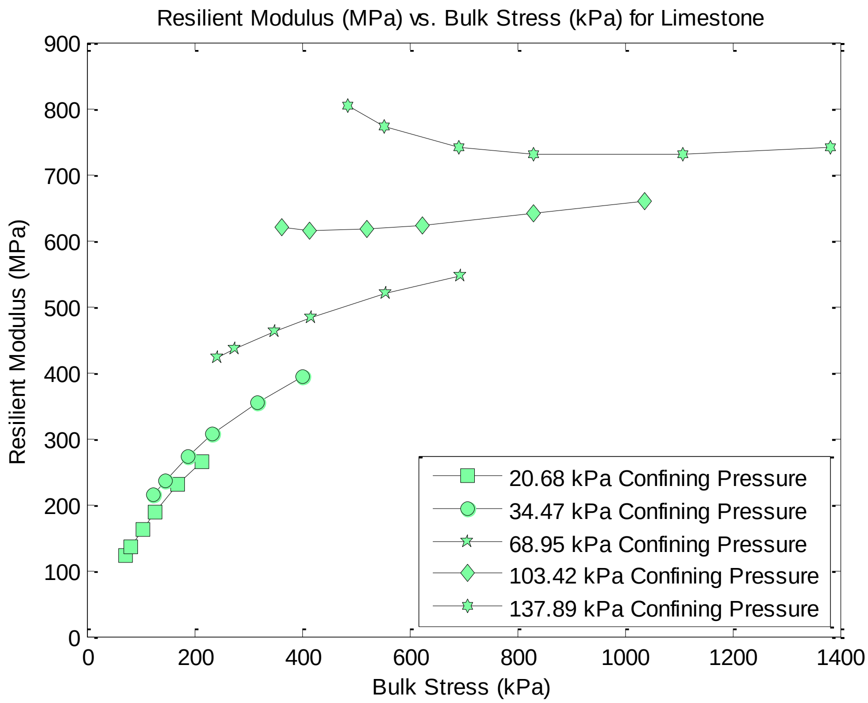


Figure 5-9 Predicted resilient modulus vs. bulk stress for limestone gradation

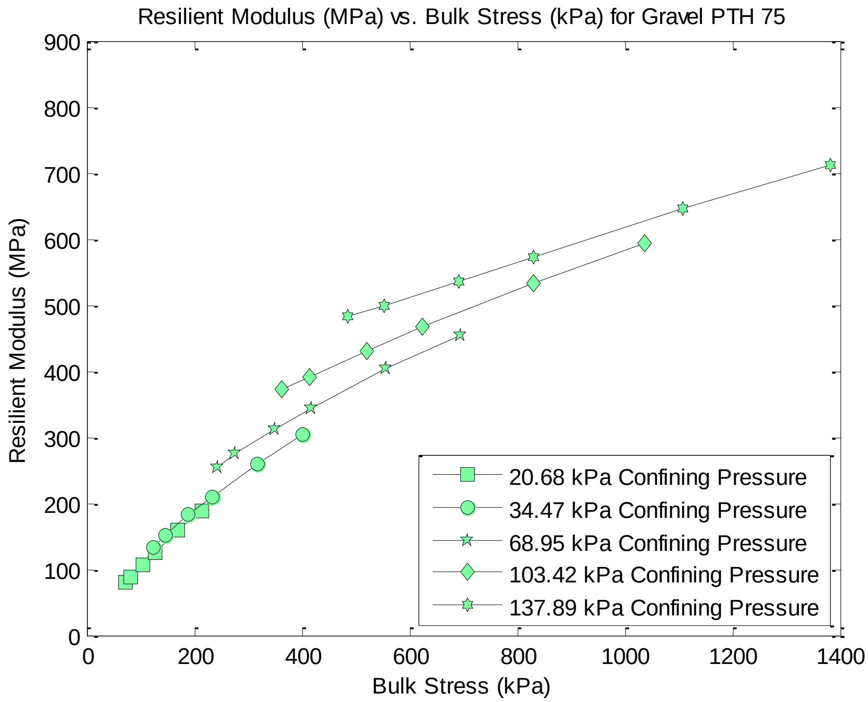


Figure 5-10 Predicted resilient modulus vs. bulk stress for gravel PTH 75 gradation

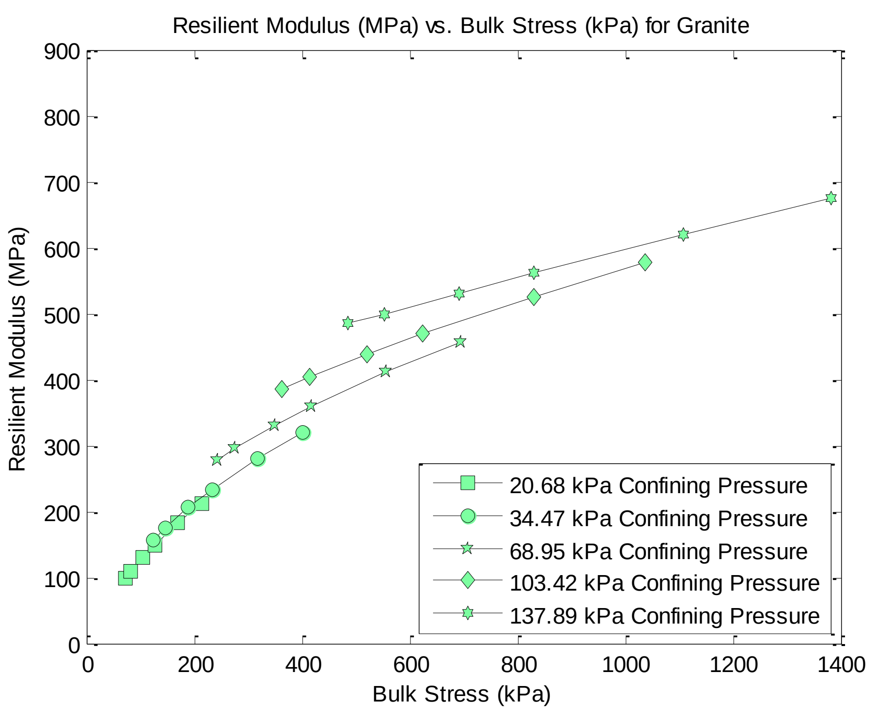


Figure 5-11 Predicted resilient modulus vs. bulk stress for granite gradation

5.3 PERMANENT DEFORMATION PERFORMANCE OF UNBOUND GRANULAR MATERIALS

Similar to resilient modulus, the resistance to permanent deformation of the tested gravel, limestone and granite was improved compared to the data reported by other researcher in the same test facility (Haithem Soliman, 2015). The improved performance over the previously reported permanent deformation performance is mainly because of increased crushed count of gravel material and the increased maximum aggregate size of the gradation. The improvement of the gravel materials has been significant because of the increased crush count. However, the gravel from PTH 75 highway project showed a higher permanent deformation rate per loading cycle in the beginning because the material has been tested at a higher moisture content then the saturation moisture content. Thus, the generation of excess pore water pressure into the sample reduced the effective stress and hence the stability. However, after few thousand loading cycles the deformation rate became stable. On the other hand, variability in the performance of the permanent deformation samples have been observed. The variability is mainly because of the sampling variability from the field. Figure 5-12 to Figure 5-15 show the permanent deformation of the tested gravel, limestone, granite and gravel from PTH 75 highway project. Table 5.3 shows the parameters obtained from the fitted model with the tested permanent deformation data.

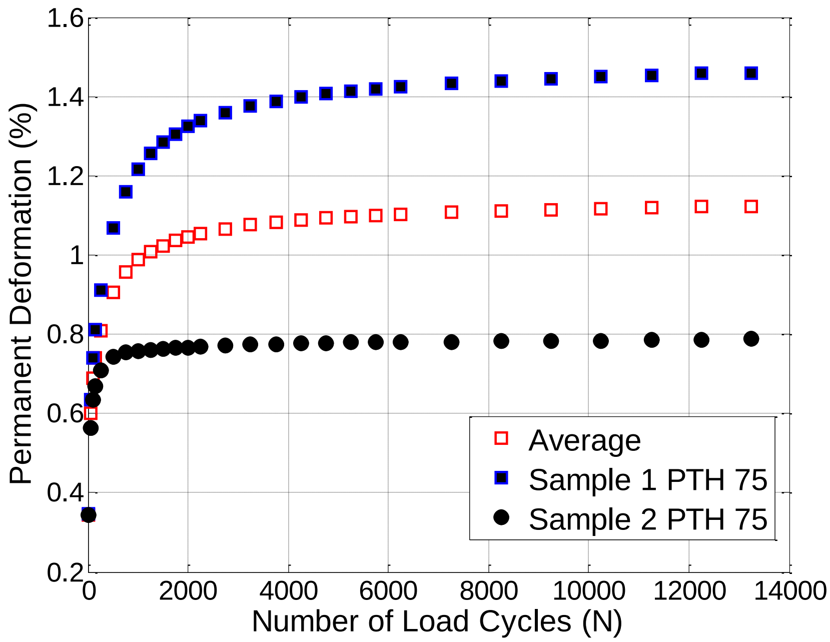


Figure 5-12 Permanent deformation of gravel PTH 75 gradation.

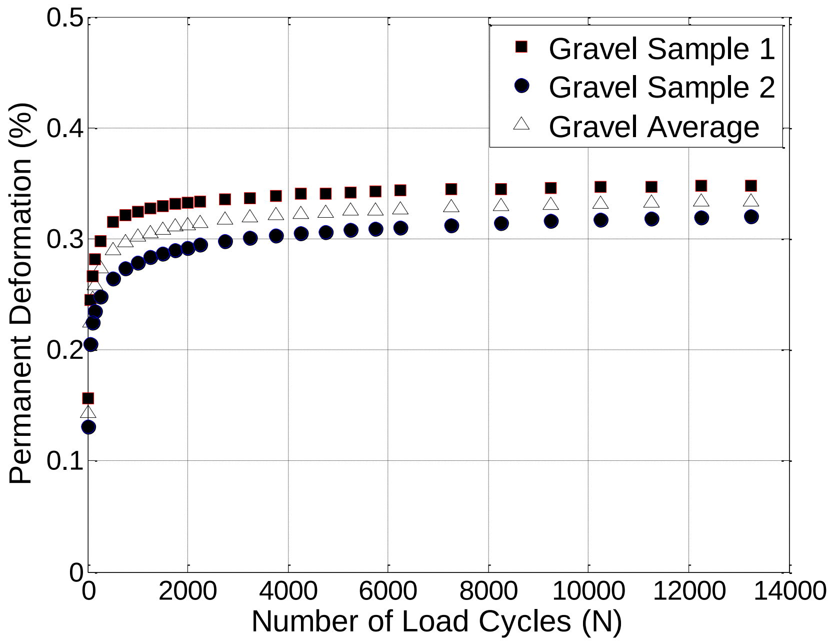


Figure 5-13 Permanent deformation of gravel gradation.

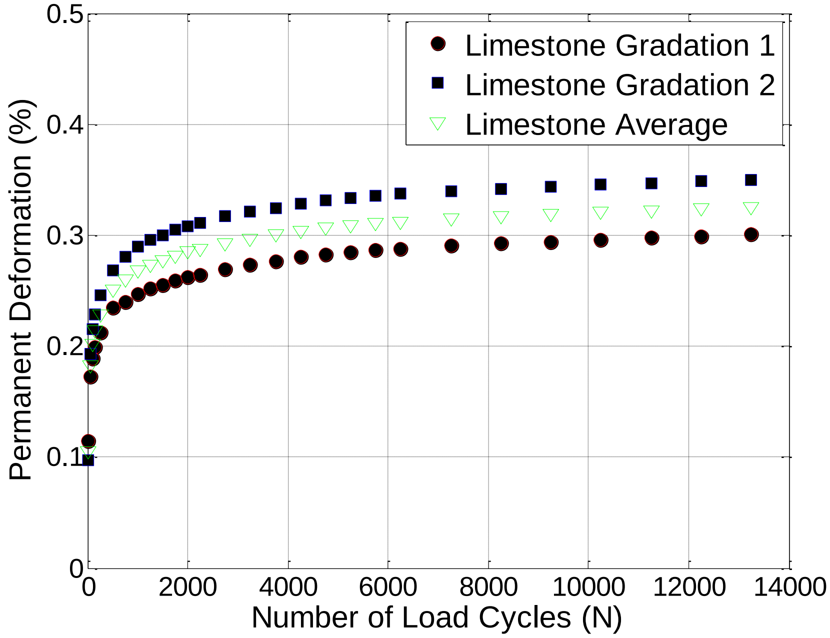


Figure 5-14 Permanent deformation of limestone gradation.

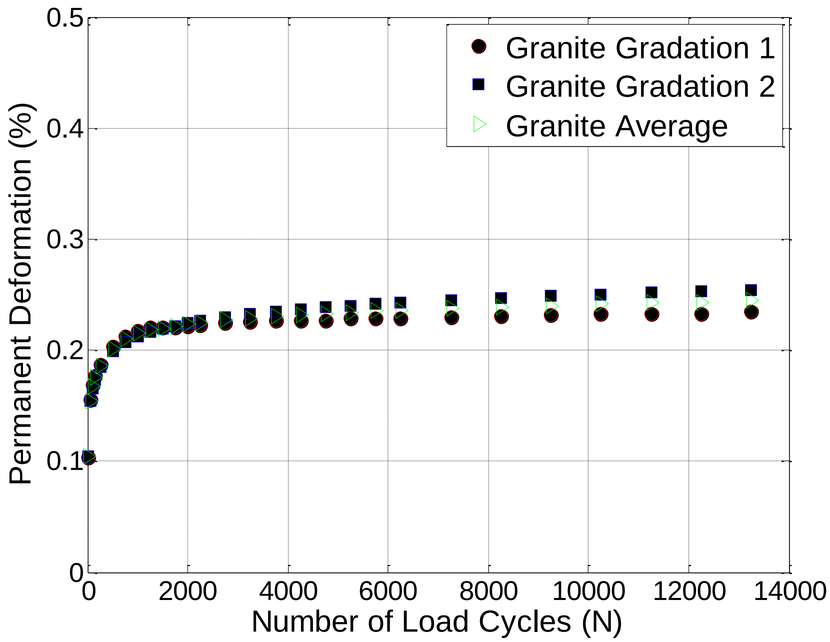


Figure 5-15 Permanent deformation of granite gradation.

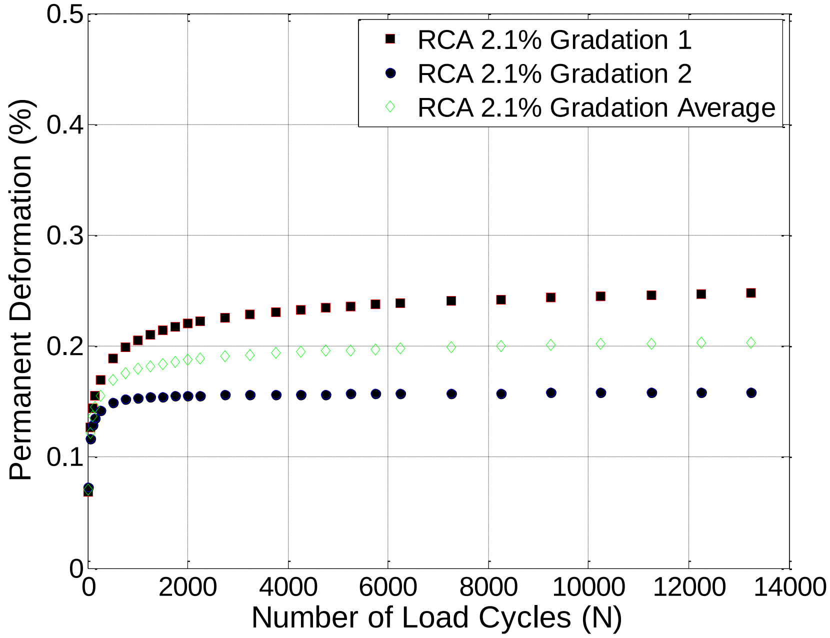


Figure 5-16 Permanent deformation of RCA 2.1% gradation.

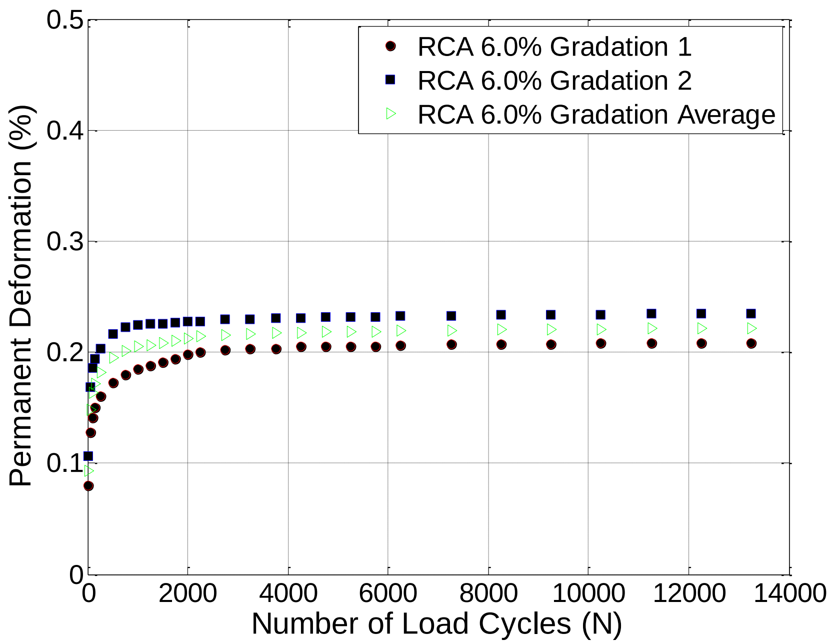


Figure 5-17 Permanent deformation of RCA 6.0% gradation.

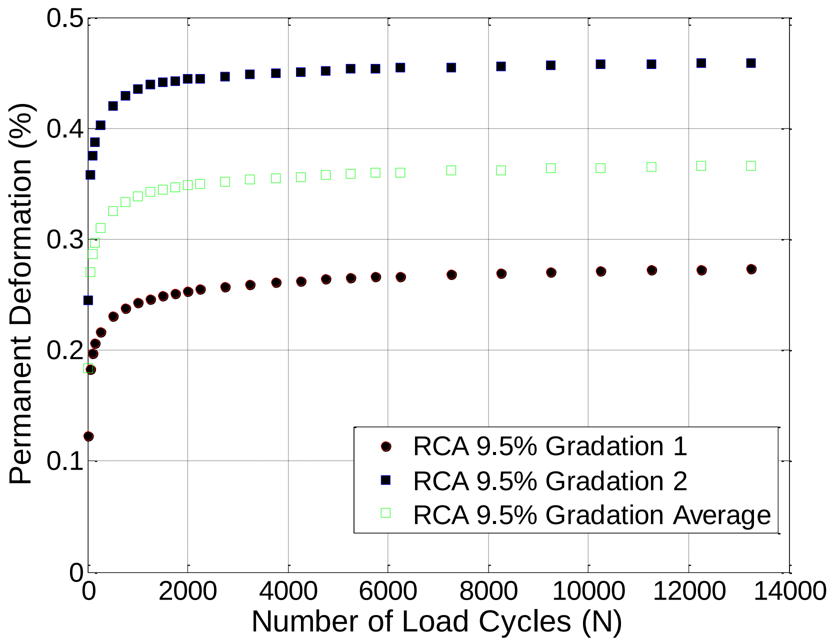


Figure 5-18 Permanent deformation of RCA 9.5% gradation.

Permanent deformation performance of recycled concrete aggregates for all the gradations have also been found to be stable (Figure 5-16 to Figure 5-18). From permanent deformation curve it can be seen that, the post compaction zone for the recycled concrete aggregate materials ends quickly since the change of deformation with increasing number of loading cycles becomes minimal.

The analyzed regression coefficient shows a very good fit with the Equation 2.3, which is used for the prediction of permanent deformation in MEPDG design.

Table 5.3 Regression coefficients obtained from Equation 2.3

Material	**Moisture Content (%)**	ε_o	ρ	β	R^2	**p-value**
Gravel	8.71	0.357	6.167	0.457	0.998	<0.05
Gravel	6.86	0.332	31.527	0.487	0.977	<0.05

Limestone	6.66	0.346	46.412	0.335	0.987	<0.05
Limestone	7.53	0.459	26.056	0.218	0.999	<0.05
Granite	6.47	0.245	8.899	0.419	0.989	<0.05
Granite	6.41	0.364	20.823	0.159	0.999	<0.05
Gravel PTH 75	8.50	1.616	106.160	0.496	0.989	<0.05
Gravel PTH 75	8.38	0.780	39.718	1.010	0.991	<0.05
RCA 2.1%	9.89	0.296	30.750	0.289	0.999	<0.05
RCA 2.1%	10.05	0.160	8.357	0.630	0.994	<0.05
RCA 6.0%	10.23	0.229	12.957	0.364	0.990	<0.05
RCA 6.0%	10.46	0.239	7.413	0.538	0.995	<0.05
RCA 9.5%	11.13	0.313	4.549	0.253	0.999	<0.05
RCA 9.5%	10.78	0.477	2.042	0.378	0.995	<0.05

5.4 RESILIENT MODULUS OF UNBOUND GRANULAR MATERIALS AND THE EFFECT OF CONDITIONING

During the resilient modulus test, conditioning is applied in order to minimize the effect of imperfect contact between test specimen and loading plate, and test sample and the base plate. The laboratory tests were performed following the NCHRP Project 1-28A, harmonized test protocol. A conditioning of 1,000 cycles have been applied to the sample at the beginning of the resilient modulus test. On the other hand, resilient modulus test has also been carried out at a different conditioning stress condition. Followed by the permanent deformation test of 13,000 cycles, another set of resilient modulus test has also been performed. The stress level of permanent deformation has been chosen from the literature, which is reported to be a representative stress level on a base layer during its operation period. Therefore, in order to evaluate the behaviour of the test samples which is already subjected to some load history, the resilient modulus test has been performed after permanent deformation test. It is noteworthy to mention that, the conditioning stress prior to resilient modulus test is higher than that the stress level applied in the permanent deformation test. Figure 5-19 shows the stress path of the two different types of conditioning before the resilient modulus test. Furthermore, it can also be seen from the Figure 5-19 that the stress ratio is different for two stress paths. In resilient modulus test, the stress ratio remains same for every five consecutive sequences and increases hereafter. Figure 5-20 shows the changes in bulk stress with change in stress sequences.

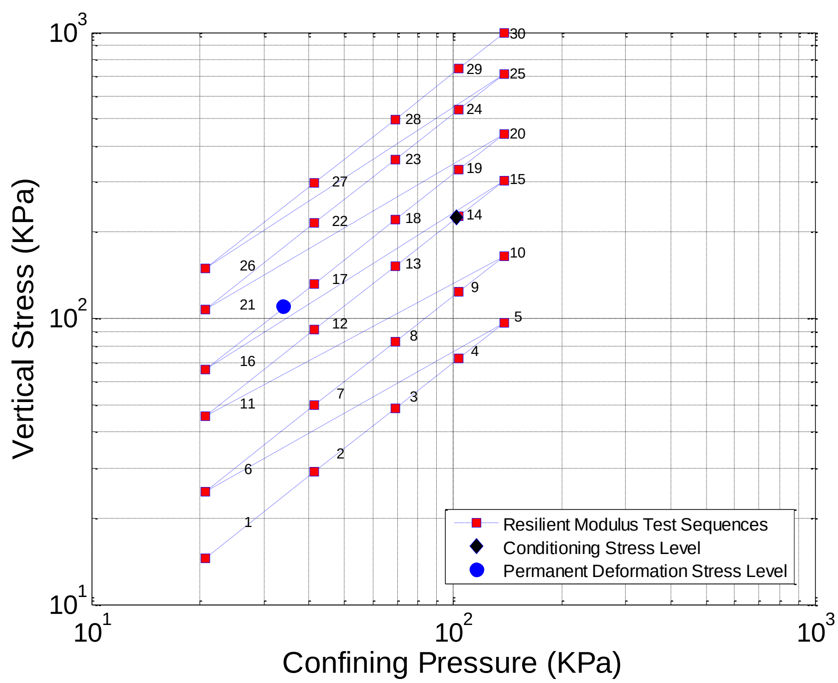


Figure 5-19 Stress path for conditioning of resilient modulus test vs. permanent deformation test (different conditioning stress have been applied to at least two samples of each gradations)

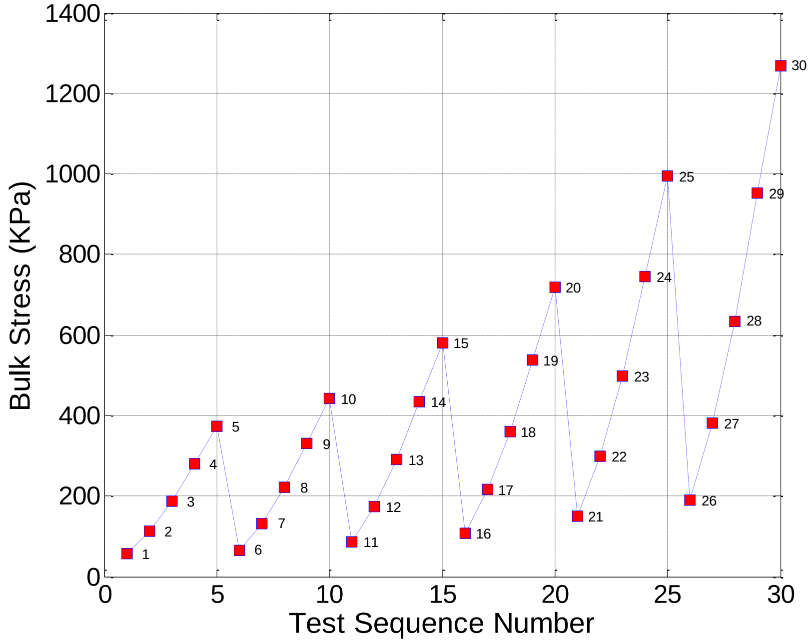


Figure 5-20 Bulk stress vs test sequences in resilient modulus test

The prediction of resilient modulus from the universal constitutive model for the resilient modulus test after permanent deformation test has been superimposed over the plot of prediction of resilient modulus test according to NCHRP harmonized protocol (Figure 5-21to Figure 5-27).

As a result of permanent deformation test before resilient modulus test, two types of behaviours have been observed. However, it is clear from the Figure 5-21 to Figure 5-27 that, there is effect of different conditions of conditioning stress and duration on the resilient modulus value of unbound materials. The curves of the prediction of resilient modulus vs. bulk stress for after permanent deformation condition for each material have been superimposed over the traditional resilient modulus test prediction of resilient modulus vs. bulk stress. Behaviour of both conditions in a same plot shows the clear difference between the mechanisms. No significant increase in resilient modulus for tested gravel and limestone samples have been observed. However, at different confining pressure, the behaviour of resilient modulus vs. bulk stress changed. In case of gravel, decreasing trend in resilient modulus with increase in bulk stress has been observed. The phenomenon can occur because of the generation of some excess pore water pressure in the small pores of a dense sample. Moreover, if the particles are densely packed, at a high confining pressure, with increase in deviator stress, the particles tend to ride on each other to move to a stable position resulting in reduction in resilient modulus value.

On the contrary, in case of limestone, the behaviour was opposite, the resilient modulus vs. bulk stress trend was higher for the resilient modulus sample after permanent deformation compared to the traditional resilient modulus test. The higher trend in resilient modulus is mainly because of the progressive densification and moisture removal as effect of permanent deformation test.

Furthermore, significant increase in resilient modulus after permanent deformation test has been observed compared to the traditional resilient modulus of the materials for the granite and PTH 75 gravel samples. The increased resilient modulus value was anticipated because of the progressive densification and moisture removal of permanent deformation test. The regression coefficients for the universal constitutive model for the resilient modulus test after permanent deformation test as conditioning has been shown in Table 5.4. The model has a good fit with the tested data and the regression coefficients obtained also have significant effect on the model prediction.

An overall increase in resilient modulus for recycled concrete aggregate have been found. However, it has also been found that, the behaviour of the material at high confining pressure changed. The changed behaviour is either due to increased excess pore water pressure or due to dilation of granular materials.

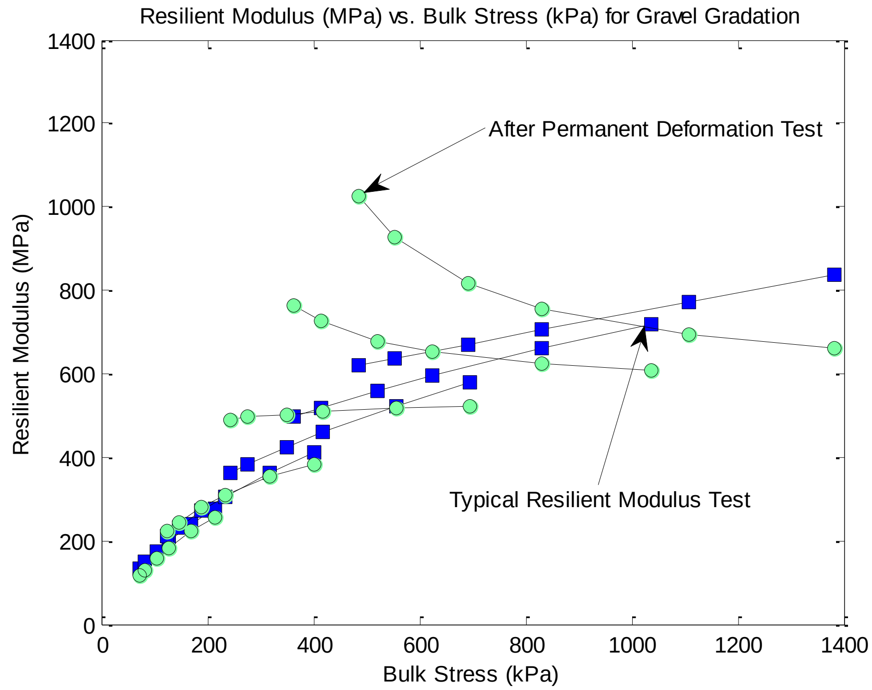


Figure 5-21 Predicted resilient modulus vs. bulk stress for gravel gradation

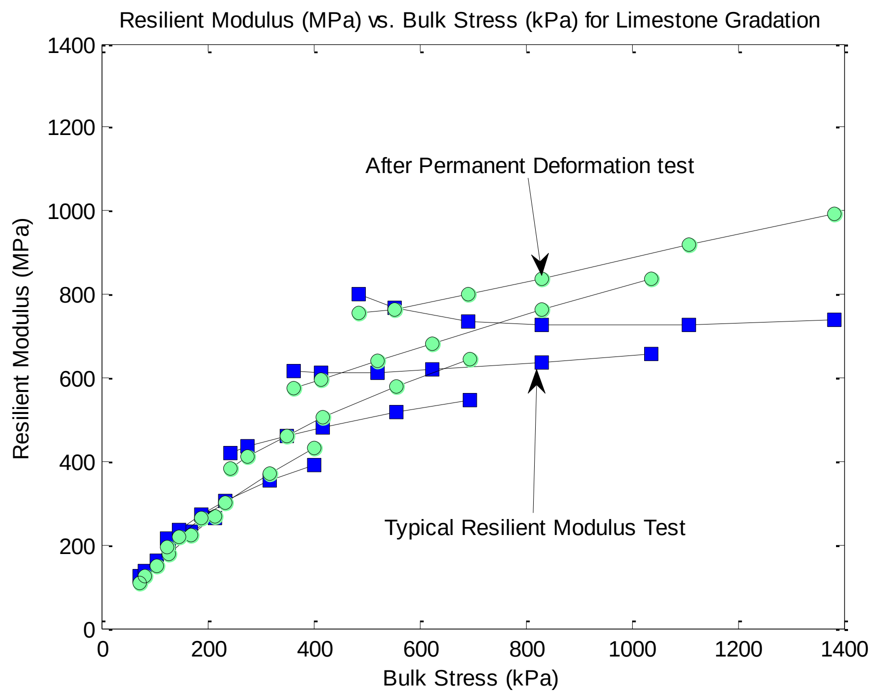


Figure 5-22 Predicted resilient modulus vs. bulk stress for limestone gradation

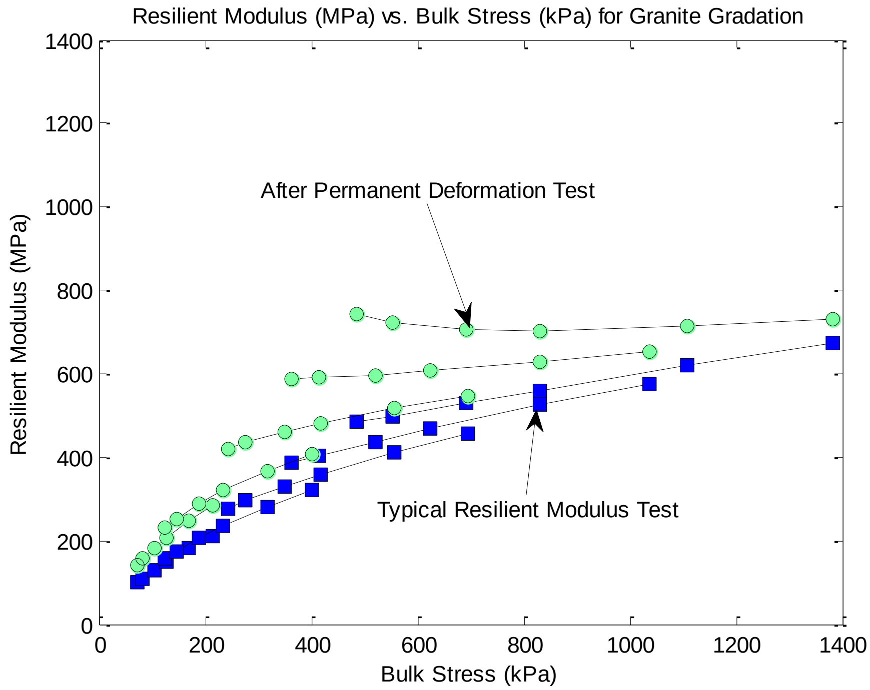


Figure 5-23 Predicted resilient modulus vs. bulk stress for granite gradation

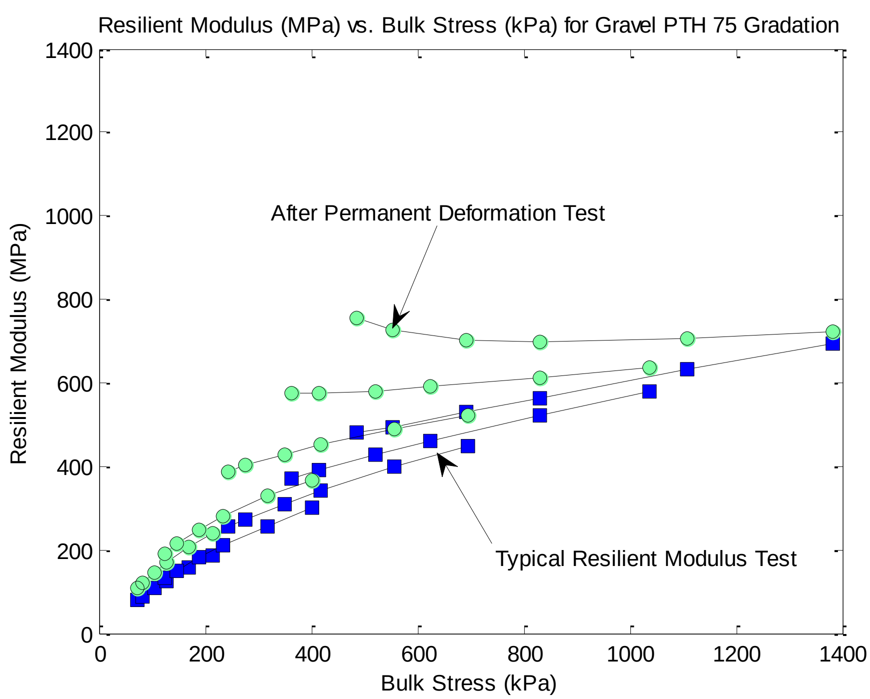


Figure 5-24 Predicted resilient modulus vs. bulk stress for gravel PTH 75 gradation

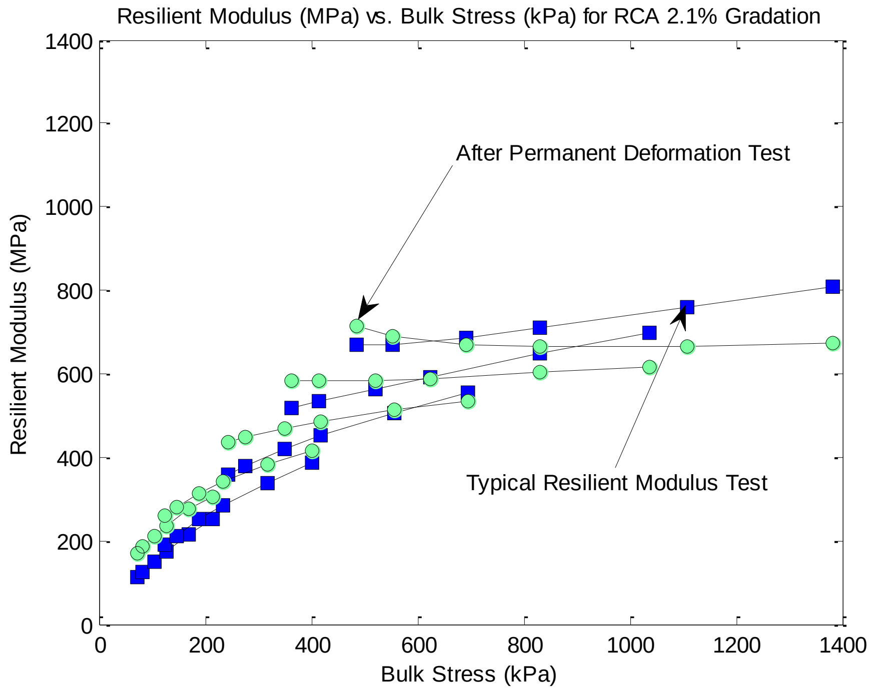


Figure 5-25 Predicted resilient modulus vs. bulk stress for RCA 2.1% gradation

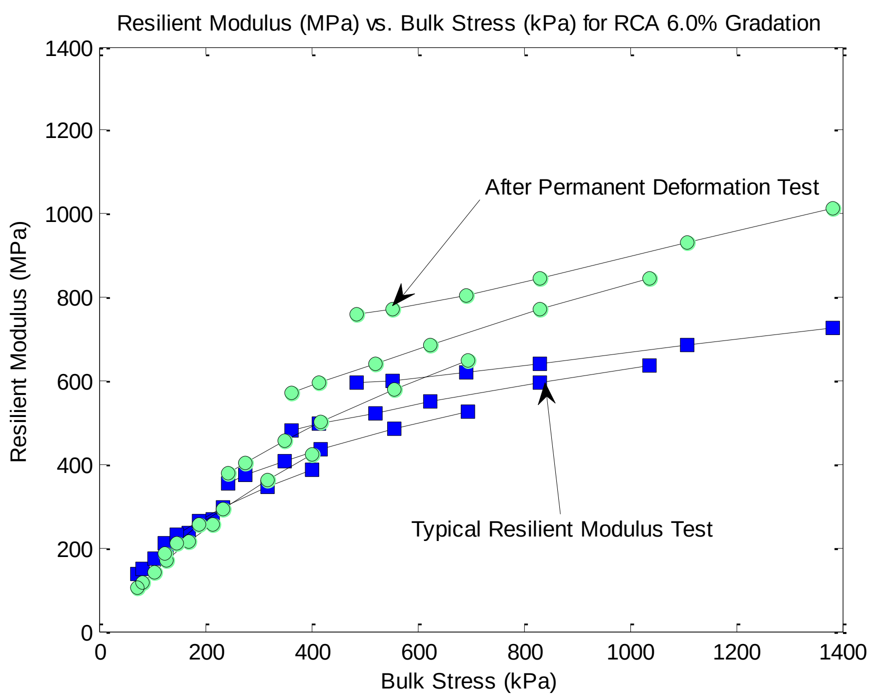


Figure 5-26 Predicted resilient modulus vs. bulk stress for RCA 6.0% gradation

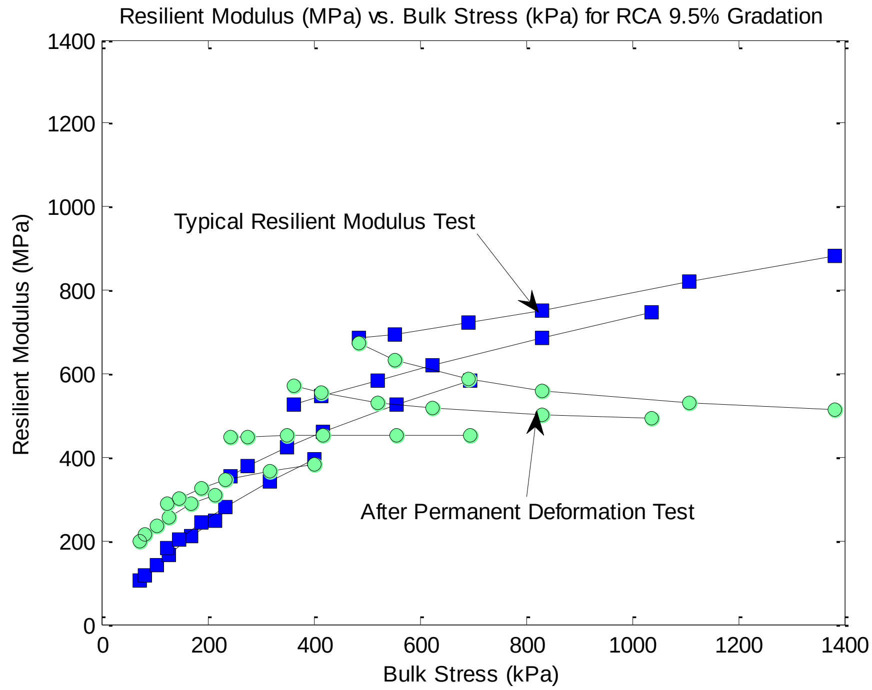


Figure 5-27 Predicted resilient modulus vs. bulk stress for RCA 9.5% gradation

Table 5.4 Regression coefficients obtained from universal constitutive equation for the resilient modulus test after permanent deformation test conditioning.

Material	**Moisture Content (%)**	**Resilient Modulus (MPa)**	**K1**	**K2**	**K3**	**R^2**	**p-value**
Gravel	6.71	302	2.21	1.28	-1.59	0.818	<0.05
Gravel	6.86	284	1.56	1.33	-0.95	0.809	<0.05
Limestone	6.66	282	1.72	1.01	-0.63	0.909	<0.05
Limestone	7.53	281	1.50	1.17	-0.59	0.767	<0.05
Granite	6.47	272	4.06	1.10	-3.09	0.967	<0.05
Granite	6.41	306	2.00	0.96	-0.72	0.936	<0.05
Gravel PTH 75	8.50	274	1.48	0.97	-0.26	0.858	<0.05
Gravel PTH 75	8.38	257	1.77	1.29	-1.47	0.775	<0.05
RCA 2.1%	9.89	301	2.11	0.98	-0.92	0.910	<0.05
RCA 2.1%	10.05	365	2.48	0.69	-0.39	0.926	<0.05

RCA 6.0%	10.23	273	1.53	1.13	-0.63	0.503	<0.05
RCA 6.0%	10.46	-	-	-	-	-	-
RCA 9.5%	11.13	327	2.37	0.71	-0.52	0.822	<0.05
RCA 9.5%	10.78	344	2.85	0.75	-0.94	0.655	<0.05

5.5 PERMANENT DEFORMATION OF UNBOUND GRANULAR MATERIALS AND SHAKEDOWN CLASSIFICATION

5.5.1 Classification in Terms of Accumulated Permanent Strain

Shakedown classification of permanent deformation behaviour is done based on the amount of accumulated vertical permanent strain due to 3,000 to 5,000 loading cycles. The boundary of the shakedown ranges 1 and 2 is 4.5 x 10-5 mm/mm vertical permanent strain and the boundary of the shakedown ranges 2 and 3 is 4.0 x 10-4 mm/mm vertical permanent strain. Table 5.5 summarizes and classifies the shakedown range of the permanent deformation of the tested unbound granular materials. Incremental collapse has been observed for one sample of gravel from PTH 75 project. This behaviour is because of the presence of excess water into the sample. The sample was compacted into the field moisture content. There was also variability in sampling from the field. Variability in results of PTH 75 project gravel material have been found.

Table 5.5 Shakedown classification in terms of accumulated permanent strain between 3,000 and 5,000 cycles.

Material	Moisture Content (%)	Accumulated Vertical Strain between 3,000 and 5,000 loading cycles	Classification
Gravel	6.71	4.24 E-5	Plastic Shakedown
Gravel	6.86	7.25 E-5	Plastic Creep
Limestone	6.66	1.07 E-4	Plastic Creep
Limestone	7.53	1.23 E-4	Plastic Creep

Granite	6.47	3.80 E-5	Plastic Shakedown
Granite	6.41	8.33 E-5	Plastic Creep
Gravel PTH 75	8.50	5.82 E-4	Incremental Collapse
Gravel PTH 75	8.38	2.87 E-5	Plastic Shakedown
RCA 2.1%	9.89	8.43 E-5	Plastic Creep
RCA 2.1%	10.05	1.05 E-5	Plastic Shakedown
RCA 6.0%	10.23	4.72 E-5	Plastic Creep
RCA 6.0%	10.46	2.19 E-5	Plastic Shakedown
RCA 9.5%	11.13	6.12 E-5	Plastic Creep
RCA 9.5%	10.78	5.03 E-5	Plastic Creep

5.5.2 Classification in Terms of Rate of Strain

The tested samples have also been classified in terms of the rate of strain. Figure 5-28 to Figure 5-29 show the classified shakedown ranges from the shape of the rate of strain. Most of the tested samples shows plastic shake down behaviour according to this classification since a progressive reduction in permanent strain rate is observed. However, the tested gravel from PTH 75 highway project samples have been classified as plastic creep range and incremental collapse range. The higher deformation is due to the moisture content of the test sample which was more than the saturation moisture content. Variability of the results in the gravel PTH 75 highway project sample was due to the variability of field sampling. All the gradations of recycled concrete aggregates have been found to be into the plastic shakedown or plastic creep limit in terms of the rate of strain.

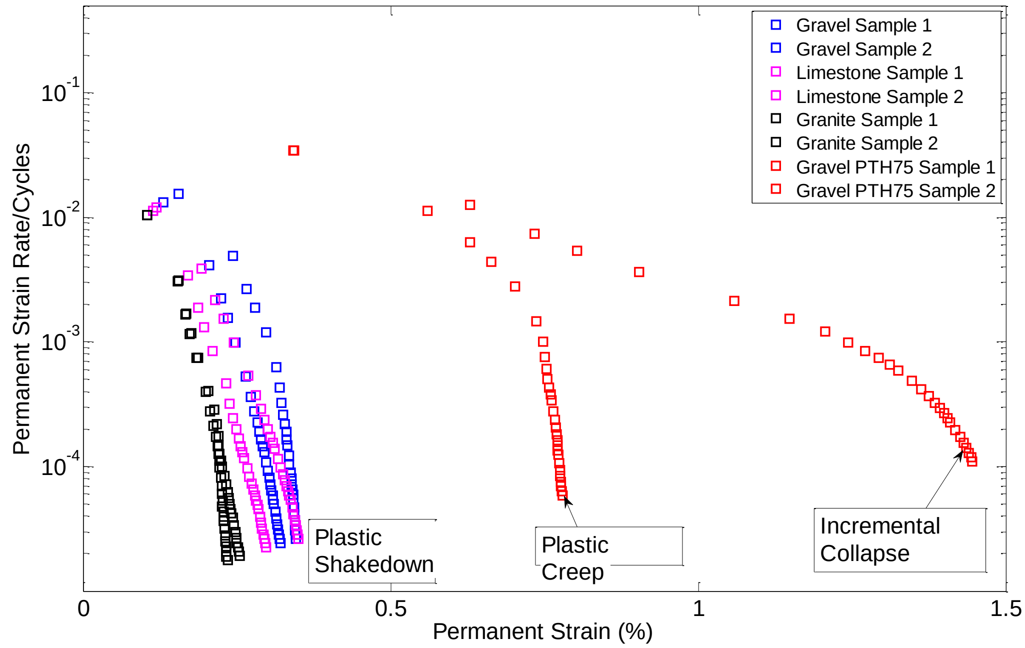


Figure 5-28 Shakedown classification for virgin UGM based on rate of strain.

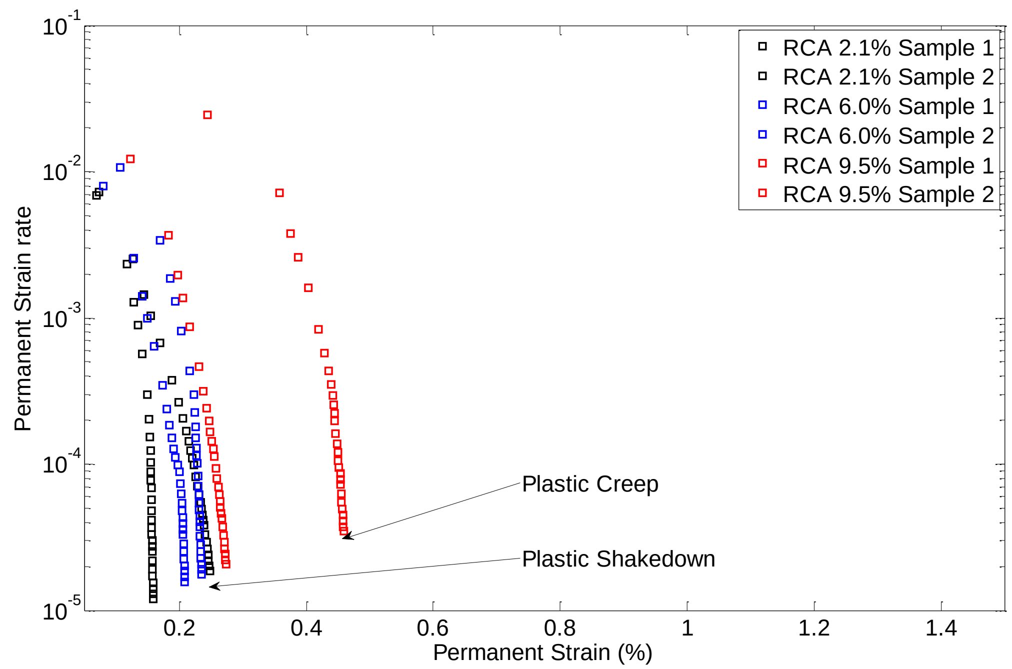


Figure 5-29 Shakedown classification for recycled UGM based on rate of strain.

5.5.3 Classification in Terms of Resilient Strain

The tested samples behaviour have also been classified in terms of resilient strain during the permanent deformation test. All the tested samples are either plastic creep or plastic shakedown range except the Gravel PTH 75 sample which has been tested at a higher moisture content. Significant change in the resilient strain value for the sample has been observed. It is a representative behaviour of the material at the incremental collapse range. Table 5.6 summarizes the classification of shakedown according to the three discussed criteria. Figure 5-30 to Figure 5-31 show the resilient strains during the permanent deformation test for gravel, limestone, granite, and gravel PTH 75 project samples. The tested recycled materials have also been found to behave in either plastic shakedown or plastic creep stage of shakedown behaviour.

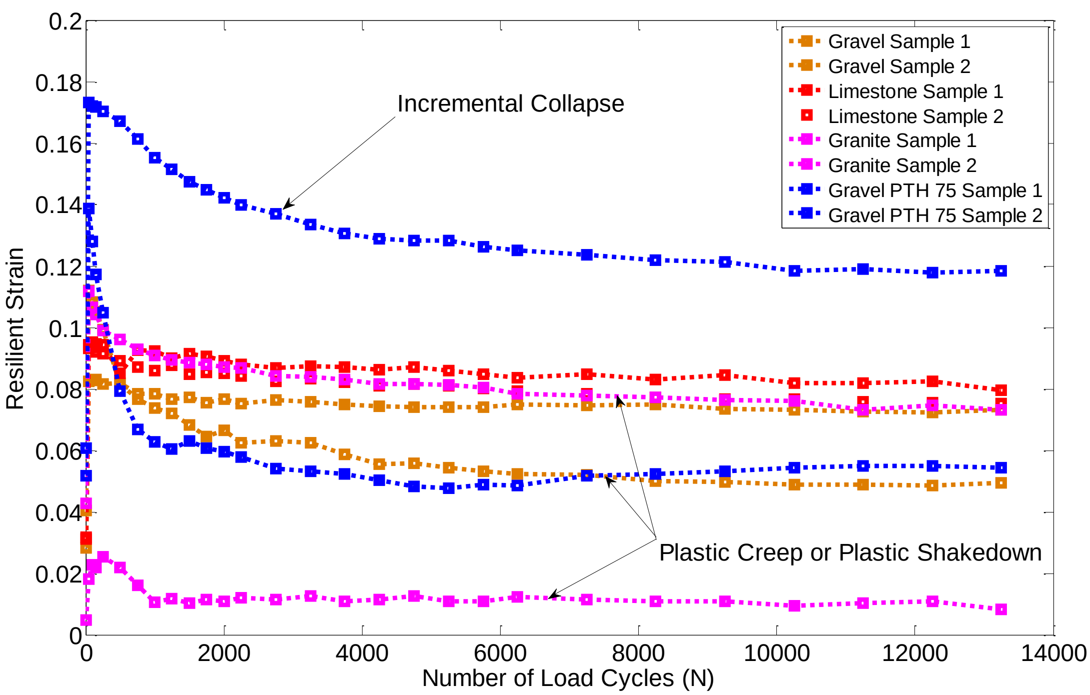


Figure 5-30 Shakedown classification of virgin UGM based on resilient strain.

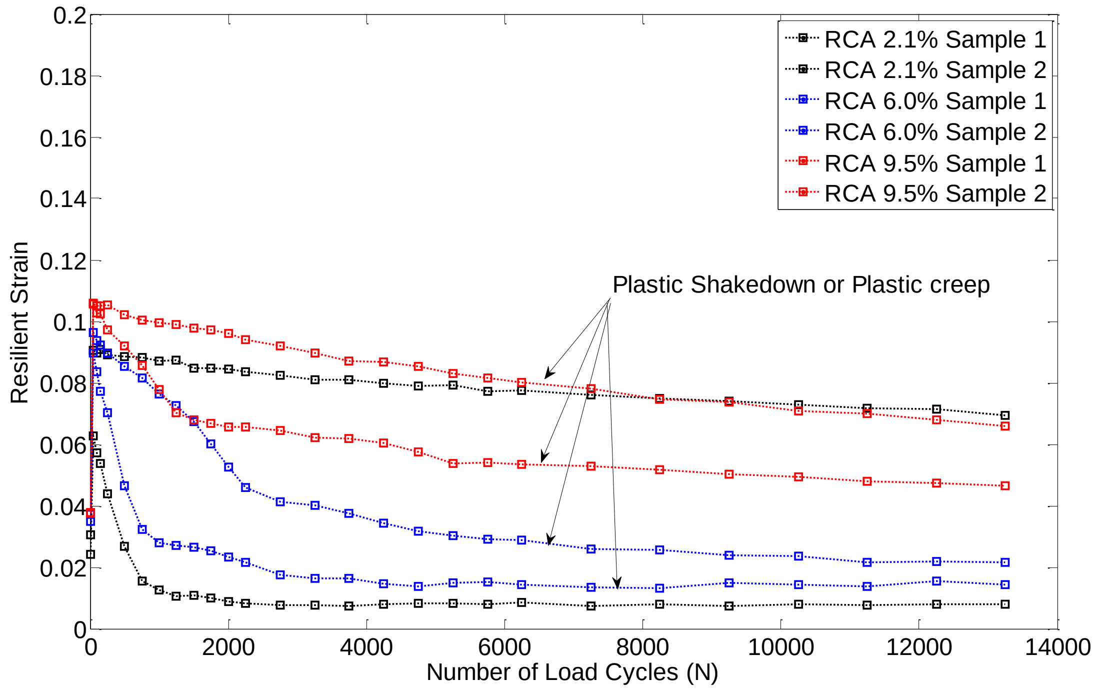


Figure 5-31 Shakedown classification of recycled UGM based on resilient strain.

Table 5.6 Shakedown classification summary

Material	**Moisture Content (%)**	**Classification Based on**		
		Accumulated Vertical Strain between 3,000 to 5,000 loading Cycles	**Resilient Strain**	**Permanent Strain Rate**
Gravel	6.71	Plastic Shakedown	Plastic Shakedown or Creep	Plastic Shakedown
Gravel	6.86	Plastic Creep	Plastic Shakedown or Creep	Plastic Shakedown
Limestone	6.66	Plastic Creep	Plastic Shakedown or Creep	Plastic Shakedown
Limestone	7.53	Plastic Creep	Plastic Shakedown or Creep	Plastic Shakedown
Granite	6.47	Plastic Shakedown	Plastic Shakedown or Creep	Plastic Shakedown
Granite	6.41	Plastic Creep	Plastic Shakedown or Creep	Plastic Shakedown

Gravel PTH 75	8.50	Incremental Collapse	Incremental Collapse	Incremental Collapse
Gravel PTH 75	8.38	Plastic Shakedown	Incremental Collapse	Plastic Creep
RCA 2.1%	9.89	Plastic Creep	Plastic Shakedown or Creep	Plastic Shakedown
RCA 2.1%	10.05	Plastic Shakedown	Plastic Shakedown or Creep	Plastic Shakedown
RCA 6.0%	10.23	Plastic Creep	Plastic Shakedown or Creep	Plastic Shakedown
RCA 6.0%	10.46	Plastic Shakedown	Plastic Shakedown or Creep	Plastic Shakedown
RCA 9.5%	11.13	Plastic Creep	Plastic Shakedown or Creep	Plastic Shakedown
RCA 9.5%	10.78	Plastic Creep	Plastic Shakedown or Creep	Plastic Creep

5.6 DISSIPATED ENERGY APPROACH

Dissipated energy approach has been used to capture the permanent deformation behaviour of unbound granular materials tested. The stress-strain for the last five cycles in each test sequences have been plotted to interpret the dissipation energy per loading cycles at different stages of permanent deformation test. As an elastoplastic material, the unbound granular materials forms a hysteresis loop during each cycle of loading pulse. The area under the hysteresis loop is the amount of energy dissipated into the materials because of application of load. Dissipation of high amount of energy dissipation means the material is undergoing higher rate of strain compared to the low amount of energy dissipation. At the beginning of the permanent deformation test, a higher amount of energy dissipation is observed. The higher energy dissipation is caused by the movement of particles at the post compaction zone of permanent deformation behaviour. However, with the application of further loading cycles, the unbound granular materials become denser and loses moisture resulting in reduction in dissipated energy. Figure 5-32 to Figure 5-59 show the stress-strain hysteresis loop and the dissipated energy with number of loading cycles gravel, gravel from

PTH 75, limestone, granite and recycled concrete aggregate materials at the test moisture contents. The dissipated energy for permanent deformation test sequences have been found to be lower than the previously reported gravel and limestone materials having finer gradation tested at the same test facility. The dissipation of energy reduced and became stable as anticipated after a few thousands loading cycles. Similar behaviour to the shakedown behaviour can also be seen into the dissipated energy approach for all the tested materials. Variability among two samples from the PTH 75 highway project materials has been observed. The variability in behaviour is caused by the variability of field sampling of the gravel materials from the project site.

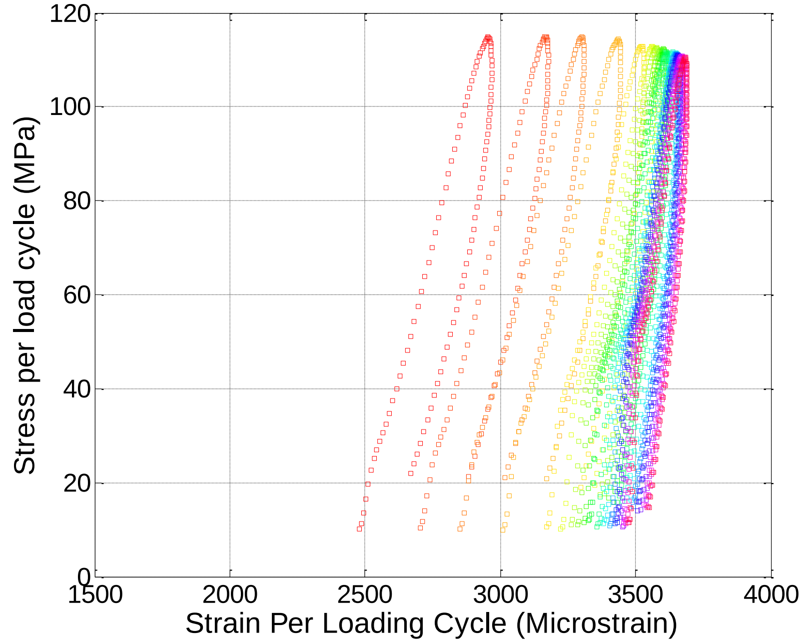


Figure 5-32 Stress strain behaviour of gravel at different permanent deformation sequences at 6.71% moisture content

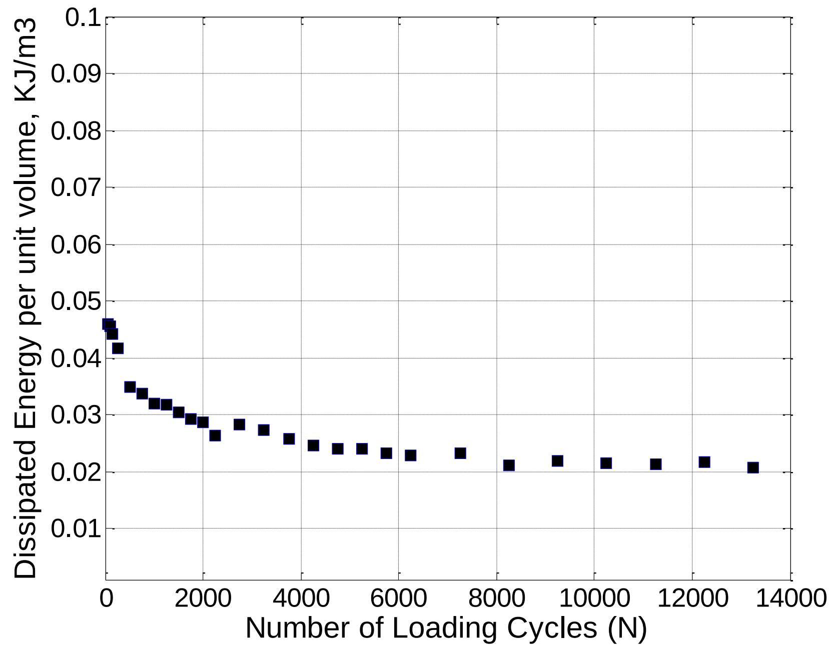


Figure 5-33 Average dissipated energy per cycle of gravel at 6.71% moisture content

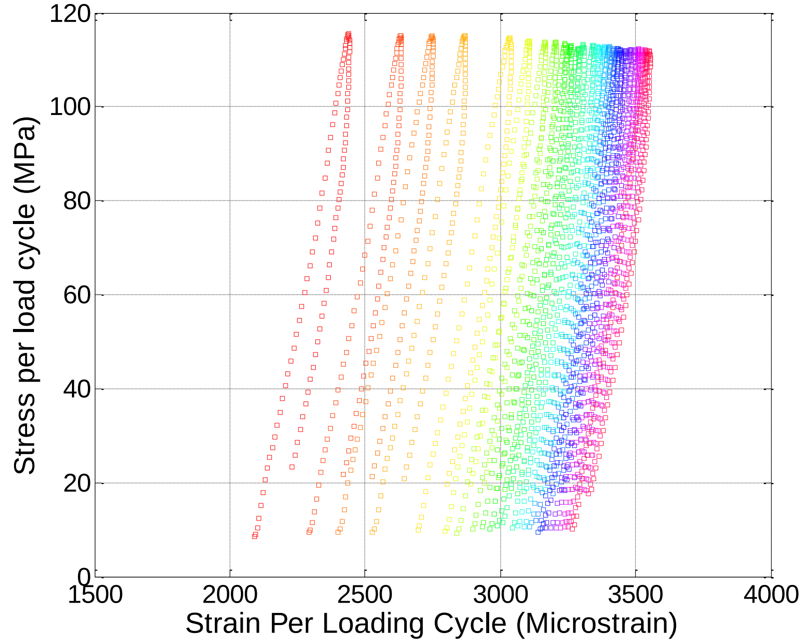


Figure 5-34 Stress strain behaviour of gravel at different permanent deformation sequences at 6.86% moisture content

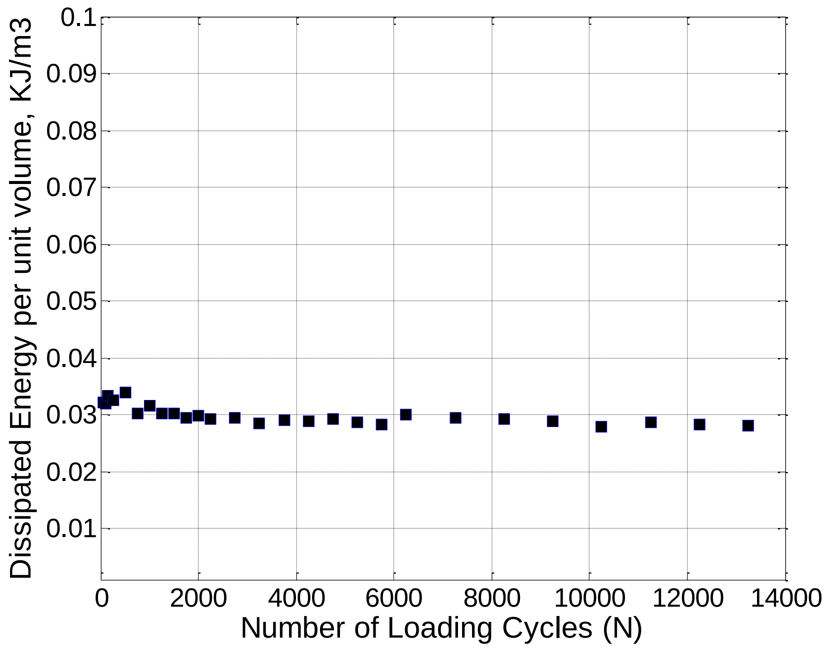


Figure 5-35 Average dissipated energy per cycle of gravel at 6.86% moisture content

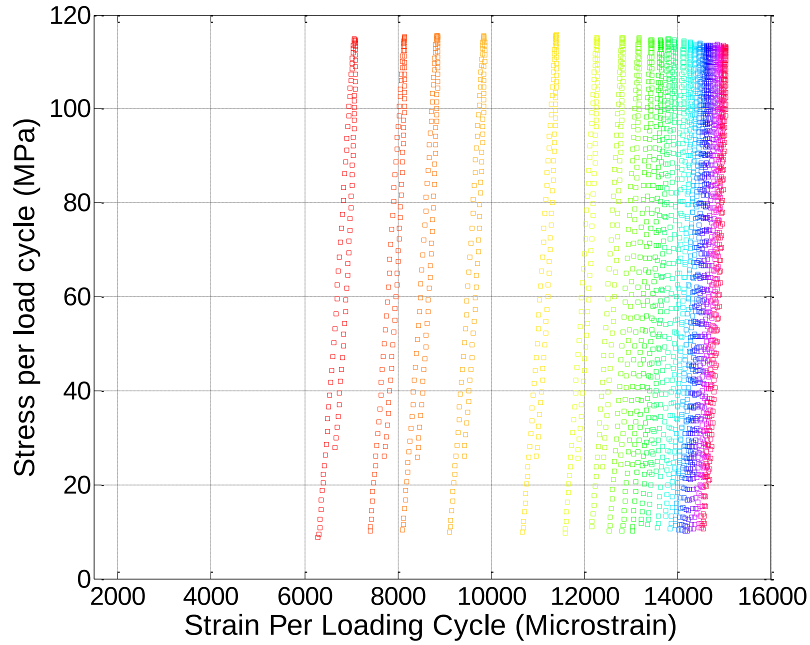


Figure 5-36 Stress strain behaviour of gravel PTH 75 at different permanent deformation sequences at 8.50% moisture content

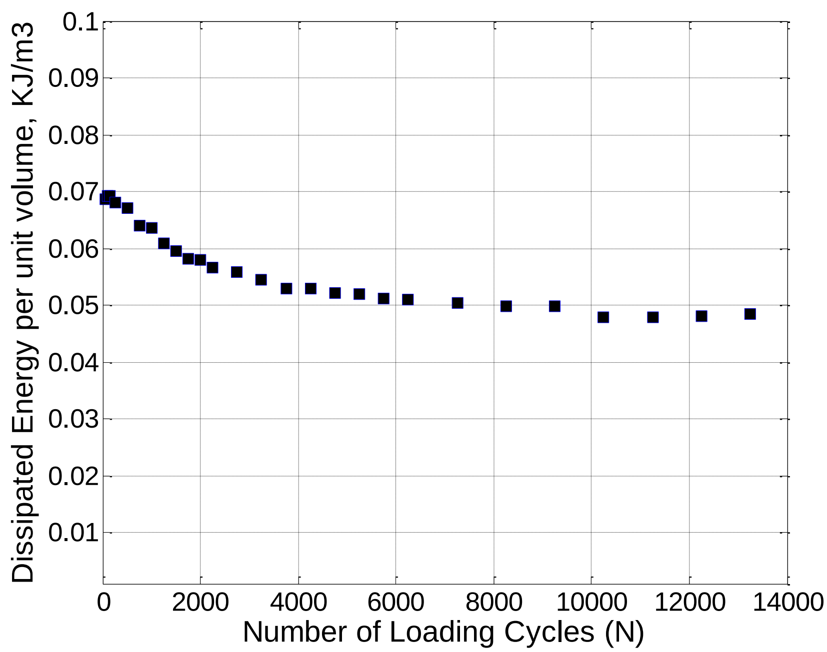


Figure 5-37 Average dissipated energy per cycle of gravel PTH 75 at 8.50% moisture content

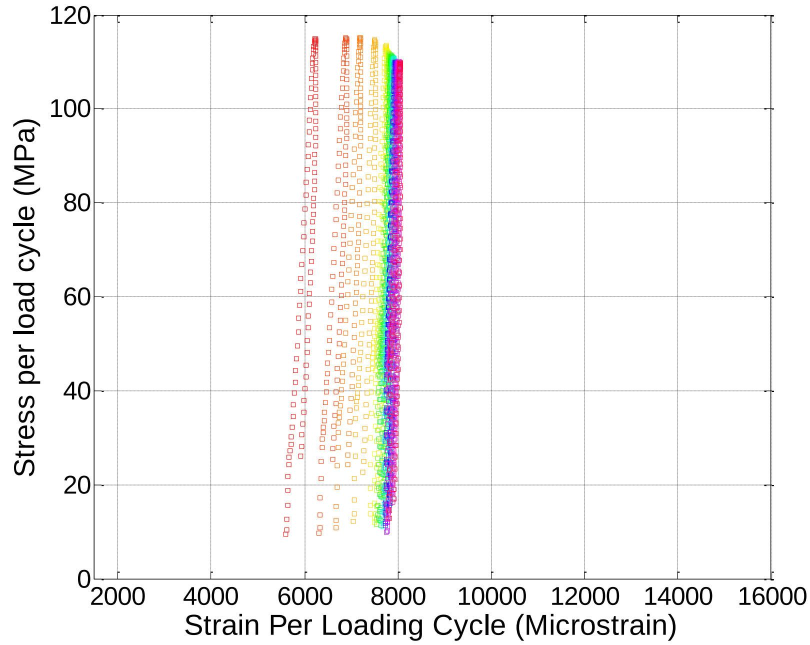


Figure 5-38 Stress strain behaviour of gravel PTH 75 at different permanent deformation sequences at 8.38% moisture content

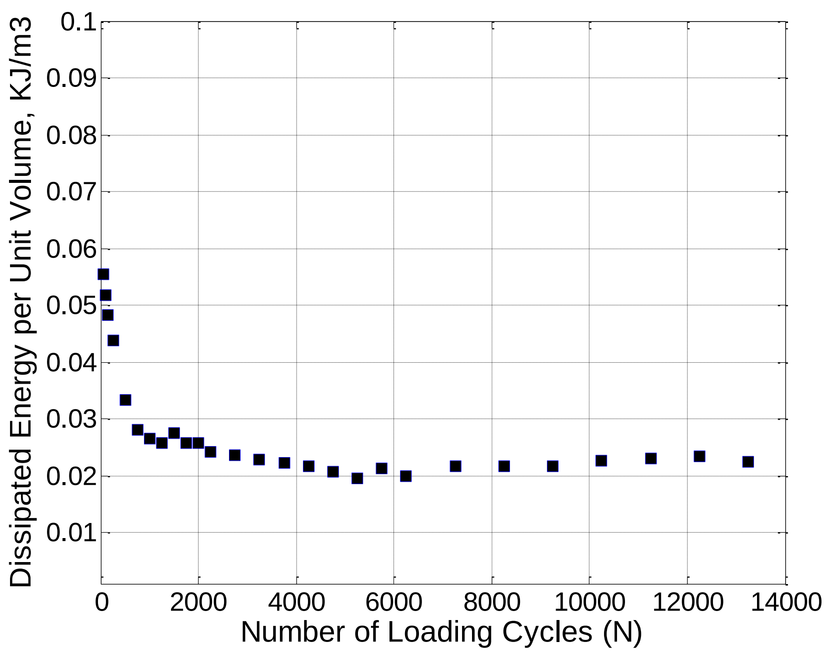


Figure 5-39 Average dissipated energy per cycle of gravel PTH 75 at 8.38% moisture content

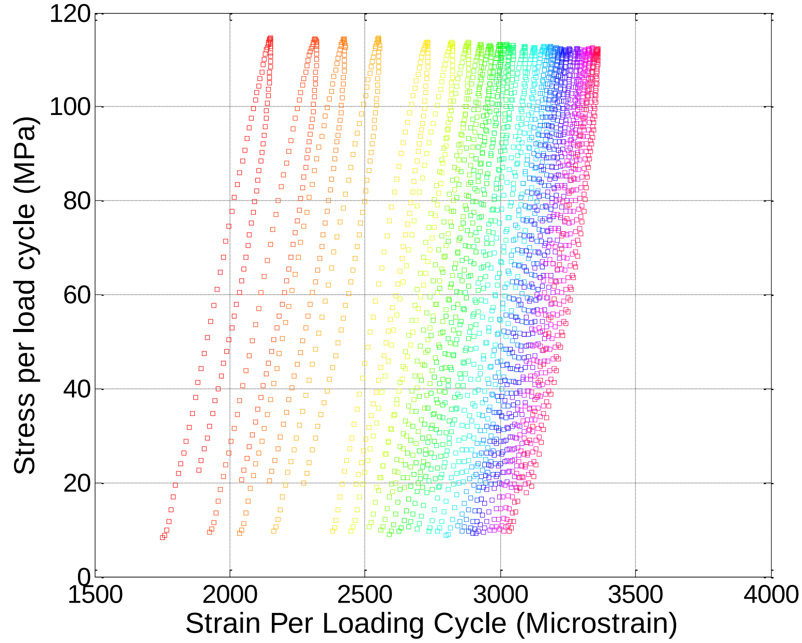


Figure 5-40 Stress strain behaviour of limestone at different permanent deformation sequences at 6.66% moisture content

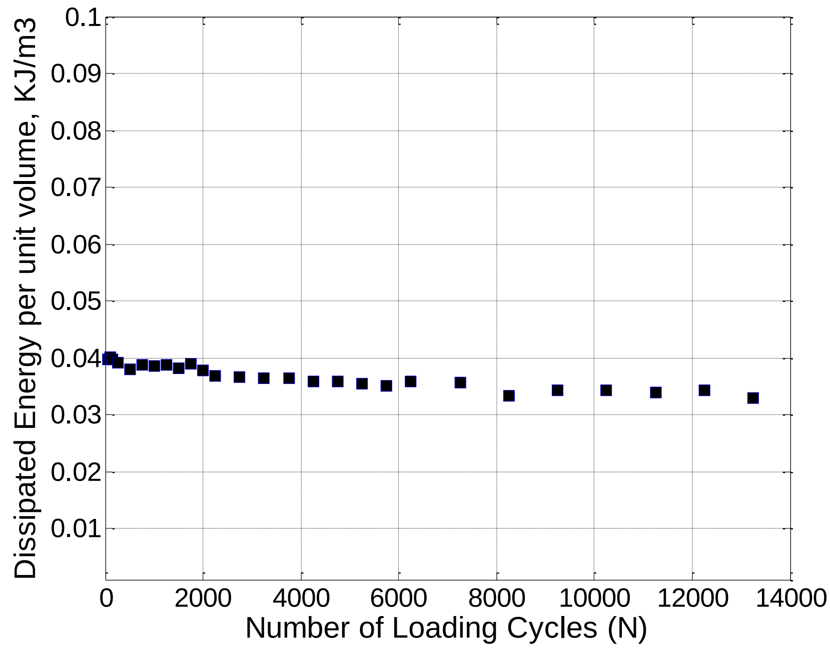


Figure 5-41 Average dissipated energy per cycle of limestone at 6.66% moisture content

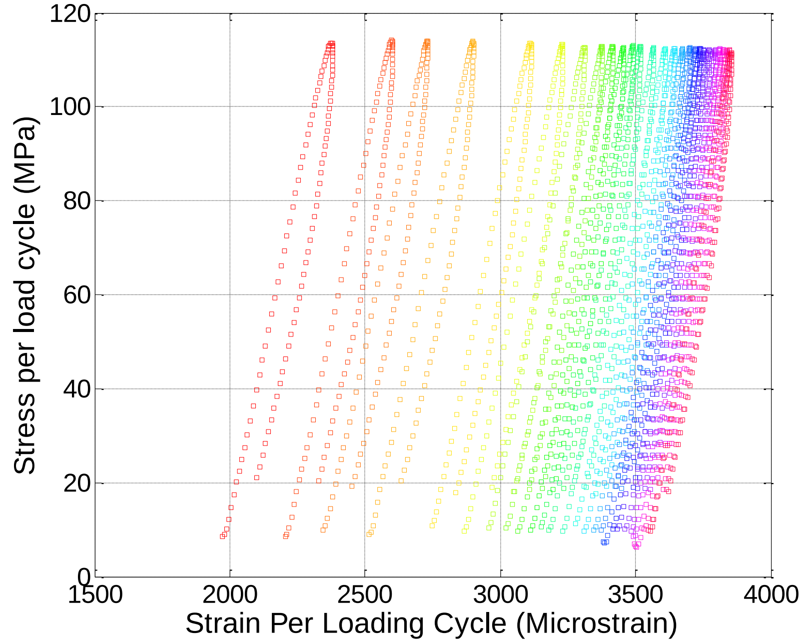


Figure 5-42 Stress strain behaviour of limestone at different permanent deformation sequences at 7.53% moisture content

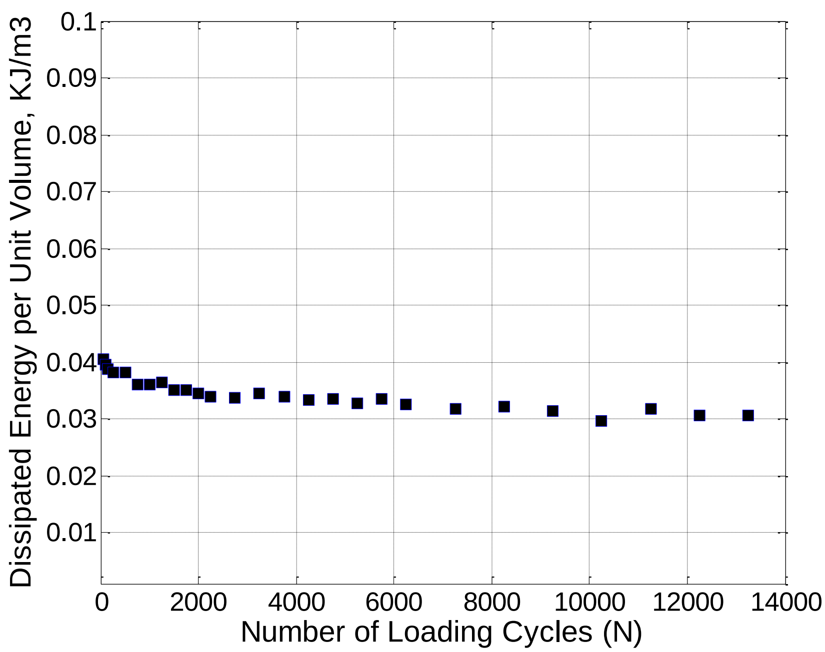


Figure 5-43 Average dissipated energy per cycle of limestone at 7.53% moisture content

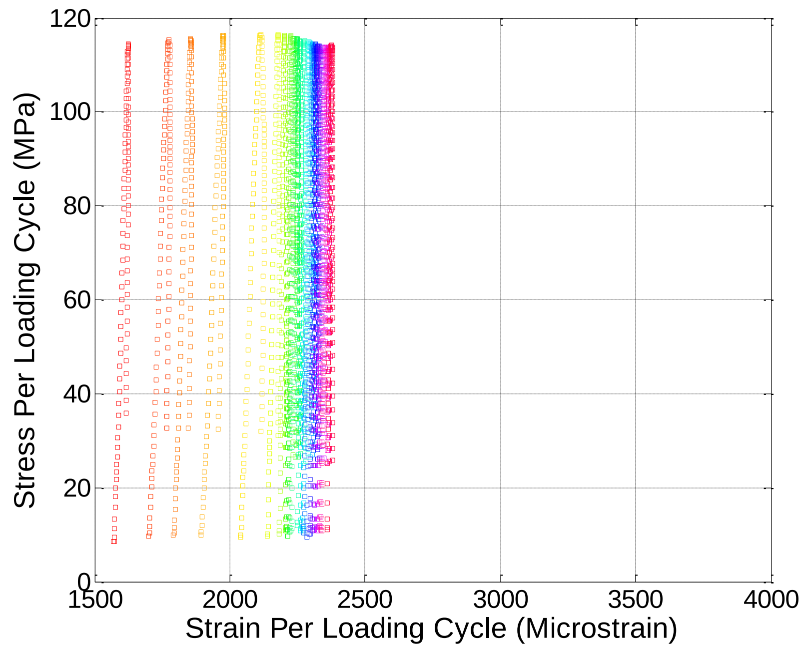


Figure 5-44 Stress strain behaviour of granite at different permanent deformation sequences at 6.47% moisture content

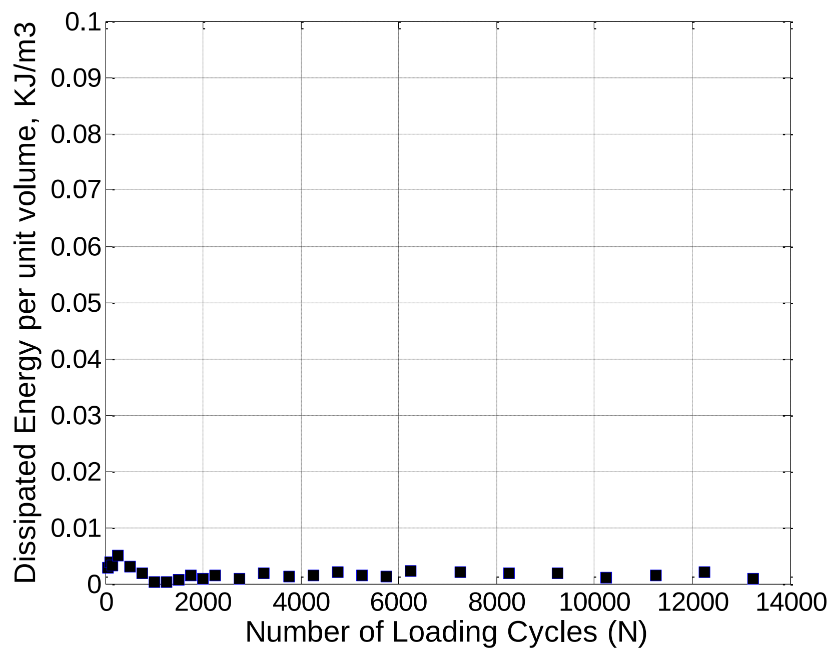


Figure 5-45 Average dissipated energy per cycle of granite at 6.47% moisture content

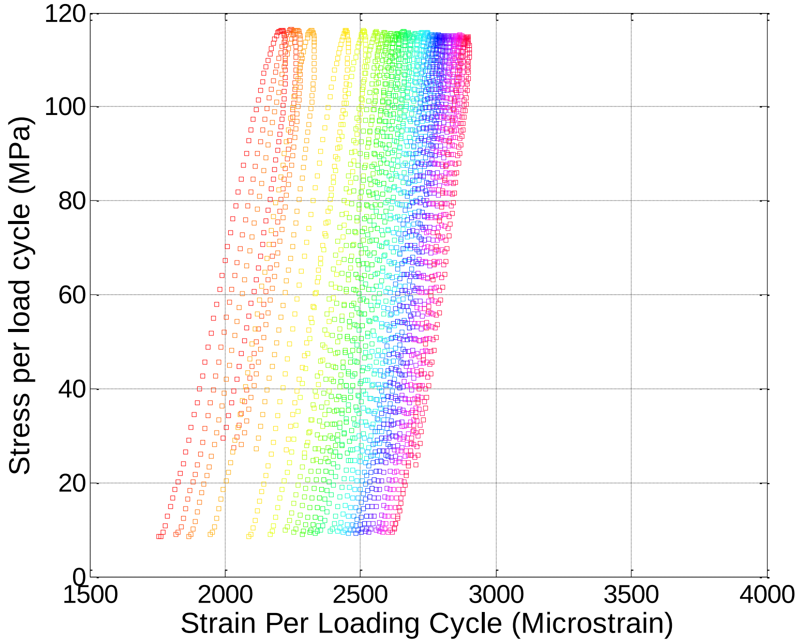


Figure 5-46 Stress strain behaviour of granite at different permanent deformation sequences at 6.41% moisture content

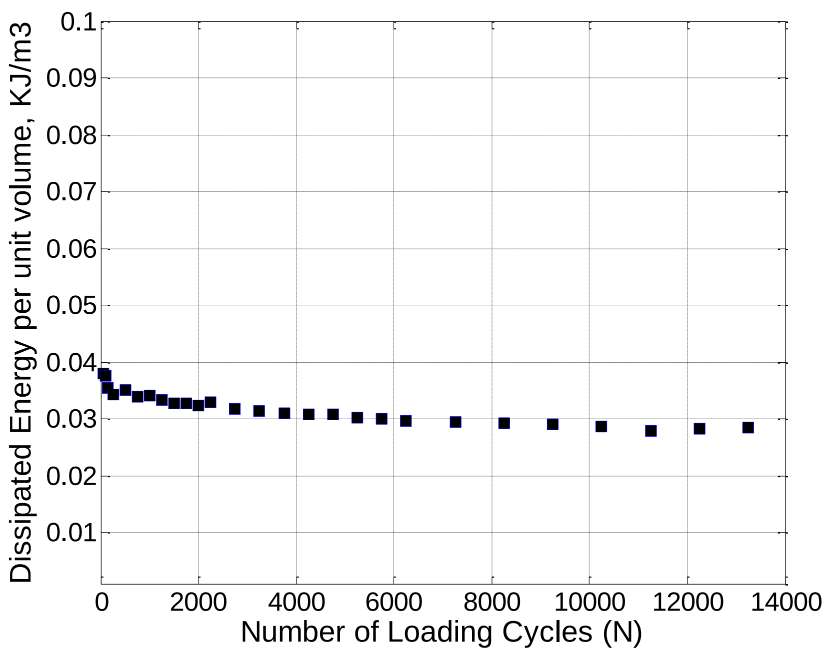


Figure 5-47 Average dissipated energy per cycle of granite at 6.41% moisture content

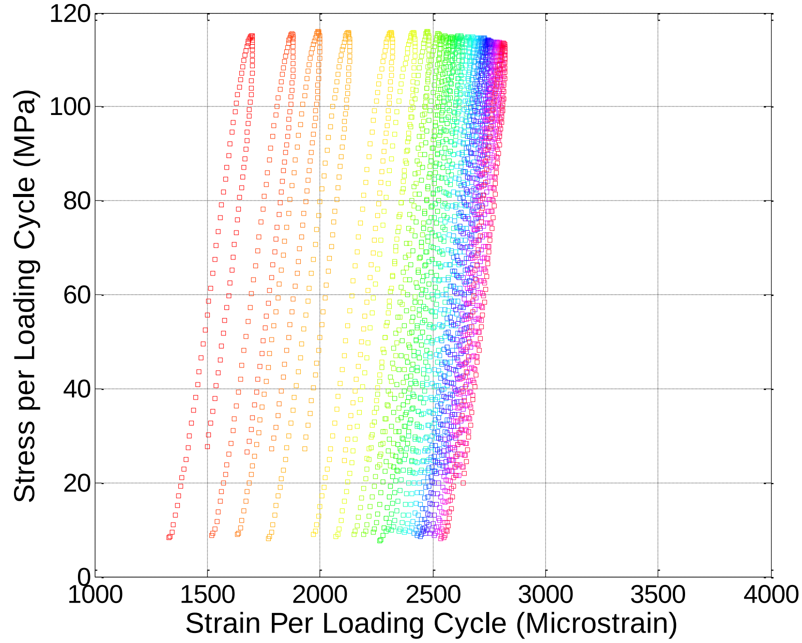


Figure 5-48 Stress strain behaviour of RCA 2.1% Fine at different permanent deformation sequences at 9.89% moisture content

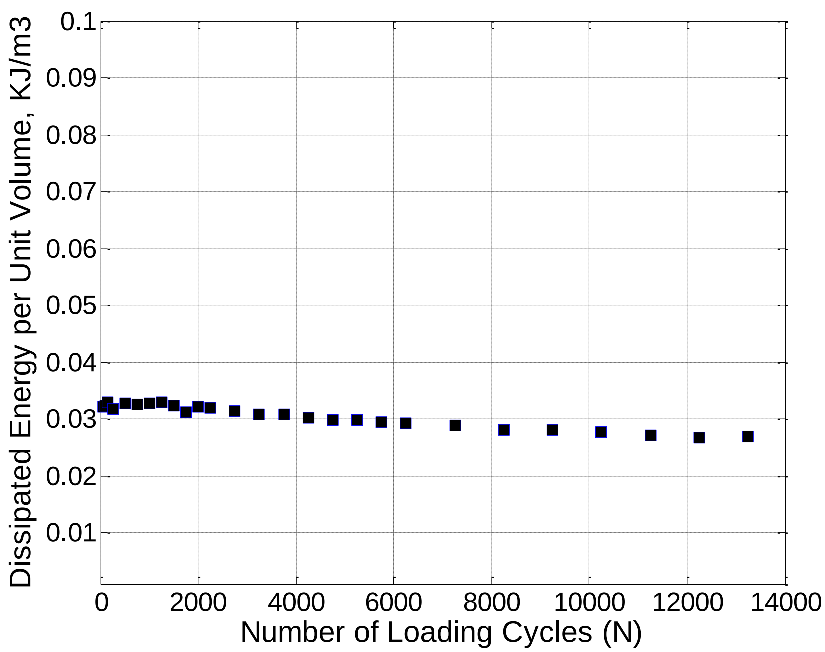


Figure 5-49 Average dissipated energy per cycle of RCA 2.1% Fine at 9.89% moisture content

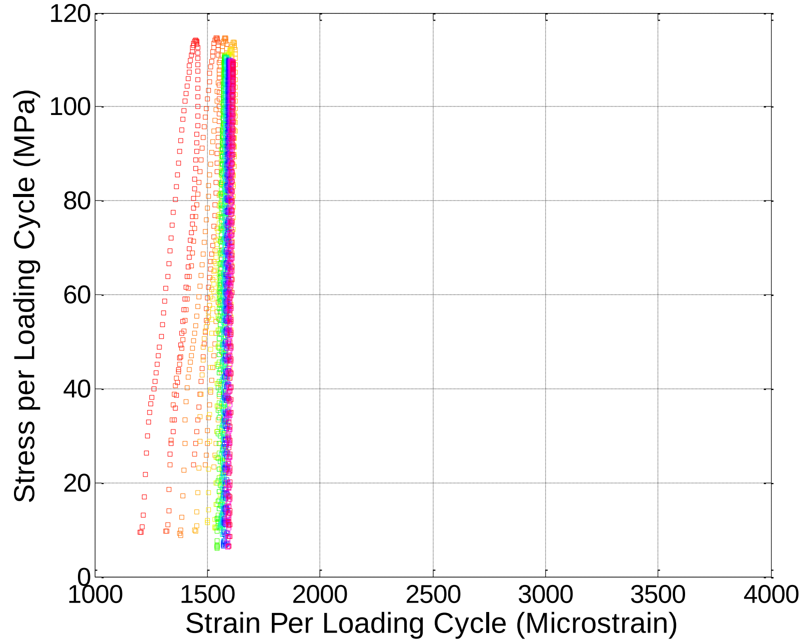


Figure 5-50 Stress strain behaviour of RCA 2.1% Fine at different permanent deformation sequences at 10.05% moisture content

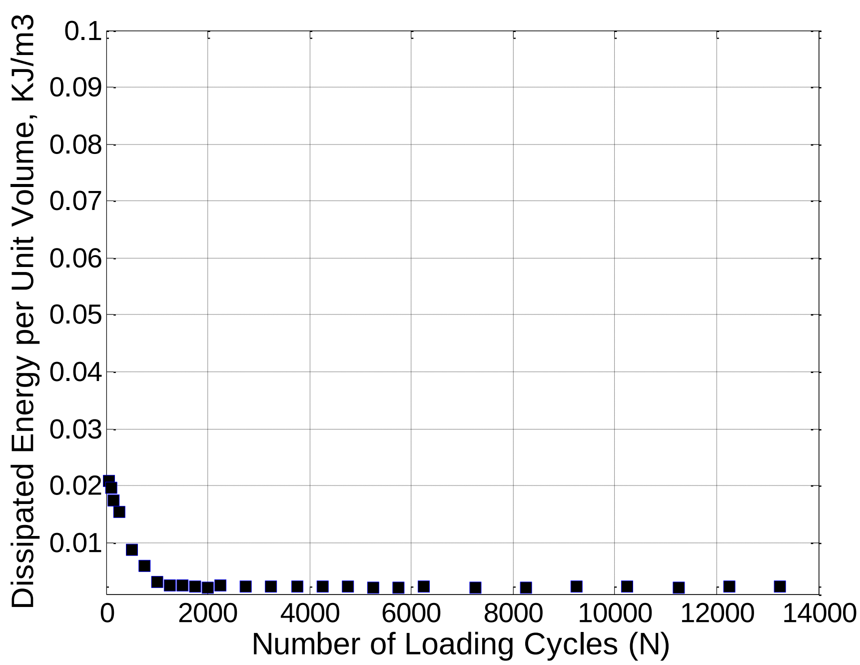


Figure 5-51 Average dissipated energy per cycle of RCA 2.1% Fine at 10.05% moisture content

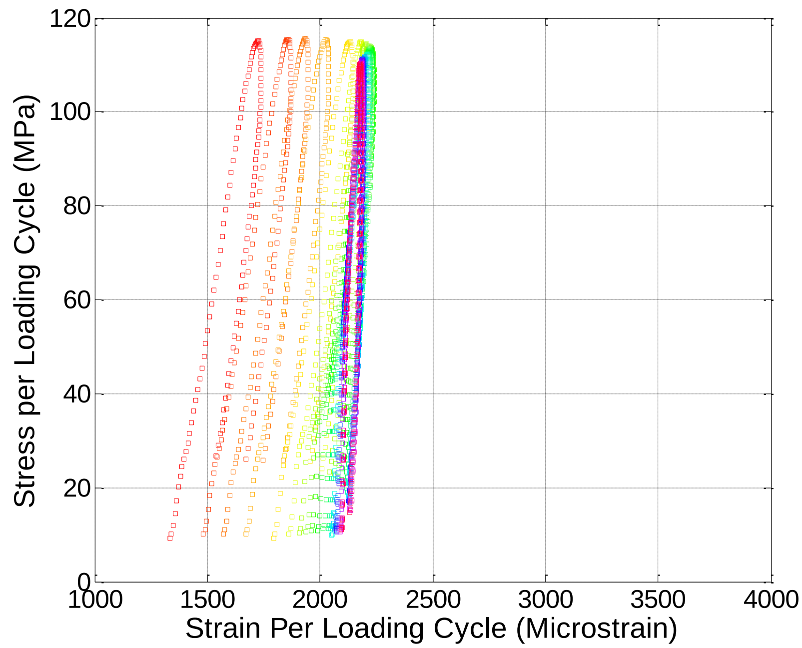


Figure 5-52 Stress strain behaviour of RCA 6.0% Fine at different permanent deformation sequences at 10.23% moisture content

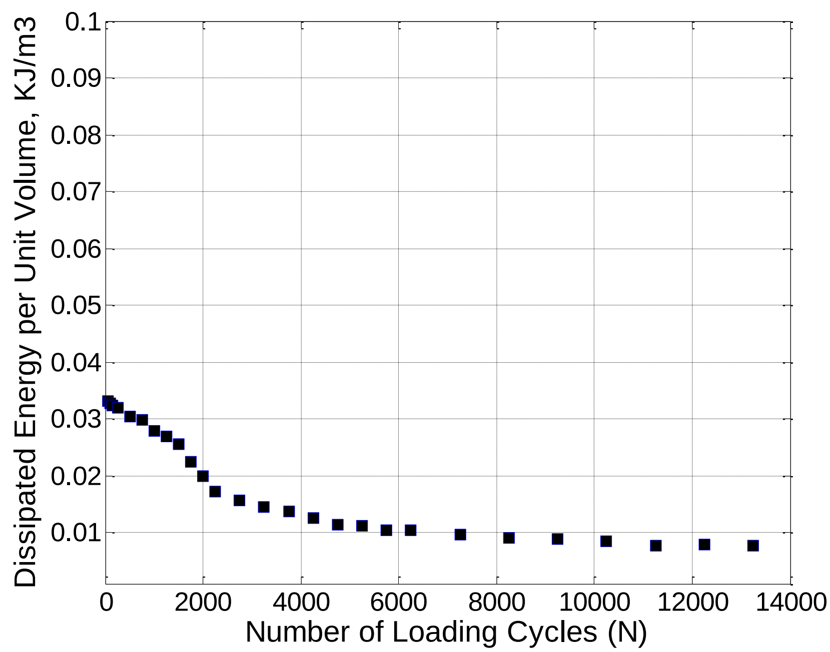


Figure 5-53 Average dissipated energy per cycle of RCA 6.0% Fine at 10.23% moisture content

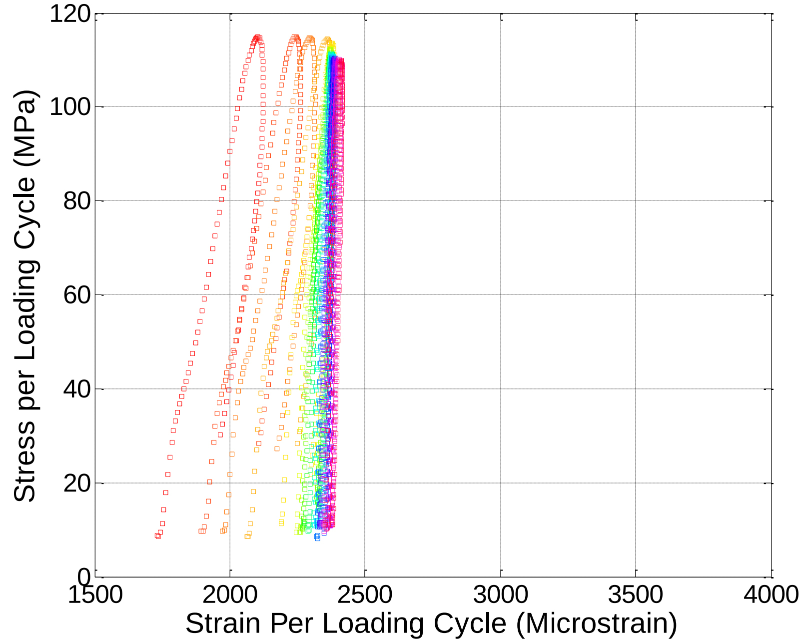


Figure 5-54 Stress strain behaviour of RCA 6.0% Fine at different permanent deformation sequences at 10.46% moisture content

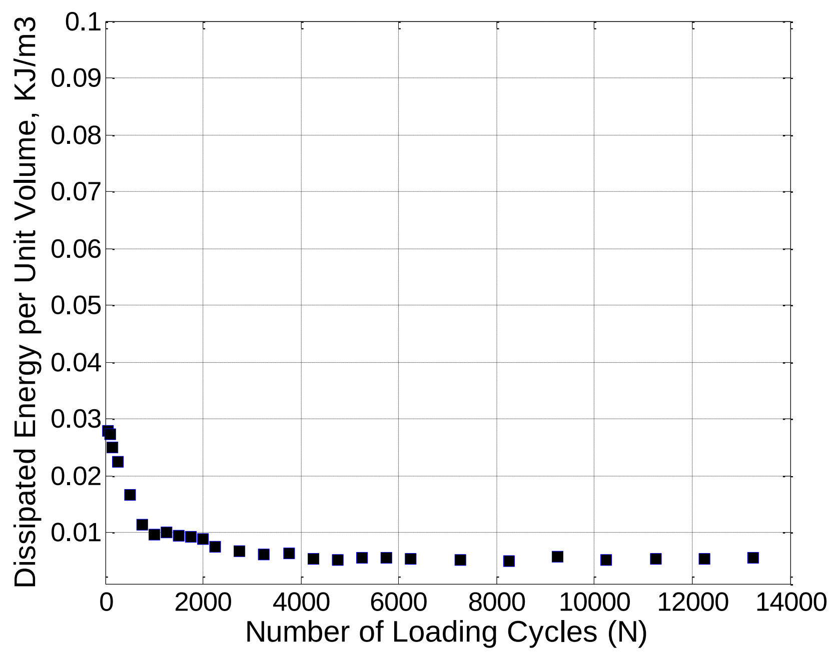


Figure 5-55 Average dissipated energy per cycle of RCA 6.0% Fine at 10.46% moisture content

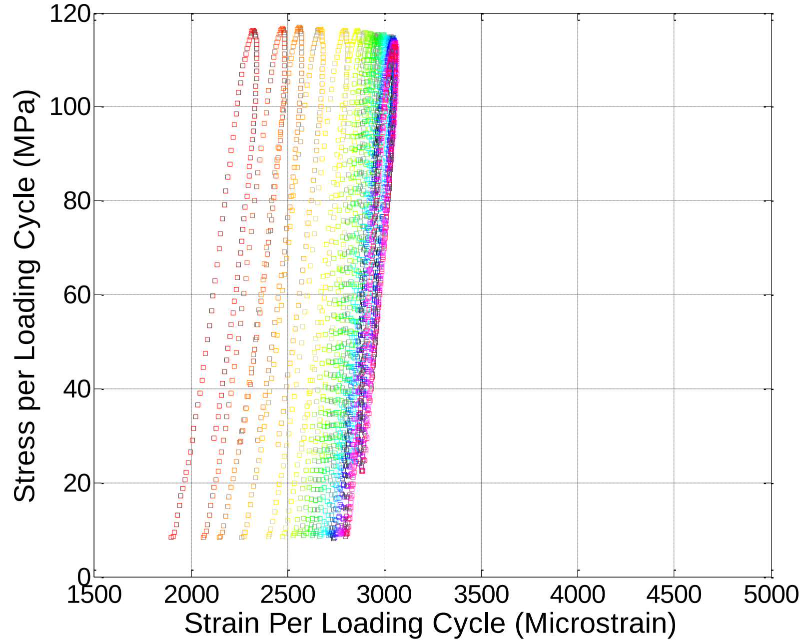


Figure 5-56 Stress strain behaviour of RCA 9.5% Fine at different permanent deformation sequences at 11.13% moisture content

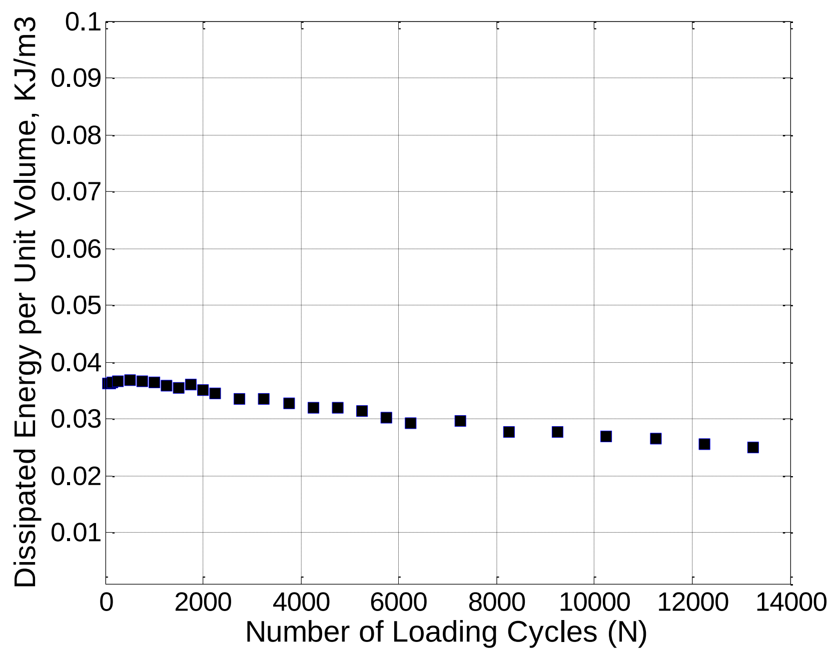


Figure 5-57 Average dissipated energy per cycle of RCA 9.5% Fine at 11.13% moisture content

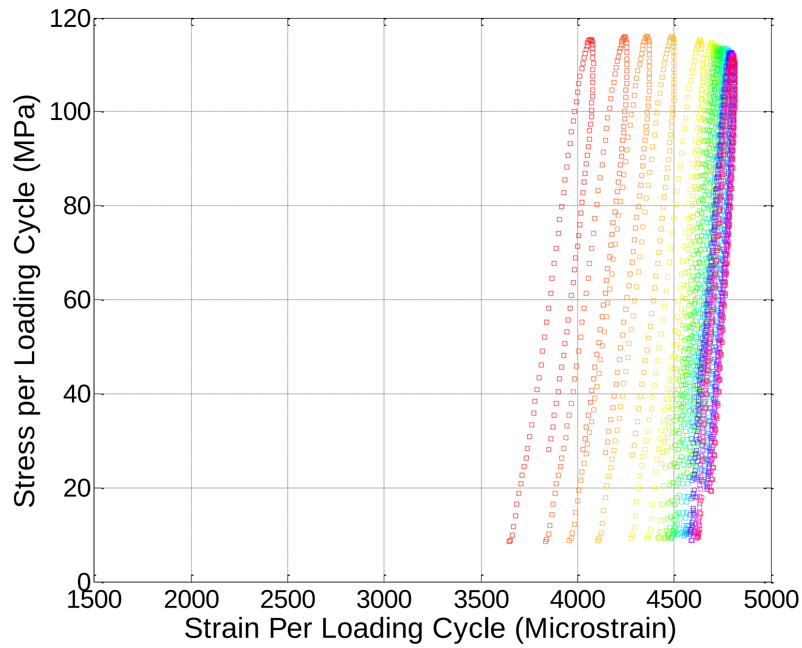


Figure 5-58 Stress strain behaviour of RCA 9.5% Fine at different permanent deformation sequences at 10.78% moisture content

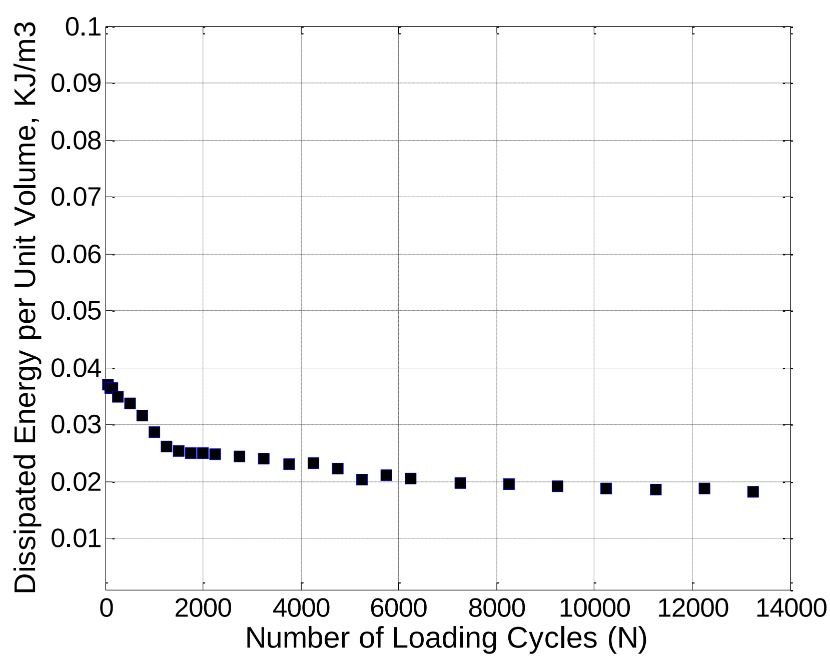


Figure 5-59 Average dissipated energy per cycle of RCA 9.5% fine at 10.78% moisture content

5.6.1 Classification of Material Behaviour Based on Dissipater Energy

From the laboratory permanent deformation test dissipated energy data, a dendrogram plot has been generated in order to find the cluster into the dataset and classify the energy dissipation curve. The obtained dendrogram plot has been shown in Figure 5-60 which shows that, there are at least 2-3 clusters into the analyzed dataset. However, in order to predict and classify the dissipated energy data and to predict a threshold value form the classification more accurately, data from more samples will be required.

In addition to that, k-means clustering has also been performed in order to classify and find the threshold value of the dissipated energy clusters. K means clustering is a data partitioning method which classifies the n number of observations to predefines k number of clusters based on the centroids. Figure 5-61 shows the classified datasets after the k-means clustering. Three clusters have been found from the datasets tested. The cluster is at the beginning of the test where there is high amount of dissipated energy per cycles of load. That cluster can represent the post compaction zone of permanent deformation test. According to the analysis, the cluster one consists of the data from beginning of the test to approximately 3,000 cycles of load. Therefore, it may represent that, the post compaction zone where relocation and rearranging of particles happen for obtaining better stability of the sample may continue up to 3,000 cycles. There is a reduced rate of change in dissipated energy at the end of the cluster one.

However, cluster two and cluster three can represent the permanent deformation behaviour of the material based on the rate of change in energy dissipation. According to cluster two, the post compaction zone ended very quickly after few hundreds load cycles followed by a rapid reduction in dissipated energy per loading cycles. Cluster three represents the material behaviour where total dissipated energy becomes almost constant with increasing number of loading cycles. However, the behaviour can vary based on change in the number of load repetitions of the permanent deformation test. The permanent deformation test consists of only 13,000 cycles which only captures the early behaviour of the permanent deformation. Therefore, in order to predict and classify the long term permanent deformation behaviour in terms of dissipated energy approach, more number of tests should be carried out with longer duration of test.

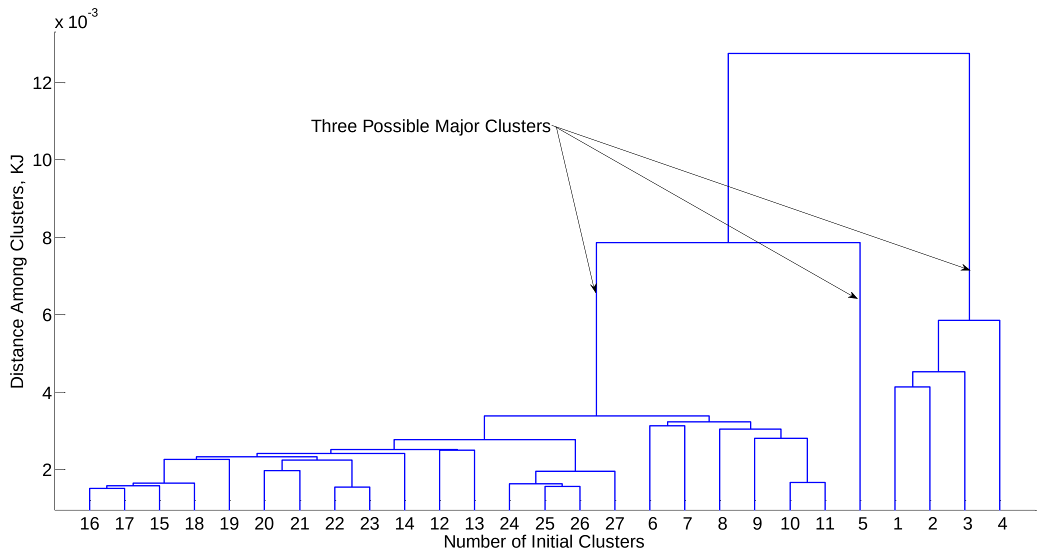


Figure 5-60 Dendrogram plot of the hierarchical binary cluster tree.

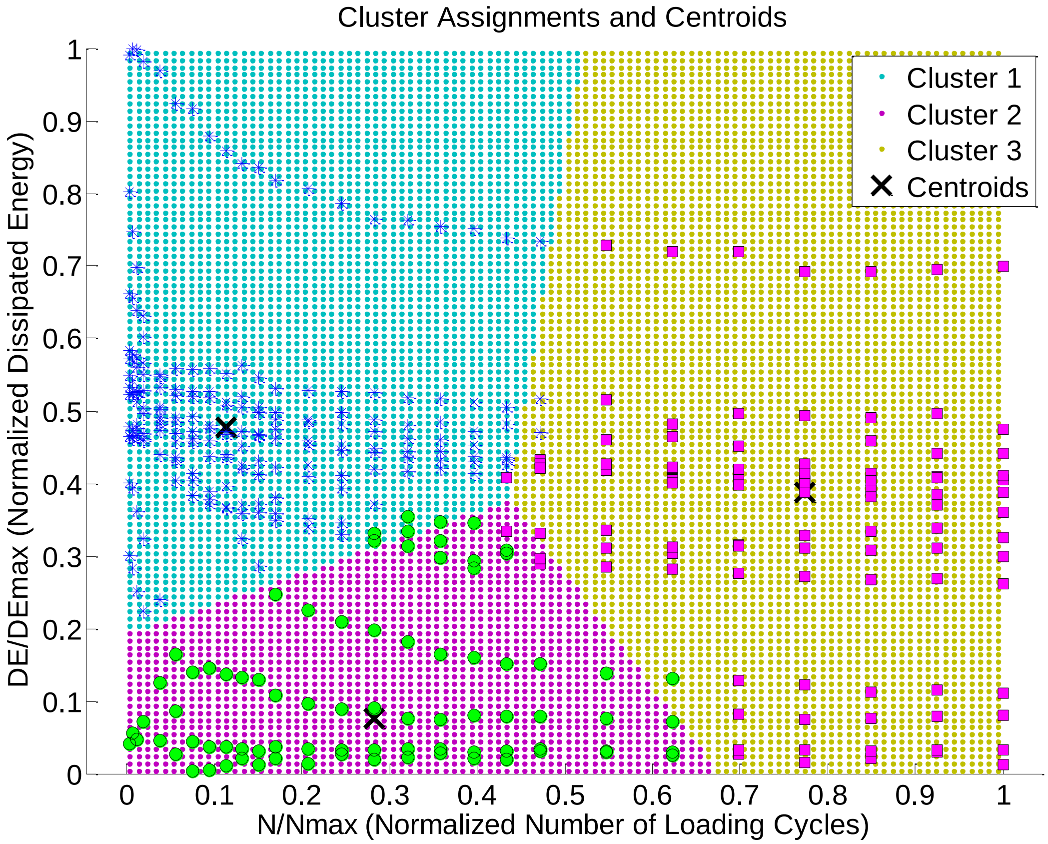


Figure 5-61 Generated clusters from k-means clustering of dissipated energy data

5.7 SIMPLIFIED APPROACH TO PREDICT PERMANENT DEFORMATION

From the cluster analysis, it has been seen that the post compaction zone can continue up to 3,000 load cycles. After the post compaction zone, the permanent deformation becomes stable. The shakedown concept of the long term permanent deformation prediction is based on the early age performance of the material. A simplified approach has been presented here for long term permanent deformation prediction. The simplified approach has been developed assuming that the long term performance of unbound granular materials is not dependent on the post compaction zone. The simplified approach does not take the creep or strain softening stage of the unbound granular materials behaviour into account. Therefore, according to the cluster analysis, the test data points up to 3,000 cycles have not been taken into account for the analysis. The permanent deformation with number of load cycles have a linear relationship for 3,000 to 13,000 load cycles. Permanent strain greater than five percent have been selected as the threshold of the failure of the unbound granular material with excessive deformation. Assuming the behaviour after the post compaction zone as linear, the permanent deformation data after 3,000 load cycles to 13,000 cycles have been fitted with straight line. Table 5.7 summarizes the linear regression fit for permanent deformation data for 3,000 to 13,000 loading cycles. The fitted data have been extrapolated up to 100 million load cycles to see the long term performance of the materials. Figure 5-63 shows the predicted permanent deformation performance. It has been seen that, it takes around one million to more than ten million cycles to reach five percent permanent deformation according to the prediction. One sample have been found to reach at five percent deformation before 1 million cycles. The simplified approach of prediction of long term deformation performance also supports the shakedown classification of permanent deformation.

Table 5.7 Linear regression summary of permanent deformation

Sample	**Bo**	**B1**	**R^2**
Gravel	0.335	8.539e-07	0.914
Gravel	0.298	1.833e-06	0.965
Limestone	0.268	2.324e-06	0.952
Limestone	0.319	2.447e-06	0.934
Granite	0.223	8.786e-07	0.966
Granite	0.229	1.982e-06	0.971
Gravel PTH 75	1.360	7.082e-06	0.932
Gravel PTH 75	0.765	1.108e-06	0.981
RCA 2.1%	0.226	1.749e-06	0.957
RCA 2.1%	0.155	2.644e-07	0.951
RCA 6.0%	0.203	4.342e-07	0.897
RCA 6.0%	0.229	4.148e-07	0.955
RCA 9.5%	0.257	1.262e-06	0.957
RCA 9.5%	0.447	8.963e-07	0.905

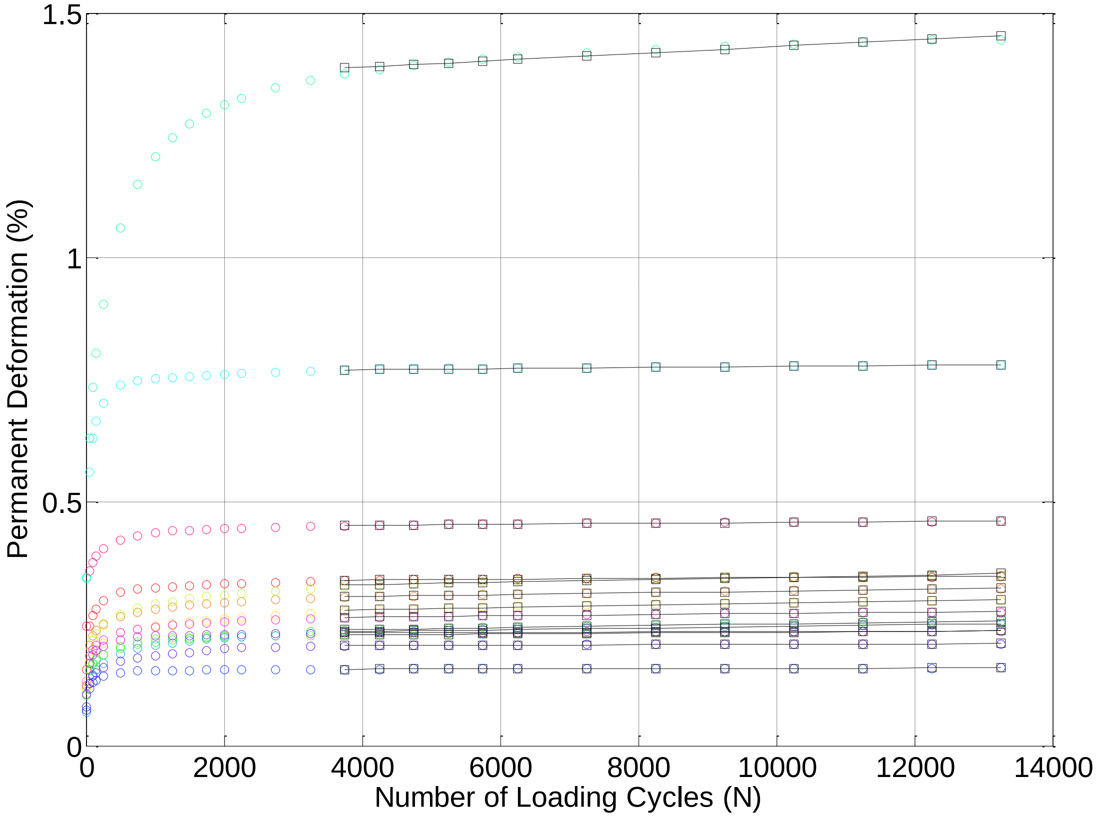


Figure 5-62 Fitted linear regression line with permanent deformation data

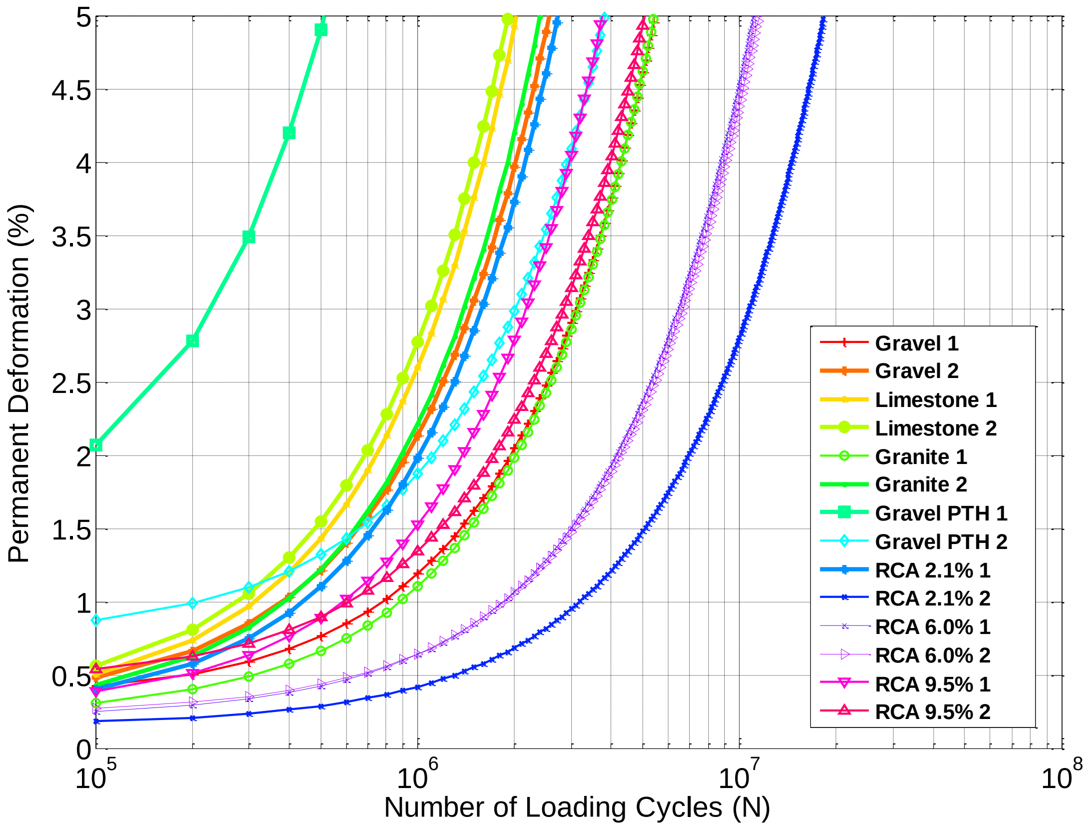


Figure 5-63 Predicted permanent deformation long term performance

5.8 PERMEABILITY

The hydraulic conductivity test results of the virgin and recycled aggregate base have been shown in Figure 5-64. The tests have been carried out according to the procedure using single ring Permeameter (section 4.5.4). It has been previously reported that, the hydraulic conductivity has an inverse relationship with increase in fine content (Figure 5-65) (Haithem Soliman, 2015). The tested materials shown in Figure 5-64 had fine content ranging from 5.5 to 6.5%. The test results have also been compared with the previously tested results (Figure 5-65) (Haithem Soliman, 2015). The permeability of the materials has been found to be low because of a high density of material (density of the materials have been reported in Table 4.3, page 62). Crushed gravel has a higher density with a different particle packing structure compared to uncrushed gravel has been found to be less permeable compared to the previous test results. The PTH 75 DSB gradation has been tested at a high moisture content. The material also has a low density due to segregation of the particles during compaction. In addition to the observation that the permeability decreases with increase in fine content, it has also been observed that, permeability dependent on the packing structure of the different size aggregates. The tested gradations of virgin unbound granular materials have a dense particle packing as it can be seen from the 0.45 power curve in Figure 5-66 compared to previously reported test results of gravel A base and limestone A base gradations (Figure 5-68 to Figure 5-69). On the other hand, the tested gradations of recycled concrete aggregates have relatively loose particle packing as it can be seen from Figure 5-67. However, the permeability of recycled concrete aggregate gradations has also been found to be dependent on the fines content. Decrease in permeability have been found with increase in fines content from 2.1% to 9.5%.

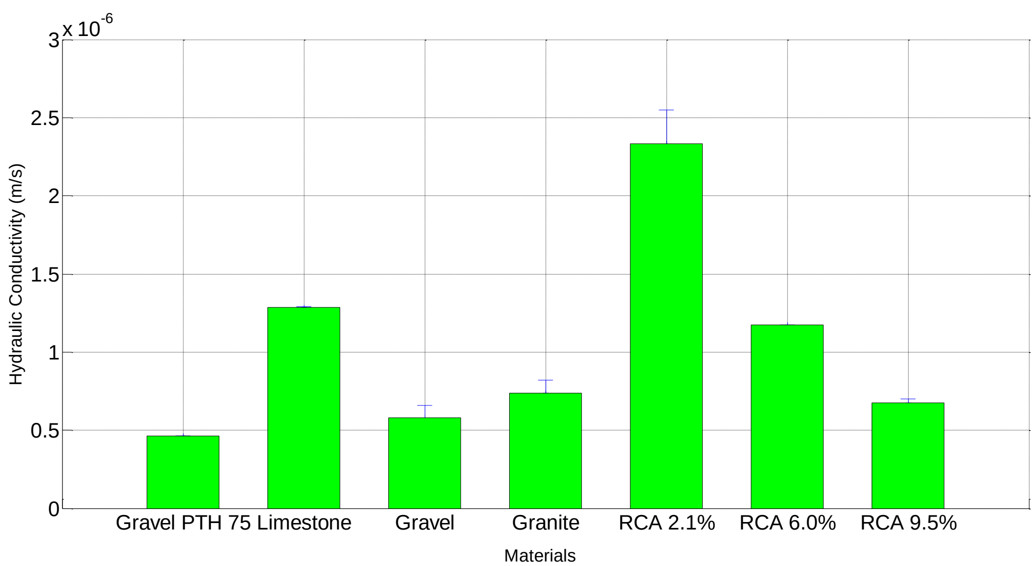


Figure 5-64 Permeability of tested unbound granular materials

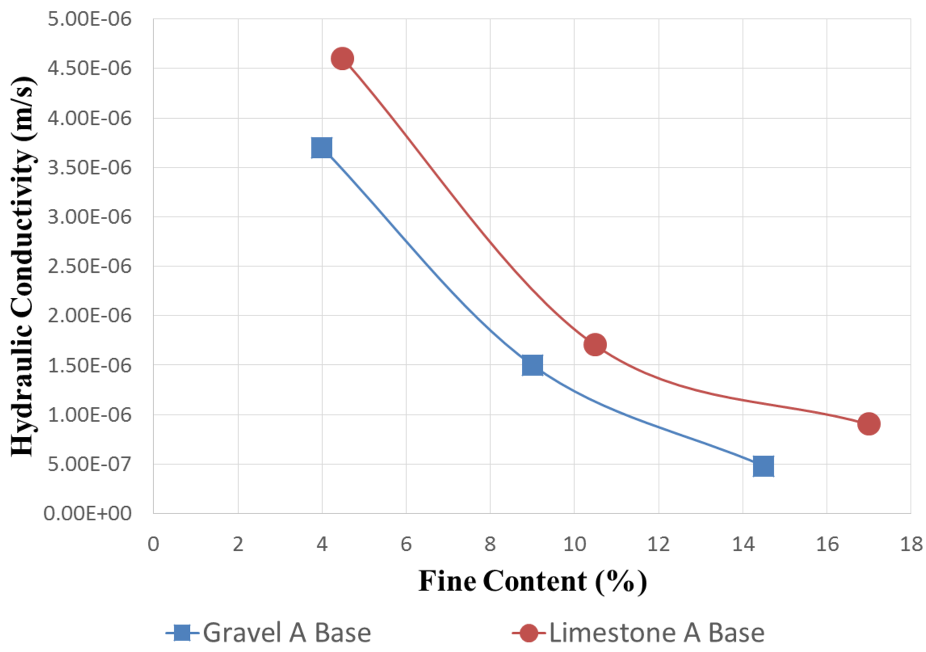


Figure 5-65 Permeability of previously reported uncrushed gravel and crushed limestone A base (Haithem Soliman, 2015).

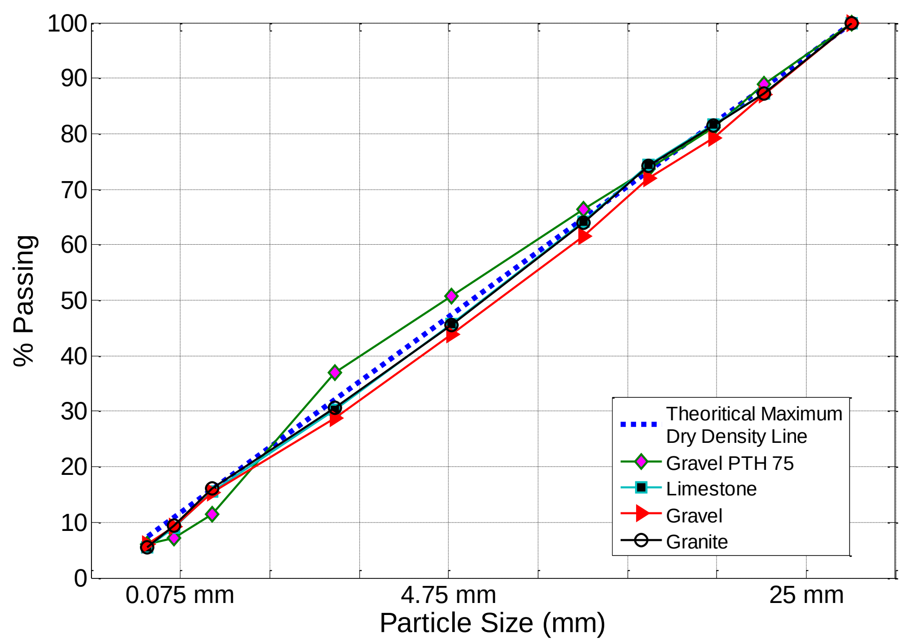


Figure 5-66 Gradation of the tested virgin unbound granular materials with theoretical maximum dry density line at 0.45 power plot.

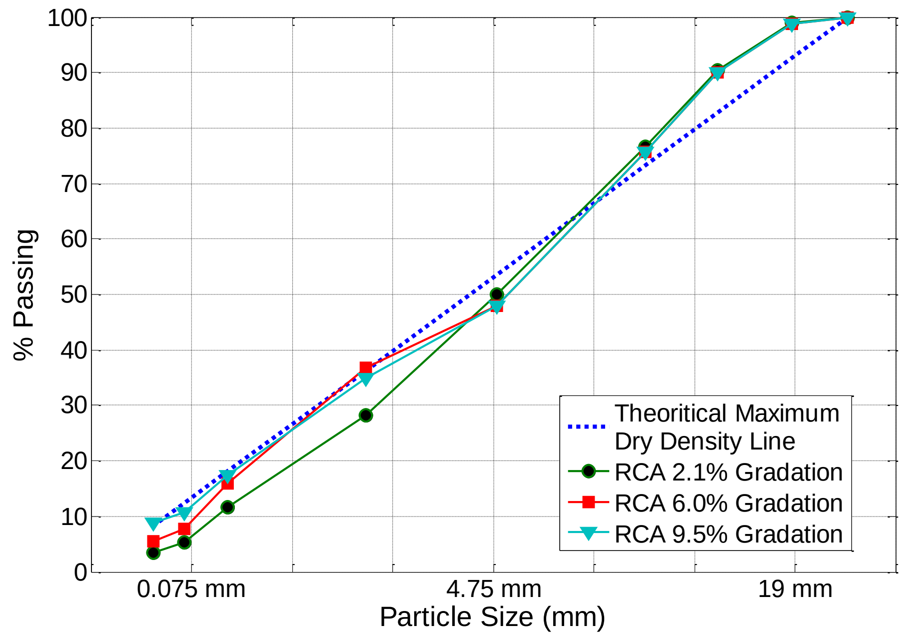


Figure 5-67 Gradation of the tested recycled unbound granular materials with theoretical maximum dry density line at 0.45 power plot.

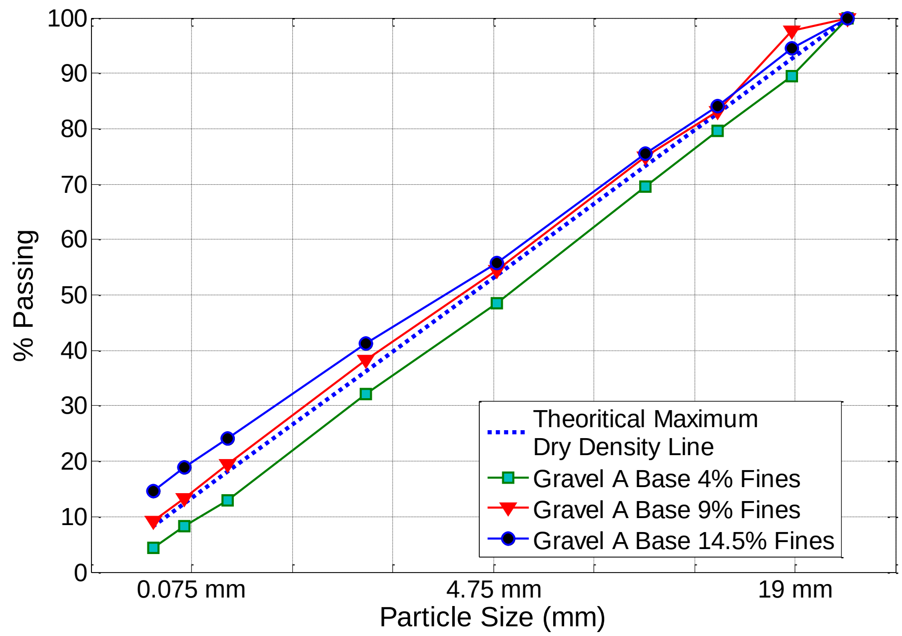


Figure 5-68 Gradation of previously tested gravel A base with theoretical maximum dry density line at 0.45 power plot (Haithem Soliman, 2015).

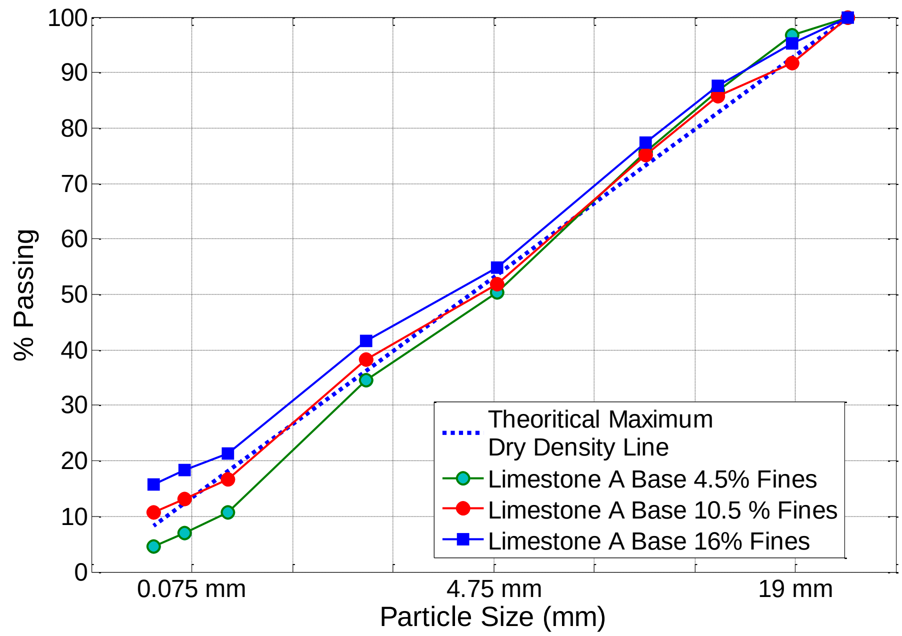


Figure 5-69 Gradation of previously tested limestone A base with theoretical maximum dry density line at 0.45 power plot (Haithem Soliman, 2015).

Chapter 6 Conclusions and Recommendations

6.1 SUMMARY OF THE STUDY

Resilient modulus, permanent deformation and permeability tests on gravel, limestone, granite and three gradations of recycled concrete aggregate with varying fines content have been carried out in order to characterize the materials. The virgin aggregate samples were collected from the existing highway project sites in Manitoba. The recycled concrete aggregates were collected from the recycled concrete aggregate producer in Winnipeg.

The province of Manitoba allows relatively higher fines content compared to neighbouring jurisdictions. Also, relatively smaller maximum aggregate size and lower crush count of coarse aggregate is specified. The tested gravel, limestone and granite have been tested with larger maximum aggregate size with higher crush count and at a lower fines content than the current specified ranges. The virgin aggregate gradations have been reconstituted with a nominal maximum aggregate size of 19 mm and with fines content ranging between 5.5-6.5%. The tested gravel materials had a minimum crush count of 60% and the tested limestone and granite had a crush count of 100%.

The current specification of City of Winnipeg allows 6-17% of fines content in the base layer. Among tested three gradations of recycled concrete aggregate materials, two gradations represent the specification limit of City of Winnipeg. In addition, the third gradation has a low amount of fines than allowed by specification. The recycled concrete aggregate materials gradations have been reconstituted with 16 mm nominal maximum aggregate size which is similar to the current specification.

Standard proctor tests have been carried out in order to evaluate the maximum dry density and optimum moisture content of the constituted gradations. For the resilient modulus test and permanent deformation tests, specimens have been compacted at optimum moisture content to 99% of the maximum dry density. For the permeability tests the samples have been compacted at optimum moisture content to 98% of the maximum dry density. The resilient modulus tests have been carried out according to the NCHRP Project 1-28A test method. The permanent deformation tests have been carried out for 13,000 load cycles at a single stress ratio level. Single ring permeameter has been used for the permeability test.

6.2 CONCLUSIONS

Following finding have been obtained from the laboratory investigation of the unbound granular materials:

- The resilient modulus performance of gravel, limestone and granite have been found to improve significantly compared to the previously reported investigations as a result of increasing the maximum aggregate size, reduced fines content and increasing the crush count of gravel. For the field gravel samples, the resilient modulus value is also higher than that of the previously reported results as a result of using coarser gradation, even though the sample was compacted at a higher moisture content (Haithem Soliman, 2015).
- The resilient modulus performance of recycled concrete aggregate has also been found to be comparable to the performance of virgin unbound granular materials. The susceptibility

of change in resilient modulus with increasing fines content has been found to be negligible within a range of 0-10% fines. At a low range of fines content, the deformation resistance and stiffness mechanism is usually controlled by the coarse aggregate matrix, aggregate to aggregate contact and interlock. The coarse aggregates of recycled concrete aggregates have crushed angular face as well as rough surface texture because of the presence of mortar with aggregates.

- The gravel, limestone and granite have been found to behave in plastic shakedown and plastic creep stage of shakedown classification. The gravel with increased crush count also has impact on the long term performance of the material. The aggregates with better interlocking capacity improved the permanent deformation resistance at low fines content. Similarly, the recycled concrete aggregates consist of crushed concrete and crushed aggregate. The resistance to permanent deformation of recycled concrete aggregate is also comparable and competitive with the performance of virgin aggregates.
- The test duration of permanent deformation in the current laboratory investigation was only up to 13,000 load cycles which could only capture the early age performance of the material. In order to predict the long term performance, shakedown approach have been used and three different criteria have been used to identify the shakedown ranges. The materials at plastic shakedown and plastic creep range is desirable to be used in order to obtain a long term sound performance of the base layer. The tested gravel, limestone and granite have been found to fall into either plastic shakedown or plastic creep stage. However, the field sample from the PTH 75 highway project exhibited incremental collapse because of high moisture content and sampling variability. The recycled concrete

aggregate gradations have also been found to be performing within plastic shakedown and plastic creep stages.

- Some effect of permanent deformation test as conditioning before the resilient modulus test compared to the NCHRP 1-28A have been found on the stiffness of the base materials. Regardless virgin aggregate and recycled aggregate, the resilient modulus has been found to be increased after the permanent deformation test as conditioning. That increase is mainly because of the moisture removal and further densification of the materials. However, the materials behaviour has changed at the high confining pressure and bulk stresses. The behaviour of the material at high confining pressure have been found to be different because of strain softening and the increased excess pore water pressure or dilation in the dense granular material.
- The fines content ranging 5-6% have been used for the gradations of virgin unbound granular materials. The permeability were lower compared to the previously reported permeability values of gravel A base and limestone A base (Haithem Soliman, 2015). The permeability is also affected by the packed particle structure of the base layer. The densely packed material with low fines content might have resulted a lower permeability. Permeability of recycled concrete aggregate is decreased with the increase in fines content.
- The dissipated energy approach has been used to calculate the dissipated energy per loading cycles. From the dissipated energy information, the post compaction zone in permanent deformation have been identified. However, for a different stress ratio, the duration of the post compaction zone may also vary. The post compaction zone could continue up to 3,000 load cycles. Linear regression has been carried out with the permanent deformation data points beyond 3,000 load cycles. The fitted lines have been extrapolated up to 10 million

load cycles in order to predict the long term permanent deformation. The simplified approach of prediction has also been compared and validated with shakedown ranges prediction of permanent deformation. Similar prediction has been found from the simplified approach of prediction and shakedown approach of prediction.

6.3 RECOMMENDATIONS FOR FUTURE WORK

The following recommendations have been proposed based on the results of the current investigation:

- Due to time and laboratory constrains, the permanent deformation test duration was only up to 13,000 cycles at a single stress ratio. However, the test can be performed up to millions of load cycles in order to capture the actual long term performance. Permanent deformation test with longer duration with different stress ratio should be performed into the laboratory to validate the shakedown prediction.
- Effect of conditioning at a different stress ratio for different duration have been found from the laboratory test results. The resilient modulus test with different conditioning with different stress ratio should be performed in order to predict the actual performance of the material with load history of different duration.
- Further investigations should be carried out in order to evaluate the influence of other parameters (shale content, different aggregate source, plasticity of fines, different crush count) on the resilient modulus and permanent deformation performance of the granular materials.

- Further research should be performed in order to improve the permeability of the stiff unbound granular material. It is often challenging to obtain a free draining layer with high stiffness. Further research is required to visualize the packed particle structure into the base layer and the influence of that on permeability.
- A major objective of the research was to provide recommendation and testing data in order to update the current specification of unbound granular materials of Manitoba. The local transportation agencies should calibrate the MEPDG empirical prediction models in order to predict the resilient modulus and permanent deformation performance. The specification of unbound granular materials should be performance and function based. Whenever, the local testing is not available, the local calibration can be used to predict the performance of the materials.

Chapter 7 References

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