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Performance of Unbound Granular Basecourse Materials under Varying Moisture Conditions

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A Thesis submitted in fulfilment of the requirements for the degree of Doctor of Philosophy in Engineering,
The University of Auckland

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ABSTRACT

In changing environmental conditions, the moisture variations play an important role in controlling the performance of unbound layers. The increased moisture decreases the strength of unbound materials and can cause early failures in low volume roads. It is important to investigate the performance of unbound granular materials (UGM) with increasing moisture levels as there is a high likelihood of water infiltrating unbound layers. The better understanding of UGM deformation behaviour as moisture levels increase can facilitate the design process of the materials and layers. This research has aimed to understand the UGM behaviour used in the basecourse layer of pavements in wet conditions. Therefore the objectives of the research can be divided into two main categories: To understand how aggregates behave in increasing moisture conditions for different aggregate gradations and fine material contents; and to investigate how well UGM behaviour can be simulated in laboratory based performance tests compared to accelerated pavement tests.

The experimental program was designed in three categories apart from the standard tests carried on the basecourse materials i.e., geological and mineralogical testing, repeated load triaxial testing, full scale accelerated pavement tests. The geology of the source rocks was tested by examining the thin sections obtained from various aggregates. X-Ray Diffraction tests were conducted which helped in finding the clay minerals present in the granular aggregates and fine particles (less than 0.10 mm). The unbound materials were tested in the repeated load triaxial (RLT) test apparatus at optimum moisture drained, saturated drained and saturated un-drained conditions to investigate the behaviour of unbound materials at elevated moisture contents. The materials were laid in the basecourse at Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) in various sections with different sealing options. The pavements were subjected to surface water runoff and standard axle loading cycles.

The results show that mineral composition of unbound materials is an important parameter which can affect the performance of compacted unbound granular aggregate materials. The results lead towards the selection of aggregates with specific range of expanding minerals, and particle size distribution with a range of Talbot’s constant. The research shows the behaviour of different grading of aggregate under elevated moisture conditions in full scale
accelerated pavement testing at CAPTIF. This research developed a new permanent
deformation model for unbound granular material based on RLT test data which includes
stress variations as well as the number of loading cycles applied on the specimen.
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1.1 Overview

Transportation systems play an important role in controlling the economy of the country by facilitating the economical movement of goods. When comparing the various modes of transportation, roads play a key role in an efficient transportation system. Competing modes of transportation i.e., railway, airway and waterway have limited ability to access land and therefore require a modal transfer and largely must connect via roadways. In a road network, there are different types of road classification e.g., the roads are classified according to the Federal Highway Administration (FHWA) United States of America (USA) as arterial highways, collector roads, and local roads; whereas a different classification is proposed by the New Zealand Transport Agency (NZTA) as National Strategic, Regional Strategic, Regional Connector, and Regional Distributor (Theme Twenty Ten, 2013). Some of these road pavements are rigid pavements whilst others are flexible pavements. In most cases the rigid or flexible pavement structures contain unbound granular material in the basecourse and subbase layers which provide stiffness to the pavement structure.

Local roads are major portions of any road network and these roads link more remote land areas to more densely populated areas. Through these roads, the agricultural crops and other products get access to national and international markets. The local roads have low to
moderate traffic volumes. A significant proportion of annual transportation budgets are allocated for the maintenance and rehabilitation of local and state highway road networks. Therefore, it is important to properly design, construct and maintain these roads to reduce the annual road transportation budget cost allocations.

The local roads in many relatively less densely populated countries are usually thin flexible pavements which are built using thin asphalt layer (less than 40mm) or chipseals as a wearing course. The unbound granular basecourse pavement layer is a prominent structural feature in local road. These unbound materials vary in geology, mineralogy, and strength from region to region and hence it is very important to conduct standard tests on the aggregate materials to enable the categorisation of their engineering performance. This performance categorisation helps a designer to design the pavements accordingly which will consequently result in a more economical and durable pavement structure.

This chapter begins with an introduction of unbound granular materials that are used in unbound pavement layers and discusses the importance of understanding the moisture susceptibility of the unbound aggregates. The objectives and scope of the research are defined followed by a brief methodology that has been adopted to achieve the objectives. The chapter ends with a section detailing the organisation of the thesis.

### 1.2 Unbound Granular Materials in Pavements

Unbound granular materials are often used in road pavements with relatively low traffic as they are the most economical material option to provide adequate strength to the pavement structure. In most cases, the locally sourced unbound aggregates are the most economical solution among the other available pavement materials. In local low traffic volume thin flexible pavements, the unbound basecourse layer is the main structural layer that absorbs the traffic loads and spreads it to the underlying subgrade layers. The unbound materials that are used in these pavements are considered strong and durable enough to bear the dynamic loads of the traffic.

As the unbound granular materials are used intensively in the pavement basecourse and subbase layers, their performance directly controls the life of a pavement. The performance of the unbound basecourse directly relates the material resilience and resistance against the deformation that is induced by the dynamic traffic loads. There are different factors that control the performance of unbound basecourse materials i.e., particle size distribution,
maximum dry density, permeability, maximum grain size, percentage and quality of fines, geology and mineralogy of the source rock, and moisture content of the material. However, there are some aggregate properties which very significantly from quarry to quarry and even from the same geological source, however often these differences are not well understood. These properties also control the performance of the materials such as the mineralogy of the aggregates and the quantity and quality of the fines present in the material mix. Moreover, the moisture content of the materials when present in the pavement under traffic loading plays a vital role in controlling the life of the pavement. Hence, in summary all of the above factors must be tested at the moisture levels that a material can be subjected to during the pavement life.

The behaviour and performance of unbound materials is very different at different moisture contents when keeping all other factors constant. In dry conditions at very low moisture levels when compared to the optimum moisture level, the denser materials with less permeability can perform better. On the other hand, when the moisture level is higher than the optimum moisture content, the material with a coarser gradation with a higher permeability is expected to perform better than a material with finer particles that can make the material more dense but can become plastic with high moisture content, thereby reducing the load carrying capacity (stiffness) of the material. There is a need to find the optimum material mixes for different moisture contents that are expected in the life of a pavement in various environmental conditions.

In real world pavements, the moisture of the unbound basecourse can change due to the moisture intrusion from the top, sides and bottom. The water can penetrate the pavement from the surface cracks in the pavement wearing course (the top). Sometimes when the pavement is constructed in cut earthwork situations, the moisture can enter from the uphill side through the pavement shoulders (the side) and can increase the moisture of the pavement structure. The increase in the water table height in various areas can cause the increase in moisture of the subgrade (the bottom) as well as in the pavement structure.

### 1.3 Research Objectives

Considering the use of unbound granular materials in pavement layers in different topographic areas and environments, the objectives of the research can be divided into two main categories:
1. To understand how aggregates behave in increasing moisture conditions for different aggregate gradations and fine material contents;

2. To investigate how well unbound granular materials (UGM) behaviour can be simulated in the laboratory based on performance tests compared to accelerated pavement tests. To understand this behaviour, in terms of the engineering performance of the pavement layers, there is a need to understand the mineral composition of the materials and moisture / drainage conditions. Moisture can react chemically with the constituent minerals of the aggregate mix and can alter the strength of the unbound materials. Moreover, the mechanical response of the material to the applied stresses needs to be understood which can help to standardise the process of selection of unbound basecourse materials in wet conditions.

The primary objectives are further subdivided into the following secondary objectives.

1. To understand the mineralogy of the unbound basecourse materials i.e., identify the differences in the selected materials with reference to dominant clay minerals;

2. To investigate the effect of change in the clay mineral proportions on the permanent deformation of the unbound basecourse materials;

3. To consider the effect of gradation and stress conditions on the permanent deformation of unbound granular materials at varying moisture;

4. To improve the application of existing statistical models to predict permanent deformation of unbound materials at different stress states and moisture levels;

5. To evaluate the response of thin flexible pavements with varying unbound granular material properties under wet conditions in terms of pavement life and mode of failure.

### 1.4 Scope of Research

The scope of this research is limited to unbound granular aggregates that are sourced from greywacke rocks which is the most common geological rock type used in pavements in New Zealand. The unbound basecourse materials are to be tested at optimum and higher moisture contents to investigate their behaviour in highly wet environments. The moisture intrusion is considered only from the top in the full scale accelerated pavement test experiments to test the permeability of the seals over the unbound basecourse materials. The clay minerals that were considered are illite, chlorite and smectite because these minerals are found abundantly...
in the local greywacke rocks and a change in their proportion may cause a difference in the basecourse material performance.

1.5 Research Methodology

The methodology of the research is briefly overviewed in this section as the detailed methodology of the respective research is explained in each chapter. To achieve the above stated objectives, the experimental program was designed in three categories i.e., geological and mineralogical testing, repeated load triaxial testing and full scale accelerated pavement tests.

The geology of the source rocks was tested by examining the thin sections obtained from various aggregates. Thin sections helped in identifying various minerals and their arrangement in the aggregates. The arrangement of minerals gave a general idea about the mineral present in the main matrix of the aggregates and the minerals present in cleavage planes. Along with the thin section analysis, the X-Ray Diffraction tests were conducted which helped in finding the clay minerals present in the granular aggregates and fine particles (less than 0.10 mm). The ratio of expanding clay minerals in the samples gave an idea about the moisture susceptibility of basecourse materials. X-Ray Diffraction tests were selected because they give an insight to the extent of weathered and prone to weathered constituent minerals.

After testing the geology and mineralogy, the basecourse materials were subjected to Repeated Load Triaxial (RLT) testing. The RLT test method was selected as it has previously been used successfully by prior researchers to predict the performance of basecourse materials. The unbound materials in this research are tested in the RLT apparatus at optimum moisture drained, saturated drained and saturated un-drained conditions to investigate the behaviour of unbound materials at elevated moisture contents.

The materials were laid as a basecourse layer at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) in various sections with different sealing options. The pavements were subjected to surface water runoff and standard axle loading cycles. The full scale accelerated pavement tests gave information about the behaviour of the basecourse materials under wet conditions.
1.6 Organisation of the Thesis

This research document is arranged such that different research papers constitute the technical chapters. These chapters are complete units which have their defined objectives, scope, methodology, results, and conclusion sections. The organisation of the thesis is as follows.

Chapter 2 reviews the behaviour of unbound granular materials that are used in unbound basecourse layers of the pavement. Common failures in flexible pavements are discussed followed by different material specifications for the unbound materials. Afterwards, various factors that affect the performance of the UGM are discussed. The causes of water entry into the pavements from the top are also discussed. The response of UGM to elevated moisture levels is discussed at the micro and macro levels.

Chapter 3 investigates the geology and mineralogy of the aggregates and the quality and quantity of the fines in the basecourse materials along with their permanent deformation obtained from RLT tests. In this chapter, the greywacke basecourse materials from the South Island of New Zealand, which were used in the CAPTIF facility, were tested to examine the differences in mineralogy of the materials and their effect on the performance of RLT tests.

Chapter 4 covers the difference in the clay minerals and their relative percentages in the CAPTIF and selected North Island basecourse materials. The results of RLT testing are discussed on the materials from both the CAPTIF and the North Island aggregates and their observed differences based on the variation in clay mineral percentages.

Chapter 5 presents the development of a new generation statistical model to predict the permanent deformation of the UGM in the RLT test by considering both stress state and the number of loading cycles. This model better follows the trend at higher numbers of loading cycles than other existing models that the RLT test results are compared against.

Chapter 6 provides a theoretical background about the variability of performance of UGM in RLT at different moisture levels, and drained and un-drained conditions. The differences in the permanent deformation of the CAPTIF materials at saturated drained and saturated un-drained conditions are focussed and are discussed theoretically.

Chapter 7 describes the results from full scale accelerated pavement test (APT) performed at CAPTIF. The effect of the continuous surface water runoff over the pavement surface under
standard axle loads on the thin chipseal surface is examined and the permeability of the seals is addressed. Moreover, the relative performance of the basecourse materials under normal sealing practice are discussed.

Chapter 8 provides a comparison of the performance of the unbound basecourse materials in both the RLT test and full scale APT. The permanent deformation in the RLT and the APT showed similarities and differences. The causes of the similarities and differences in the permanent deformation are discussed.

Chapter 9 overviews the research objectives and discusses what was undertaken in this research to achieve the stated objectives. Chapter 10 gives the final conclusions of the research and areas for further research.

### 1.7 References

Chapter 2 Literature Review

CHAPTER 2.

LITERATURE REVIEW

2.1 Introduction

The infrastructure of a country is based on many systems and transportation systems are one of the most important systems. The primary mode of transportation in most regions of the world is the land transportation. The highway pavements play an important role in controlling the movement on land. The good condition and excellent serviceability of the pavements ensure the in time delivery of goods from the remote places of production to big markets, timely movement of people from city to city, and economical movement of freight across the country. Hence the well-developed highway infrastructure is important for the reliable movement of people and goods on land. The safety and reliability of the transportation system develops a confidence in people about the system and this improves the economy of the Country. Hence the condition of the pavements indirectly links to the prosperity of a nation.

It is very important to build the long lasting and reliable pavements keeping the costs of construction and maintenance to a minimum while considering key factors controlling the life of the pavements such as the volumes of traffic on the road, and the climatic region of the area. The causes of the pavement failures that are learned from the past experience should be properly addressed in the new design methodology to overcome the obvious problems, and to
increase the expected life of the pavement. The costs of construction and maintenance can be optimised if the local materials are properly used in the pavements after testing their performance considering the previous failure causes, the climatic conditions of that area, and expected traffic on the road.

There are many types of pavements which build a whole transportation network. There are variations in structure of different pavements which belong to the strength of the soil in the area, the volumes of expected traffic, and the climatic conditions such as mean annual rainfall and average temperature. Thick pavements (with wearing course more than 40 mm, a deep basecourse layer along with thick subbase layer) are required where the strength of soil is relatively weaker, high volumes of heavy traffic are expected, and location experiences extreme weather conditions. In contrast, thin pavements (wearing course less than 40 mm, average thickness of base along with nominal thickness of subbase or no subbase) are provided where the low volumes of traffic are expected and weather conditions are mild. These variations in pavement structure keep the budget, allocated to the construction and maintenance of the pavements, to an optimum and ensure that the pavements are not over designed (for low volumes of traffic, mild weather conditions) and not under designed (for the high volumes of traffic, and extreme climates).

The low traffic volume roads are thin pavements and provide access to the remote areas and connect them with big cities and markets. These thin pavements make a considerable portion of the road network and therefore it is important to properly design, construct, and maintain these roads to maintain the minimum level of service. The structure of thin pavements consists of a thin surfacing with unbound basecourse and subbase layers over the subgrade. The main load bearing layer in the pavement structure is the unbound basecourse which transfers the traffic load to the subgrade such that stresses are minimised to a level which is not exceeding the load bearing capacity of subgrade. The functional failures (such as stripping, potholes, bleeding) in thin pavements occur in surfacing and the structural failures (such as rutting) occur in the basecourse. The functional failure can be corrected during regular maintenance processes of the pavements whilst the structural failure demands the rehabilitation or reconstruction of the pavement which is relatively more costly than regular routine maintenance. Hence, major portions of budgets can be saved if the causes of unbound basecourse failure in low volume roads with thin pavements are properly understood and remedies are well incorporated.
Chapter 2 Literature Review

The evaluation of unbound granular materials (UGM) is very important as they build the basecourse layers and their behaviour determines the performance of the thin flexible pavements (low volume roads). There are certain material specifications for these UGM made by the local authorities in the world which control the performance of these materials in the real pavement. The behaviour of UGM depends on many factors which will be discussed later in this chapter. There are different procedures adapted to evaluate the performance of these materials. The mineralogical makeup of the source rock from which the aggregates in UGM are obtained plays an important role in determining the strength of the aggregates in various loading and environmental conditions. To evaluate the performance of the aggregates in real world pavements, the full scale accelerated pavement test (APT) is the most reliable test which imposes the maximum field-like conditions on the UGM being tested. The laboratory methods are also available to measure the performance of UGM. The laboratory methods try to replace the APT as the cost of full scale testing is very high and is impractical to perform repeatedly.

The main focus of this research was to identify the performance of UGM in basecourse in elevated moistures and predict the performance using statistical models based on laboratory tests. There are many ways by which the moisture can enter into the pavements which can elevate the moisture content of UGM in the basecourse. It is very important to properly understand the performance of UGM at high moisture contents. The factors that are involved in controlling the performance of UGM in high moisture are to be focused to improve the performance of UGM. The prediction models are very important in defining the behaviour of a material. A number of statistical models have been proposed to predict the performance of UGM at a different number of loading and stress levels. These suggested models give an insight and compare the performance of various UGM materials.

The literature review starts from an introduction to various types of pavements and continues to discuss the structure of flexible pavements. As the emphasis of this research is on the performance of UGM used as basecourse materials, the modes of failure in unbound basecourse layers are discussed followed by mentioning various specifications and factors affecting the performance of UGM. The causes of an increase in moisture of basecourse layers in actual pavements and the performance of UGM at high moisture are described. The chapter ends with the findings and conclusions which identify the needs of research.
2.2 Structure of Pavements

The pavement is a structure that transfers the load from the traffic and distributes it to the subgrade. This structure should be strong enough to ensure that the subgrade is not over stressed.

“The pavement is that portion of the road placed above the design sub-grade level for the support of, and to form a running surface for, vehicular traffic” (Transit NZ, 2005).

There are three major types of pavements (Huang, 2003) as shown in Figure 2.1:

- Flexible or asphalt pavements;
- Rigid or concrete pavements; and
- Composite pavements.

Rigid or concrete pavements distribute the vehicle loads to the subgrade by bridging action due to the rigidity of the concrete surface layer in contrast to flexible pavements where the loads are transferred to the subgrade through grain to grain contact. There are usually four main types of concrete pavements (Huang, 2003): Jointed Plain Concrete Pavements (JPCP), Jointed Reinforced Concrete Pavements (JRCP), Continuous Reinforced Concrete Pavements (CRCP), and Pre-stressed Concrete Pavements (PCP). The concrete pavements have a drainage layer of basecourse or subbase between the concrete surface and subgrade however, they are sometimes built over the subgrade directly depending on the strength of the subgrade.

The composite pavements consist of both asphalt and concrete surface layers. Plain cement concrete (PCC) is used as a bottom layer and the asphalt layer as a top layer. In these types of pavements, the PCC provides a strong foundation base and the asphalt provides a smooth riding quality. Figure 2.1 shows that this research is limited to flexible pavements. Rigid and composite pavements are outside the scope of this research and will not be discussed further.
The flexible pavements are layered systems in which traffic loads are transferred by grain to grain contact and the stress induced by the traffic loads in the surface and basecourse layers is distributed to the sub-base and subgrade. The intensity of stress decreases with the depth of the pavement. High strength materials are used in top layers as they are subjected to higher relative stresses and subsequently, lower strength materials are used in layers beneath. Flexible pavements are further classified into thin flexible pavements, and thick flexible pavements. Thick flexible pavements are usually bound asphalt pavements and are built by placing more than 80 mm of asphalt over the basecourse layer. These pavements are provided where medium to high volumes of traffic are anticipated. One of the main purposes of providing an asphalt layer over the top of unbound basecourse is to provide greater durability and a better riding quality for high volumes of traffic. Thin flexible pavements have various types of surfacing layers including chipseals of 15-25 mm thickness and asphalt layers of less than 40 mm. These pavements are constructed for low to medium traffic volumes. The flexible pavements are distinguished generally by their surfacing and thickness. The structure of a typical flexible pavement and types of surfacing for flexible pavements are discussed further in this section.
2.2.1 Structure of Flexible Pavements

The flexible pavement consists of usually three layers over the subgrade. The top layer which is directly in contact with the vehicle tyres in sealed flexible pavements is known as the wearing course. The layers beneath the wearing course are the basecourse and the subbase, respectively. The subbase layer lies over a prepared natural ground surface known as subgrade. A typical cross-section of a flexible pavement is shown in Figure 2.2.

The chipseal or asphalt layer is commonly used as a wearing course in flexible pavements. Some of the important functions of wearing courses are to provide adequate friction between the tyre and road surface, smooth riding quality, and to stop the surface water intrusion in the pavement.

The basecourse is the main structural layer in thin flexible pavements. This is an unbound layer containing a percentage of coarse aggregates. Granular materials are used in the construction of the basecourse layer. The other important function of the basecourse layer is to provide drainage for the water that may intrude from the surface or sides of the road. Materials used in the base course are specified not to be moisture sensitive.

The sub-base layer is also unbound usually are of lower strength materials when compared to the base course material. Its main functions are to provide extra strength to the pavement structure and to minimise the intrusion of fine materials from the subgrade in to the base layer so that the drainage properties of base course are not affected.
As the basecourse and subbase layers are usually composed of UGM in low volume roads, the variations in the layers come with the different geology of source rocks for the aggregates used. The different grading and thicknesses of basecourse layers also affect the performance of these layers.

2.2.2 Types of Surfacing in Flexible Pavement

In New Zealand, roads are functionally classified by their degree of serving a national importance as state highways, and local roads. Local roads are further sub-divided in to urban roads and rural roads. The pavements in New Zealand can be further divided into two main classes, sealed and unsealed. The unsealed pavements fall outside the scope of this research. According to the network statistics annual report (NZ Transport Agency, 2008), the total road network in New Zealand consists of 93,804 Kms (total length) of which 61,503 Kms are sealed pavements. Sealed pavements can be further divided into four broad classes of surfacing: chipseal surfacing, slurry seal surfacing, asphalt surfacing, and specialised surfacings.

There are many types of chipseal surfacing. The chipseal surfaces are considered as either first coats or reseals. Some of these types are explained in (Transit NZ, 2005). The slurry seal surfacing possesses the characteristics of a chipseal as well as asphalt as they fail like chipseal surfacing, however are produced like asphalt. The slurry seal mixes are designed using specific proportions of aggregates, emulsified binders and additives. There are two types of slurry seal surfacing, slurry seals and cape seals.
The slurry seal also known as microsurfacing in North America and Europe consists of specially graded aggregate, emulsion binder, filler and water. The mix is prepared in a truck-mounted mixing plant and is spread on the road surface by a spreader box behind the truck. There are four aggregate gradings which form four types of slurry seals such as Type 1, Type 2, Type 3 and Type 4. The nominal size of the aggregates in Type 1 is 3 mm and is usually applied on foot paths and for void filling on airport runways. Type 2 consists of 5mm nominal sized aggregates used on low volume roads, car parks, foot paths and residential streets. Type 3 consists of aggregates with nominal maximum particle size of 7mm and is
applied on roads with higher traffic volumes. The macrotextures resulting in Type 1, Type 2 and Type 3 are of 0.3mm, 0.5mm and 0.7mm respectively. Type 4 has the coarsest macrotexture and can be applied on high traffic volumes as a surface course (Transit NZ, 2005). Common uses of the slurry seals are; as a thin wearing course, to reduce noise, as reseals over ageing porous asphalts, repairing ruts and as an alternative to chipseals. A cape seal is a two coat seal where the first coat is a chipseal and the second coat is a slurry seal. The slurry seal is applied soon after the application of the chipseal which fills the texture of chipseal. These types of seals are used where high resistance to traffic stress, reduced traffic noise and a smooth texture are required, especially in the case when chipseals or asphaltic concrete are not acceptable options.

*Asphalt* is a surfacing treatment often applied over the existing chipseals, and in many cases constructed directly over the newly constructed chipseal. In this instance the chipseal layer is termed as a membrane seal and functions as a waterproofing layer. The *specialised surfacings* are associated with providing adequate skid resistance and for example the coloured bus lanes. These surfacing can be used as chipseals, asphalt mixes and colour surfacing. Some types of these special surfacings are rejuvenating seals (fog seal, enrichment seal) and geotextile seals. Rejuvenating seal is the application of binder on a surface with no chip applied. These seals are used to enrich an existing seal, applied to old and coarse seals where a binder is becoming brittle, and to prevent chip loss. The geotextile seals incorporate a synthetic fabric composed of flexible polymeric materials under the chipseal (single coat, two coat or racked in seal). These types of seals are used on soft, flushed and severely cracked pavements. They extend the life of the pavement by delaying reconstruction (Transit NZ, 2005).

The structure of pavements is designed such that the life of the pavement can be extended to the maximum with optimum usage of resources. Failures in pavements can occur in an unbound basecourse layer of flexible pavements due to traffic moving over it or due to environmental degradation of the pavement. The type and causes of failure in flexible pavements are discussed in the next section.
2.3 Failures in Thin Flexible Pavements

2.3.1 Failure Mechanisms, Distress Types and Causes of Failure

When a pavement is not able to perform its intended functions; to provide a smooth and safe travelling surface for road users, and/or is not capable of bearing the repetitive loads that are applied by the moving vehicles, the road is considered to be failed and therefore requires corrective action. This stage may occur before the pavement reaches its design life. There are many types of failures possible in the pavement due to the distresses induced. These distresses may appear due to repeated loading of the traffic, aging of the binder and/or intrusion of water in the base layer. Some of the failure mechanisms, distress types and causes of failure are discussed in the subsequent paragraphs.

There are two general types of failure mechanisms in pavements: functional failure and structural failure (Figure 2.3). Functional failure occurs when the pavement is not able to deliver the vital functions such as riding quality and safety but may be still structurally sound. The failure in which a pavement deteriorates to such extent that it is unable to sustain moving loads imposed on its surface, is termed as a structural failure (Yoder and Witczak, 1975).

Deterioration in flexible pavements may be indicated by poor riding quality and various surface distresses (Ricketts et al., 2003). Surface distress is defined as the indication of poor pavement performance or an indication of impending failure (University of Washington, 2005).
Surface distresses are categorised in the following three modes: fracture, distortion and disintegration as shown in Figure 2.4. The fracture can be in the form of the cracks or spalling that appear on the surface of the flexible pavements. Possible causes of the fracture are excessive loading, fatigue and thermal changes. Cracking is the most common type of distress in asphalt wearing courses. Depending on the traffic loading and considering the environmental factors, fatigue cracking is most commonly observed in the surface courses. The cracks appear probably due to accumulated damage caused by repeated traffic loading. Various techniques are used to predict the fatigue cracking such as the fatigue damaging model based on a continuum damage approach that describes the micro-cracks and crack propagation in wearing courses (Suo and Wong, 2009). The other types of cracking include block cracking, joint reflection cracking, longitudinal cracking, slippage cracking, and transverse thermal cracking. Distortion is the surface deformation in which the pavement surface turns rough. Potential causes of the distortion are densification, consolidation, swelling, and creep. Typical forms of surface distortion are rutting, corrugation, shoving, and depression. Disintegration is in the form of stripping and ravelling. It can be caused by ageing of the binder resulting in a loss of bonding ability of the binder, chemical reactions (between intruding water and contaminants, binder and aggregates), and poor consolidation/compaction.

![Figure 2.4 Distresses in Flexible Pavements](image)

**Type of Distresses in Flexible Pavements**

- **Fracture**
  - Fatigue Cracking, Block Cracking, Joint Reflection Cracking, Longitudinal Cracking, Slippage Cracking, Transversal Thermal Cracking, etc.

- **Distortion**
  - Rutting, Corrugation and Shoving, Depression, etc.

- **Disintegration**
  - Stripping, Ravelling, etc.
Chapter 2 Literature Review

There are three most probable causes of road failures as shown in Figure 2.5. The first cause is the overloading that includes gross loads of vehicles, repeated loading and high tyre pressures. A Second cause of failure is a change in climatic and environmental conditions which result in structural weakness of the pavement. Some of the failures caused by the changes in climatic conditions are volumetric changes of soil due to moisture variation, frost heave, freezing and thawing, and improper drainage. The third cause of failure is deterioration of the pavement materials in different layers such as aging of asphalt in the wearing course and deterioration in the base course material due to freeze thaw cycles (Yoder and Witczak, 1975).

![Figure 2.5 Failures in flexible pavements](image)

2.3.2 Failures in Wearing Course-Chipseal

Most of the low volume road pavements are thin surfaced pavements with chipseal surfacing. Generally chipseals have an average life of six to eight years but some of the chipseals have shown lives more than twenty years, which is rare but have been recorded. Some of the factors which prevent chipseals surviving their design lives are micro-texture polishing, chip loss, cracking and flushing (Transit NZ, 2005).
The reason for polished surfaces found at various sites was the use of aggregates with very low Polished Stone Value (PSV). Sections of only two years of age have required resealing due to polishing of the aggregates.

Premature chip loss has also been a problem for pavement surfacings. The causes for the early chip loss may be the weather, improper quality control at the time of construction and the choice of wrong seal type. In the case of early chip loss, repairs are carried out instead of applying a reseal which do not last for the intended design life.

Major causes of premature cracking in the chipseals are excessive pavement deflections and too hard and brittleness of the binder. Binder hardness causes the premature failure through cracking on pavements subjected to freeze-thaw cycles such as on the Desert Road (SH-1) in the central North Island. Due to these cracks water seeps into the base, sub base layers and then reaches the sub-grade, resulting in reduced stiffness and load bearing capacities. This results in pavement failure demanding major maintenance or rehabilitation. High deflections can be associated with the improper drainage of the pavements as well as high tyre pressures. Edge cracking on pavements is generally found in the New Zealand due to mountainous topography and accordingly narrow shoulders (Saleh and Patrick, 2006). Edge failures have been recorded on these roads due to lack of lateral support and high axle loads.

Bitumen Flushing is also a problem in chipseals resulting in early failure of the chipseal pavements. Some of the mechanisms causing the flushing are forcing of the aggregate chip in to the substrate, instable pavement layers and rising of binder due to moisture vapour.

### 2.3.3 Failures in Granular Basecourse

The type of basecourse failures and the factors causing these failures are discussed in this section. Rutting is the main surface distress indicating failure in the granular basecourse which is caused by excessive deformation in granular material under repeated loads of traffic, aggregate breakdown, and the improper draining capability of the granular material.

One of the failure mechanisms in thin flexible pavements is the excessive plastic deformation in the unbound granular pavement layers (Werkm eister et al., 2003a). Every passing vehicle induces some permanent strains in the unbound basecourse layer of the pavement which accumulate with the passage of time. This ultimately results in rutting of the pavement surface. As the basecourse is made of unbound granular material, the resistance to tensile stress that develops at the bottom of the basecourse is negligible. As a result, the thickness of
the base course directly controls the stiffness of the pavement section as well as it prevents the early rutting due to traffic loading (Gopalakrishnan and Thompson, 2008).

The mechanism of rutting in flexible pavements can be due to the compaction of the aggregates in basecourse layer or due to the shear failure of the materials under the traffic loading. The compaction of the basecourse layer under the traffic loading occurs when the compaction of basecourse in the pavement is not done properly during the construction process. The shear failure occurs when the basecourse is subjected to the loads that exceed its load bearing capacity and the material is pushed on the sides from the path of moving loads. The pavement load bearing capacity can be reduced due to many reasons and this also causes a rut that result from a shear failure.

The process of deterioration can be accelerated by other factors such as a lack of weathering resistance of the basecourse material and/or lack of proper drainage (Diyaljee, 1985). The permeability and moisture susceptibility of the granular materials used in the basecourse layers affect the pavement performance as it is concluded by Bejarano & Harvey (2002) the well-drained pavement sections fail due to rutting while the undrained pavement sections fail first due to fatigue cracking. The use of coarse gradation enhances the permeability in the granular material (Freeman and Anderton, 1994). The granular materials are arranged into two groups according to the gradation criteria i.e., dense graded and open graded, which also predict their permeability. The adequacy of the drainage layer which was introduced in the pavement structural design by The Corps of Engineers in both flexible as well as rigid pavements was investigated (Grogan, 1994) and it is concluded that the introduction of a drainage layer in the flexible pavement improves its performance. The exposing of base course material to elevated moisture contents, decreases the stiffness, load carrying capability and resistance to permanent deformation (Chen and Scullion, 2008). Some effort has been made by previous researchers to formulate the benefits of drainage in granular materials. The drainability of granular materials can be defined in terms of the degree of saturation of that material under gravity drainage. Drainage of basecourse materials depend on the cross-section of the pavement as well as the geometry of the road. A formula was presented for the minimum degree of saturation. The method proposed included one dimensional analysis of the saturated flow in granular base materials (McEnroe, 1994). An Equation 2.1 to predict the permeability of the untreated granular base material was developed considering the factors such as maximum aggregate size, porosity and aggregate gradation (Elsayed and Lindly, 1996).
Chapter 2 Literature Review

\[ K = -0.251 + 0.92 \text{VOID RATIO} + \frac{2.68}{\text{PASS 30}} - 0.005 \text{PASS} \quad 2.1 \]

Where:

\[ K \quad = \quad \text{Coefficient of Permeability (cm/sec);} \]

\[ \text{VOID RATIO} \quad = \quad \text{Ratio of volume of voids to ratio of volume of solids;} \]

\[ \text{PASS}30 \quad = \quad \text{Percentage by weight of aggregates that Pass 0.6 mm (No. 30)} \]

sieve in aggregate specimen; and

\[ \text{PASS}200 \quad = \quad \text{Percentage by weight of aggregates that Pass 0.075 mm (No. 200)} \]

sieve in aggregate specimen.

The equation resulted from the regression of the data, obtained from the rigorous testing of these materials for permeability. The equation gave better results compared to existing equations at that time.

The common failure mechanisms in the thin flexible pavements have been discussed especially for the wearing course and the basecourse layers as most of the failures occur in these two layers in low volume roads. The causes of failures for unbound basecourse are discussed generally in this section and are discussed in more detail in the next sections describing the factors that affect the performance of UGM. The properties of UGM basecourse materials, commonly addressed in specifications by the local authorities, keep the basecourse layers performing to an optimum level and are explained in the next section.

### 2.4 Specifications for Unbound Basecourse Materials

In pavement design process, structural layers are not supposed to fail. To ensure this fact, road controlling authorities prepare specifications for the qualification of road pavement materials used in structural layers such as the basecourse layer (Austroads AGPT04A/08, 2008; Department of Transport UK Volume 1 Series 0800, 2009; Florida DOT 204, 2010; Transit New Zealand M/4, 2006). The aggregates used as basecourse materials are evaluated for their source properties and production properties in these specifications, and include various quality and performance tests for the road materials. The source properties are important as the characteristics of the aggregates depend upon the source rock. The properties of source rocks that are usually considered are mineralogy, grain size and texture, rock structure, and degree of weathering of the rock. The production specifications control the properties of aggregate blends that are to be used in the basecourse layer. The production
properties that are usually addressed in the specifications are maximum size, particle size distribution, particle shape, nature of fines, clay and silt content, particle strength and durability, compacted density, and optimum moisture content of the aggregate blend. The layout of basecourse aggregate properties that are addressed in specifications is shown in Figure 2.6.

![Figure 2.6 Aggregate properties addressed in specifications](image)

### 2.4.1 Source Properties

The base course aggregate is obtained by crushing or breaking either: water worn gravel; quarried rock, or from other sources accepted by the respective authority. The source material is normally free from soft disintegrated stones, and deleterious materials. It contains hard and sound material of uniform quality. The geology of basecourse aggregates is an important aspect which can influence the performance of these aggregates in pavement and the engineering properties of aggregates (i.e., mineralogy, grain size and texture, rock structure, degree of weathering) can be assessed if the type of the source rock is known as the rocks are classified based on their mineralogy, grain size, and texture. The type of the rock along with a petrographic description provides a sound foundation to judge the engineering properties of the source rocks (Austroads AGPT03/09, 2009).
The type of the source rocks can be identified by the microscopic investigation of the rock by making thin sections (Higgins, 2000; Ikuta et al., 2007). A standard thin section is prepared from the rock on the glass slide which is about 30 μm thick. Thin sections can be inspected under the electron microscope to identify various types of minerals present in the rock. There are minerals which are strong (i.e., quartz, feldspar) while some minerals are relatively weaker (i.e., chlorites, smectites) and can reduce the strength of the rocks (Black, 2009). The information about the minerals present in the aggregates can be useful considering the type and location of the pavement in which these aggregates are to be used. Thin section technique can be further utilised to find the arrangement of grains in the rock, their texture, and overall rock structure. The strength of aggregates is dependent on the shape of grains, arrangement of grains, and the cementatious nature of the abundant mineral found in the matrix of the aggregate. An example of this type of petrographic investigation are detailed in Austroads Standard AGPT03/09 (2009).

The source properties are checked for the periods not exceeding two years and when the aggregate source or the processing method is changed. Source properties are sampled for every 10,000 m³ of the source material. The source property tests included in Transit NZ standard M/4 (2006) are Crushing Resistance Test, California Bearing Ratio Test, and Weathering Quality Index Test.

### 2.4.2 Production Properties

The representative samples of the aggregates for testing the production properties of aggregates are obtained from conveyor belts, bin, stockpile and trucks. Representative samples are obtained in accordance with local specifications. Table 2.2 shows the minimum sampling rate for the production property tests.

**Table 2.2 Minimum sampling rate for production property test (Transit New Zealand M/4, 2006)**

<table>
<thead>
<tr>
<th>Lot size</th>
<th>Number of Samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>From</td>
<td>To</td>
</tr>
<tr>
<td>1 m³</td>
<td>400 m³</td>
</tr>
<tr>
<td>400 m³</td>
<td>1500 m³</td>
</tr>
<tr>
<td>1500 m³</td>
<td>4000 m³</td>
</tr>
</tbody>
</table>
The production properties of the aggregates give an indication of performance of the aggregates in the actual pavement layers, and are briefly introduced here in the subsequent paragraphs. The specific requirements can be found in the local specifications developed by local authorities. The aggregate size distribution or aggregate gradation is considered to be the measure of how the material might mix and compact. The gradation specifications are developed by the national/local authorities based on the performance of available materials and include an upper and a lower limitation curve. The gradation curve of aggregates to be used in road construction should lie within these curves.

The shape of the particles also affects the permeability of the aggregates as well as the mechanical stability of unbound material. The Flakiness Index and Elongation Index tests are used to measure the shape of the granular aggregates. The permeability of aggregates is also an important parameter when discussing the drainage in basecourse layers. The permeability of the coarse aggregates can be measured by Falling Head Permeameter test.

The aggregate cleanness determines the performance of the aggregates when moisture conditions are considered. The fine particles on the aggregates affect the bonding of bitumen with the aggregates as well as reduce their draining capability. To measure the aggregate cleanness, Sand Equivalent test and Cleanness Value tests are used.

The durability and soundness of the aggregates are determined in many ways by the following tests. The Weathering Quality Index (WQI) determines the nature of matrix of a material and its degree of lithification. The Sulphate Soundness test and Micro-Deval test measure the volume changes caused by the water in pores during freezing. The Los Angeles Abrasion test gives the abrasion resistance and impact resistance of the aggregates. The California Durability Index estimates the degree of abrasion when aggregates are mixed with sand.

The clay fractions in the aggregate material affect the strength and moisture susceptibility. The Clay Index test determines the surface area of fine particles and the Plasticity Index tests give us the indication of moisture content where the material starts to be plastic.

Strength of the aggregates determines the ability of pavement layers to carry the traffic loads. For strength assessment of the road aggregates, the Crushing Resistance (CR) or Aggregate Crushing Value test and the California Bearing Ratio (CBR) test are used.
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The specifications of basecourse UGM are for the characterisation of the materials which ensure that the used materials conform to standard quality control measures. To measure the performance of the aggregates in the field, there are some laboratory tests which try to simulate the stress conditions on the sample of these aggregates which they are subjected to in actual pavements. The stiffness and the resulting strains are used to evaluate the performance of mix in actual pavements. The stiffness of the compacted aggregates is determined by the confined sample which is subjected to axial loading. The data for axial stress and resulting strains is collected to investigate the stress strain relationship of the material. The Repeat Load Triaxial (RLT) test is common and reports the resilient modulus and permanent deformation in the samples. The RLT test will be explained later in this chapter. The factors which affect the performance of unbound basecourse materials are discussed in next section.

2.5 Factors Affecting the Performance of UGM

The behaviour of the UGM can be defined as the response of the materials to the applied dynamic loading. The UGM in the basecourse layer of a pavement are subjected to moving loads of traffic which induce stresses in the pavement and the induced stresses produce the recoverable and non-recoverable deformations in the pavements. The recoverable deformations are often referred as resilient deformations and show the strength of the materials to bear the dynamic loads. The rate of accumulation of non-recoverable or permanent deformations in unbound pavement layer hints about the current condition and remaining life of the pavement. The other properties of UGM such as anisotropy and Poisson ratio can help to predict the performance of the materials in the pavement. The factors that affect the performance of UGM are the applied stress condition and physical properties of individual grains as well as the mixture. The magnitude of applied stresses on UGM determines how the material is going to perform as the proportion of applied shear stress to the failure stress is very important. The proportion determines whether the UGM is going to either behave elastically or will keep deforming gradually. Along with stresses, the physical properties of mix such as gradation curves, percentage of fines, shape of individual grains, density, and moisture control the performance of UGM. These factors are explained in detail in this section mainly addressing the permanent deformation of the UGM as it is the dominant mode of failure in such materials.
2.5.1 Stress

The magnitude of stresses produced in unbound basecourse pavement layers depends upon the load carrying capacity of above layers. The higher is the load carrying capacity of top layers, lower will be the stresses transferred to the basecourse layer. However, there is provided a thin wearing surface in low volume roads and this thin wearing course is not designed to dissipate the loads applied by the moving traffic. Hence the higher stress conditions prevail in thin pavement basecourse layer compared to the basecourse of thick pavements. The unbound materials in basecourse show a stress dependent behaviour and the stress levels to which UGM basecourse are subjected in thin pavements should be considered to predict the performance of thin pavements (Van Niekerk et al., 2002). Heavy traffic produces higher stresses in the pavements and the rutting in the unbound basecourse of thin flexible pavements is directly proportional to the magnitude of stress applied (Zakaria and Lees, 1996). The higher mean stress levels on UGM cause these materials to deform at higher rates leading to relatively quick failure of unbound basecourse (Dawson et al., 2000).

At lower level of stresses, the permanent deformation accumulation in UGM eventually levels off after the application of some loading cycles and UGM show a perfectly resilient behaviour. But the accumulation of the permanent deformation continues at higher stresses and leads to the failure of the pavement due to rutting. Hence there lies a threshold level of stress which differentiates between two responses of the UGM and is defined as shake down limit of the stresses (Lekarp and Dawson, 1998). The response of UGM is elastoplastic at lower stresses that are well below the failure stresses however, it becomes plastic as the stress level increases to the failure state (Wolff et al., 1994). The stress level has an effect on the permanent deformation of UGM but there is a need to relate the permanent deformation behaviour to an ultimate stress level which may be less or greater than the failure stress condition (Lekarp et al., 1996).

In low volume roads and thin asphalt pavements, there is a need to measure the stress in UGM layers as they may lie in shake down stage where the layer is continuously compressed with each loading cycle (Werkmeister et al., 2003b). Hence number of loading cycle before the required rut depth should be calculated. The level of stress from shake down approach can be found out for a specific material that if the rutting in the pavement will be of some concern or not (Dawson et al., 2003). The level of stress is defined by shakedown stages in granular materials. Shakedown A is the stress stage where stresses are lower so that materials behave
elastically after initial loading cycles. Shake down B is the stress state where permanent strains keep accumulating at slower rates, while shake down state C is where materials keep deforming at higher rates (Werkmeister et al., 2001).

The traffic loading applies a shear stress in the unbound basecourse layer but it is not only the shear stress that affects the permanent deformation. The ratio of shear stress and confining stress is also an important aspect to consider while predicting the permanent deformation of UGM. The constant and variable confining stress on UGM can affect the permanent deformation but if the constant stress is equal to the mean of cyclic stress, the similar value of permanent deformation is obtained in both cases of cyclic and constant confining stresses (Brown and Hyde, 1975). Confining stress plays an important role in controlling the permanent deformation (Khogali and Mohamed, 2004) however, the combined effect of confining and shear stress also affects the permanent deformation of UGM (Theyse, 2002b). For giving confining stress, the permanent deformation can be controlled by limiting the applied shear stress ratio to the failure shear stress (Huurman and Molenaar, 2006).

### 2.5.2 Loading History and Number of Loading Cycles

The traffic loads are repeatedly applied on the pavement and the pavement response varies with the number of loading cycles, and history of loading. The history of the loading on UGM affects the permanent deformation behaviour (Brown and Hyde, 1975) and the sample tested in RLT tests with some loading history shows less permanent deformation than the sample with no loading history (Dawson et al., 2000). Hence the stress history of the UGM should be considered while predicting the permanent deformations in unbound basecourse layers (Kim and Tutumluer, 2006).

The number of loading cycles are also important as the UGM experience a small permanent deformation with each loading cycle (Lekarp and Dawson, 1998). There is a nonlinear behaviour of UGM as the rate of permanent deformation decreases with increase in number of loading cycles (Lekarp et al., 1996). The deformation in UGM is not linear in the initial phase of loading and becomes linear after certain loading cycles. The initial phase is attributed to the densification of the material (Wolff et al., 1994).

If a sample is subjected to a single stress state in RLT test, it will produce similar amount of permanent deformation compared to the sample subjected to staged stress states provided; keep the level of stress state in previous stages less than the stress state applied in single stage
test, and keeping final stress same as in single stage test (Gidel et al., 2001). This is very important finding as this allows one to test a single UGM sample in one RLT test at various stress states, reducing the testing time, and avoiding the variance due to different samples.

### 2.5.3 Gradation of Materials

Gradation is an important factor when studying permanent deformation of UGM (Thom and Brown, 1988; Van Niekerk et al., 2002) as gradation of UGM controls the extent of rutting produced in the unbound basecourse layer in the pavement (Zakaria and Lees, 1996). The permanent deformation of UGM is more affected by the packing characteristics that are controlled by grading than the compaction effort (Dawson et al., 1996).

There can be two principle types of gradation that can be obtained in an unbound granular mix; well graded mix, and uniformly graded mix. Well graded mix has comparatively shown lesser permanent deformation when compared to the uniformly graded mix (Dawson et al., 2000). The well graded aggregates showed higher shear strength than the open graded aggregate mix (Nunez et al., 2004). Comparing the open graded mix materials, it was noticed that greater particle size in one mix open graded gives higher angle of internal friction than the other with smaller maximum aggregate size. The greater aggregate size gives higher shear strength at higher confining stress (Nunez et al., 2004).

The gradation of unbound materials control their performance in different moisture conditions as; coarser gradation allow the moisture to drain more easily and not creating higher pore pressures; finer gradations show less permanent deformation as they show more stiffness in dry conditions. The coarser gradation with Talbot’s constant ‘n’ value 0.8 performed well in wet conditions while the material with gradation constant 0.3 performed well in dry conditions (Arnold et al., 2007).

### 2.5.4 Density

Density is the property of aggregate mix which affects the permanent deformation produced in UGM basecourse layer (Theyse, 2002b; Thom and Brown, 1988). A certain level of maximum dry density of the unbound materials from laboratory is to be achieved in basecourse layer by compaction and is mentioned in local specifications. To achieve the minimum level of density is important as change in density varies the performance of UGM and pavement may face early failures. The basecourse with low density show increased rates of permanent deformation however, the rate of deformation decreases as the density of the
basecourse increases (Cheung and Dawson, 2002; Dawson et al., 2000). More over if the density of the UGM increases 100% of maximum dry density (MDD), the effect of change in density on the permanent deformation is minimal (Vuong, 1992).

The ratio between applied shear stress and failing shear stress which controls the rutting depends upon the degree of compaction of UGM which is controlling the density of the unbound material in basecourse layer (Huurman and Molenaar, 2006). Hence, Degree of compaction plays an important role in controlling the permanent deformation in UGM layers in a pavement (Van Niekerk et al., 2002). Relatively more compaction effort produces more stiff unbound granular mix which shows more resistance to permanent deformation (Magnusdottir and Erlingsson, 2002).

2.5.5 Percentage of Fines

The fine particles are referred to the clay particles which pass through 0.075 mm sieve. The fine particles are added to a basecourse material to increase the density and stiffness of the unbound granular aggregate mix. But an increase in percentage of fine in UGM decreases the load bearing capacity and resistance to permanent deformation of the mix (Dawson et al., 2000; Khogali and Mohamed, 2004). This might be due to the reason that the increased proportions of fine particles cause the aggregate particles to flow under the action of high stress conditions. The addition of fines makes the mix dense and below a certain proportion does increase the ability of mix to resist permanent deformation. However, if the percentage of fines gets higher than a certain proportion, endangers the unbound granular layer to poor drainage and subsequent saturation which leads to increased rate of rutting and early failures (Thom and Brown, 1987). The increased percentage of fine can decrease the stiffness. The dense graded materials perform well if the percentage of fines is less than 9%. The dense graded materials with high fine content than 9% have reduced strength (Magnusdottir and Erlingsson, 2002).

2.5.6 Aggregate Shape

The aggregates physical characteristics such as shape, angularity, surface roughness, and roundness affect the resistance to permanent deformation of these aggregates in UGM when subjected to stresses in the basecourse layer (Barksdale and Itani, 1989). In unbound layers of flexible pavements, the load is distributed through grain to grain contact. The stiffness that is required to sustain traffic loads is obtained through interlocking of the aggregates which is
directly dependent on the physical properties of aggregates such as shape, and surface roughness. A good level of interlocking is achieved if the surface is rough and shape is angular, compared to when the surface of aggregate is smooth and shape is round.

The value of applied shear stress which controls the rutting depends upon the shape of particles in UGM (Huurman and Molenaar, 2006). The angular aggregates require higher stresses to deform the unbound granular mix than the round aggregates due to decreased interlocking in round aggregates. The gravel which has rounded surface increases the susceptibility of mix to permanent deformation compared to the angular aggregate which reduces the deformation (Dawson et al., 2000). The angularity and roundness of the stone particles influence the resistance to permanent deformation. The surface roughness and surface friction of the stone particles are important parameters in determining the performance of the UGM (Cheung and Dawson, 2002).

2.5.7 Moisture

Moisture content of UGM plays an important role in controlling their resistance to deformation as increase in moisture content increases the accumulation rate of permanent deformation in UGM (Dawson et al., 2000; Khogali and Mohamed, 2004). Water content of UGM is the highest influencing factor which controlled the resistance to permanent deformation (Gidel et al., 2001). An increase in moisture from pessimum to optimum level increases the permanent shear deformation in UGM (Dawson et al., 1996). The presence of moisture increases the permanent deformation in UGM by lubricating the aggregates when there is apparently no pore pressure (Thom and Brown, 1987).

The level of moisture is benchmarked from the optimum moisture level where a material shows maximum density. There is least change in permanent deformation behaviour below 70% of optimum moisture content (Vuong, 1992). The increase in moisture increased stiffness when the degree of saturation was very low. When the moisture increased from certain degree of saturation, stiffness decreased leading to total collapse in some cases. This can be attributed to high pore pressures generated leading to reduction in magnitude of effective stress (Magnusdottir and Erlingsson, 2002).

Moisture content can also be presented in the form of degree of saturation which affects UGM resistance to permanent deformation (Theyse, 2002b). In a pavement, unbound basecourse may attain higher degree of saturation. Saturation of poorly drained base course
layer reduces the load carrying capacity of the pavement and can cause increased
deformations (Thom and Brown, 1987). If the moisture is drained before reaching the UGM
layer, the well-drained pavement section shows less deformation than the un-drained section
(Bejarano and Harvey, 2002). Moisture can be more severe than the applied loading
considering the permanent deformation in unbound flexible pavements. Change in moisture
causes greater deformations than the change in loading hence the moisture of the UGM
basecourse layer must be kept checked (Dawson et al., 1996). The amount of rut produced in
UGM basecourse layer increases with an increase in moisture of the pavement (Zakaria and
Lees, 1996). An increase in moisture reduces the plastic shakedown limit of the material.
There can be a threat of rutting at higher moisture contents in basecourse (Werkmeister et al.,
2003b).

The factors affecting the performance of UGM in basecourse layer are discussed in detail in
this section. This research further continues to discuss the performance of UGM in varying
moisture conditions and is discussed in section 2.7 in detail. It is obvious that a pavement can
perform better in dry conditions but water can still get penetrated in the pavement structure.
The entry of moisture in the pavement structure can be possible in many ways but the
moisture intrusion through surface is very important as the conditions required to enter
moisture from top are most common in pavements and are detailed in next section.

2.6 Water Entry in Pavements through Surface

The flexible pavements networks face early failures due to the water intrusion into the
pavement layers. The water intrusion in the pavement layers increases the degree of
saturation of the pavement materials in the basecourse, sub-base and subgrade which results
in reduced load carrying capacity and rutting of these layers (Chen and Scullion, 2008;
Ekblad and Isacsson, 2008). The entry of moisture through surface is possible in almost every
pavement at some stage of pavement life and hence is particularly addressed in this section.
The quantity of water entering pavement from the wearing course layer determines the life
and performance of the flexible pavement. The quantity of water penetrating in a pavement
depend upon many factors including water film thickness over the pavement surface, sealing
characteristics of wearing course, and traffic loading.
2.6.1 Water Film Thickness

The rainfall is the major source of water that comes on the pavement surface and builds a water film. The thickness of this water film depends upon the amount of water and drainage path provided. Various researchers have developed some methods to find the water film thickness over the pavement surface and to find the solution to the hydroplaning problems that are created by the water film over the road surface. Hydroplaning can increase water penetration in the pavement through surface as the water in front of tyre is pushed hard against the pavement surface.

The rainfall is a major source of water runoff on the pavement surface and the rainfall frequency in an area govern that how frequently a pavement is subjected to surface runoff. The surface runoff builds a film of water on the surface of pavement and the thickness of this film is controlled by duration and intensity of rainfall. The longer duration and high intensity of rainfall produce a thick water film over the road surface that affects the infiltration of water into the pavement subsurface layers.

The thickness of water film on the pavement surface determines how much water intrudes the pavement surface through cracks or under the tyre pressure. There will be a greater probability of intruding water through wearing course if there is a deep water film which will provide more water. Water film depends upon the path followed by the road surface runoff which in turn depends on the geometry of the road such as grades of the road and cross fall. The length of surface drainage path control the water film thickness directly as the water film thickness increases with the length of the drainage path.

Various researchers have tried to measure the water film thickness on the pavement surface (Becchi et al., 2001; Chesterton et al., 2006; Huebner et al., 1997; Langanier, 1976). The water film thickness on pavement surfaces develops due to the rain fall, longer surface drainage paths leading to the side drains, and the texture of the road surface. A method was proposed to measure the water film thickness on the pavements (Langanier, 1976). This method was based on the use of device which measured the moisture indirectly by rapid neutron deceleration. Geometric data, gradient and ruled surface of the pavement treatments were considered for measuring water film thickness. It was concluded from a research that water accumulating on the surface of pavements could be calculated by the water film thickness and average texture depth of the pavement (Agrawal and Henry, 1977).
Water films on the road create safety problems as they reduce the friction between the tyre and the road and may also cause hydroplaning. Hydroplaning on the pavements was linked to the water film thickness and speeds of vehicle. Empirical equations were developed to relate these two criteria (Huebner et al., 1986). It was found after an investigation that tire pavement friction was reduced significantly considering very thin water films ranging from 0.025 to 0.230 mm. New equipment were developed and used for the rubber surface friction as well as water film thickness on the pavements (Kulakowski and Harwood, 1990). A computer model PAVDRN was developed to determine the speed of the vehicle on which hydroplaning effect would initiate, considering some specific cross-sections. This model was limited to five pavement sections including tangent, super-elevated, transition, vertical crest and sag sections (Huebner et al., 1997). A wedge that is created in front of the tyre that causes hydroplaning also affects the water to be pushed into the pavement beneath layers at relatively higher rates through cracks in the pavement surface.

The duration and intensity of rainfall, and the length of drainage path which is controlled by the geometry of the road, control the thickness of water film which is calculated by different methods described by the researchers. The water film thickness provides water over the pavement surface but the intrusion of that water depends upon the condition of cracks and sealing ability of wearing course.

### 2.6.2 Sealing Characteristics of Wearing Course

Type and condition of pavement surface control the amount of water which penetrates into the pavement. The asphalt layer is usually more water proof than the chipseal layer while newly constructed. With the passage of time, cracks develop in asphalt layer as well as chipseal which allow the intrusion of water into the pavement layers through them.

One of many ways the moisture enters the pavement is from the top through wearing course and even into the intact chipseals. Some efforts have been done to observe the moisture ingress into the pavement through the wearing course (Ball et al., 1999; Button, 1996; Kutay and Aydilek, 2007; Towler and Ball, 2001; Vuong, 2007). In a study by Button (1996) the permeability of slurry seal and micro-surfacing was found less than $1 \times 10^{-5}$ cm/sec. It was recommended that asphalt layer should be covered with these surfaces initially for 2 to 4 years after the pavement construction. This recommendation was due to the fact that rate of aging of asphalt was found substantially reduced after four years. Hydraulic conductivity of asphalt was studied by Kutay and Aydilek (2007) and the velocities of moisture movement
were calculated throughout the depth of the pavement section under dynamic loading. It was found that the top 10 to 40 mm of sample surface was affected by higher pressure gradients and velocities.

Most of the chipseal pavements are susceptible to moisture intrusion causing the distress in pavements hence the permeability of the chipseal needs to be addressed. Two possible mechanisms by which the water may ingress in the chipseals were considered: The vapour pressure of water present in base course pushes the bitumen through the seal layers; and Water penetrates from the top into chipseals and with the temperature rise; it vaporises which cause the bubbles to produce on the surface (Ball et al., 1999). After the experimentation, it was concluded that both of the mechanisms were true. From the tests performed in the laboratory, it was observed that chipseals were highly permeable under pressure (Towler and Ball, 2001). It was also concluded that most chipseals are permeable to water more than expected and the amount of water which penetrates through chipseals depends upon the rainfall and the heavy traffic.

2.6.3 Traffic Loading

The traffic loading applies the pressure to the water comes between tyre and the road surface which cause water to penetrate through the chipseal and the cracks in pavement surface. Intensity and frequency of the traffic loading are a few of many factors which determine how much water enters the subsurface layers of the pavement.

Tyre pressures of vehicles passing over the pavement apply enough pressure on the water flowing on the pavement surface to enter into the pavement through wearing course especially in chipseals. It was concluded after dynamic pressure tests on seals that water is forced into the pavement under traffic loading (Ball et al., 1999). Water was recorded to enter the chipseal from top at fault sites which were 110 in 1 sq meter. These fault sites enlarged under the heavy loading i.e., truck tyres. It was found by Ball et al. (1999) that more than expected amount of water can ingress through the intact seal layers under low tyre pressures, even at pressures below those in car tyres.

The factors that affect the water to enter the pavement from the surface are discussed to understand and control the water ingress into the pavements from top. The control of moisture becomes very critical in case of unbound flexible pavements where the basecourse layer is constructed from UGM. The performance of the pavements is considerably affected
by the moisture increase in the unbound pavement layers. Hence the response of the UGM is also very important to be studied and is discussed in next section.

### 2.7 Response of UGM in High Moisture Condition

It is generally known that the dry basecourse materials perform better than the wet materials but there is need to investigate the probable causes of the underperformance of the wet basecourse materials. Once identified the root cause of the problem, the materials can be selected suitably for different environmental condition, the design of the pavement can be improved, and selection of surface course can be optimized. The factors affecting the performance of basecourse materials have been discussed in section 2.5. The effect of moisture on basecourse materials is discussed in detail in this section.

The basecourse materials can be obtained from different available sources and can have different gradation compositions suiting the local conditions. Different sources induce the variability in the mineralogical composition of the aggregates and the gradation variability can affect the stiffness of the basecourse material. Therefore, the response of the unbound basecourse materials to the elevated moisture content needs to be understood at two different levels i.e., at micro level which is at an aggregate level, and at macro level which is the response of the basecourse materials which are mixture of aggregates and fine particles. The micro level investigations provide information about the minerals composition, the arrangement of minerals in aggregates, and help to understand the response of minerals at the surface and in the matrix of an aggregate when it comes in contact with water. The micro level investigations involve the examination of thin sections of the aggregates of thickness about 30 microns on a glass slide (Higgins, 2000), the X-Ray Diffraction (XRD) test (Wilson, 1987) on the aggregates powder conducted in a standard XRD equipment, and other standard tests such as clay index tests (Sameshima, 1977) conducted in the laboratory. Thin section technique details the arrangement of minerals in an aggregate i.e., minerals in the matrix of the aggregate, and in veins of the aggregate. The minerals in matrix of the aggregate give strength to the aggregate while the veins in aggregate can be planes of failure if consist of weak minerals like chlorite and smectite.

The macro level investigations provide information about the strength of the basecourse material under the expected traffic loading and its deformation characteristics at different moisture levels. These investigations include Repeated Load Triaxial (RLT) test, static shear tests, and if possible full scale accelerated pavement tests on basecourse materials (Arnold,
2004; Werkmiester, 2003). There are different standards for RLT tests having variation in the sequence of stress paths, magnitude of the deviator and confining stresses, frequency of loading, and difference in staged and un-staged testing. Static shear tests are conducted in standard triaxial cell with static loading. The full scale accelerated pavement tests can be performed in the available facility by constructing a full pavement and applying different axle loading at different speeds. The response of unbound basecourse materials to elevated moisture conditions varies with the variation in mineralogy of the source rocks of the basecourse aggregates, and the variation in basecourse material mixtures, which are discussed further.

2.7.1 Micro Level Effect of Moisture (Clay Mineralogy)

The aggregates face degradation when exposed to excessive moisture due to reaction of constituent clay minerals with water. The porosity and compactness of the aggregates determines the depth to which water can reach in the aggregates. The water can be entrapped in the aggregate structure of longer time if the aggregate is porous. The Greywacke aggregates are most common basecourse materials in New Zealand and they are usually light weight and Greywacke are slightly porous rocks. The clay minerals commonly found in Greywacke aggregates are mainly quartz and feldspar along with small proportions of kaolin, illite, chlorite, and smectite (Black, 2009). The quartz and feldspar minerals give strength to the aggregates and form the matrix of the aggregate. The other minerals are found in small proportions and can cause disintegration in the basecourse material either by expanding, becoming highly plastic, and degradation. The structure and composition of these minerals is responsible for their different behaviour and is explained by Murray (2006).

The disintegration process makes the aggregates and clay proportions weaker in strength. The high proportions of kaolin and montmorillonite groups in the basecourse materials can cause early failures in the basecourse materials at high moisture conditions (Sameshima, 1977). When the basecourse is subjected to moisture for longer periods of time, the clay minerals such as chlorite, albite, and illite disintegrate to kaolin and montmorillonite group minerals. Similarly the smectite group minerals expand quickly when subjected to moisture causing an aggregate to disintegrate and makes the basecourse material expand causing decrease in density leading to quick failures.
2.7.2 Macro Level Effect of Moisture

The macro level investigation deals with the moisture movement in the basecourse aggregate mixes and hydrostatic pressures develop under the traffic loading.

Resilient and Permanent deformation characteristics of the basecourse aggregate materials help in determining their performance in the pavement under the moving vehicles. It has been investigated that resilient modulus of the basecourse aggregates increases with a decrease in degree of saturation (Hicks and Monismith, 1971). The increase in moisture lubricates the aggregates in the basecourse aggregates and increases the resilient and permanent deformation in the basecourse (Dawson et al., 1996; Thom and Brown, 1987). Similarly, the Poisson’s ratio was found decreasing when the degree of saturation of the basecourse materials was increased. The effect of saturation increase and number of loading cycles on permanent deformation is shown in Figure 2.7.

![Figure 2.7 Effect of degree of saturation on permanent strain (McLachlan and McLarin, 1995)](image)

Drainage is also an important parameter which in combination with moisture affects the performance of unbound basecourse. If we compare the extent of damage by moderate levels of moisture in normal and poorly drained basecourse, the same moisture level can affect the poorly drained basecourse to higher extent than the normal basecourse (Thom and Brown, 1987). During high rainfall seasons, the water gets penetrated into the basecourse and heavy vehicles movement cause increase in pore water pressure which reduces the life of the pavements considerably. The abundant free water on road can accelerate the non-load bearing
deterioration processes (Cedergren, 1988). Therefore proper drainage can help the water to keep out of pavement structure and can slow the decrement process of pavement.

The dynamic loading on unbound basecourse materials creates an excessive pore water pressure in the basecourse which reduces the effective stress. The generation of pore water pressure depends on the gradation of the basecourse materials and open graded basecourse materials show more resistance to the deformation (McLachlan and McLarin, 1995; Raad et al., 1992) and the effect of degree of saturation on pore pressure is shown in Figure 2.8.

The effect of moisture content varies with the density of basecourse materials. The basecourse materials show best performance at the higher density and lower moisture contents (Vuong, 1992). The effect of moisture content and density on resilient modulus is shown in Figure 2.9.

Figure 2.8 Change in pore pressure with degree of saturation (McLachlan and McLarin, 1995)

![Figure 2.8 Change in pore pressure with degree of saturation](image-url)
The materials are to be selected in such a way that they perform to optimum level as the high densities do give the material the required stability but can decrease the permeability of the materials. The open graded materials are very hard to compact and require a lot of compaction effort during which materials can get broken down and can change the gradation of the materials.

### 2.8 Summary

The low volume pavements often have unbound basecourse layer which is the main structural layer. The UGM used in basecourse have different properties which affect the performance of basecourse in pavements under traffic loading. There are different types of failures in thin pavements and are controlled by the properties of basecourse materials. For controlling the basecourse properties, there are many specifications available from national and local highway authorities which suit locally available materials and environmental conditions. Water on the pavement surface can enter into the pavement structure which can increase the moisture content of the basecourse layer. The change in basecourse moisture can affect the performance of basecourse layer significantly and has been discussed in the literature review. Moreover, the more related literature review is presented in each chapter for that specific topic.

There is a need to find that how the UGM perform at and above optimum moisture content in the laboratory and in the field. The results from laboratory tests can provide the initial information about the materials which can be used in the design of the pavements. Moreover,
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the laboratory tests are economical and easy to conduct as compared to the full accelerated pavement tests. The performance of UGM in field can be evaluated by the full accelerated pavement tests. The results from full scale tests can be correlated with the laboratory tests so that laboratory tests become sufficient to categorise the UGM used in basecourse layer.

2.9 References


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Florida DOT 204. (2010). Standard Specifications for Road and Bridge Construction, Division II - General Construction Operations Section 204 Graded Aggregate Base.


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Research Board (pp. 73-82). Washington, D.C.: Transportation Research Board of the National Academics.


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CHAPTER 3.

INVESTIGATING THE PERMANENT DEFORMATION OF UNBOUND GREYWACKE ROAD BASE CONSIDERING GEOLOGY, GRADATION AND MOISTURE CONDITIONS

Abstract

This research investigates the effect of geology and nature of fines (<75 μm) on the permanent deformation of unbound flexible pavement greywacke basecourse materials along with the change in gradation and moisture of the materials. The investigation is part of a New Zealand Transport Agency (NZTA) project which is being carried out at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) and investigates the water proofing of various surfaces along with the performance of different basecourse materials under wet conditions. The water can intrude into the unbound basecourse layer and in the worst situation can saturate the layers leading to premature pavement failures. An investigation is required to find the relative performance of different basecourse materials at optimum moisture and saturated conditions. The materials used in this research were constructed and tested in the New Zealand Accelerated Pavement Testing Facility located in Christchurch.
New Zealand (CAPTIF). The aggregate mineralogical composition and quality of fines were investigated by preparing thin sections of the selected coarse aggregates and performing X-Ray Diffraction (XRD) tests. The permanent deformation as obtained by Repeated Load Triaxial (RLT) testing was analysed to investigate the effect of grading and moisture on the relative performance of the selected unbound greywacke basecourse materials. The results of this study indicate that gradation of the materials and moisture conditions affect the permanent deformation of unbound granular materials if the geology of the aggregates and quality of fines are similar in all materials.

3.1 Introduction

The permanent deformation under repeat loading cycles is considered as one of the key performance parameters for unbound granular basecourse materials. This research investigates key factors responsible for the performance of unbound basecourse pavements being the geological nature of the materials and the environmental effects of water intrusion into the pavement layers. The research is part of a larger New Zealand Transport Agency (NZTA) project which is investigating various types of thin surfacing over varying quality of basecourse materials. Testing of the materials were completed in both the laboratory and at Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) at Christchurch New Zealand. In this NZTA project, water has been introduced in the indoor CAPTIF facility on the pavement surface to investigate the water proofing of chipseals as well as the performance of various basecourse materials under wet conditions.

Locally sourced greywacke sandstone aggregates in the Christchurch region were used as the chosen basecourse materials for the CAPTIF research project. In New Zealand approximately 75% of all aggregates produced are sourced from the Permian – Mesozoic greywacke sandstones that constitute approximately 25% of the exposed rocks of the New Zealand landmass. The aggregates used in this study have all been produced from gravel pits sited on the Quaternary alluvial braid plane deposits of the lower Waimakariri River, immediately west of Christchurch. A small amount of windblown fine silt (loess, clay size particles less than 75 µm) was added to one of the aggregates (Fine Graded (FG) in Table 1). The alluvial gravels have all been sourced from the weakly metamorphosed sandstones of the Permian - Triassic Rakaia sub-terrane of the Torlesse terrane that forms the foothills of the Southern Alps (Forsyth et al., 2008).
The presence of swelling clay minerals in the fines of aggregate source materials can result in premature pavement failures. Smectite, a swelling clay mineral which increases its volume when it comes in contact with water (Black, 2009) is a minor constituent of many types of New Zealand greywacke aggregate. If there are smectite clays present in the fines of these alluvial gravels, they can decrease the resistance of these basecourse materials to permanent deformation under repeated loading. The quantity of smectite family minerals should be known in the fines of the materials because elevated proportions of these minerals can convert a good basecourse into a moisture susceptible basecourse. Basecourses of flexible pavements constructed of moisture susceptible materials may undergo premature failures that manifest in defects such as rutting and alligator cracking (Chen, 2009).

The other factors such as stress states, grading and moisture affect the plastic deformation of the unbound basecourse materials. At high moisture content levels, the open graded aggregates show higher resistance to the applied stresses than the dense graded aggregates (Lachlan, 1996; Raad et al., 1992). The increase in moisture of basecourse materials in rainy seasons can cause excessive deformations in granular aggregates (Thom and Brown, 1987). The Repeated Load Triaxial (RLT) test has been used in a number of research studies to investigate different factors influencing the permanent deformation of unbound granular materials under repeated loading (Arnold et al., 2007; Thom and Brown, 1988; Werkmiester, 2003).

There is a need to test the quality of fines and mineralogical composition of coarse aggregates along with performance tests such as permanent deformation to properly understand the behaviour of the basecourse materials. The purpose of this research is to investigate the relationship of geology and percentage of swelling clays in the fines of the CAPTIF basecourse materials as related to the plastic deformation in Repeated Load Triaxial tests, and to explore the possible causes of the induced unbound basecourse failures. This article is further divided into four parts. In the first part, the materials used in this research along with their engineering properties are explained. The second part describes the methodology used to perform geological and performance testing of these materials. In the third part the results of the tests are discussed and finally the fourth part draws conclusion from the test results.

3.2 Materials

As part of the CAPTIF experiment, three different types of greywacke base course material were used under different surface conditions, resulting in different moisture regimes of the
base course layers of the tested pavement. The intent of this part of the research was to establish how sensitive these material types are to the varying moisture conditions in terms of their loading resistance. The three base course materials represent three different grading types that could potentially be used in road construction throughout the South Island of New Zealand. The three material types used for this experiment include:

- Coarse Graded (CG) – this material represents a commonly used basecourse material that complies with the premium New Zealand standard state highway specification (TNZ M4) for unbounded basecourses (Transit New Zealand M/4, 2006).
- Medium Graded (MG) – this material is a slightly denser material than the M4 specification but it was believed to still comply as a suitable basecourse material;
- Fine Graded (FG) – This material would be considered as a weak material which falls well outside of the M4 specification.

There were other differences in these materials predominantly related to the maximum particle size, percentage of fines and gradation. The materials CG and FG had a maximum aggregate size of 37.5 mm while the MG material had a maximum particle size of 20 mm. The extra fines (particle size less than 75 µm) were added to the FG material shown in Figure 1 to enable a comparison of the results for excessive fines in a material with the least percentage of fines. The differences in the grading curves of the three materials are shown in Figure 3.1. The particle size distribution is expressed by Talbots grading curve with a constant ‘n’ shown in Equation 3.1.
\[ p = 100 \left( \frac{d}{D} \right)^n \]  

Equation 3.1

Where:

- \( p = \) percent passing sieve size‘d’
- \( D = \) maximum particle size and
- \( ‘n’ = \) a coefficient that commonly has a range between 0.3 (fine grading) and 0.6 (coarse grading) (Arnold et al., 2007).

When selecting the materials for the CAPTIF experiment there was some deliberate shifting in the grading percentages as explained by Talbot’s grading value exponent ‘n’. This was undertaken to ensure that differences in the material grading would result in the materials performing differently in their drainage characteristics. The values of ‘n’ for CG, MG and FG materials were approximated as 0.5, 0.43 and 0.37 respectively. The maximum aggregate size of CG and FG materials is 37.5 mm whereas the maximum aggregate size in MG material is 19.0 mm.

**Table 3.1 Various Engineering Properties of the Basecourse Materials**

<table>
<thead>
<tr>
<th>Engineering Property</th>
<th>CG</th>
<th>MG</th>
<th>FG</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unified Classification System</td>
<td>GW</td>
<td>SW</td>
<td>GW</td>
</tr>
<tr>
<td>AASHTO Classification System</td>
<td>A-1-a</td>
<td>A-1-a</td>
<td>A-1-a</td>
</tr>
<tr>
<td>Cone Penetration Limit (Moisture in %age)</td>
<td>21</td>
<td>19</td>
<td>21</td>
</tr>
<tr>
<td>Sand Equivalent (% age of sand to clay)</td>
<td>36</td>
<td>24</td>
<td>17</td>
</tr>
<tr>
<td>Clay Index (volume in ml of methylene blue absorbed by 1 g of material)</td>
<td>1.4</td>
<td>2</td>
<td>2.2</td>
</tr>
<tr>
<td>Permeability (Average of Head Difference 2 kPa and 5 kPa, m/s)</td>
<td>0.00001</td>
<td>0.000002</td>
<td>0.000008</td>
</tr>
</tbody>
</table>

Notes:
- Cone Penetration Limit: Test 3.2, NZS 4407:1991
- Sand Equivalent: Test 3.6, NZS 4407:1991
- Clay Index: Test 3.5, NZS 4407:1991
- Permeability: Triaxial test with back pressure technique

The engineering properties of the materials used are given in Table 3.1 Various Engineering Properties of the Basecourse Materials. From the table it is observed that although the material classifications according to AASTHO are similar, there are some significant differences observed for the sand equivalence and clay index. From sand equivalent values,
the materials seem to consist of excessive portions of clays as the values for the materials are under 40% but from the clay index test, it looks like the fines of these materials may have little deleterious clay materials.

3.3 Research Methodology

There are two distinct testing phases used in the methodology of this research:

- Geological testing; and
- Repeated load triaxial (RLT) testing.

The geology testing includes the investigation of the coarse aggregates and the fine particles of the three materials. The repeated load triaxial tests investigate the effects of various stress states at varying moisture on permanent deformation of these materials. The results from the two portions are studied together to find the effects of geology and quality of fines on the performance of the unbound materials.

3.3.1 Geological Testing

Two types of tests were conducted in the laboratory to investigate the aggregate formation and the quality of fines present in the three materials used in the CAPTIF experiment i.e., thin sections and X-Ray Diffraction tests. Thin sections were used to identify minerals and their arrangement in the matrix of aggregates. The specific arrangement of mineral particles such as minerals in faults of the aggregate gives an idea about the performance of the aggregate when in contact with moisture. XRD tests were performed to identify and quantify the mineral weight percentages in aggregates and fines of unbound granular materials. The XRD tests are important as they indicate the presence of minerals like chlorites and illites which are slightly weathered and convert to expanding minerals e.g., smectite mineral when they are in continuous contact with moisture demonstrate significant expansion. Representative aggregates from the samples were selected for thin sections. The thin sections were prepared in the laboratory and are generally less than 30 microns in thickness. The prepared thin sections were investigated under an electron microscope (under normal and polarised light) for their geological makeup and the identification of various constituent minerals.

The fines from these samples were put in an aluminium folder and X-Ray Diffraction (XRD) tests were performed on multiple samples from each of the three materials. The test conditions for the XRD tests were:
Chapter 3 Investigating the Permanent Deformation of Unbound Greywacke Road Base considering Geology, Gradation and Moisture Conditions

- Copper anode X-ray tube running at 40 kV, 20 mA;
- Divergence slit: 1 degree;
- Receiving slit: 0.2 mm;
- Scatter slit: 1 degree;
- Scanning speed 2 or 3 degrees 2-theta per minute. Range 2 degree per min;
- Typical step size 0.02 degrees per step.

3.3.2 Repeated Load Triaxial Testing

Repeat load triaxial (RLT) tests were performed to examine the influence of the gradations, percentage of fines, nature of fines and geology of aggregates on the simulated permanent deformation. The materials were tested at various saturation levels listed in Table 3.2. The optimum moisture content was selected to find the base line performance of the materials and these values were compared with the deformations at saturated drained and saturated undrained conditions for the medium graded material to find their effect on permanent deformation.

<table>
<thead>
<tr>
<th>Materials</th>
<th>Specimen No.</th>
<th>Maximum Dry Density (MDD) t/m³</th>
<th>Moisture (%)</th>
<th>Measured Specimen density (95% of MDD)</th>
<th>Test Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>CG</td>
<td>Cf</td>
<td>2.34</td>
<td>4.3</td>
<td>2.223</td>
<td>Optimum Moisture Content (OMC)</td>
</tr>
<tr>
<td>MG</td>
<td>A</td>
<td>2.38</td>
<td>4.8</td>
<td>2.261</td>
<td>Saturated, undrained</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>2.38</td>
<td>4.8</td>
<td>2.261</td>
<td>Saturated, drain</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>2.36</td>
<td>4.8</td>
<td>2.242</td>
<td>OMC</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Materials</th>
<th>Specimen No.</th>
<th>Maximum Dry Density (MDD) t/m³</th>
<th>Moisture (%)</th>
<th>Measured Specimen density (95% of MDD)</th>
<th>Test Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>FG</td>
<td>C</td>
<td>2.36</td>
<td>4.8</td>
<td>2.242</td>
<td>OMC</td>
</tr>
</tbody>
</table>

Table 3.2 Properties and Conditions of Testing Samples

Table 3.3 Different Stress States Applied at the Samples in RLT Test

<table>
<thead>
<tr>
<th>Stress State</th>
<th>( \sigma_1 ) (kPa)</th>
<th>( \sigma_3 ) (kPa)</th>
<th>( p = (\sigma_1 + 2 \sigma_3)/3 ) (kPa)</th>
<th>( q = \sigma_1 - \sigma_3 ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>210</td>
<td>120</td>
<td>150</td>
<td>90</td>
</tr>
<tr>
<td>2</td>
<td>166.7</td>
<td>66.7</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>3</td>
<td>141.7</td>
<td>41.7</td>
<td>75</td>
<td>100</td>
</tr>
<tr>
<td>4</td>
<td>270</td>
<td>90</td>
<td>150</td>
<td>180</td>
</tr>
<tr>
<td>5</td>
<td>470</td>
<td>140</td>
<td>250</td>
<td>330</td>
</tr>
<tr>
<td>6</td>
<td>530</td>
<td>110</td>
<td>250</td>
<td>420</td>
</tr>
</tbody>
</table>
The RLT tests were performed as per the NZTA draft specification TNZ T/15 (2010). The size of samples was 150 mm in diameter and 300 mm in height. The back pressure technique was used to saturate the samples in the triaxial cell in accordance with BS 1377-6: 1990. All of the samples were compacted by a vibrating hammer at optimum moisture content to 95 percent of Maximum Dry Density (MDD). The densities of the samples at compaction and their relative moisture contents are shown in Table 3.2. All of the materials were tested at the stress states (based on different principle stresses) shown in Figure 3.2 and Table 3.3. The stress tensors ‘p’ and ‘q’ are the average and difference of vertical and confining stresses respectively. In the first three stress states, the values of ‘q’ and ‘p’ decrease while the ‘q/p’ ratio increases. In stress state ‘4’, ‘q/p’ ratio has decreased while values of ‘q’ and ‘p’ have increased. In stress state ‘5’ and ‘6’, the ‘q/p’ ratio has increased again as with the value of ‘q’ but the value of ‘p’ remained the same. An imaginary static failure line is drawn based on static failure tests undertaken on similar materials used in previous CAPTIF projects (Arnold, 2004). The distance of stress state points in Figure 3.2 decreases rapidly from the static failure line. The test procedures applied 50,000 load cycles at each stress condition to each of the three materials. The same samples were used in successive stress states. The loads on stress states were applied successively on the same sample. The values of deformation were omitted for the first 100 repeated load applications.

![Figure 3.2 Stress tensors (q and p) at different stress states.](image-url)
3.4 Results & Discussion

3.4.1 Geological Nature

Thin-sections were made of representative chips taken from coarse aggregate portions for each of the samples representing the three material types used in this study. The coarse aggregate chips all conformed to the available descriptions of Rakaia sub-terrane sediments in that they are monotonous quartzofeldspathic sandstones that would be classified as arkosic arenites under the Pettijohn sandstone classification scheme (Folk et al., 1970; Pettijohn et al., 1987). The sandstones contain lithic fragments of felsic volcanics and rare basic volcanics. Detrital mica grains are relatively common. The sandstones have been metamorphosed to prehnite-pumpellyite facies. Thin veins of quartz and less commonly prehnite and calcite are common. Microphotos (plane polarised light) illustrating common features of the source rocks are shown in Figure 3.3 (a) and (b); scale bars are 1000μm.

![Figure 3.3 (a & b) Representative thin sections from aggregate samples of tested materials.](image-url)
Each of the rock chips that had been thin sectioned and representative splits of the fine sand fractions of the aggregates were subject to X-ray diffraction analysis. Diffractograms of each of the rock chip and fines samples were obtained from bulk random samples packed into aluminium holders and the minerals present were indentified using an automised search/match programme and powder diffraction data files. The weight percentages of the minerals present in the samples were also calculated using SIROQUANT XRD software (Sietronics (pty) Ltd). In the case of the fines up to three duplicates were run for each sample. The clay size fraction of each of the fines samples were also sedimanted onto a glass slide to enhance the basal spacings of any sheet silicates and clay minerals present and the oriented samples were also glycolated to determine if any swelling clays were present.

The major minerals in all the sandstone chips and the fine sand fraction of the three aggregates were quartz and feldspar (Albite) shown in Figure 3.4. Other minerals detected in the samples were chlorite, illite, mica (biotite and muscovite), a mixed layer illite – chlorite, kaolinite and pumpellyite; these in total constituted less than 10 percent of the sample by weight. Smectites were not found in detectable amounts in any of the bulk diffractogrammes. Figure 3.4 shows that the variations in percentages for each material are within 4% of weight percent.

![Figure 3.4 Weight percentages of minerals in fine contents of aggregate mixes.](image)
A statistical analysis of the above data was performed to investigate whether there is significant difference in the materials and their weight percentages of constituent minerals. A non-parametric analysis was done for the purpose as the data was not normally distributed. The Kruskal Wallis test was performed as this method is used to find the non-parameteric variances between more than two groups (three materials are included in this research). The p-value (Table 3.4) is greater than 0.05 shows that all the materials have approximately similar mineral percentages.

Table 3.4 Kruskal Wallis test to compare UGM materials

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Values for Comparison of CG, MG and FG</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chi-Squared Value</td>
<td>0.05</td>
</tr>
<tr>
<td>Degrees of Freedom</td>
<td>2</td>
</tr>
<tr>
<td>p-value</td>
<td>0.97</td>
</tr>
</tbody>
</table>

Results indicate that the three aggregate materials were almost identical in terms of the geological and mineralogical compositions and that the observed differences between the three samples could not be expected to affect the performance of the aggregate. Further, the fines present in these aggregates are non-expanding and non-plastic. The high quartz content in clasts and matrix cement of the aggregate source rock sandstones give the material high strength (Crushing Resistance to produce 10% fines were found at loads greater than 400 kN); CI and PI values are consistently low and thus are not routinely measured.

### 3.4.2 Factors Affecting Permanent Deformation in RLT Test

#### 3.4.2.1 Stress States

The stress state of the unbound granular material sample under repeated loading determines the extent of permanent deformation in the sample. If the permanent deformation of the three materials at optimum moisture content is compared as shown in Figure 3.5 (a, b and c), all the materials are showing maximum deformation at state ‘5’ and ‘6’ which indicates that high stresses in all three unbound materials may cause more damage. The results are consistent with previous studies by Lekarp and Dawson (1998) and Gidel et al., (2001) that found an increase in stress state can cause excessive permanent deformation in unbound materials.

One of the factors affecting the trends in low stress state (1-4) is the ‘q/p’ ratio and the distance of stress state from the static failure line. In CG and MG materials, the order of
stress states in terms of increasing permanent deformation observed was ‘3’, ‘4’, ‘2’, and ‘1’. The permanent deformation (PD) for CG and MG materials is decreasing as the ‘q/p’ ratio is increasing but PD increases as the ‘q/p’ ratio drops a little in state ‘4’.

The grading and percentage of fines determine how the stress states affect the permanent deformation. The trend of deformation from least to maximum stress state in FG material is ‘1’, ‘2’, ‘4’ and ‘3’. As the ‘q/p’ ratio increases in FG material from state ‘1-3’ and its distance to static failure line decreases, the deformation increases but as the distance from static failure line increases in stress state ‘4’, the deformation decreases. The difference of PD at the first four stress states in the three materials at Optimum Moisture Content (OMC) is due to the difference of grading and percentage of fines.

The increased stresses are causing more permanent deformation even when the ‘q/p’ ratio does not change significantly but the distance from failure line is decreasing. The ‘q/p’ ratio at state ‘3’ and state ‘5’ are approximately the same but the deformation at state ‘5’ is significantly larger than at the state ‘3’ in all the materials. This may also be due to the increase in values of both stress tensors ‘p’ and ‘q’ at state ‘5’ which is significantly larger than at state ‘3’ and the value of stresses are closer to the static failure line.

3.4.2.2 Grading

The gradation and percentage of fines in the unbound basecourse materials significantly affect the permanent deformation (Arnold et al., 2007; Molenaar and Van Niekerk, 2002). The PD of samples which were prepared and tested at Optimum Moisture Content (OMC), is shown in Figure 3.5 (a, b and c) for all three materials CG, MG and FG respectively. The FG material showed the least resistance against vertical deformation at OMC compared to CG and MG materials, whilst the CG material showed the least permanent deformation in comparison to the three materials, for each stress state in the RLT test. The MG material demonstrated PD values in between CG and FG materials deformations. The possible reasons for the least deformation of CG and most PD for FG are varying cohesion and angle of internal friction due to change in the Talbot’s gradation constant for the CG materials as ‘n’ is greater in CG compared to MG and FG materials.

The percentage of fines affects the behaviour of materials at different stress states. If stress states ‘3’ and ‘1’ are compared, the CG and MG materials have less deformation at state ‘3’ while in the FG material the deformation is less at state ‘1’. The confining stress supports the material against deviator stress which depends upon the percentage of fines present in that
aggregate mix, provided that the fines present in the material are non-plastic. As there is a
greater percentage of fines in FG material, the decrease in confining stress may be the factor
which is increasing the PD at stress state ‘3’ compared to state ‘1’.

Figure 3.5 shows that under various stress conditions especially at stage 5 and 6, at OMC, the
CG material performed the best, resulting in the least deformation than the other two
materials, MG and FG. The possible cause for the better performance of CG is its grading and
smaller percentage of fines (particles less than 75 μm). FG and MG materials have a finer
gradation as compared to CG material. The added fines fill in between aggregate particles
and reduce the contact stresses between aggregate particles as fine particles easily move and
rotate. This effect is more visible at higher stress states as the unbound material resist as their
particle to particle contact stresses and contact stresses reduction is more visible at higher
stresses which results in increased permanent deformation.

3.4.2.3 Moisture

An increase in moisture of the unbound basecourse material decreases the resistance to the
permanent deformation (Dawson et al., 1996; Vuong, 1992). If the PD of MG material is
compared at the end of the 50,000 cycles at moisture conditions tested (shown in Figure 3.6),
the final value of PD at all states was higher for saturated un-drained (SUD) samples than the
saturated drained (SD) and OMC samples. This comparison shows that if the water is
entrapped in the voids of the material (as in saturated conditions), the unbound basecourse
materials will fail much earlier than if they were at OMC.

The deformation of MG material at various stress states was not uniform at the moisture
conditions tested. The PD values of OMC samples are less than SD samples at state ‘1’, ‘4’,
‘5’ and ‘6’ but for state ‘2’ and ‘3’, the PD values of OMC samples are slightly greater than
SD samples. The results show that both the addition of water and low permeability of these
materials have increased the resistance to deformation at lower stress levels. The reason for
this behaviour is that at low stress conditions, the water is taking stresses in the saturated
sample while not reaching a failure state, but at the optimum moisture content sample, the air
voids are being compacted under repeated loading causing more deformation at that state.

In summary, the results showed that the MG samples showed the least deformation at
optimum moisture content in comparison to the saturated conditions. This shows that the
basecourse materials will perform much better in terms of permanent deformation at dry
conditions than at wet conditions.
Figure 3.5 Permanent deformation at six stress states at optimum moisture content of three materials: a) CG, b) MG and c) FG.
Figure 3.6 Permanent deformation at Six Stress States; MG material at different moisture conditions: a) SUD, b) SD c) OMC.
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3.5 Summary and Conclusions

This research used both geological and engineering test methods to characterise three similar sourced greywacke aggregate materials with quite different grading and therefore expected variance in performance under repeated loading cycles. It was found that it is very important to comprehend the material formation (geologic) properties of the aggregates, the minerals present in the aggregate as well as the minerals in the fine material of the whole pavement layer matrix, to better understand the engineering performance of an unbound granular aggregate mix. The thin sections of the aggregates used in this research were studied along with XRD tests showing the percentage of various clay minerals in the fine aggregate contents. Furthermore, the RLT tests have shown the performance of the materials at different stress states and moisture conditions. This combined geological and engineering data and analysis allows the researcher confidence in understanding the causality or material failure mechanism, thereby allowing the possibility of a better prediction of engineering performance of the materials. On the basis of the results, the following conclusions can be drawn:

- The geology of aggregates showed that the quartz and albite formed the major structural matrix of the aggregates in the tested materials making the aggregates reasonably sound and durable. As the coarse aggregates of all three materials were geologically sound it can be concluded that the PD and therefore the durability of the aggregates is not affected by the coarse aggregate fractions tested at various moisture contents.

- The minerals found in the fines of the aggregates did not demonstrate any significant proportions of expanding clay minerals (e.g., smectite clays) suggesting that the fines of these materials were non-swelling in nature. Therefore the permanent deformation will not be affected by the quality of fines in the tested materials. This is also shown in the difference in the results of the engineering test methods of the quality of the finer aggregate fractions. The sand equivalent (SE) generally failed the specification and was quite variable in performance between the three samples, however the clay index (CI) test which is a better method to determine the quality of the fine fraction all passed typical engineering aggregate specification criteria.

- The permanent deformation of the materials as determined from RLT tests varied with the change in the gradation of the materials i.e., the materials performed best at
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Talbot’s grading value constant ‘n’ equal to ‘0.5’ and worst at ‘0.37’ while the material with ‘n’ value ‘0.43’ showed permanent deformations in between the two.

- An increase in the percentage of fines in a material increases the amount of permanent deformation as the FG material that had additional fines resulted in the highest levels of deformation.
- The addition of moisture in basecourse material causes the variance in permanent deformation at different stress states especially at optimum and higher moisture contents.

3.6 References


Chapter 3 Investigating the Permanent Deformation of Unbound Greywacke Road Base considering Geology, Gradation and Moisture Conditions


Chapter 3 Investigating the Permanent Deformation of Unbound Greywacke Road Base considering Geology, Gradation and Moisture Conditions

CHAPTER 4.

THE EFFECT OF MOISTURE AND RELATIVE PROPORTIONS OF CLAY MINERALS (SMECTITE, CHLORITE, AND ILLITE) ON THE PERFORMANCE OF UNBOUND GRANULAR BASECOURSE

Abstract

An increase in moisture accelerates the deterioration of pavement and this is especially so for unbound granular materials below pavement surfacings. One of the possible reasons for this accelerated deterioration can be the mineralogical makeup of the aggregates that can include clays. This chapter discusses the effect of the relative proportions of clay minerals on the performance of different basecourse materials in repeated load triaxial (RLT) tests considering moisture variation. The materials selected for this research were sourced from greywacke sedimentary rocks from both the North and South Islands of New Zealand (NZ). The mineralogical makeup of the clay content present in the basecourse materials was determined using the X-ray powder diffraction (XRD) method as it indicates the presence of
moisture susceptible and weathered minerals. The performance of these materials were further tested in RLT tests at different moisture and drainage conditions. The results of the XRD and RLT tests showed that aggregates with high relative proportions of smectite clay minerals can decrease the load bearing capacity of the basecourse material when moisture is introduced into the pavement materials.

4.1 Introduction

The unbound basecourse material has an important load bearing function in flexible pavements. In high traffic volume roads, bound bituminous or concrete layers are laid over this unbound basecourse to reduce the stresses at the top of the basecourse layer. However, in chip sealed or very thin bituminous asphalt wearing courses over unbound granular layers, the function of the wearing course is not primarily as a structural layer but more for waterproofing and mainly to provide adequate skid resistance, drainage, maintenance and dust reduction. The unbound aggregates must therefore transfer the dynamic load stresses of traffic from the top layer to the subgrade layer such that the stresses are reduced to a level where the top of the subgrade can withstand the distributed loads and thus prevent fatigue failures. These unbound materials when appropriately selected and designed demonstrate good resistance against permanent deformation when they are drier than the optimum moisture content. However, the resistance to permanent deformation can decrease considerably with an increase in moisture above optimum moisture contents.

There can be many reasons for the decrease in the strength of these unbound basecourse materials with an increase in moisture such as:

- the expanding clays in the matrix of the compacted basecourse can absorb water and become plastic which can cause excessive deformation,
- the lubrication of the aggregates thus decreasing the friction at the aggregate particle to particle contact area, and
- pumping of the finer fraction (fines) from the aggregate that creates cavities within the basecourse material, decreases the density and increasing interstitial stresses.

In this chapter only the expanding clays are addressed. The expanding clay is purely the material property which can be linked to a specific quarry, or can be linked to weathering of the unbound material in which the existing clay and other unstable minerals present in the clay size fraction are converted into expanding clays. The relative proportion of sand to clay
The effect of moisture and relative proportions of clay minerals (smectite, chlorite, and illite) on the performance of unbound granular basecourse

in the finer aggregate fractions and the presence of expanding clays are currently tested in New Zealand (NZ) by performing the Sand Equivalent test (SE) and/or the Clay Index (CI) test respectively (Standards NZ, 1991). These current NZ test methods have been shown to be inadequate in characterising the effect of the clay on the performance of the aggregate as they do not provide detailed insight into the mineralogy of the clays (Lowe et al., 2010). Lowe recommended in his research that the current NZ tests (SE and CI) that were based upon US standards needed to be revised to better account of the clay mineralogy found in NZ and to take account of both the quality and quantity of the clay fines in the aggregate. Knowledge of minerals present in the clay size fractions can help to identify the nature of the problems with the basecourse material. Some clays when immersed in or saturated with water, expand by absorbing water in the interlayer position and this expansion can reduce the dynamic load bearing capacity of the material when it is subjected to highly moist conditions for long periods.

The strength of the aggregate and moisture susceptibility of the clays contained in the aggregates that are used as basecourse materials are also affected by the amount of geological weathering of the source rock (Hodder and Hetherington, 1991; Velde and Meunier, 2008) as well as the aggregates’ susceptibility to further weathering when in soaked condition while in service. The common clay minerals found in the fine particles of NZ greywacke basecourse materials are chlorite, smectite, illite, and kaolin. This research was mostly concerned with the swelling properties of these minerals which result in a weakening of the basecourse material thereby reducing the pavement life under loading. The smectite group minerals are highly swelling and can exist either as a separate mineral or as mixed interlayer mineral, usually with chlorite or illite (Murray, 2006). There are many species of smectites for example sodium montmorillonite, calcium montmorillonite, saponite (Mg-rich), nontronite (Fe$^{3+}$-rich), and beidellite (Al-rich). Sodium montmorillonite has a 10-15 times swelling potential for swelling capacity when placed in water. Calcium montmorillonite has a swelling index of 2-3 which is very much less than the sodium form with a swelling index of 24-36 however it can still be sufficient to cause an increase in volume of the basecourse material when subjected to moisture that will accelerate pavement failure. In contrast, kaolin, illite, and chlorite have very low absorption and swelling properties (Murray, 2006).

This research tested unbound basecourse materials in two phases, the first, to identify the mineral composition of the material at phase ‘1’ and secondly to compare the performance at
Chapter 4 The effect of moisture and relative proportions of clay minerals (smectite, chlorite, and illite) on the performance of unbound granular basecourse

Phase ‘2’. X-Ray Powder Diffraction (XRD) data was collected from oriented samples of the clay fraction of samples in their untreated and then glycolated state to identify expanding minerals, such as smectites, in the clay size fractions of the unbound granular basecourse (Wilson, 1987). After performing XRD tests on the materials (5 greywacke aggregates – three from the South Island that were used in coincident APT tests and 2 North Island greywackes), repeated load triaxial (RLT) tests (TNZ T/15, 2010) were performed to compare their relative engineering performance. From the XRD tests, the relative percentage area under the curve of smectite mineral with the chlorite, illite and kaolin minerals were calculated and investigated to determine whether this relative proportion adversely affects the engineering performance of the unbound basecourse material.

The South Island aggregate materials used in this research are taken from the test pavements of the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) and from a known road pavement failure from the North Island of New Zealand. In the following sections, the research objectives and scope will be discussed along with the methods and materials, methodology of assessment results and discussions, conclusions and recommendations.

4.2 Objectives and Scope

The main objective of this chapter was to investigate if there was a difference in the mineralogy of the materials from the South Island aggregates that were used in the CAPTIF experiments and materials from a road failure that occurred in the North Island of NZ. If a difference can be shown then the effect of different clay mineral proportions on the performance of unbound granular materials in RLT tests will be assessed. A particular aim will be to verify whether the smectites, which are more prone to expansion, if found in a high percentages could cause a difference in the load carrying capacity of unbound basecourse.

4.3 Materials and Methods

The aggregates used in the basecourse materials taken from the CAPTIF pavement and the road failure was all sourced from greywacke. The CAPTIF materials were sourced from the South Island of NZ while the well-known road failure material was sourced from North Island NZ greywacke source rocks. The major geological difference between these materials was found to be the the proportion of the various minerals. In the South Island materials, a
significant proportion (intensity) was found of chlorite and illite minerals whilst for the North Island materials, the smectite and kaolin minerals were found in higher ratios.

Initially, the unbound basecourse materials used were obtained from the test pavements constructed in the CAPTIF test pavement, Christchurch. The materials had three different particle size distributions categorised as coarse graded (CG), medium graded (MG) and fine graded (FG) and were used as the basecourse in the CAPTIF test pavements. These three materials were subjected to different laboratory tests to find out if there were any significant mineralogical differences in the materials both in aggregates and in the clay contents of the mix. The test results showed that the mineralogical composition of the materials used in the CAPTIF pavements were largely the same, hence the difference in the performance of CAPTIF materials could not be explained by their mineralogical composition. To investigate whether the mineralogical composition of the materials can affect the performance of the basecourse, the materials from a well-known but politically sensitive road material failure in the North Island (the location and material sources could therefore not be identified), were collected and tested for their mineralogical composition. In this material there was a significantly high proportion of smectite mineral in the materials obtained from the pavement failure in comparison to the CAPTIF pavement materials.

The particle size gradation of the materials used in the CAPTIF and sampled from the North Island pavement failure is shown in Figure 4.1a) and b) respectively. The gradation curves show that the materials used in CAPTIF test pavements have different gradation curves. However, all the sample materials obtained from the road failure are in the same range of particle distribution and met NZTA M/4 specification psd envelopes (Transit New Zealand M/4, 2006). A comparison can be readily made between the mineralogical compositions of all of the aggregate materials. However, as the psd distributions for the CAPTIF test pavement are quite varied only the closest psd sample from the CAPTIF test pavement will be compared to the North Island road pavement failure samples (i.e., the CG coarse grained sample). RLT results that were taken as a post construction material failure test were obtained for the 11C material road failure material. The 11C material test results were labelled as 3010 when the RLT tests were conducted in dry and drained conditions whilst labelling of 3011 was used when the RLT tests conditions were saturated and un-drained.

The tests methods usually specified for testing the quality of the fine fraction of the basecourse materials are the clay index (CI) test, and the sand equivalent (SE) test (Standards
Chapter 4 The effect of moisture and relative proportions of clay minerals (smectite, chlorite, and illite) on the performance of unbound granular basecourse

NZ, 1991). The results of both of these tests are shown in Table 4.1 for the basecourse materials tested and analysed in this research. The specification for M/4 basecourse is that the material should have value of CI < 3, and value of SE >40. In the specifications materials have to satisfy either CI or SE test. As CAPTIF materials (CG, MG, FG) are virgin materials, they are selected on the basis of CI. However, in pavement failure materials, only 4B material passes the M/4 requirements. The test results show that there is no obvious correlation between Clay Index value and Sand Equivalent value. For example the sand equivalent value shows that CG has a higher clay-size content w.r.t. 11C although the clay index of CG 1 is less than for the 11C material.

Figure 4.1 Gradation curves of basecourse materials (a) CAPTIF; (b) Road failure

<table>
<thead>
<tr>
<th>Table 4.1 Test Results of Basecourse Materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
</tr>
<tr>
<td>----------</td>
</tr>
<tr>
<td>South Island from CAPTIF Pavements</td>
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<td></td>
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<tr>
<td></td>
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<tr>
<td>North Island from a Road Failure</td>
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</tbody>
</table>

*Values for the material near pits as exact values are not found in data. Light shaded cells show pass value, medium shaded cells show near failure, and dark shaded cells show failure values.
Chapter 4 The effect of moisture and relative proportions of clay minerals (smectite, chlorite, and illite) on the performance of unbound granular basecourse

The methodology and analysis followed in this research was divided into two stages. In the first stage, the mineral composition of the source rocks and clays present in the fines of the basecourse material are identified using petrographic thin-section studies and the standard XRD test for identifying clay minerals (Wilson, 1987). The clay size fraction is separated using the standard density separation method and deposited on a silica glass slide and allowed to sediment and air dry to provide an oriented sample mount. The oriented sample is then subject to X-Ray diffraction analysis in three stages:

- the sample is first examined in the un-treated state during stage one;
- the same sample is exposed to ethylene glycol vapour at 40 degrees centigrade (ie glycolated) for 24 hours at stage two; and
- the glycolated sample was then re-heated to 550°C for at least one hour at stage three.

The untreated XRD sample results show the current clay mineral state present in the material prior to construction or loading. The glycolated sample results show the these minerals might behave when subjected to long contact of basecourse materials with moisture.

The RLT tests were proposed at the second stage of the research, the setup is shown in Figure 4.2. The different materials were tested in accordance with the draft NZ standard T/15. The materials were subjected to different stress states in the staged RLT test and for 50,000 loading cycles applied to the sample for each of six stress states (each with an increasing deviator load). The materials were tested at Optimum moisture drained conditions and Saturated un-drained conditions. The aggregate clay minerals come in contact with water in the RLT specimen and can show the effect of the quality and proportion of the fine clay component when tests are conducted in saturated un-drained conditions under cyclical fatigue loading conditions.
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4.4 Results and Discussion

4.4.1 Mineralogy of the source rocks

The source for the South Island aggregates used in the CAPTIF programme is quartzofeldspathic (i.e., arkosic) greywackes in which micas dominate as both a detrital and metamorphic mineral. In contrast the source quarries for the greywacke aggregate from the North Island are volcaniclastic sandstones in which chlorite is the dominant clay mineral. The difference in the mineralogy of the clay fraction can be seen by comparison of the relative intensities of the 14Å and 7Å peaks at c.6 and c. 12.5 degree 2 theta (the first and second order basal spacings of chlorite respectively) with the 10Å peak at c.9 degree 2 theta peak (first order basal spacing for illite/mica in Figure 4.3a with Figure 4.4a.

All the greywacke source rocks contain prehnite, pumpellyite and occasionally epidote minerals. The arkosic greywackes contain abundant metamorphic albite + white mica and occasionally stilpnomelane and/or K-feldspar, while the volcaniclastic North Island aggregate rocks contain abundant albite + chlorite as well as illite and occasionally actinolite. In terms of metamorphism the North and South Island greywackes have been metamorphosed under very-low metamorphic, sub –greenschist conditions. Estimates of the conditions of metamorphism for rocks containing the observed assemblages would be 250-300°C and 2-4 Kbar (Frey et al., 1991).
4.4.2 X-ray diffraction study of clay size fraction of samples

The results of XRD analysis for minerals present in the clay size fraction of the basecourse materials are shown in Figure 4.3 and Figure 4.4. The first peak in the diffractogram at circa 6 degrees 2 theta corresponds to the basal spacings (ie the 001 plane) of both chlorite and air-dried smectite minerals (at c. 14-15 Å). The second peak at approximately 9 degrees 2 theta is the illite / mica basal spacing (c. 10Å) while the third peak at approximately 12.5 degrees 2 theta (c. 7Å) coincides with the 002 lattice plane of chlorite and the first basal spacing of the kaolin group minerals.

One of the South Island aggregates used in the CAPTIF programme (FG) had weathered material added to it to provide contrast to the otherwise good quality aggregates used. The effect of weathering on the aggregate can be seen in the high background on the low angle side of the illite and chlorite peaks and the shoulder on the low angle side of the chlorite peak (ie hydration swelling of the basal spacing to higher d-spacings) of the untreated sample, and the general lack of sharp expansive peak in the glycolated sample. These are common features of weathered chlorites and illites where interlayer minerals, and also the expanding mineral known as vermiculite formed by leaching of potassium from illite, are common (Moore and Reynolds, 1997).

North Island volcaniclastic greywackes that are altered or weathered contain kaolin and smectite (Bartley et al., 2007). It is very difficult to detect kaolin in samples containing chlorite since the first order kaolin basal peak overlaps with the second order chlorite basal peak. One of the characteristic features of iron-rich chlorite is the very notable increase in the intensity of the 14Å peak and corresponding decrease in intensity of the 7Å peak on heat treatment. This feature is very notable in the diffractograms shown in Figure 4.3c and Figure 4.4c. There are small but significant differences in the position of the 002 and 004 peaks for iron-rich chlorite (at c. 12 and 25 degrees two theta respectively) and the 001 and 002 peaks for kaolin, which allow discrimination of the two minerals (Moore and Reynolds, 1997) but also contribute to broadening of the observed peaks (as seen particularly in Figure 4.3 a and b). Step heating from temperatures at 50 degree intervals from 400 to 550 degrees also show the presence of small amounts of kaolinite in the North Island samples.

Figure 4.3 and Figure 4.4 show a clear difference in clay mineralogical composition between the CAPTIF and the North Island failure samples. In the CAPTIF samples the 14Å basal spacing develops a slight shoulder on the low angle side of the peak on glycolation (Figure
4.3b) but is unaffected by heat treatment (Figure 4.3c). The 10Å illite peak shows little change in intensity although sharpens with heat treatment. The exception is the sample FG which on heat treatment develops a clear shoulder on the low-angle side of the illite peak (between 7 and 9 degrees two theta) indicating the presence of interlayered clay minerals. This shows the presence of weathered minerals that can absorb moisture in the long run and cause material to decrease its resistance to loading, earlier than expected. The North Island diffractograms show clearly that the untreated sample peaks at 14Å (c. 6 degrees two theta) shown in Figure 1.4 a are the combination of a smectite which expands on glycolation to c.16Å (Figure 1.4 b) and collapses to 10Å (c. 9 degrees two theta) on heating (figure 1.4c) and a non-expanding 14Å chlorite peak (Figure 4.4 a and b).

Figure 4.3 CAPTIF materials XRD test clay fraction oriented (a) Untreated, (b) Glycolated, (c) Heated at 550 °C
Chapter 4 The effect of moisture and relative proportions of clay minerals (smectite, chlorite, and illite) on the performance of unbound granular basecourse

4.4.3 Percentage of Relative Area under Curve for XRD Intensity Graphs

As the smectite minerals are known to absorb water and reversibly expand when they come into contact with moisture, in this section the focus is on the relative percentage of smectite with respect to chlorite and illite. The area under the curve for each mineral has been calculated and then the relative percentage of the area under the curve of the respective minerals is shown in Figure 4.5 and Figure 4.6.

Figure 4.5 shows that for all CAPTIF test materials, the relative percentage of the smectite mineral is approximately 20% for all XRD test conditions. The dominating minerals i.e., chlorite and illite mineral are found in approximately equal proportions. In these, the chlorite of first and second basal spaces are added. In the untreated (Figure 4.5 a) and glycolated (Figure 4.5 b) diffractograms area under the curve for the smectite basal peak is notably less than that for the chlorite and illite minerals.
Chapter 4 The effect of moisture and relative proportions of clay minerals (smectite, chlorite, and illite) on the performance of unbound granular basecourse

Figure 4.5 CAPTIF materials XRD test percentage of area under curve (a) Untreated, (b) Glycolated

The Figure 4.6 shows the area under the curve for the basal spacings of the different minerals in the road failure materials. From the results (Figure 4.6) it is clear that the percentage intensity of smectite mineral is less than that of chlorite and higher than that for illite in samples 2A and 4B but is at the highest level in samples 8C and 11C.

4.4.4 Permanent Deformation Results from RLT Tests

The RLT test results for the CG materials from the CAPTIF tests is shown (Figure 4.7) for the purpose of comparison with the RLT test results of the sample material 11C from the North Island road failure specimens (Figure 4.8). The mineralogy of the CG material has shown that it has lesser proportions of smectite mineral in comparison to the chlorite, illite, and kaolin minerals. The RLT test results (Figure 4.7) show that the material performed well at optimum moisture content in drained conditions resulting in a 0.3% permanent deformation. The highest intensity ratio of the smectite was found when comparing the remaining minerals that included the smectite that is known as a problematic swelling mineral. The 11C material showed a permanent deformation of 0.4% and is shown as 3010 material respectively in Figure 4.8 b. The performance of the CG material is better than the 11C material in terms of resisting the cyclic loads.

In saturated drained conditions, the CG material resisted the dynamic loading till the sixth stress state and resisted failure up till more than 260,000 loading cycles (Figure 4.7). In comparison the 11C material which is shown as 3011 in Figure 4.8 a, failed after only 100,000 loading cycles. In the saturated drained condition, the CG material has clearly performed better than the 11C material. The apparent reason for the good performance of the CG material in comparison to the 11C material is the relative lower percentage of smectite to the other considered minerals. The other possible reasons for the difference in performance
Chapter 4 The effect of moisture and relative proportions of clay minerals (smectite, chlorite, and illite) on the performance of unbound granular basecourse

are the gradation and maximum dry density which were 2.34 and 2.28 t/m³ for CG and the 1IC materials respectively.

![Figure 4.6 Pavement failure materials XRD test percentage of area under curve (a) Untreated, (b) Glycolated](image)

**4.5 Summary and Conclusions**

This research was conducted to investigate the type and quantity of minerals present in the unbound basecourse materials and the effect of the relative proportion of these minerals on the RLT performance of various materials. XRD tests were also conducted on the greywacke basecourse aggregates from the CAPTIF accelerated pavement test pavement sections in the South Island and a well-known road pavement failure in the North Island. The relative proportions of the minerals were calculated by using the area under the curve technique from the results of untreated oriented clay samples, glycolated samples, and heated samples at 550°C. A further two greywacke materials were subjected to RLT tests and a comparative analysis undertaken from the North and South Islands of NZ. The following conclusions can be drawn from the test results and analysis:

- The XRD results of materials from the CAPTIF test pavement and the road pavement failure showed the presence of similar minerals in the greywacke rocks of the North and South islands (e.g. chlorite, illite, smectite) but with varying proportions.
- The smectite mineral was seen to expand when in contact with moisture by absorbing water and contracting when heated. Smectite was found in double the proportions in the North island aggregates taken from a road failure relative to the South island aggregates which were taken from CAPTIF test pavements. This higher proportion of smectite in the materials that failed is one of the causes of the rapid deterioration in stiffenes found in the RLT tests.
The RLT test results demonstrate that the road failure materials from the North Island greywacke showed relatively higher deformations than the South island materials when tested in saturated un-drained conditions due to the presence of the smectite mineral in relatively higher proportions in North island aggregates in comparison to South island aggregates.

Figure 4.7 Permanent deformation of CG M/4 AP40 basecourse material used in CAPTIF
Chapter 4 The effect of moisture and relative proportions of clay minerals (smectite, chlorite, and illite) on the performance of unbound granular basecourse

Figure 4.8 Permanent deformation of materials taken from a pavement failure (a) saturated undrained RLT (b) dry drained RLT

Figure 4.8 Permanent deformation of materials taken from a pavement failure (a) saturated undrained RLT (b) dry drained RLT
The calculated smectite relative percentage area under the curve for CG was 20% and for the 11C material was 40%. The smectite present was double the percentage of the 11C sample relative to the CG material. High proportions of smectite mineral can cause a significant reduction in the performance of the same material in high moisture environments as the North Island material failed after only 100,000 loading cycles in saturated un-drained condition while it survived more than 300,000 loading cycles at optimum moisture content.

The conclusions in this research are based on a programme of limited experimental work due to limited resources; however the results evidently do show that the presence of smectite minerals in higher relative proportions can lead to the earlier failure of a basecourse material in high moisture environments. There is further need of research on the comparison of the performance of the aggregates with different relative mineral proportions as it needs extensive experimentation to determine the point where a change in performance behaviour is based on the relative proportion of smectite mineral. The XRD results clearly show the relative proportions of smectite mineral to the other minerals present in a basecourse and can help the construction industry in selecting appropriate basecourse material.

4.6 References


Chapter 4 The effect of moisture and relative proportions of clay minerals (smectite, chlorite, and illite) on the performance of unbound granular basecourse


CHAPTER 5.

MODELING THE PERMANENT DEFORMATION OF UNBOUND GRANULAR MATERIALS USING REPEATED LOAD TRIAXIAL TEST

Abstract

The use of unbound granular materials (UGM) in road pavements is very common in New Zealand (NZ) and sparsely populated countries as they are economical and provide a good load bearing foundation for the wearing course. The load bearing capacity of UGM materials defines the performance of these types of pavements, which are normally used on the lower traffic volumed roads spectrum. The most common performance test for unbound granular materials is the repeated load triaxial (RLT) test that simulates the dynamic loading of traffic on UGM in actual pavements. Given that this test can be a good indicator of the actual performance of these materials under simulated cyclic loading, forecasting models have been developed using RLT data. In this chapter, various statistical models based on single input criteria (number of loading cycles, or stress state) as well as combined effects of loading and stress criteria are assessed for the RLT test data obtained in this research. A comparison of how existing models fit the empirical data is undertaken. Furthermore, an alternative model is proposed that takes into account both the number of loading cycles and stress conditions in
the sample. This new model shows promising results, especially since it is able to predict aggregate performance for different moisture and drainage conditions.

5.1 Introduction

5.1.1 Background

Road engineers since Roman and even earlier periods have realised the importance of providing sufficient drainage for road pavements (Babi et al., 2000; Cedergren, 1988). This is particularly important for roads constructed using unbound granular base courses such as crushed rock. These roads are often associated with shear failures in wet conditions, thus emphasising the importance of using appropriate aggregates that are not susceptible to change in high moisture conditions especially for the upper pavement layers and using water tight surfacing such as an asphalt surface or thin chip or sprayed seal surfaces.

In a recent New Zealand Transport Agency (NZTA) research project undertaken at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF), two main aspects were investigated. Firstly, experiments were undertaken to establish how much water seeps through the surface layers that were constructed according to different sealing techniques (Hussain et al., 2011d). The second aspect was to investigate the performance of road building aggregate in wet conditions, while being subjected to expected traffic loading (Hussain et al., 2011c). The unbound aggregates after being compacted and placed at the CAPTIF facility were subjected to loading in wet surface flow conditions. Previously CAPTIF research had been only in dry conditions. In addition, the properties of these aggregate types were also tested in the laboratory in order to understand its geological properties (Hussain et al., 2011d) plus its performance behaviour according to repeated load triaxial tests.

This chapter documents the statistical modelling of the data obtained from the testing of the aggregate using the Repeated Load Triaxial (RLT) test in which the materials are subjected to axial stress simulating the vehicle load on the material at various stress states. Different stress states were investigated since that represents different depths of the material within the pavement structure, plus it may also represent different levels of saturation.
5.1.2 Objectives and Scope

The main objectives of the over-all research project are twofold. Firstly, to investigate the permeability characteristics of different surface technologies and secondly, to investigate the performance of various base course materials with different engineering properties at different moisture levels. Ultimately, the outcome of this research is to develop the ability to forecast the performance of both the surface and pavement layers under different moisture conditions and varying aggregate properties.

This research documents results from RLT tests and the forecasting of the expected plastic deformation. Different models were assessed as part of this work and an alternative model is proposed. A review of the empirically derived models that are either principally derived from predicting the deformation from the number of loadings or from the applied stresses. In the context of this research these models are referred to as First Generation Models. Second Generation Models incorporate both the loading cycles and the stresses in forecasting the permanent deformation. These models were tested based upon the data from the RLT tests conducted in this research. A brief assessment of the strength for the First Generation Models is presented, while results from model tests of the Second Generation Models are discussed in more detail.

5.2 Experimental Methodology

5.2.1 Aggregate Properties

The three unbound granular materials used in this research are Greywacke obtained from the South Island of New Zealand. An example material is selected to demonstrate the modelling process. This material in its various forms is the most commonly used as basecourse material. The particle size distribution (gradation) curve presented in Figure 5.1 shows a small percentage of clay fractions in the material, thus it has been classified as largely non-plastic. Other engineering properties of the material are listed in Table 5.1. According to these properties, the material complies with the specifications for basecourse according to the TNZ M/4 (Transit New Zealand M/4, 2006) standard.
Chapter 5 Modeling the Permanent Deformation of Unbound Granular Materials using Repeated Load Triaxial Test

Figure 5.1 Gradation Curve for Unbound Material with Upper and Lower Limits

Table 5.1 Engineering Properties of the Basecourse Material used

<table>
<thead>
<tr>
<th>Engineering Property</th>
<th>AP-40</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unified Classification System</td>
<td>GW</td>
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<tr>
<td>AASHTO Classification System</td>
<td>A-1-a</td>
</tr>
<tr>
<td>Cone Penetration Limit (Moisture in %)</td>
<td>21</td>
</tr>
<tr>
<td>Sand Equivalent (% age of sand to clay)</td>
<td>36</td>
</tr>
<tr>
<td>Clay Index (volume in ml of methylene blue absorbed by 1 g of material)</td>
<td>1.4</td>
</tr>
<tr>
<td>Permeability (Average of Head Difference 2 kPa and 5 kPa, m/s)</td>
<td>0.00001</td>
</tr>
</tbody>
</table>

Notes:
Cone Penetration Limit: Test 3.2, NZS 4407:1991 (Standards NZ, 1991)
Sand Equivalent: Test 3.6, NZS 4407:1991 (Standards NZ, 1991)
Clay Index: Test 3.5, NZS 4407:1991 (Standards NZ, 1991)
Permeability: Triaxial test with back pressure technique

5.2.2 RLT Testing Method

Background to the RLT Tests

In an RLT test, a specimen is compacted and placed in a triaxial cell (shown in Figure 5.2) where it is confined with air or water that applies the confining stress ($\sigma_3$). A load cell is
placed at the top of the sample which applies the axial load on the sample which is known as the deviator stress ($\sigma_1$). The application and release of vertical load over the compacted sample completes one loading cycle. The sample deforms with each load application and a portion of the deformation is recovered when the loading is relaxed. The un-recovered portion of the deformation contributes to the permanent plastic deformation of the sample. This plastic deformation behavior of the UGM causes failure in the field when the deformation accumulates in the basecourse with each passing vehicle. The permanent deformation or plastic deformation is the property of UGM that the material settles in increments with each loading cycle. Hence it is very important to formulate the phenomenon of UGM plastic deformation.

Note that later sections refer to the deviator stress ($q$) and mean normal stress ($p$), given by Equations 5.1 and 5.2 (Brown and Hyde, 1975).

\[ q = \sigma_1 - \sigma_3 \]  
\[ p = \frac{\sigma_1 + 2\sigma_3}{3} \]

**Figure 5.2 Repeated load triaxial test**

Testing Undertaken for this Research

The samples are compacted in layers into a cylindrical shape 290 mm height and 150 mm in diameter. All samples were compacted at optimum moisture content (4.8%) using a vibratory hammer and a maximum dry density (2.34t/m$^3$) was achieved. The RLT tests were conducted
at different levels of saturation of this material. In addition, some of the samples were subjected to an increase in saturation in the triaxial cell by applying back pressure. This was carried out to, as best that could be best practically undertaken, saturate the basecourse material. The three different moisture conditions for testing were:

1. Samples were compacted and tested at optimum moisture under **drained** conditions;
2. Samples were compacted at optimum moisture. Following compaction, the moisture was increased in the triaxial cell through backpressure technique. **Drained** conditions were allowed during the loading cycles; and
3. Lastly some samples were prepared in the same way and saturated to more than 90 percent, the samples were then tested in **un-drained** conditions.

The RLT deviator stress \((q)\) was applied at a frequency of 4 Hz. The stress states used for the stage tests were taken from the draft New Zealand Standard TNZ T15 (2010). These stresses have been recommended from the previous CAPTIF projects and have been formulated in the new standard. The stress paths for these stress states are shown in Figure 5.3. A total of 50,000 loads were applied for each stress level. The values of different stresses at different test stages are shown in Table 5.2.
### Table 5.2 Stress States for RLT Test

<table>
<thead>
<tr>
<th>Stress State</th>
<th>Axial Stress $\sigma_1$ (kPa)</th>
<th>Confining Stress $\sigma_3$ (kPa)</th>
<th>Deviator Stress $q$ (kPa)</th>
<th>Mean Normal Stress $p$ (kPa)</th>
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<td>180</td>
<td>150</td>
</tr>
<tr>
<td>5</td>
<td>470</td>
<td>140</td>
<td>330</td>
<td>250</td>
</tr>
<tr>
<td>6</td>
<td>530</td>
<td>110</td>
<td>420</td>
<td>250</td>
</tr>
</tbody>
</table>

### 5.3 Assessing First Generation Permanent Deformation Models

#### 5.3.1 Background to Different Modelling Approaches

To date, the RLT test remains the most popular and accurate test method for the estimation of permanent deformation of aggregates. It successfully simulates the repeated load characteristics associated with road pavement material under traffic loading. In addition to that it can also simulate the aggregate being under different levels of confining stress similar to road aggregates being under different stress due to their varying depth within a pavement.

Many researchers have attempted to model permanent deformation of aggregates on the basis of RLT results by using both empirical and mechanistic models (Allou et al., 2007; Allou et al., 2010; Chazallon et al., 2006; Chen et al., 2010; Habiballah and Chazallon, 2005; Klisinski et al., 1991; Lekarp and Dawson, 1998; Lekarp et al., 2000; Pérez and Gallego, 2010; Pérez et al., 2010). Most of the empirical models use input parameters such as the number of repeated axial loadings, confining stress, deviator stress, and elastic deformation to predict the permanent deformation of granular materials used in basecourse construction. The plasticity theories also involve some empirical models, that utilise finite element modelling to predict the deformation of the unbound granular materials. This technique is commonly known as constitutive modeling. A number of constitutive models follow the shake down theory concept (Allou et al., 2010; Boulbibane et al., 2005; Garcia-Rojo and Herrmann, 2005; Krabbenhoft et al., 2007; Werkmeister, 2006) while some of the others are based on high cycle plasticity theory (Niemunis et al., 2005; Wichtmann et al., 2010) and fuzzy set plasticity theory models (Chen et al., 2010; Klisinski, 1988; Klisinski et al., 1991).
5.3.2 Forecasting Permanent Deformation on the Basis of Loading Cycles (N)

Forecasting permanent deformation (PD) on the basis of load repetitions is certainly one of the favoured methods of previous researchers. Table 5.3 lists some of these models and the parameters utilised in the models. These models were tested on the basis of RLT test data for this research and a qualitative assessment of its strengths and limitations are discussed in subsequent paragraphs based upon how the model output fits the RLT data.

The Paute model (Paute et al., 1996) (Equation 5.3) uses three parameters: $\varepsilon_{p(100)}$ is the PD after 100 load cycles; and, A and B are model constants. This model performs well for the materials that achieve a stable condition after a certain number of loads. It however does not follow the pattern seen in empirical test data for the various materials/stress states where the permanent deformation increases with the number of loading cycles.

The Sweere model (Sweere, 1990) (Equation 5.4) and Barksdale model (Lekarp et al., 2000) (Equation 5.5) both have two regression parameters: a; and b. These models effectively predict PD up to a certain load repetition. From this point onwards, it constantly under-predicts the PD. It is suspected that the dataset for these models only covered lower repetition ranges.

The Wolff model (Wolff et al., 1994) (Equation 5.6) and Theyse model (Theyse, 2002a) (Equation 5.7) tend to constantly over-predict the PD values. The model given by Pérez (Pérez et al., 2010) (Equation 5.8) is the combination of Sweere Model (Equation 5.4) and Wolff Model (Equation 5.6). This combined model seems to be more accurate in predicting the PD for the RLT test data.

5.3.3 Forecasting Permanent Deformation Models Based on Stress

Some of the models that used stress states to predict the permanent deformation of the UGM are listed in Table 5.4. The axial ($\sigma_1$) and confining ($\sigma_3$) stresses are directly used in some of the models, while in other models, the deviator stress and average stresses are used to predict the permanent deformation of UGM.
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Table 5.3 Permanent Deformation Models using Number of Loads as Predictor

<table>
<thead>
<tr>
<th>Model</th>
<th>Equation</th>
<th>Parameters</th>
<th>Eq. No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Paute Model (Paute et al., 1996)</td>
<td>( \varepsilon_p = \varepsilon_{p(100)} + A \left[ 1 - \left( \frac{N}{100} \right)^{-B} \right] )</td>
<td>A, B, ( \varepsilon_{p(100)} )</td>
<td>5.3</td>
</tr>
<tr>
<td>Sweere Model (Sweere, 1990)</td>
<td>( \varepsilon_p = aN^b )</td>
<td>a, b</td>
<td>5.4</td>
</tr>
<tr>
<td>Barksdale Model (Lekarp et al., 2000)</td>
<td>( \varepsilon_p = a + b \log N )</td>
<td>a, b</td>
<td>5.5</td>
</tr>
<tr>
<td>Wolff and Visser Model (Wolff et al., 1994)</td>
<td>( \varepsilon_{1p} = (mx + a)(1 - e^{-bx}) )</td>
<td>m, a, b</td>
<td>5.6</td>
</tr>
<tr>
<td>Theyse Model (Theyse, 2002a)</td>
<td>( pd = mN + a(1 - e^{-bN}) )</td>
<td>m, a, b</td>
<td>5.7</td>
</tr>
<tr>
<td>Pérez Model (Pérez et al., 2010)</td>
<td>( pd = a1N^{b1} + (mx + a2)(1 - e^{-b2x}) )</td>
<td>m,a1,a2,b1,b2</td>
<td>5.8</td>
</tr>
</tbody>
</table>

Table 5.4 Permanent Deformation Models using Stresses as Predictors

<table>
<thead>
<tr>
<th>Model</th>
<th>Equation</th>
<th>Parameters</th>
<th>Eq. No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hyde Model (Gidel et al., 2001)</td>
<td>( \varepsilon_1^p = a \frac{q}{\sigma_3} )</td>
<td>a</td>
<td>5.9</td>
</tr>
<tr>
<td>Shenton Model (Gidel et al., 2001)</td>
<td>( \varepsilon_1^p = K \left( \frac{q_{\text{max}}}{\sigma_3} \right)^\alpha )</td>
<td>K,( \alpha )</td>
<td>5.10</td>
</tr>
<tr>
<td>Lekarp Model (Lekarp and Dawson, 1998)</td>
<td>( \varepsilon_1^p \left( \frac{N_{\text{ref}}}{L/p_0} \right) = a \frac{q}{p_{\text{max}}}^b )</td>
<td>a, b, ( N_{\text{ref}} ), ( L = \sqrt{q^2 + p^2} ) ( p_0 = 100 \text{ kPa} )</td>
<td>5.11</td>
</tr>
<tr>
<td>Paute Model (Paute et al., 1996)</td>
<td>( A = \frac{q}{b \left( m - \frac{q}{p + p^*} \right)} )</td>
<td>b, m ( p^* ) is the stress parameter defined by the intersection of the static failure</td>
<td>5.12</td>
</tr>
</tbody>
</table>

The Hyde model (Gidel et al., 2001) (Equation 5.9) and Shenton model (Gidel et al., 2001) (Equation 5.10) use deviator stress and confining stresses to predict the PD. The Hyde model is a linear model and tends to fit the trend of this research dataset well, but does not fit the absolute data points.

The Shenton (Gidel et al., 2001) model is a non-linear model that when fitted to the RLT data both the absolute forecasts and the trend fitting performed well. The Lekarp Model (Lekarp and Dawson, 1998) (Equation 5.11) was based on the shake down approach. It tended to
predict the PD trend well but not the absolute values from this dataset. The Paute model (Paute et al., 1996) (Equation 5.12) had the best correlation with this research’s dataset for both the trend and absolute values compared to all the other models presented in Table 5.4.

It has been concluded from these two sections that both the number of loadings and the stresses are relevant in the forecasting of permanent deformation. The next section discusses tests that have been conducted on models that incorporate both these aspects.

5.4 Testing Second Generation Models on the Basis of Data from this Research

The permanent deformation in the UG material has been predicted by the regression models which take into account both the effect of stresses and number of repeated loading cycles. Diagnostic model tests were undertaken on these models for the RLT data completed for different load cycles repeated on each individual stress state.

Two of the most recent available models (from Gidel (2001) and Wekmiester (2003)) have been selected from the literature and are compared in greater detail. In both cases model coefficients were determined using a least square approach fitted on the dataset from this research. Results from these tests are presented in the following sections.

5.4.1 Werkmeister Model

The shakedown theory of the UGM has been discussed in detail by Werkmeister (Werkmeister, 2003). There are three stages defined by Werkmeister: 1) Range A - Plastic Shakedown Range; 2) Range B - Intermediate Response - Plastic Creep; and 3) Range C - Incremental Collapse. The material from the RLT tests most closely represents that from the Range B of the shakedown state. The model presented by Werkmeister that links the permanent deformations with the number of loading cycles and stress states is expressed in Equation 5.13:

\[
\varepsilon_p(N) = \left[ \left( a_1 \frac{\sigma_2}{\sigma_3} \right)^2 + \left( a_3 \sigma_3^a \right) \left( a_4 \sigma_3^b \right) \right] (N)^\left( b_1 \sigma_2^a \sigma_3^b \right)
\]

where:

- \( \varepsilon_p(N) \) = Permanent deformation at number of loading ‘N’ (µm)
- \( \sigma_1 \) and \( \sigma_3 \) = axial and confining stresses respectively (kPa)
- \( a_1, a_2, a_3, a_4, b_1, b_2, b_3, b_4 \) = regression parameters
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The Werkmeister model Equation 5.13, is an eight parameter model involving axial and confining stresses. In this model, the parameters which addressed the range ‘B’ behavior have been used as there are some other equations for range ‘A’. The outcome from the fitted model is presented in Figure 5.4. It shows the fitted model compared to actual data points for three different moisture conditions. The estimates of the regression parameters for Equation 5.13 are shown in Table 5.5.

![Figure 5.4 Fitted Werkmeister Model in Different Moisture States of an Unbound Material](image)

Figure 5.4 Fitted Werkmeister Model in Different Moisture States of an Unbound Material
Chapter 5 Modeling the Permanent Deformation of Unbound Granular Materials using Repeated Load Triaxial Test

Table 5.5 Regression constants for Werkmeister Model

<table>
<thead>
<tr>
<th>Stress-St</th>
<th>a1</th>
<th>a2</th>
<th>a3</th>
<th>a4</th>
<th>b1</th>
<th>b2</th>
<th>b3</th>
<th>b4</th>
</tr>
</thead>
<tbody>
<tr>
<td>11x</td>
<td>1.38E-06</td>
<td>3.00</td>
<td>5.00</td>
<td>0.685</td>
<td>-0.180</td>
<td>-0.895</td>
<td>0.073</td>
<td>-0.101</td>
</tr>
<tr>
<td>12x</td>
<td>8.85E-07</td>
<td>3.00</td>
<td>6.05</td>
<td>0.718</td>
<td>0.012</td>
<td>-0.0678</td>
<td>0.0015</td>
<td>0.545</td>
</tr>
<tr>
<td>13x</td>
<td>0.318616</td>
<td>1.00143</td>
<td>2.00</td>
<td>-2.00</td>
<td>-4.76E-5</td>
<td>1.201</td>
<td>0.015</td>
<td>0.417</td>
</tr>
</tbody>
</table>

The first digit in stress states in Table 5.5, Table 5.6, and Table 5.7 show the number of material which selected i.e., CG material. The second digit shows the moisture state i.e., 1 shows OMC Drained, 2 shows Saturated Drained and 3 shows Saturated Undrained conditions. The third digit or ‘x’ shows for all stress states. This model predicts the behaviour of the material while changing the stress states in repeated load triaxial (RLT) tests. The Werkmeister model tends to slightly under-estimate the PD values towards the end of each stress state.

5.4.2 Gidel Model

Gidel et al. (2001) presented the model which used the mean average stress ‘p’ and deviator stress ‘q’ to predict the plastic deformation in the unbound granular materials (Equation 5.14). This model contains two portions; the first part predicts the deformation with respect to the number of loadings applied while the second part is used to shift the model according to a change in mean and deviator stresses. Gidel fitted two types of stress models to the data: one fitted the data hyperbolically with change in stress state and the other fitted the data exponentially. The model that changed hyperbolically with the stress state fitted better in the RLT test data and is incorporated in the final model.

\[
\varepsilon_p = [a(1 - N^{-b})] \left[ \left( \frac{L}{p_a} \right)^n \frac{1}{m + \frac{s}{p} - \frac{q}{p}} \right]
\]

5.14

where:

- \( \varepsilon_p \) = Permanent deformation (µm)
- N = Number of loading cycles
- q and p = deviator and mean stresses respectively (kPa)
- L = Length of stress path \( (L = \sqrt{q^2 + p^2}) \)
\( p_a = 100 \text{ kPa} \)
\( a, b, n, m, s = \text{Regression parameters} \)

The model was fitted to the RLT test results obtained from the tested materials at three different moisture conditions (refer to Figure 5.5 and Table 5.6). The fit in Figure 5.5 shows that the Gidel model generally follows the change in stress state well compared to the RLT test data. However, it is shown that at a high stress state the model fit is less accurate, especially for the saturated drained moisture condition.

![Gidel Model](image)

**Figure 5.5 Fitted Gidel Model in Different Moisture States of an Unbound Material**

**Table 5.6 Regression constants for Gidel Model**

<table>
<thead>
<tr>
<th>Stress-St</th>
<th>a</th>
<th>b</th>
<th>n</th>
<th>m</th>
<th>s</th>
</tr>
</thead>
<tbody>
<tr>
<td>11x</td>
<td>4278.82</td>
<td>0.279</td>
<td>0.811</td>
<td>17.83</td>
<td>-16.15</td>
</tr>
<tr>
<td>12x</td>
<td>1244.70</td>
<td>0.181</td>
<td>0.890</td>
<td>4.37</td>
<td>-25.67</td>
</tr>
<tr>
<td>13x</td>
<td>687.57</td>
<td>0.165</td>
<td>1.072</td>
<td>3.99</td>
<td>-17.38</td>
</tr>
</tbody>
</table>
5.4.3 Suggested Hussain Model

Taking the lessons from Gidel (Gidel et al., 2001), an alternative Hussain model is proposed. Gidel’s model was used as a base model but the ‘N’ term from Equation 5.14 is replaced by Equation 5.8 (Pérez and Gallego, 2010), resulting in:

\[
\varepsilon_p = \left[ a_1 N^{b_1} + (m_1 N + a_2)(1 - e^{-b_2 N}) \right] \left[ \left( \frac{L}{P_a} \right)^n \frac{1}{m_2 + \frac{s}{p} - \frac{q}{p}} \right]
\]

\[
5.15
\]

\( \varepsilon_p \) = Permanent deformation (µm)  
\( N \) = Number of loading cycles  
\( q \) and \( p \) = deviator and mean stresses respectively (kPa)  
\( L \) = Length of stress path \( (L = \sqrt{q^2 + p^2}) \)  
\( P_a \) = 100 kPa  
\( a_1, b_1, a_2, b_2 \)  
\( m_1, n, m_2, s \) = Regression parameters  

Therefore, the new model is based on the combination of two models; a) the model presented by Pérez which predicts the permanent deformations with respect to number of loads only; and b) the second part of this model is taken from the second portion of the Gidel model that predicts the UGM behaviour with respect to stress states. The resulting model outcome is presented in Figure 5.6 and Table 5.7.
Chapter 5 Modeling the Permanent Deformation of Unbound Granular Materials using Repeated Load Triaxial Test

Figure 5.6 Fitted New Hussain Model in Different Moisture States of an Unbound Material

Table 5.7 Regression constants for Hussain Model

<table>
<thead>
<tr>
<th>Stress-St</th>
<th>a</th>
<th>b</th>
<th>A</th>
<th>B</th>
<th>m1</th>
<th>n</th>
<th>m2</th>
<th>s</th>
</tr>
</thead>
<tbody>
<tr>
<td>11x</td>
<td>3073.17</td>
<td>0.021</td>
<td>110.55</td>
<td>0.00017</td>
<td>0.0037</td>
<td>0.81</td>
<td>17.68</td>
<td>-15.24</td>
</tr>
<tr>
<td>12x</td>
<td>734.36</td>
<td>0.023</td>
<td>43.73</td>
<td>0.00014</td>
<td>0.0023</td>
<td>0.89</td>
<td>4.35</td>
<td>-25.68</td>
</tr>
<tr>
<td>13x</td>
<td>341.29</td>
<td>0.023</td>
<td>79.72</td>
<td>0.00012</td>
<td>0.00029</td>
<td>1.04</td>
<td>3.63</td>
<td>-6.46</td>
</tr>
</tbody>
</table>
5.4.4 Model Diagnostic Comparisons

In addition to the promising visual results depicted in the previous three graphs a detailed statistical comparison was also undertaken in order to assess the predictive power of the three modelling approaches. This section reports on the:

- Graphical fit for the maximum stress state;
- Akaike’s Information Criterion;
- Residual standard error and R-squared.

![Comparison of Models- Sat Dr-Stress State 6](image)

**Figure 5.7 Comparison of Fits for the presented models**

5.4.4.1 Graphical Fit

From previous work it has been established that most existing models are capable of predicting PD relatively accurate at low stress states. However, it becomes more challenging to forecast the PD at higher stress states. Figure 5.7 shows the comparison of the three models.
compared to the actual data for the highest stress state. It appears that the alternative model format is more accurately fitting this dataset.

5.4.4.2 Akaike’s Information Criterion (AIC)

The AIC value can be defined as “an estimate of the distance from the model fit to the true but unknown model that generated the data” (Ritz and Streibig, 2008). It is a function of:

- Number of observations;
- Residual sum of squares;
- Best estimate of the parameter; and,
- Number of regression parameters.

The AIC values for some of the models are shown in Table 5.8.

<table>
<thead>
<tr>
<th>Stress-State</th>
<th>Werkmeister Model</th>
<th>Gidel Model</th>
<th>Hussain Model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Akaike Information Value</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11x</td>
<td>11617.54</td>
<td>12416.42</td>
<td>12012.7</td>
</tr>
<tr>
<td>12x</td>
<td>13811.84</td>
<td>14286.48</td>
<td>13237.2</td>
</tr>
<tr>
<td>13x</td>
<td>12662.54</td>
<td>12883.04</td>
<td>12728.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Residual Standard Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>11x</td>
</tr>
<tr>
<td>12x</td>
</tr>
<tr>
<td>13x</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>R-Squared Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>11x</td>
</tr>
<tr>
<td>12x</td>
</tr>
<tr>
<td>13x</td>
</tr>
</tbody>
</table>

Note that if a model has additional number of parameters, it increases the AIC value. However, for the Hussain model, the increase in number of parameters (from Gidel Model)
has not increased AIC value, suggesting a potential better fit to the data. The comparison of the respective AIC values revealed that the models did not differ significantly.

### 5.4.4.3 Residual Standard Error (RSE) and R Squared (Coefficient of Determination)

RSE is the second measure for assessing the best fit of the nonlinear model. Lower values of RSE indicate a better model fit. It is a function of:

- the ‘distance’ between the forecasted and actual data points;
- the number of observations; and,
- the number of regression constants used in the equation.

The values of RSE (Table 5.8) show that there is not a significant difference in the model accuracies, with the alternative model resulting in the best outcome. One of the most commonly used regression fit assessment for linear models is the R-Square. The R-Squared also confirms the Hussain model equally best fitting the RLT data from this experiment as the other two.

### 5.5 Conclusions

This research is a part of a CAPTIF project which is full scale indoor testing facility. The materials used in this research were tested using RLT tests. This chapter has reviewed two regression models that used the number of loading cycles and the stress states to predict the permanent deformation in the UGM. The assessments of available models resulted in the compilation of an alternative model that combines principles used by Pérez and Gidel (Gidel et al., 2001; Pérez et al., 2010). The loading cycle component of the Pérez model was combined with the stress state model suggested by Gidel. Conclusions from the results are:

1. Aggregates used in road pavement layers are subjected to different stress states (as a result of its depth and relative position to the wheel loading) plus the number of loading cycles. Therefore second generation models, taking account of both the stress state and loading cycles, more closely represent the actual field situation compared to the first generation models that uses either one of these factors in isolation to predict deformation;

2. The Hussain model presented in this chapter resulted in a closer fit with the research dataset, especially at higher stress levels when compared to other existing models.
Chapter 5 Modeling the Permanent Deformation of Unbound Granular Materials using Repeated Load Triaxial Test

The method used to fit the new model along with the other compared models give the regression parameters for the unbound granular materials to be used in the basecourse and sub-base layers. The deformation can be predicted at various stress states in these materials which is an important criterion to judge the material performance in real in field pavements. The model work results related in an improved model that simulates the PD at different stress states. In addition, it can also reflect the varying behaviour of the material at different moisture conditions, which contributes to the understanding of material behaviour in in-service pavements which are often subjected to different moisture regimes.

5.6 References:


Chapter 5 Modeling the Permanent Deformation of Unbound Granular Materials using Repeated Load Triaxial Test


CHAPTER 6.

FUNDAMENTAL BEHAVIOUR OF UNBOUND ROAD AGGREGATES UNDER CYCLIC LOADING FOR DRAINED AND UN-DRAINED CONDITIONS

Abstract

The use of unbound granular materials (UGM) is an economical solution for the construction of a base layer in road pavements. Research has shown that water seeps through surface cracks leading to a high degree of saturation in the UGM layers. The pavement layers may act under drained or undrained conditions at various times of the year depending upon various factors such as the techniques used and the quality of construction, maintenance and the geometric alignment and environmental and climatic conditions of the road, Little is known about the performance of UGM under high degrees of saturation and the factors influencing their expected behaviour. In order to understand the behaviour of partially saturated UGM under drained and un-drained conditions, a series of RLT tests were conducted and are described and discussed in this chapter. The results show that permanent deformation of UGM under different drainage conditions is mainly a function of grading, void ratio and degree of saturation.
6.1 Introduction

Moisture significantly influences the performance of road pavements, irrespective of whether the pavement is flexible or a more rigid structure (Dawson, 2008; White et al., 2008). An increase in moisture causes a significant decrease in the strength of pavement materials (Toros and Hiltunen, 2008) and it is most prominent for unbound granular materials (UGM). UGM are mostly used in the basecourse layer of the pavements and thus absorb most of the traffic loading. The load carrying capacity of basecourse layers controls the life of the pavement structure (Chen, 2009).

Moisture intrusion of the base layer results in the reduction of the bearing capacity of the UGM resulting in excessive deformation under repeated loading of the traffic. An increased knowledge of behaviour under varying moisture conditions of these material types will significantly contribute towards more appropriate selection of material types for different climatic conditions.

This chapter documents research that is a part of New Zealand Transport Agency (NZTA) project “Are chip seals waterproof enough on high volume roads”. In this project, trial pavements consisting of unbound basecourses and surfaced using chip or sprayed seals have been constructed in the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF). The pavements have been tested for wet conditions by artificially adding water to the surface. Different surface and pavement configurations resulted in different moisture and drainage conditions. The same material has also been characterised using the Repeated Load Triaxial Test (RLT) in order to understand deformation characterisation under different moisture and drainage conditions. These tests assisted in understanding the permanent deformation behaviour of these aggregate.

6.1.1 Objectives of the Research

This chapter involves a continuation of developing a greater understanding of the performance of unbound aggregates used in basecourse. The specific research outcomes of the CAPTIF project include:

- Examination of performance based on mineralogical makeup of aggregates and how that influences their behaviour under wet conditions (Hussain et al., 2011c; Hussain et al., 2011d);
Chapter 6 Fundamental Behaviour of Unbound Road Aggregates under Cyclic Loading for Drained and Un-Drained Conditions

- Examining the effect of different stress states on the behaviour of the material (Hussain et al., 2011d); and
- Evaluation of an improved deformation prediction model based on RLT data (Hussain et al., 2011a).

6.1.2 Objectives of this Research Document

The objective of this chapter is to explain the fundamental behaviour of the UGM under cyclic loading and link the theory with the results observed in RLT tests. By linking the fundamental understanding of aggregate behaviour to the laboratory results, an increased understanding of aggregate behaviour in road pavements is obtained. The following goals were developed to better understand aggregate behaviour:

- To investigate the difference in permanent deformation of UGM under drained and un-drained conditions. Of particular interest is a better understanding of factors that cause a reduction in effective stress; and,
- To better understand the effect of aggregate gradation on permanent deformation of UGM. The particle size distribution of aggregates may differ which could affect their permanent deformation. For this study three materials with different grading all from the same sedimentary greywacke parent material in the South Island of NZ were used in this research.

6.2 Methodology of Assessment

6.2.1 Description of Materials

This section discusses the materials used in the CAPTIF experiments and for the subsequent laboratory tests. The materials used were obtained from South Island New Zealand quarries and are sourced from Greywacke rocks. Further information of these materials about the mineralogical make-up of these aggregates can be found in Hussain et al. (2011d). The aggregate gradation curves of the three UGM materials used are shown in Figure 6.1. The material classification used in this chapter is:

- Coarse Graded (CG) – this material represents a commonly used basecourse material that complies with the premium New Zealand (NZ) standard state highway M/4 specification (Transit New Zealand M/4, 2006) for unbound basecourses. The Talbots constant for this material was n=0.5 and maximum aggregate size was 37.5 mm.
Medium Graded (MG) – this material has Talbots constant of $n=0.43$ with maximum aggregate size of 19.0 mm. The maximum aggregate size of 19.5 makes this material denser. We call it medium graded on the base of Talbots constant value of 0.43 as it is in between CG and FG. This material falls outside the M/4 specification envelope.

Fine Graded (FG) – This material would be considered as a non-compliant M/4 material as whilst it falls within the envelope above the 1mm sieve fraction it does not comply below the 1mm fraction having a greater proportion of finer materials. It also has a more ‘S’ shaped psd profile that researchers (Salt et al., 2009) have shown to be more prone to fatigue failure than a well graded material. The Talbots constant value for this material is 0.37 and maximum aggregate size is 37.5 mm.

The permeability tests were also conducted on these samples using a triaxial cell (BS 1377-6:1990, 1990). The values obtained at constant 5 kPa were found to be 0.01 cm/sec for CG material, 0.002 cm/sec for MG material, and 0.006 cm/sec for FG material.
6.2.2 RLT Test Method

The specimens were compacted and placed in a triaxial cell (shown in Figure 5.2). The confining stress ($\sigma_3$) is applied by water pressure within the enclosed triaxial cell. A rubber membrane around the specimen prevents the surrounding water to penetrate the test specimen. The vertical deviator stress ($\sigma_1$) at the top of the sample is applied and then released accounting for one loading cycle. In the RLT test, a stress pulse is applied with a predetermined amplitude and dwell time. As a result of this loading, both resilient and permanent deformations occur. Resilient deformation is recovered following the release of the stress, whilst the permanent deformation of the specimen remains in the specimen. This permanent deformation in the basecourse results in rutting of the unbound layer in the pavement.

![Figure 6.2 Repeated load triaxial test setup](image)

The deviator stress ($q$) and isotropic mean normal stress ($p$) are defined by the following Equations 6.1 and 6.2 (Brown and Hyde, 1975).

\[
q = \sigma_1 - \sigma_3
\]  
\[
p = \frac{\sigma_1 + 2\sigma_3}{3}
\]

The test stress states followed in this research are explained in the NZ Draft standard (TNZ T/15, 2010). Various stress paths are shown in Figure 6.3. The frequency of deviator stress loading was 4 Hz in which the vertical loading phase exists for 0.1 sec with a dwell time of
0.15 sec in one load cycle. A total of 50,000 load cycles were applied for each loading stress level. The specimen properties tested are further explained in Table 6.1. The results of these tests will show the behaviour of unbound materials in different stress and moisture conditions. The specimen testing conditions include:

**Drained Conditions:** Samples were compacted by the (TNZ T/15) method at optimum moisture content. Following compaction, the moisture was increased in the specimen through a backpressure flushing technique (BS 1377-6:1990). Draining of the sample was allowed during the loading cycles; and,

**Un-Drained Conditions:** The samples were compacted and moisture in the specimen was increased in the same way as in drained tests but the specimens were tested in un-drained conditions.

![Figure 6.3 Total stress paths for RLT test](image-url)
Table 6.1 Properties and Conditions of Testing Samples

<table>
<thead>
<tr>
<th>Materials</th>
<th>Max. Dry Density (MDD) (t/m³)</th>
<th>Optimum Moisture (%)</th>
<th>Pre-test Moisture (%)</th>
<th>Post-test Moisture (%)</th>
<th>Post-test Deg. Of Sat. (%)</th>
<th>Density (95% of MDD)</th>
<th>Test Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>CG</td>
<td>2.34</td>
<td>4.3</td>
<td>5.9</td>
<td>5.9</td>
<td>98.30</td>
<td>2.223</td>
<td>Saturated, un-drained</td>
</tr>
<tr>
<td></td>
<td>2.34</td>
<td>4.3</td>
<td>5.9</td>
<td>3.9</td>
<td>77.78</td>
<td>2.223</td>
<td>Saturated, drained</td>
</tr>
<tr>
<td>MG</td>
<td>2.38</td>
<td>4.8</td>
<td>5.9</td>
<td>5.9</td>
<td>99.09</td>
<td>2.261</td>
<td>Saturated, un-drained</td>
</tr>
<tr>
<td></td>
<td>2.38</td>
<td>4.8</td>
<td>5.9</td>
<td>4.7</td>
<td>89.32</td>
<td>2.261</td>
<td>Saturated, drained</td>
</tr>
<tr>
<td>FG</td>
<td>2.36</td>
<td>4.8</td>
<td>6.1</td>
<td>6.1</td>
<td>99.79</td>
<td>2.242</td>
<td>Saturated, un-drained</td>
</tr>
<tr>
<td></td>
<td>2.36</td>
<td>4.8</td>
<td>6.1</td>
<td>4.5</td>
<td>83.96</td>
<td>2.242</td>
<td>Saturated, drained</td>
</tr>
</tbody>
</table>

6.3 Regression Model for Permanent Deformation of UGM

Hussain et al. (2011a) proposed a model (Equation 6.3) which successfully predicts the permanent deformation results in the RLT of the same unbound aggregate materials presented in this research chapter, considering the variation of loading cycles and stress states simultaneously at different degrees of saturation. This expression was developed taking equations from Pérez & Gallego (2010) and Gidel et al. (2001) that forecast deformation as a function of loading cycles, and different stress states respectively. The model fit is presented in Figure 6.4 along with the experimental data which excludes permanent deformation for the first 100 loading cycles at each stress state. This figure shows the model fit plotted against actual deformation from the RLT tests. The RLT results presented are on the coarse graded (CG) material for three different drainage and moisture conditions.

\[
\varepsilon_p = \left[ a_1 N^{b_1} + (m_1 N + a_2) (1 - e^{-b_2 N}) \right] \left[ \left( \frac{L}{p_a} \right)^n \frac{1}{m_2 + \frac{s}{p} - \frac{q}{p}} \right] \quad 6.3
\]

\[
\varepsilon_p = \text{Permanent deformation (µm)}
\]

\[
N = \text{Number of loading cycles}
\]

\[
q \text{ and } p = \text{deviator and mean stresses respectively (kPa)}
\]

\[
L = \text{Length of stress path } (L = \sqrt{q^2 + p^2})
\]
Chapter 6 Fundamental Behaviour of Unbound Road Aggregates under Cyclic Loading for Drained and Un-Drained Conditions

\[ p_a = 100 \text{ kPa (approximately atmospheric pressure)} \]

\[ a_1, b_1, a_2, b_2 \]

\[ m_1, n, m_2, s = \text{Regression parameters} \]

![Hussain Model](image)

**Figure 6.4** Fit of the modelled behaviour to test behaviour for CG material (Hussain, 2011b)

Observations from these plots are:

- The model is capable for forecasting permanent deformation for:
  a) Different loading cycles;
  b) Under varying stress conditions; and,
  c) For different drainage conditions.

- The CG material was tested at optimum moisture level and higher degrees of saturation at drained and un-drained conditions;
The maximum permanent deformation was observed for the CG material under saturated and drained condition, whilst the same material under undrained conditions failed prior to reaching the full loading of 300,000 cycles. The specimen is considered failed when permanent deformation is more than 1% of the total height of specimen.

The difference in behaviour of these conditions is discussed in more detail in the following sections.

6.4 The Effect of Different Material Gradings and Drainage Conditions on the Performance of Aggregates at High Degree of Saturation

The RLT tests were conducted on the materials that were used in the CAPTIF test pavements. This section presents the permanent deformation results from the RLT tests for materials at a high degree of saturation under different drainage conditions.

6.4.1 Coarse Graded Material

After compacting at OMC, the material was brought close to a high degree of saturation (more than 90 per cent). The RLT test was then repeated under drained and un-drained conditions. The simulated drained condition was only partially achieved given that the sample was allowed to drain at one end of the cylinder only. Results are presented in Figure 6.5.

It was observed that the drained sample (top graph of Figure 6.5) was able to deform for the entire load cycle spectrum (up to 300,000). Similar trends are observed for each stress state (50,000 load cycles). Initially the sample displays a rapid deformation that quickly stabilises to a uniform plastic deformation for increased loading cycles. This process repeats at each increase of the stress states. The slope of the stable deformation is relatively flat for the lower stress states, but at the higher stress states (5 and 6) the stable rate of deformation increases significantly.

These observations correlate well with earlier CAPTIF (Henning, 2008) research and field observations (Henning et al., 2009). A newly constructed pavement will display high initial rut rates shortly after construction due to further densification of the pavement layer. Soon after, the rut rate stabilises until the number of ESA axle loading significantly increases. At this point, there is a sudden increase in rut rate that stabilises again soon after.
The un-drained sample displayed almost identical deformation to the drained sample up to approximately 250,000 load cycles (stress state 5). After this point, the material becomes unstable and fails instantaneously showing deformation higher than 1% of the sample size.

### 6.4.2 Medium Graded Material

The permanent deformation results of highly saturated RLT drained and un-drained tests for MG materials are shown in Figure 6.6. The behaviour of this material was similar to the coarse graded material. Again low deformation rates were observed at low stress levels, with increasing deformation commencing at 200,000 cycles (stress state 5). The undrained sample again failed soon after stress state 5 (250,000 cycles).
6.4.3 Fine Grade Material

Figure 6.7 depicts the deformation of the fine graded material for the drained and un-drained conditions respectively. Compared to the other aggregate gradings, this material generally displayed higher rut rates. Rapidly increased rut rates started at lower stress states and both drained conditions failed at stress states 4 and 5 for drained and undrained conditions respectively.
6.4.4 Comparative Results

Figure 6.8 and Figure 6.9 presents a comparison of the permanent deformation at the end of each total stress state path number shown in Figure 6.1 for all three material gradings and for drained and un-drained conditions respectively.
It is observed that under drained conditions, the fine graded material displayed higher permanent deformation for all the stress states. In addition to that it fails during stress state 5. At lower stress states (below stress state 5) it appears that the medium graded material has lower permanent deformation if compared to the coarse graded material. However, at the high stress state, the ultimate deformation of the medium grade material is more significant than the coarse graded material.

The undrained conditions (Figure 6.9) displayed different trends. Firstly, in all stress states, the coarse graded material displayed the lower deformation. Medium graded material had the
highest deformation at lower stress states. The fine graded material displayed significant increased rates of deformation at stress state 3 and failed during stress state 4.

6.5 Explaining Laboratory Results through Fundamental Aggregate Behaviour Principles

The pavements can be subjected to a high degree of saturation especially during the rainy seasons along with different drainage conditions (Brown, 1996). Mostly the basecourse layers have drained conditions as they are specifically designed for drainage purposes however due to choking of these layers water can become entrapped in the pavement structure and can create undrained conditions. Whether drained or undrained conditions basecourse materials in NZ are mostly found in partially saturated conditions with different degrees of saturation depending upon the moisture content of the basecourse layer. In this testing program, RLT tests have been conducted on the specimen with high degrees of saturation under both drained and undrained conditions. The main factors which affect the performance of basecourse materials in partially saturated conditions include the pore water pressure (Arab et al., 2011; Bouferra, 2007; Yegian et al., 2007), suction (Côté and Konrad, 2003; Kolisoja et al., 2002; Kolymbas, 1998; Saarenketo et al., 2001; Scullion and Saarenketo, 1997), and degree of saturation (Dawson et al., 2000; Dawson et al., 1996; Gidel et al., 2001; Khogali and Mohamed, 2004; Thom and Brown, 1987).

The principle of effective stress for saturated soils (Terzaghi, 1943) can be applied on unsaturated soils (Bishop and Blight, 1963) and the proposed model predictions agree well with the experimental results by other researchers (Khalili et al., 2004; Nuth and Laloui, 2008). Terzaghi (1943) suggested that stress in soil can be computed from the total stress acting at the point and if the voids in the soil are filled with water under stress ‘u’, the total principle stress can be computed from two parts: one part u acts equal in all directions, and the second part is the difference of total and neutral stress represents the excess stress over neutral stress (known as effective stress).

\[ \sigma' = \sigma - u \]  

where:
- \( \sigma' \) = the effective stress;
- \( \sigma \) = the total stress,
- \( u \) = pore water pressure
The above Equation 6.4 holds true for shear strength if two conditions are assumed: 1) the solid particles in aggregates are incompressible, and 2) the yield strength of individual grains is independent of confining stresses applied. But these conditions are normally not fulfilled by aggregate mixes hence the equation can be modified in the form:

\[ \sigma' = \sigma - ku \]  

and \( k = \left(1 - \frac{a \tan \psi}{\tan \phi'}\right) \) for shear strength where \( a \) is the contact area of particles per unit gross area of material, \( \psi \) is the angle of intrinsic friction of soil grains, \( \phi' \) is angle of shearing resistance of the granular material. When this equation is extended to the mix having two fluids, it takes the form:

\[ \sigma' = \sigma - k_1 u_w - k_2 u_a \]  

where \( u_w \) is the pore water pressure and \( u_a \) is the pore air pressure. If neutral stress concept applies for fluid pressures, then the simultaneous change in total stress, pore-water pressure and pore-air pressure will not cause considerable change in shear stress. If \( k_1 \) and \( k_2 \) remain constant,

\[ 0 = \Delta \sigma - k_1 u_w - k_2 u_a \]  

and

\[ \Delta \sigma = \Delta u_w = \Delta u_a \]  

then

\[ k_2 = 1 - k_1 \]  

And if \( k_1 = \chi \) then we can easily find the famous Equation 6.10

\[ \sigma' = \sigma - u_a + \chi (u_a - u_w) \]  

where:

- \( \sigma \) = the effective stress;
- \( \sigma \) = the total stress,
- \( \chi \) = constant, \( \chi = 0 \) for dry and \( \chi = 1 \) for saturated materials
- \( u_a \) = the pore air pressure in the sample
- \( u_w \) = the pore water pressure in the sample

In Equation 6.10, the value of \( \chi \) is assumed to be depending on degree of saturation of saturation of partially saturated material considering the wetting-drying cycles of material,
structure of the material, and stress path being applied. The portion \((\sigma - u_a)\) represents the net stress which is the difference of total stress and the pore-air pressure. The part \((u_r - u_w)\) represents the value of suction in the partially saturated material.

The Equation 6.10 shows the dependence of effective stress on degree of saturation, and the suction i.e., \((u_r - u_w)\). The effective stress controls the strength of the unbound granular basecourse layer in the pavements i.e., the strength and resistance to deformation in basecourse materials will be more in case of higher effective stress. In unbound granular materials when they are unsaturated, there are always two fluid mediums air and water. Hence this equation can directly be used to explain theoretical effective stresses/ shear strengths of the materials.

The other main factor that controls the performance of basecourse materials is the inter aggregate pore space which is controlled by the particle size distribution, and percentage of fines. The higher inter aggregate pore space allows water to drain out easily from the basecourse. From the above discussion we can safely infer that the behaviour of granular material under different drainage conditions is controlled by the following factors:

- Pore-water pressure;
- Degree of saturation;
- Permeability and grading distribution of the aggregate.

### 6.5.1 The Influence of Pore Water Pressure

Excess pore pressures exist in UGM and the magnitude of the pore pressure in partially saturated granular basecourse is directly proportional to the degree of saturation (Bouferra, 2007; Yegian et al., 2007). Previous research has shown that in undrained conditions, the pore pressure increases at a higher rate in the initial loading cycles and then the rate of change decreases after a finite number of loading cycles (Bouferra, 2007; Indraratna, 2009; Indraratna et al., 2010). Further, suction is a property of basecourse materials that keeps the water in the basecourse layer even in drained conditions. In Equation 6.10 the difference of pore air pressure and pore water pressure shows the value of suction. At higher degrees of saturation, the suction decreases and goes negative as the pore water pressure increases more than the pore air pressure making the production of suction and saturation negative. This decreases the effective stress and as a result, higher permanent deformation can be observed.
It can be interpreted from the above references that in the RLT tests conducted in this research, pore water pressure existed in both drainage conditions (i.e., drained and undrained). However the undrained specimen had higher pore-water pressures as the degree of saturation is higher in the undrained tests (Table 6.1) and consequently resulted in positive pressures. From the permanent deformation of the MG and FG materials in the RLT tests (Figure 6.6 and Figure 6.7), it can be inferred that due to higher pore-water pressure in the undrained tests in comparison to the drained tests, the effective stress reduced (Equation 6.10) and more deformation was observed in the undrained RLT tests.

The Influence of Pore Water Pressure –Drained Conditions

Under drained conditions (reduced pore water pressure), the density of material plays a more significant role in the plastic deformation of UGM. The highest density was achieved in this research for the medium graded material. The higher density aggregates are resisting the deformation more effectively with each loading cycle. This is shown when a comparison is made between the CG and MG material where the MG material has a higher density (deformation process is shown in Figure 6.10). Under drained conditions, the MG material will display a smaller vertical deformation at less intense stress states than the CG material (Figure 6.8). At lower stress states there is a gradual decrease in volume (and void ratio) up to stress state 5. The pore pressure increase is very small due to the drainage condition and the density of the material restrains the rate of permanent deformation especially under low stress states (state 1 to state 5).

The situation changes at higher stress states as the deformation increases in the specimen. For stress state 6, the rate of permanent deformation is higher and the drainage of pore water is low due to lesser permeability in the MG material. Therefore, local pore water pressures increase which ultimately decreases the shear strength of the specimen resulting in higher permanent deformation observed in the MG material in comparison to the CG material even though it has higher permeability. The effect of pore sizes and permeability are discussed in the next section.
6.5.2 The Influence of Degree of Saturation

The moisture content of the UGM also plays an important role in controlling their resistance to deformation as an increase in moisture content increases the accumulation rate of permanent deformation in UGM (Dawson et al., 2000; Khogali and Mohamed, 2004). The moisture content of UGM is the factor which controls the resistance to permanent deformation (Gidel et al., 2001). An increase in moisture from dry of optimum to optimum level increases the permanent shear deformation in UGM (Dawson et al., 1996). The presence of moisture at these levels increases the permanent deformation in UGM by lubricating the aggregates when there is apparently no pore pressure (Thom and Brown, 1987).

From Equation 6.10 it can be inferred that pore air pressure will decrease and pore water pressure will increase with an increase in the degree of saturation of the material. The increased degree of saturation will cause a decrease in effective stress which in turn reduces the resistance to permanent deformation of the basecourse material. From Table 6.1 we know that the degree of saturation in an undrained test condition is higher than in the drained test condition at the end of the RLT test. If we compare the permanent deformation of each material at the end of the test, in Figure 6.5, Figure 6.6, and Figure 6.7, the deformation in the undrained tests are higher than the drained test due to the higher degree of saturation.

Figure 6.10 Height of sample at different stages of RLT test
6.5.3 Permeability and Grading Distribution

The permeability of the UGM enables drainage from the pavement structure. The permeability controls the movement of water in the material and therefore controls the pore water pressure, compressibility of voids, effective stress and ultimately the strength of the UGM under repeated loading.

The permeability is the property of soils that allows water to pass through its structure. The velocity \( v \) of the fluid in the soil is explained by Darcy’s Law.

\[
v = \frac{k \Delta h}{\mu n \Delta l}
\]

Where:

- \( \Delta h \) = difference in heads
- \( \Delta l \) = length of drainage path
- \( k \) = coefficient of permeability
- \( \mu \) = viscosity of fluid
- \( n \) = porosity of material

The coefficient of permeability depends upon the viscosity of the fluid in the material and the porosity of the material. Consider an example of two soils with similar void ratios: the sample with bigger voids will have greater permeability than the sample with smaller individual voids (refer to Figure 6.11). This shows that the grading distribution controls the water movement in the sample. The increase in pore pressure depends inversely on the void ratio, which explains the performance of the various gradings of the materials under un-drained conditions and at higher stress states. The excess pore pressure generation in the un-drained RLT can cause liquefaction due to cyclic loading (Kolymbas, 1998). Observing the permanent deformation trend in Figure 6.9 confirms this theory as the amount of final deformation is ranked in the same order as the void ratio for the soils. The CG material has the highest void ratio and lower pore pressure, thus resulting in the lowest permanent deformation.
Moreover, the quantity and size of fines determines how the inter aggregate voids and spacings are filled. As the percentage of fines increases, water movement and permeability decreases and suction increases. The increase in fine content and decrease in permeability is shown in Figure 6.12.
MG and FG material which is as would be expected. Theoretically, the resistance to permanent deformation in the CG materials at high degrees of saturation and under undrained conditions should be the greatest when compared to MG and FG materials due to the maximum pore space available for the movement of water. It is very clear from Figure 6.9 that the CG material has the least permanent deformation of all RLT stress states.

6.6 Conclusions

This chapter presents the results of a series of RLT tests that have been conducted for drained and un-drained conditions for various aggregate material gradings. The results of the RLT tests have been discussed on the basis of fundamental principles related to factors such as effective stress in unsaturated aggregates, development of pore water pressure, permeability and particle size distribution. Based on the discussion of the results, the following conclusions were made.

- Previous research demonstrated that pore water pressure exists in partially saturated basecourse materials under drained and undrained conditions and furthermore that the degree of saturation controls the intensity of pore water pressure in the partially saturated basecourse materials. It can be concluded from this previous knowledge when combined with the results of this research that the undrained RLT tests on MG and FG materials showed higher deformations than the drained RLT tests due to the higher pore water pressure generation. The higher pore water pressure decreases the effective stress in the specimen which controls the shear strength.

- At lower stress states in the drained RLT tests, the aggregate density played a more significant role in resisting the permanent deformation as the rate of measured deformation was less. Hence, the denser MG material performed better than the CG material with a lower density.

- At higher stress states, there is an increase in pore water pressure of the MG material with low permeability (in drained conditions) causing higher permanent deformations as compared to the CG material which has a higher permeability. This conclusion is also supported by the results of the undrained RLT test where due to higher permeability and void ratio the CG material performs the best when compared to the MG and FG materials.

- From Equation 6.10 it is clear that the effective stress is controlled by the product of the degree of saturation and suction which is the difference in pore air and pore water
pressures. With an increased degree of saturation, this term becomes negative decreasing the effective stress. From these results it is concluded that a higher degree of saturation produces higher permanent deformation.

- The void space available in a basecourse material controls the pore water pressure generation in the samples. The coarse graded materials have more void but less density whilst the dense graded materials have less void space but higher density. The CG material having the highest percentage of voids available shows the least permanent deformation under undrained conditions as the greater proportion of voids keep the pore water pressure lower causing a significant decrease in effective stress. However, the MG material with the highest density and lowest voids shows lesser permanent deformation under drained condition and at lower degrees of saturation, compared with the lower density materials (CG and FG).

This chapter has established a link between the theoretical behaviour of aggregates with the laboratory test results from RLT tests on unbound granular materials. The next step would be to link these findings with the findings on the accelerated loading tests at the CAPTIF test track and this will be discussed in further chapters.

6.7 References


CHAPTER 7.

WHAT HAPPENS WHEN IT RAINS?
PERFORMANCE OF UNBOUND FLEXIBLE PAVEMENTS UNDER ACCELERATED PAVEMENT TESTING

Abstract

Surface dressings (chipseals) over unbound granular pavements are used extensively in most southern hemisphere countries with low population densities and therefore relatively low traffic volumed roads in comparison to more dense European and northern hemisphere countries. It has been thought for many years that chipseals provide a water proof barrier to the underlying unbound granular basecourse being the main structural layer in unbound flexible pavements. The stimulus for this research is that some chipseal surfacings in New Zealand in recent years have not lasted their minimum expected design life for a combination of relatively high traffic volumes and wet climatic areas. Therefore, more regular and intensive maintenance is required thereby increasing the maintenance budget for maintaining minimum levels of service for road networks.
A research project was conducted in the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) in Christchurch, New Zealand to determine the suitability of the previously used prime coat in chipseal technology as a means to compare, for the same loading regime, chipseal surfaces that were applied with and without a prime coat. Prior to this research project the CAPTIF facility excluded environmental factors including water in the accelerated pavement loading process. The relative change in moisture in unbound granular pavement layers as water seeps through the surface chipseals was monitored during the research project. In this research project, water was allowed to flow over the sealed pavement sections with accelerated loading allowing water to enter only from the surface and not the sides of the pavement.

The research results demonstrated very dramatically that water penetrated the underlying layers even under no loading. Furthermore the sections with medium graded basecourse with and without a prime coat lasted significantly longer in wet conditions under standard axle loading than the coarse graded and fine graded basecourses. Results also showed that the addition of a prime coat to the chipseal significantly extended the life of unbound pavements. With the recent development of emulsion seals that are much safer to construct than earlier ‘cut back’ bitumen prime coats it is expected that emulsion seals will allow prime coats to be re-introduced where appropriate during sealing construction to better waterproof basecourse layers prior to chipsealing.

7.1 Introduction

7.1.1 Background

Chip seals (surface dressings) are commonly used as a cost effective maintenance strategy to prolong the life of pavements all around the world. These chip seals are a significant proportion of low traffic volume transportation networks especially in Australia, South Africa and New Zealand (NZ) characterised by large land masses, low population density and extensive transportation networks. This demonstrates that they are still one of the best economic surfacing practice options for low traffic volumed roads (Gransberg and James, 2005). The majority of NZ rural highways consist of thin flexible unbound granular pavements surfaced with either surfacing dressings (chip seals) or thin asphalt surfaces (thickness $\leq 40\text{mm}$). These roads would typically carry between 4,000 to 10,000 vehicles per day which makes the unbound pavement and chipseal surface technology affordable for these
traffic volumes. Although still classified as low volume roads according to world standards, these roads may sometimes carry traffic loadings in excess of their design standards.

Different zones in Australia and NZ are subjected to high mean annual rainfall where this can exceed 1700 mm (Bureau of Meteorology, 2011; Mackintosh, 2001). The recent floods in Australia (Hurst, 2011) showed that three quarters of Queensland, Australia was declared a disaster zone due to surface flooding. The high rainfall increases the available amount of water around the pavement which can increase the moisture within the pavement. An increase in surface runoff and rainfall intensity adds to the water film depth over the pavement and moisture can infiltrate through cracks present in the wearing course of a pavement. The prime functions of the chip seal is to form a waterproof layer to prevent intrusion of moisture into the basecourse and subbase layers and to bind the top part of the granular basecourse, thereby reducing dust and increasing skid resistance. Additional functions are to reduce the rolling resistance and pavement wear. The thin surface layers do not have a structural capacity in themselves to bear the traffic loads. The granular basecourse layers in an unbound pavement perform as the main structural strength component thereby distributing the traffic loads such that the subgrade does not get over stressed. Previous research has shown that if the granular structural layers (which lie beneath the wearing course) become wet or saturated under loading the structural strength and pavement life in terms of expected traffic loads or Equivalent Standard Axles (ESAs) are significantly reduced (Ekblad and Isacsson, 2006; Ekblad and Isacsson, 2008; Toros and Hiltunen, 2008). Therefore, the ability of the surface wearing layer to keep the water out of the pavement alongside the moisture sensitivity of the unbound granular materials used in the pavement layers together determine the strength and durability of the road pavement. Earlier research had shown that chipseals are relatively water proof under normal circumstances (Button, 1996; Solaimanian and Kennedy, 1998), however they can become partly permeable under traffic loading with water on the pavement surface (Ball et al., 1999; Towler and Ball, 2001). Furthermore, when capeseals (an extra layer over the chipseal to make it waterproof) are used along with chipseals, the permeability of the surfacing is approximately equal to a thin asphalt layer (thickness ≤40 mm) (Solaimanian and Kennedy, 1998). Hence the successful use of chipseals over relatively low traffic volumed roads without any additional protection seems an appropriate expectation.
Chapter 7 What happens when it rains? Performance of unbound flexible pavements under accelerated pavement testing

The primary failure cause of most flexible pavements is the failure of the unbound basecourse layer (Chen, 2009). The basecourse layer is subjected to repeated traffic loads and the response of this structural layer depends upon the degree of compaction, gradation of the material, number of loads, and moisture content (Huurman and Molenaar, 2006; Khogali and Mohamed, 2004; Nunez et al., 2004). The unbound basecourse materials perform well under dried conditions but as the moisture reaches near and above saturation level, there is a significant decrease in the load carrying capacity as well as an exponential increase in permanent deformation (Chandra et al., 1989; Dawson et al., 1996; Thom and Brown, 1987).

The pavements tested in accelerated pavement tests have shown a good stable performance in dry conditions. In accelerated pavement tests different gradings for basecourse with thin asphalt surfaces have lasted more than 1 million axle loads whilst not exceeding the rutting limit of 20 mm (Arnold et al., 2001; Chazallon et al., 2009). These accelerated pavement tests were under controlled environmental conditions and there were no intentional variation/addition of moisture added to the pavement. However, as has been known by highway engineers for many centuries, an increase in moisture in the pavement reduces its life (Bejarano and Harvey, 2002) resulting in the reduction of load repetitions from one million to less than half a million before failure.

This research documents one of the first accelerated pavement testing experiments involving the monitoring of water entering the base course through the chip seal surface. This research was part of a New Zealand Transport Agency (NZ Transport Agency) project “Are chipseals waterproof enough on high volume roads”. This NZTA project was a joint venture with The University of Auckland, OPUS International Consultants and the NZTA owned and operated accelerated pavement research facility in Christchurch - Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF).

7.1.2 Problem Statement

It is a well-known fact that unbound granular pavements carrying high traffic volumes do not perform well under wet conditions. Certain types of granular aggregates perform worse than others in wet conditions (Dawson et al., 1996). It is therefore imperative that chip seals fulfill their primary purpose of keeping the base course dry. Practitioners often assume that chipseals are water tight (Button, 1996). However, there is an indication of water ingress through chipseals in research by Ball (1999).
To better understand the engineering performance of chipseal surfaces, it is necessary to answer questions such as:

- how can it be ensured that the constructed seal is working as intended; preventing water entering into the basecourse layer from the sides or bottom?
- how can the intrusion of water be monitored from different chipseal options into the pavement under surface runoff conditions?; and
- how can the performance of different particle size distribution (PSD) basecourse gradations be measured in wet conditions under moving traffic?

It would be very difficult to answer these questions while undertaking tests on existing or newly built pavements due to the requirements of proper instrumentation and data collection facilities at multiple pavement sections with different surfacing options and basecourse materials.

To answer these questions and to help find the causation of premature pavement failures, this research investigated the effect of introducing surface water runoff across the indoor CAPTIF accelerated pavement testing research facility with a test pavement constructed with various surfacing techniques and PSD gradations of basecourse.

7.1.3 Objectives of the Research

The primary objective of this research was to understand the interaction between surface technology and its ability to waterproof the underlying basecourse layers. In order to achieve this understanding, the research set the following goals:

a) To develop a testing regime that would determine the amount of water entering the base course through the surface under accelerated conditions. The CAPTIF facility has been successful in answering a number of pavement behavior research questions in the past. However, it has never been used to simulate water entering the pavement through the surface;

b) To determine whether different chipseal design and construction techniques would have a different outcome in terms of the amount of water entering through the surface; and,

c) To test a range of aggregate source properties and their respective susceptibility to moisture conditions and thereby determine their suitability as base course materials in wet conditions.
7.2 Track Design

7.2.1 Materials

In the CAPTIF facility, six road sections were prepared with a combination of two types of chipseal surfacings and three types of basecourse materials. The wearing course used over all test sections was a NZ Grade 3 & 5 two coat chipseal laid respectively. Grade 3 aggregates have an average least dimension range equal to 7.5-10 mm chip size. For the grade 5 chips, there is an envelope specification defined in TNZ M/6 specification with a maximum of 9.5 mm chip size. One of the chipseals was laid without a prime coat (UP) above the basecourse layer and the other with a prime (P) coat. The prime coat was an additional layer of emulsion asphalt sprayed over the basecourse before laying the chipseal layer.

![Figure 7.1 Gradation curves for three granular materials.](image)

The three types of basecourse used were selected according to the percentage of fine particles and their maximum aggregate size. The gradation curves for the materials are shown in Figure 7.1. These basecourse materials were categorized on the basis of gradation as;

- BC1 as non-sensitive (low-L) basecourse with the least percentage of fines;
Chapter 7 What happens when it rains? Performance of unbound flexible pavements under accelerated pavement testing

- BC2 as averagely (medium-M) sensitive basecourse with intermediate percentage of fines; and
- BC3 as (high-H) moisture sensitive basecourse with high percentage of fines.

The permeability of the materials was tested at two constant heads for comparison at different pressures. There was an insignificant change in the permeability constants for BC1 and BC2 at 2kPa and 5kPa, however a change was noted for BC3 where the permeability constant was higher for 2kPa. The permeability of the BC1 aggregate was the greatest and the BC2 aggregate was the lowest among the three materials. Figure 7.2 shows the permeability of the materials.

![Figure 7.2 Permeability for three granular materials](image)

### 7.2.2 Pavement and Instrumentation

The total testing length (circumference) of the CAPTIF test track is approximately 59 meters. A plan view of the CAPTIF test pavement is shown in Figure 7.3. The section description from A to F and their respective chainage is shown in Table 7.1.
Table 7.1 Description and Chainage of Sections

<table>
<thead>
<tr>
<th>Description of the section</th>
<th>Chainage of the section</th>
</tr>
</thead>
<tbody>
<tr>
<td>A=un-primed least sensitive (UP-L)</td>
<td>A 56-8</td>
</tr>
<tr>
<td>B=primed least sensitive (P-L)</td>
<td>B 08-18</td>
</tr>
<tr>
<td>C=primed medium sensitive (P-M)</td>
<td>C 18-28</td>
</tr>
<tr>
<td>D=primed most sensitive (P-H)</td>
<td>D 28-37</td>
</tr>
<tr>
<td>E=un-primed most sensitive (UP-H)</td>
<td>E 37-47</td>
</tr>
<tr>
<td>F=un-primed medium sensitive (UP-M)</td>
<td>F 47-56</td>
</tr>
</tbody>
</table>

Figure 7.3 Layout plan for CAPTIF sections.

The un-primed sections were at the poor end of the construction quality scale and were probably typical of the very quick failures experienced in the field. The basecourse was too dry and dusty and the bitumen was not viscous enough (which was a compromise made to get...
a good seal on the primed section). The dust and lack of cutters meant that the bitumen on the un-primed sections “balled up” leaving gaps in the first layer of bitumen. It looked reasonable at the end of construction but was not the best construction practice.

Moisture monitoring using time domain reflectometry (TDR) gauges has been successfully used in the past (Ekblad and Isacsson, 2007; Liang et al., 2006) and was therefore applied in this research. A total of eight TDR gauges were installed at each of two stations in each pavement section. A data logger was installed and attached to the 16 TDR gauges in each of the two sections. Three data loggers were installed in total. The plan and side view of a TDR gauge is shown in Figure 7.4 Time Domain Reflectrometer plan and side view. At a station, two TDR gauges were placed under the wheels whilst the other is placed on the inner side of the circular track. The surface camber collected the surface water to the inner circumference of the track where runoff was collected in the tanks and pumped back into the water supply tanks. The pavement cross section and location of TDR gauges is shown in Figure 7.5.

### 7.2.3 Water Application

The water application and collection system was designed such that water could be collected from the surface as well as from the basecourse layer. On six sections, the assembly of water tanks, collecting tanks and pumps were arranged for each section to collect and pump water back to the water supply tank so that the applied water could be used repeatedly. The submerged pumps were installed in the collection pits for the water coming from the basecourse and sensors continuously measured the outflow from the basecourse layers.
To spread the water on the pavement surface, the water delivery manifolds were used to apply a uniform layer of water whilst distributing the water on the pavement surface. One of the manifolds is shown in Figure 7.6. There were slot discs in the T-joint of the manifold that controlled the volume of water and the water film thickness over the pavement surface.
7.3 Results and Discussion

7.3.1 Water Inflow in Sub-Drains

The portion of water applied on the pavement surface which seeped into the pavement, was collected through perforated pipes on the inner circumference of the pavement into subsurface drain pipes which was then pumped out by submerged pumps. The amount of water pumped from the subsurface drains was recorded by flow meters which were fixed at the end of the discharge pipe of the submerged pump. Water was collected after the specified water level was reached in the sub-drain collection pipes. The horizontal bars in Figure 7.7 show the time duration between the pump out of water from the pits.

The water was allowed to run on the surface of the pavement for four hours and the data shows that the water that seeped into the basecourse kept discharging from the pavement for more than twelve hours. This shows the water that seeped into the pavement basecourse layer drained out at a slower rate to the collecting pit where it was pumped out and measured.

![Test 06, Subsurface Flow](image)

*Figure 7.7 Sub-surface drain flows from basecourse in different pavement sections*
The results in Figure 7.7 shows that significantly less water was collected through the subsurface drains from sections with a prime coat (i.e., section B, C and D) as compared to the sections without a prime coat (section A, E and F). Comparing the water discharged from various un-primed sections, the section ‘A’ drained the highest volume of water compared to sections ‘E’ and ‘F’. The section ‘A’ basecourse material grading was characterised by the least amount of fines resulting in the highest basecourse permeability than the rest of the basecourse materials therefore explaining the greater discharge volumes. The performance of the surface treatment of all three un-primed sections ‘A’, ‘E’ and ‘F’ was reasonably comparable and significantly worse than the primed sections. Thus, it can be assumed that approximately equal volumes of water had seeped into the basecourse layers for these un-primed sections.

The CAPTIF pavement for these tests had been recently constructed and these initial tests began prior to any accelerated wheel loading history over the chipseals. The quantity of water that seeped into the pavement basecourse layer through the chipseal shows that a chipseal surface over an unbound basecourse material may look like as if it has been constructed well. However, the chipseal had underlying construction quality problems (as can occur in the field) as it was unable to keep the surface water out under standard axle loading or even with no wheel loading.

### 7.3.2 Relative Change in Moisture through TDR Gauges

The TDR gauges were placed in the pavement so that the movement of moisture in the basecourse could be studied transversely as well as longitudinally from the point of water application. There were two stations in each section (shown in Figure 3) where TDR gauges 1-4 were located at the point of surface water application while TDR gauges 5-8 were located on the second station which was offset from the point of water application. In a three dimensional view (Figure 7.8), TDR gauges 1 and 2 at the first station and TDR 5 and 6 gauges at the second station, were nearly under the loading zone. TDR 3 and TDR 7 gauges were located in the middle of the base course layer and in the inner quarter of the track. The TDR 4 and TDR 8 gauges were located in the inner circle of the track.
According to the factory instructions, the TDR gauges used in the research require an individual calibration for each TDR for each type of soil/aggregate that they are placed within. The moisture data shown in Table 7.2 has been calibrated for only a sample of TDR gauges for all soil/aggregate types. The calibration equations of TDR gauges were obtained for each material and a correction factor was added to each TDR placed in the CAPTIF pavement after comparing the TDR output with Nuclear Densometer results.

The results of the TDR output are shown in Table 7.2 that are taken from the static tests i.e., there was no loading applied on the pavement. The water was applied on the surface at specified locations on each section for a time span of four hours and the moisture intrusion into the basecourse under two different surface treatments (primed and un-primed) was investigated. It was initially assumed that water would not get through the surface in the absence of wheel loading as two coat seals are usually considered as providing a water proof layer.
Table 7.2 Moisture data from TDR gauges placed in all sections

<table>
<thead>
<tr>
<th>Section</th>
<th>Moisture From TDR (%) **</th>
<th>Static Test 06</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>TDR 1</td>
<td>TDR 2</td>
</tr>
<tr>
<td>A</td>
<td>Initial</td>
<td>3.12</td>
</tr>
<tr>
<td></td>
<td>Max</td>
<td>11.07</td>
</tr>
<tr>
<td></td>
<td>Min</td>
<td>2.49</td>
</tr>
<tr>
<td></td>
<td>Rel. Change</td>
<td>8.57</td>
</tr>
<tr>
<td></td>
<td>Time Lag*</td>
<td>00:03:23</td>
</tr>
<tr>
<td>E</td>
<td>Initial</td>
<td>4.12</td>
</tr>
<tr>
<td></td>
<td>Max</td>
<td>10.75</td>
</tr>
<tr>
<td></td>
<td>Min</td>
<td>3.32</td>
</tr>
<tr>
<td></td>
<td>Rel. Change</td>
<td>7.42</td>
</tr>
<tr>
<td></td>
<td>Time Lag*</td>
<td>00:07:03</td>
</tr>
<tr>
<td>F</td>
<td>Initial</td>
<td>1.60</td>
</tr>
<tr>
<td></td>
<td>Max</td>
<td>2.09</td>
</tr>
<tr>
<td></td>
<td>Min</td>
<td>1.60</td>
</tr>
<tr>
<td></td>
<td>Rel. Change</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>Time Lag*</td>
<td>00:00:02</td>
</tr>
<tr>
<td>B</td>
<td>Initial</td>
<td>1.95</td>
</tr>
<tr>
<td></td>
<td>Max</td>
<td>1.95</td>
</tr>
<tr>
<td></td>
<td>Min</td>
<td>1.42</td>
</tr>
<tr>
<td></td>
<td>Rel. Change</td>
<td>0.53</td>
</tr>
<tr>
<td></td>
<td>Time Lag*</td>
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</tr>
<tr>
<td>C</td>
<td>Initial</td>
<td>2.15</td>
</tr>
<tr>
<td></td>
<td>Max</td>
<td>2.64</td>
</tr>
<tr>
<td></td>
<td>Min</td>
<td>2.15</td>
</tr>
<tr>
<td></td>
<td>Rel. Change</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>Time Lag*</td>
<td>00:00:10</td>
</tr>
<tr>
<td>D</td>
<td>Initial</td>
<td>3.46</td>
</tr>
<tr>
<td></td>
<td>Max</td>
<td>3.46</td>
</tr>
<tr>
<td></td>
<td>Min</td>
<td>2.66</td>
</tr>
<tr>
<td></td>
<td>Rel. Change</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>Time Lag*</td>
<td>00:00:02</td>
</tr>
</tbody>
</table>

* Time lag is for maximum value. It is time after start of test. Its units are days: hours: minutes (dd:hh:mm)

** Due to signal variation in all TDR gauges, there is a tolerance of 0.50-0.60 % change in moisture.
However, if vertical movement of water through the chipseal surface to the basecourse layer occurred, then there would be an associated rapid change in the moisture levels detected in the basecourse layer during the time of water application. This is especially so as the water application was fully controlled in the CAPTIF facility and no water would be present at the surface after the water application had stopped. The moisture movement observed from the TDR gauges show some interesting results:

- The TDR gauge results in Table 7.2 for un-primed sections (A, E and F) showed that water was getting into the pavement bottom layers even in the absence of standard axle loading which was planned to be applied in the presence of surface water.

- The results of TDR 1 and TDR 2 in section A and E demonstrate the maximum increase in the moisture within 4 hours of application which represents an almost immediate wetting of the pavement basecourse layer. These TDR gauges were directly under the point of wheel load application of water on the respective sections. The relatively quick change shows that water increased in the basecourse layers due to vertical movement of water which is possibly due to the presence of cracks in the seals.

- In section F, there was no immediate change in the TDR 1 and 2 gauge moisture data (there is a tolerance of 0.5% with moisture levels in the pavement layers). This means that the water was not getting through to the TDR gauges in this section. One of the possible reasons is that the permeability of BC2 in section F was the lowest in comparison to the other two basecourse materials. This may have prevented the rapid vertical movement of moisture to the depth of the TDR gauge. The other possible reason may be a good seal condition near the point of water application which may have prevented the vertical intrusion of moisture into the basecourse layer.

- In the primed sections (B, C and D) the relative change of moisture was minimal in the TDR 1 and TDR 2 gauges which were directly under the point of water application. In primed sections there was an additional bitumen seal under the applied surfacing. This additional layer provided a further barrier for the vertical movement of moisture through the chipseal surfacing to the basecourse layer. The signals of TDR 4 in section B and C show only a very little increase in moisture after 4 hours which may be due to moisture coming laterally from a distance or from drains near these TDR gauges as they are located near to the surface and the subsurface drains.
For the longitudinal movement of water in the base course, the TDR gauges show a moisture change after some time when water was applied on the pavement surface. The following are results showing water has moved laterally in the basecourse layer:

- The movement of water in longitudinal sections can be seen in the un-primed section A from the results (Table 7.2) of TDR gauges 5, 6, 7 and 8 which are longitudinally offset from the point of water application. The results of section A show that water travelled longitudinally in the basecourse to increase the moisture around the TDR gauges which were not directly under the point of water application. Similarly, TDR gauges 3 and 7 in section E (Table 7.2) show an increase in intensity of signals after some time when the water application had stopped.

- However, there were no horizontal movements of moisture in the primed sections of the CAPTIF pavement except in TDR gauge 8 of section B where the TDR gauge signals increased after a long duration of water application. This moisture may have infiltrated into the basecourse from cracks in the seal which were away from the location of TDR 1 and 2 (as there is no prominent change in their signals). This moisture may have travelled through the basecourse to the point where the TDR gauge 8 of section B then detected the change in gravimetric moisture.

The wet and dry cycle was prominent in the unbound basecourse materials BC1 and BC3 under un-primed chipseal surfacing as the intrusion of water was more visible in these sections. The TDR gauge readings of section A and E from Figure 9 show different patterns of wetting and drying of the same materials in the CAPTIF pavement.

- In section A, that has the most permeable basecourse (BC1), the TDR gauges 1 and 2 that were located directly under the point of water application showed an immediate wetting and drying of the basecourse. The readings also show that it took 4-5 days after water application had stopped for the basecourse to dry back to the original moisture levels.
Figure 7.9 Moisture data obtained from TDR ‘1’ and ‘2’ of Section ‘A’ and ‘E’
In section E, the TDR gauges 1 and 2 also showed the immediate wetting and then drying effects for 6 days of the surface runoff being applied for 4 hours per day. The moisture levels in this case did not recover their original moisture levels after 6 days due most likely to the materials very low permeability.

There was a drying trend in the TDR gauge readings of section A and E that showed a decrease in moisture levels. The fast drying trend in section E and lesser in section A may be due to the grading difference in these materials. A greater proportion of fines in a material resist the movement of water in the basecourse and this is clearly shown in Figure 9.

### 7.3.3 Failure of Pavement Sections

After the CAPTIF pavement was tested for static water application (applying water on the pavement surface without loading) and then subsequently and sufficiently dried, a number of laps (1054) of standard axle loads were applied on all pavement sections without the application of water on the surface with little effect. However, with the application of water on the chipseal surface coupled with standard axle loading at 40 km/hr, the un-primed sections failed very quickly. The failure criteria was a rut of more than or equal to 20 mm. Section A was the first to fail after only 132 further laps with water and loading being applied (i.e. 1186 cumulative laps that included the 1054 laps without water application). After the repairs on section A, standard axle loading was applied again for a further 215 laps (a cumulative lap total of 1401 laps) and section E failed. The last un-primed section F failed after a further 574 laps and a cumulative lap count of 1975.

The pavement was repaired after every section failed with a thin asphalt layer and allowed to dry. With the further application of water on the surface, the chipseal sections with the prime coats were expected to last significantly longer. However, primed sections B, E and F failed after cumulative laps of 2244, 3501 and 4917 respectively. It is considered that less water seeped into the unbound basecourse layer of the primed sections but the surface water still caused premature failures to the primed sections as well.

In the un-primed sections and primed sections, the basecourse that lasted longer with the chipseal was the BC2 section. The BC2 section was categorised as a medium moisture sensitive basecourse with the least permeability. The reasons for considering BC2 as the medium moisture sensitive material were the grading and maximum aggregate size (20 mm instead of 40 mm).
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7.4 Conclusions

This chapter presents the findings from introducing water on different surfaces constructed on an accelerated pavement test track (the CAPTIF facility in Christchurch, New Zealand). The aim of the research was to investigate the ability of different surface techniques to keep the basecourse layer dry. It also looked at the hydrological performance of different grading in the basecourse materials.

The research was successful in developing a testing methodology that quantified the amount of water entering the pavement through the surface with and without loading. This was a significant outcome, because for the first time, it was possible to demonstrate the effect of water entering the pavement through different surfacing techniques.

The most surprising observation of the research was that unprimed chipseals are not as water proof as originally expected. The research demonstrated that at times, poorly constructed chipseals may appear robust but when it is subjected to loading, it fails very rapidly under elevated moisture conditions in the basecourse layers. Water can seep into the pavement structural layers through that chipseal even under no wheel loading. Furthermore, basecourse layers with insufficient permeability may lead to water being retained within the layer as opposed to it being discharged from the layer. When such a pavement is subjected to traffic, rapid failure is expected given the complete lack of bearing capacity of the soaked material.

The water proofing of chip sealed low trafficked volumed roads can be improved by applying prime coats. This involves spraying of a bituminous layer directly on the basecourse before laying the chipseal layer. This creates an extra water proofing membrane beneath the chipseal that helps to keep the water out of the pavement and in this case prevent the balling of the bitumen as was seen in the un-primed sections.

It has also been shown that even with an additional prime coat layer, water can still penetrate pavement layers, especially under traffic loading. Therefore, the combination of applicable surface technologies and the specification of the basecourse materials is very important in the construction and design of durable pavement surfacings, especially in high rainfall areas.

From the results of the TDR gauges, it was concluded that drying of the basecourse depends upon the amount of water entering the pavement and the length of the drainage path. It will take longer to dry the material present at the end of the drainage path compared to the material present in the beginning.
It was also concluded after dynamic tests, performed on the CAPTIF track in the presence of a water film over the chipseal surface, that the BC2 basecourse material performed the best in highly wet conditions. This material was characterised as a medium moisture sensitive basecourse based on the permeability, grading and maximum aggregate size. It was concluded that an increase in moisture significantly reduces the performance of unbound flexible pavements.

7.5 References


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Chapter 7 What happens when it rains? Performance of unbound flexible pavements under accelerated pavement testing

CHAPTER 8.

COMPARING RESULTS BETWEEN THE REPEATED LOAD TRIAXIAL TEST AND ACCELERATED PAVEMENT TEST ON UNBOUND AGGREGATE

Abstract

Unbound granular materials (UGM) are extensively used as basecourse materials around the world as they are capable of bearing relatively high traffic loads and are an economical option in comparison to bound materials. The unbound materials performance as basecourse determines the life cycle costs of a pavement. The extent to which the Repeated Load Triaxial test can predict the performance of unbound granular materials in the laboratory is an important parameter for the road designers. Moreover, the performance of the unbound basecourse materials depends upon the moisture conditions when they are being loaded, gradation curve of the material, in-situ density, permeability, and the nature of the aggregate fines (clays). There is a need to find the factors that cause the variation in the performance of the materials both in the laboratory and in-field pavement conditions to enable appropriate selection and use. This research utilises accelerated pavement tests (APT) alongside repeated
load triaxial (RLT) tests to test differently graded unbound granular materials at higher moisture contents. The objective of the research is to find the similarities and contrasts in basecourse material in both tests and to find the root causes of variation in the aggregate performance. If results of the RLT test can truly represent the results from full scale APTs for basecourse materials then the pavement materials industry can use the much lower costing RLT tests with confidence knowing it is representative of performance tests from APTs. The accelerated tests are performed on test pavements at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) with the introduction of surface water which replicates typical rainfall events. An increase in moisture in the basecourse layers is observed during the APTs due to entry of surface water through the surface seal which allowed the performance of the basecourse material at higher moisture contents to be assessed. The RLT tests were conducted in parallel with the same basecourse materials at higher moisture conditions. The relative performance ranking of APTs and RLT tests is found to be the same for some materials however it differs in some cases. The similarities and differences in the rankings from both procedures are highlighted and the causes of these similarities and differences are inferred and discussed.

8.1 Introduction

All laboratory based and accelerated loading tests on aggregates aim to quantify and rank the material behaviour of the aggregate in order to make more appropriate selection of materials for varying traffic loading and environmental conditions. The most effective manner to test the impact of traffic loading and environmental conditions on pavement configurations remains the simulated situations in full scale accelerated pavement tests (APT) (Al-Qadi et al., 2011; Patrick et al., 2010; Wu et al., 2010). However, these tests are both expensive and time consuming compared to laboratory tests that are in comparison relatively inexpensive. However, both methods remain only a simulation of actual behaviour of the material. The main question thus is the confidence that engineers have in laboratory test results as an indicator of actual engineering performance of pavement layer materials. This research investigates the performance of unbound granular aggregates used in basecourse layers of low traffic volumed chipseal roads on the basis of laboratory and APT results.

The primary failure criterion for unbound granular pavement is wheel track rutting or permanent deformation (Arnold, 2004; Werkmiester, 2003). The most commonly used test for deformation resistance of aggregates is the repeated load triaxial (RLT) test (Pérez and
Chapter 8 Comparing Results between the repeated Load triaxial test and accelerated pavement test on Unbound Aggregate

Gallego, 2010; Theyse et al., 2007; Werkmeister, 2006). The actual loading conditions of the pavement material is simulated in the triaxial cell with water pressure applied on the cell (confinement stress) to represent the pressure from the soil matrix at a given depth, whereas the repeated wheel load is simulated by the vertical loading plate that applies an impact load at a given frequency. This research is therefore a useful validation between the RLT and the APT results, especially for the unbound material under varying moisture conditions.

This chapter is an extract of findings from a larger research that also included:

- The effect of mineralogical makeup on the performance of the UGM materials in high moisture conditions (Hussain et al., 2011d),
- the performance of Unbound Granular Material (UGM) materials in the RLT at different moisture and stress states (Hussain et al., 2011a; Hussain et al., 2011d); and,
- the performance of these basecourse materials in relatively low traffic volumed chipseal roads under standard axle loading and surface runoff conditions (Hussain et al., 2011b; Hussain et al., 2011c).

8.2 Objectives and Scope

The main objective of the research is to validate the results of the RLT test on unbound basecourse materials in comparison to the APT test results undertaken at the CAPTIF facility on the same unbound basecourse materials. The factors that cause the variances in the observed behaviour of the unbound basecourse materials in the RLT and APT tests are highlighted.

8.3 Materials and Methods

In this research, three different graded basecourse materials were selected from the greywacke rocks of the South Island of New Zealand. The grading curves of these materials are shown in Figure 8.1. The mineralogy of the materials was found to be similar and to not adversely affect the performance of either the APT or RLT test (Hussain et al., 2011d). The materials are categorised as Coarse Graded (CG), Medium Graded (MG), and Fine Graded (FG) for identification purposes.
The three greywacke materials were all classified as A-1-a according to the AASHTO Classification system; and passed the basic qualification tests such as the cone penetration limit, clay index, and sand equivalent tests (Standards NZ, 1991). As the permeability and density properties are a function of the gradation of the material, these two properties can play an important role in the engineering performance behaviour of the materials. The materials were tested for permeability (BS 1377-6:1990, 1990) and the 95% of maximum dry density (MDD) (TNZ T/15, 2010) and the results are shown in Figure 8.2. The figure clearly shows results that are expected as the material having the highest compacted density (MG) results in the lowest permeability and the lowest density material (CG) achieves the highest permeability.
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The pavements for full scale APT are constructed in a concrete circular tank with a central circumference of approximately 59 meters. The circular track is divided into 59 stations which are 1m apart and are identified accordingly by their station numbers. Three basecourse materials were laid in six segments under chipseal layers and were labelled from A to F. Three sections including one of each basecourse material type were constructed with an extra prime coat sprayed before the laying of the chipseal layer, called primed sections (B, C, and D). The rest of the sections were constructed without a prime coat and labelled as un-primed sections (A, E, and F). To measure the vertical deformation of the pavement under the moving standard axle loads, emu coils were installed at a station in each APT CAPTIF test segment. The emu coil details is shown in Figure 8.3. To measure the final end of life deformation at the surface, the surface profiles are taken at the end of each section failure.

The test experiment was developed for the application of water on the surface of the pavement sections. Six water tanks were placed at the corner of each pavement segment to apply water on the surface of the pavement during APT simulating surface water runoff. The water was collected from the surface into the underground concrete tanks from where it was pumped back to the water tanks for recycling the water usage. The subsurface water was collected and measured separately in side tanks as it was important to know the proportion of water that ran off the surface in comparison to that which entered the pavement through the surface layer. This methodology is further explained in Hussain et. al. (2009).
Along with the full scale accelerated pavement testing at CAPTIF, these materials were tested in the laboratory. The RLT tests were conducted on the CAPTIF pavement unbound materials. A staged approach was used for the RLT tests by varying the confinement and vertical stresses for each stress state. The New Zealand standard (TNZ T/15) was followed for the RLT test methodology. Six stress states and 50,000 loading cycles were applied at each stress state with a frequency of 4Hz, and the samples were compacted by vibratory hammer to achieve MDD. All of the samples were prepared at optimum moisture content conditions. The materials were then tested at optimum moisture drained (OM), saturated drained (SD), and saturated un-drained (SUD) conditions.
8.4 Results and Discussions

8.4.1 Results from full scale APT

The pavement sections were subjected to moving standard axle loads at a speed of 40 km/hr for initially 1000 loading cycles in dry conditions and then subsequently loaded with water flowing over the surface for the remainder of the APT tests. The presence of the water depth flowing over the pavement surface represents extreme weather conditions that results in water being forced into the surface layer under wheel loads (Ball et al., 1999; Towler and Ball, 2001). In order to continue testing all sections to failure, the failed sections were repaired by reconstructing with an asphalt layer in that section until all sections had failed.

The emu coils were installed to capture permanent deformation at various positions and depths in the constructed base layer. The emu coil data from each pavement section and position were collected and analysed (Figure 8.3). The coil pair numbers 1, 2, 3, 8, 9, 10, 12, 13, and 14 were placed vertically within the basecourse layers while coil pair numbers 4, 11 and 15 were placed as the uppermost vertical pairs within the subgrade.

It was found that there was significantly less deformation below the top $1/3$rd of the basecourse compared to the bottom $2/3$rd of the basecourse layer. This observation was supported by cross sectional profiles taken at failure of each of the sections. Typical cross sectional failures are shown in Figure 8.4 that illustrates the movement of material (shoving failure) in the upper part of the base course. The material heaves up alongside the rut as opposed to a deep seated rut that is associated with a compaction (layer densification) or failure of the deeper underlying layers.

The failure type which was observed in the CAPTIF test section pavements is typical of shear failure, within the basecourse layer which is often observed on road pavements that fail due to drainage inefficiencies.

The UGM materials in the APT have two types of wearing course (i.e., chipseal surface with and without a prime coat). Hence, the two surfacing types should be separately analysed when to comparing the relative performance of the materials. The failure sequence of the CAPTIF sections were:

1. All un-primed sections failed first according to the following sequence:
   a. Coarse graded (1,186 load repetitions);
b. Fine Graded (1,401 load repetitions);  
c. Medium Graded (1,975 load repetitions);  

2. The primed sections lasted longer due to lower moisture levels within the pavement and failed according to the same sequence as the un-primed section:  
a. Coarse graded (2,244 load repetitions);  
b. Fine Graded (3,501 load repetitions); and,  
c. Medium Graded (4,917 load repetitions).

The primed sections lasted longer due to lower moisture levels within the pavement and failed according to the same sequence as the un-primed section. The priming treatment was applied to stabilize the pavement and improve its performance.

The results from the APT test thus suggest that the material fails according to the amount of water penetrating the surface and the pavement. Therefore the worst case in terms of pavement performance would be a permeable surface on top of a course graded base course as in this case the most of the surface runoff will be penetrating the pavement structure. Moreover, the increased moisture content will decrease the point to point particles friction by lubrication in flexible pavement that is responsible for transferring the load by grain to grain contact resulting in settlement of the unbound layer.

8.4.2 Results from RLT tests

The unbound granular materials were also tested in the RLT test at different moisture levels and drainage conditions. The permanent deformation in the RLT test results are shown in Figure 8.5. The staged RLT tests are performed on the UGM which means that the same sample is subjected to various stress states. The higher values of permanent deformation in the RLT test show that the material is showing greater deformation under repeated loading in specific stress, moisture and drainage conditions. A total of 50,000 loading cycles were applied on each stress state. Each graph shows a noticeable change in permanent deformation after every 50,000 loading cycles which is due to the change in stress state.
Figure 8.5 shows the test results completed on the respective material and moisture states. The MG material demonstrates lower deformation than the FG material at all three different moisture and drainage conditions. This is especially noticeable at later stages of the RLT loading stages. However, in saturated un-drained conditions, the FG material shows lower deformation than MG material for the first two stress states. The CG material shows the least amount of permanent deformation compared to MG and FG materials at the last stage of the RLT test (at higher stress states) for all moisture and drainage conditions.

The resilient modulus was derived from the RLT results and a comparison of these results is shown in Figure 8.6. The resilient modulus is obtained by dividing the applied deviator stress with the recoverable strain. The resilient modulus shows the flexibility and strength of the material and simultaneously links the ability of the material to resist plastic deformation. The higher resilient modulus shows that the material is more elastic and shows higher resistance to permanent deformation in the RLT test. Note that for Figure 8.6, the change in resilient modulus for the respective (50,000) loading cycles is due to the change in stress state of the sample. The MG material demonstrates a higher moduli than the FG material at all moisture...
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levels and drainage conditions. The CG material shows higher moduli compared to both MG and FG materials for all stress, moisture and drainage conditions.

From the permanent deformation results obtained from the RLT test, there is a clear categorisation of the test materials. The CG material shows the smallest deformation in all tested materials. The FG material shows the highest permanent deformation in all cases of moisture level and drainage when compared to CG and MG materials. From the permanent deformation results it can be inferred that the CG material can be categorised as the most resistant to permanent deformation followed by MG and then FG. Naturally the resilient modulus yields the same categorisation of the material’s behaviour.

8.4.3 Comparing APT and RLT Tests Results

A comparison of results from the APT and RLT tests is shown in Table 8.1. This table ranks the performance of the various aggregate gradings according to its failure sequence. Both the primed and un-primed results are shown for the APT. The results clearly demonstrate that although the primed sections have lasted slightly longer than the un-primed sections, the sequence of failure for the respective gradings is the same. The table also correlates the
results in terms of the failure sequence for the fine and medium graded material between the APT and RLT tests. In both cases the medium grade material performed the better.

### Table 8.1 Rankings of UGM in APT and RLT tests

<table>
<thead>
<tr>
<th>Material</th>
<th>APT Ranking Un-Primed</th>
<th>APT Ranking Primed</th>
<th>RLT Test Ranking</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>“Ranking”</td>
<td>Load Rep</td>
<td>“Ranking”</td>
</tr>
<tr>
<td>CG</td>
<td>3</td>
<td>1,186</td>
<td>3</td>
</tr>
<tr>
<td>FG</td>
<td>2</td>
<td>1,401</td>
<td>2</td>
</tr>
<tr>
<td>MG</td>
<td>1</td>
<td>1,975</td>
<td>1</td>
</tr>
</tbody>
</table>

Note: Ranking 1 represents the material with least deformation/ failing last, and Ranking 3 represents the material with highest deformation/ failing earliest.

However, an opposite trend was observed for the course graded material. During the APT loading, the course graded sections failed first, whilst it lasted the longest during the RLT test.

It is believed that these observations could be explained by the following factors:

- The main difference in the performance of the course graded material is related to the difference in confinement stress between the RLT and APT methods. During the RLT test, the confinement stress is relatively high, while the permanent deformation is measured. This permanent deformation is similar to the rutting failure observed in CAPTIF pavements. Both the TDR readings and the visual observation of the failures confirmed that the CG material failed due to shallow shear under the APT loading. Shallow shear normally happens in the upper pavement layers or on the side of the road in the absence of lateral support. This type of failure is prevented during the RLT test due to the increased confinement stress. Therefore the other options of less confining stresses may be tested which can represent the extreme loading in wet conditions at the top of basecourse where vertical stress is highest and confining stresses are least;

- The grading and the consequential basecourse aggregate permeability govern the behaviour of the aggregates tested within the APT. With an increase in moisture entering the pavement, the failure resistance of the material decreases;
The Medium graded material achieved a much higher density compared to the fine graded material. This increased density is believed to be the result of the better performance of the MG material when compared to the fine graded material.

8.5 Summary and Conclusions

This chapter documents the comparative results between the accelerated loading of pavements and the repeated load test method widely used in laboratories around the world. The aim of this comparison was to investigate whether the RLT can sufficiently quantify the behaviour of material used for the construction of road pavements. Of particular interest for this research was the behaviour of the material with different grading distributions and varying moisture conditions.

The following conclusions can be drawn from the research:

- The boundary conditions in the RLT do not represent the true failure for the materials in the field which can fail at low confinement pressures i.e., the materials with low density and compaction and in the case of this research the coarse graded material. As the material fails at low confinement pressure, the shear failure often occurs due to shear at the top layer and at higher moisture contents and much more quickly (i.e., at less traffic loads (ESAs) to point of failure). However, in the RLT test used in this research, the loading piston diameter is equal to the sample diameter and the confinement stress is the same from the top to the bottom of the specimen which does not allow the material to move at the sides or the top of the sample. Hence, the coarsely graded materials that have lesser maximum dry density may last longer in the RLT test than the APT test. The main objective of RLT test is to simulate the field conditions in laboratory, therefore, the other options of stresses in the high moisture conditions should be used, i.e., less confining stress along with maximum vertical repeated loading which simulates the top of the basecourse. Moreover, the size of the sample also affects the results and bigger size sample can simulate the conditions more realistically.

- The grading of the UGM, which controls the density and permeability, plays an important role in the relative performance of the UGM under high moisture conditions in both the APT as well as the RLT tests;
- Both the RLT and the APT provided similar results for the fine and medium graded materials. The density achieved with the medium graded material gave it a superior performance due to lower permeability and greater deformation characteristics;

- The course graded material (CG) gave contradictory results between the RLT and the APT. For the RLT test, the course graded material outperformed the fine and medium graded materials. However, it performed the worst under the accelerated tests (APT). The main failure mode for the APT tests was shallow shear failure which is more prone to occur for course graded material. However, the increased confinement stress during the RLT prevents this failure mechanism during the laboratory tests.

It can therefore be concluded that the RLT test and the methodological procedures set out in the TNZ T/15 specification are not appropriate for all material types and moisture conditions. It is important to realise that the RLT test currently only simulates a deformation type failure and does not simulate other failure mechanisms such as shallow shear and/or cracking.

The experiment also confirmed well-acknowledged road building principles such as well graded materials that are appropriately compacted are the strongest materials under wet conditions. Also, the value of keeping the road pavement material as dry as possible was emphasised and this results in an increased pavement life.

### 8.6 References


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CHAPTER 9.

RESEARCH SUMMARY AND CONCLUSIONS

9.1 Research Problem

The strength of unbound granular materials (UGM) is an important aspect in road design where these materials control the design life of the pavement. UGM are common materials in road construction worldwide and mostly used as basecourse and subbase materials. These UGM layers act as the principal load bearing layers in low volume roads where a thin asphalt layer or chipseal layer is laid over these unbound layers. The strength of the unbound granular materials depends on a number of factors such as stress magnitude, stress reorientation, gradation, fine content, aggregate, drainage, moisture, maximum dry density/achieved construction density, maximum particle size, and number of loading cycles. In changing environmental conditions, the moisture variations play an important role in controlling the performance of unbound layers. The increased moisture decreases the strength of unbound materials and can cause early failures in low volume roads. A number of recent failures in New Zealand have been observed (McKay’s Crossing (Wellington) on State Highway One (SH1) where a relatively good quality basecourse was used, and another around Taupo (also on SH1) where rehabilitation failures occurred where traffic volumes were lower but the marginal basecourse material was known to be moisture sensitive. Both of these failures occurred within the pavement basecourse layers after it was observed that moisture had infiltrated the unbound layers (Alabaster, 2008).
Chapter 9 Research Summary and conclusions

It is important to investigate the performance of unbound granular materials with increasing moisture levels as there is a high likelihood of water infiltrating unbound layers. The better understanding of UGM deformation behaviour as moisture levels increase can facilitate the design process of the materials and layers. These investigations help us to understand what it is that makes UGM moisture sensitive and these materials and their engineering performance can then be improved and/or designed accordingly. The materials that are less sensitive to moisture can be used in high rainfall areas where there is a higher probability of water infiltrating the unbound layers.

This research highlights the behaviour of unbound granular materials in elevated moisture conditions and has shown how mineralogy and physical properties of materials affect the permanent deformation of UGM at high saturation levels. The permanent deformation in the basecourse layer results in the rutting of pavements especially in chipseal pavements when there is free runoff over the surface. The permanent deformation which is commonly known as rutting by densification within the basecourse layer is the primary layer failing structurally in thin pavements. The influence of the level of saturation of basecourse materials is very important and it can dramatically change the load bearing capacity of thin pavements. Hence, the basecourse material gradation and surfacing of thin pavements are very important to be selected appropriately considering the environmental conditions of an area.

9.2 Research Aims and Objectives

This research has aimed to understand the UGM behaviour used in the basecourse layer of pavements in wet conditions. Therefore the objectives of the research can be divided into two main categories:

1. To understand how aggregates behave in increasing moisture conditions for different aggregate gradations and fine material contents;
2. To investigate how well UGM behaviour can be simulated in laboratory based performance tests compared to accelerated pavement tests. To understand this behaviour, in terms of the engineering performance of the pavement layers, there is need to understand the mineral composition of the materials and moisture / drainage Conditions. Moisture can react chemically with the constituent minerals of the aggregate mix and can alter the strength of the unbound materials. Moreover, the mechanical response of the material to the applied stresses needs to be understood
which can help to standardise the process of selection of unbound basecourse materials in wet conditions.

The research utilises the results from both accelerated pavement tests and laboratory investigations to address the following aims:

i. To understand the mineralogy of the unbound basecourse materials i.e., identify the differences in the selected materials with reference to dominant clay minerals;

ii. To investigate the effect of change in the clay mineral proportions on permanent deformation of the unbound basecourse materials;

iii. To consider the effect of gradation and stress conditions on permanent deformation of unbound granular materials at varying moisture;

iv. To improve the application of existing statistical models to predict permanent deformation of unbound materials at different stress states and moisture levels;

v. To evaluate the response of thin flexible pavements with varying unbound granular material properties under wet conditions in terms of pavement life and mode of failure.

### 9.3 Understanding Material Behaviour

#### 9.3.1 Influence of Mineralogy

The materials selected for the basecourse layers of the CAPTIF pavements were selected from South Island of New Zealand sedimentary geological sources, in this case from crushed river gravels. All three of the materials were greywacke aggregates and were subjected to XRD testing to identify the clay minerals present in the materials. The main purpose of the XRD testing was to find the differences in mineralogical composition of the selected materials which can affect the engineering performance of the materials. The other objective of these experiments was to find the presence of certain minerals in the materials that can weaken the material in wet conditions.

The XRD tests were undertaken on two different size portions of the unbound materials, i.e., the full aggregate proportion and the fine particle fraction less than 75 μm. It is assumed that when the aggregates are produced by crushing from one geological source, the fine particles are generated due to the crushing process. Hence, it is expected that there would be no difference in the mineralogy of the aggregates and fine particles, used in a pavement taken
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from the same source. However, due to weathering of the fine particles, clay minerals can convert into new minerals which are different from the aggregates, e.g., chlorites convert into smectite minerals which are more water absorbing and can cause accelerated deterioration in the unbound layers.

There were two types of tests carried out on the CAPTIF basecourse materials i.e., random XRD tests and oriented XRD tests. The random tests were undertaken to quantify the weight percentage of the minerals present in selected materials. The oriented tests were more targeted towards the identification of clay minerals. The unbound material samples from aggregates and fines were prepared such that the orientation of the particles is the same. The standard XRD test was undertaken by subjecting the samples to three different types of treatments: No treatment; Glycolated; and Heated to 550°C. The purpose of undertaking these treatments in the stated order is that in the first stage, the general identification of minerals is carried out and in the second stage with glycolation, the clay minerals which can expand on contact with water can show changes in the peaks of the XRD results. In third stage with heat, various minerals can collapse and this can be demonstrated in the XRD results with peaks that disappear between the stages which helps to confirm the presence of certain clay minerals.

From the results of the random XRD tests on the three CAPTIF aggregate materials, the dominating minerals were Quartz and Albite which consisted of more than 85% weight percentage. The other minerals found in relatively lower percentage were Chlorite, Illite and Kaolin. The Quartz and Albite are the minerals which impart strength to the materials and make the matrix of the aggregates whereas the Chlorite, Illite, Kaolin and Smectite are the minerals which are present in the veins of the aggregates and are relatively more prone to react with moisture. For a more detailed study of the moisture susceptible clay minerals, the oriented tests results were analysed. It was found that along with Chlorite, and Illite, Smectite minerals were also present in the materials. Smectite is known to be highly moisture susceptible and was present in the tested materials.

In such small samples, only the relative proportions of these weak clay minerals can help to infer the mineralogical composition of the materials. To find the relative proportions of these clay minerals, the graphs of the CAPTIF materials were plotted for oriented XRD tests and the area under the curve for clay minerals was calculated. The relative percentage of each clay mineral (Smectite, Chlorite, and Illite) was calculated which showed that these minerals
were found almost in the same proportions for all of the CAPTIF materials. The material for the CAPTIF tests originated from one geological source, with one of the test materials artificially modified with additional clay added from the same geological area. It was therefore expected that the mineralogical difference between the test samples would not vary significantly. The slight difference in material composition was confirmed using the XRD tests, however, as the three aggregate samples used in the CAPTIF tests were geologically so similar, it was not possible to test the impact of mineralogy on the engineering performance of wider unbound granular materials used in New Zealand in a meaningful manner.

To determine the effect of clay mineral proportions on the permanent deformation of unbound granular materials, a few more samples of unbound basecourse materials were selected from greywacke sources in the North Island of NZ. These aggregate samples were expected to be different in mineralogy from the CAPTIF materials which were taken from the South Island of NZ. The same methodology was followed for the North Island unbound granular materials as for the CAPTIF materials. Two tests methods (XRD and RLT) were used to compare the mineralogical differences and their respective performance. To test the engineering performance of the CAPTIF and North Island Materials in both dry and wet conditions, staged RLT tests were conducted. Only one material was selected from the CAPTIF aggregates for comparison purposes. The CAPTIF selected aggregate and the North Island materials had similar particle size distributions. The samples were compacted at optimum moisture content to achieve maximum dry density and were then tested at two moisture levels: optimum moisture level; and a moisture level higher than optimum. A staged RLT test was conducted in which the same sample is subjected to different stress states.

In the North Island materials, the amount of smectite mineral, which absorbs water and expands in volume, was found to be in relatively higher proportions than the Chlorite, and Illite, minerals, unlike the CAPTIF aggregates. The presence of smectite minerals in higher proportions shows that the North Island material selected in this research can show instability in wet conditions under applied loading.

It was observed from the RLT test results that the CAPTIF material and the North Island materials both demonstrated higher permanent deformation in moisture conditions higher than optimum. Moreover, the relative change in permanent deformation for the CAPTIF material at optimum and moisture higher than optimum was much lower as compared to the same in the North Island materials. One of the reasons for the relatively higher deformation in
the North Island materials is the presence of smectite in relatively higher proportions than the other minerals which has the property of absorbing moisture and can increase in volume which reduces the load carrying capacity of the unbound basecourse material. The research results demonstrated that mineralogy of materials obtained from the same geologically sourced rocks but different locations can differ and this difference in mineralogy can change the engineering performance of the unbound basecourse materials in different moisture conditions.

9.3.2 Influence of Particle Size Distribution

With a better understanding of the influence of the mineralogy on the performance of the aggregate, the next aspect to consider was the influence of grading. As the unbound basecourse materials from the CAPTIF pavement sections had approximately the same clay mineral compositions and they were all obtained from the same greywacke source rocks, the major difference in the materials was their particle size distribution (psd). This distribution varied from Talbot’s constant (n) value of ‘0.37’ to ‘0.50’. To find the difference in performance of these materials at different saturation levels and in varying psd gradation, the materials were subjected to laboratory based RLT tests in different drainage conditions.

To observe the effect of moisture change with change in stress, samples were subjected to the same stress states at varying saturation level and drainage conditions. The staged RLT tests were undertaken on the CAPTIF materials in which the same sample is subjected to various stress stages. Six stress states were applied on each sample and ‘50,000’ loading cycles were applied at each stress state. The samples were compacted at optimum moisture level to achieve their maximum dry density and then were subjected to three different testing conditions: saturation level at optimum moisture in drained condition; approximately saturated and drained condition; and approximately saturated and un-drained condition. The effect of moisture with varying stress was analysed by comparing the same material being subjected to the same stress level in different saturation and drainage conditions. The effect of moisture with varying gradation was observed by comparing the results of the materials at the same saturation and drainage conditions.

All CAPTIF materials at the highest stress state resulted in the maximum permanent deformation in saturated un-drained conditions and the least in optimum moisture drained conditions. The increase in permanent deformation with an increase in saturation can be explained by the lubrication of the aggregates, and the development of pore water pressure.
within the materials that can then result in a reduction in effective stress. The lubrication of the aggregates helps the grains to rotate and slide against each other that then cause further compaction. Moreover, the fine particles in the material become plastic further helping the deformation process. The increase in saturation creates pore water pressure in the material which reduces the effective stress, which is responsible for the strength of the specimen. Hence, when the stress is higher on the materials, the material becomes weak at higher saturation levels resulting in greater permanent deformation.

However, the permanent deformation behaviour in different saturation and drainage conditions was different for certain materials at lower stress states as compared to the highest stress state. The CG material showed relatively less deformation at saturated un-drained conditions than saturated drained conditions for the first five stress states. The permanent deformation of FG material at saturated drained and saturated undrained conditions was the same until the third stress state. For the rest of the stress states the FG material showed greater permanent deformation with increasing saturation. The MG materials showed higher deformation at saturated undrained condition than the saturated drained condition at all stress states.

The major cause of the relatively lower permanent deformation for CG material at saturated un-drained conditions than the saturated drained in the RLT test is thought to be the combined effect of particle size distribution, stress state and drainage conditions. It can be inferred from the results that at lower stresses in the un-drained RLT test, the water was able to absorb the applied cyclic loading due to two possible reasons: first is the coarser gradation of the CG material and secondly, the volume of the sample was not decreasing due to the un-drained conditions. Hence, the height of the sample in the saturated un-drained condition showed a lower decrease than the sample compacted in saturated drained condition. But this case is specific for the lower stress states. The magnitude of pore water pressure at lower stress states is less and cannot cause the increased vertical deformation. However, the removal of water in the drained conditions causes the reduction in height more than the undrained conditions. These results show that a coarser gradation can delay the development of high pore pressures that cause early failures due to more inter-particle spaces.

The results of RLT tests on MG and FG materials demonstrate and therefore infer higher deformation at saturated undrained conditions than saturated drained conditions in contrast to the behaviour of CG material. The main reason for the greater deformation in saturated
drained condition is thought to be that MG and FG materials have the finer gradations in comparison to the CG material which comes in to play when undrained conditions are applied. The pore water pressure that is created in the MG and FG materials even at lesser stress states is considered to be enough to cause higher deformation in the specimen than in saturated drained conditions.

### 9.3.3 Modelling the UGM Behaviour

Previously developed nonlinear statistical models are fit to the RLT test results data by using non-linear regression curve fitting techniques. The models that included both the stress states and the number of loading cycles as input parameters were especially considered. After a detailed analysis it was found that the existing models were unable to predict the permanent deformation trend at higher stress states and at higher number of loading cycles.

The nonlinear statistical models that include both the stress and the number of loading cycles simultaneously as input parameters, presented in the thesis are the Werkmeister Model (2003) and Gidel Model (2001). Both the models showed good fits in terms of coefficient of determination (R Squared), Residual Standard Error and Akaike Information Criterion. However, the prediction of permanent deformation of these models with a higher stress state and an increased number of loading cycles was found to be less meaningful in this research. The data of only 50,000 cycles per stress state was used. The model predictions deviated significantly from the original data at higher numbers of loading cycles.

A new model was fitted to the RLT test results data which was obtained by a mix of the Perez (2010) and Gidel (2001) Models. The Perez Model only predicts the permanent deformation using the number of loading cycles. The Gidel model uses both the number of loading cycles and stress variation as the model input. The new model presented in this research has a better fit to the test data compared to models presented by other researchers.

### 9.4 Simulating Material Behaviour

#### 9.4.1 CAPTIF Tests

A full scale accelerated pavement testing was undertaken under surface water runoff and the influence of standard axle loading. The pavement sections were built in the CAPTIF facility and had a chipseal surfacing with unbound granular basecourse which is typical of low to medium traffic volumed roads in New Zealand. A total of six pavement sections were
constructed with three unbound basecourse materials and two chipseal surfacing options. The pavement sections were instrumented for measuring the moisture intrusion into the pavement layers lying under the chipseal and movement of moisture through the basecourse layer. For this purpose Time Domain Reflectometry (TDR) gauges were installed at different depths in the basecourse layers near the wheel path and on the lower side of the cross fall in each pavement cross section. Moreover, TDR gauges were installed at two stations in each pavement section. A set of emu coils was also installed at a station in each pavement section to determine the movement of the material under standard axle dynamic loading.

The results from these tests suggested that all chipsealed surfaces are much more permeable than originally expected. Furthermore, the results confirmed engineering expectations that unbound granular materials do not perform well under high moisture conditions.

It was found that moisture intruded through the chipseal layer through imperfections in the chipseal layer. The moisture intruding depth in the basecourse layer depended upon the density of the basecourse. The denser basecourse did not allow water to penetrate deep into the layer however in the less dense basecourse material, the depth of penetration was higher. This fact caused an earlier failure in the less dense basecourse material compared to the highly dense material. The density of the materials was linked with the particle size distribution as the coarser graded materials achieved less maximum dry density when compared to the relatively fine graded materials. It was observed that there was minimal deformation in the subgrade layer however, there was significant deformation recorded in the basecourse materials.

This demonstrated that when water seeps into the pavement, the load carrying capacity of the basecourse layers decreases to such an extent that in this research, the basecourse layers failed before ‘5,000’ standard axle load repetitions. Moreover, the failure was not deep as the bottom layers like the subgrade layer and the lower portion of the basecourse did not fail and was strong enough to bear the applied loading in the given conditions.

This research demonstrated that even new chipseal surfacings are far more permeable than expected. The water seeps in to the unbound layers through chipseals during a water surface runoff event even if there is no loading. The depth that the moisture can penetrate in an unbound layer depends on its gradation, compactness and permeability. During a rainfall event, water gets into the unbound layer where it is hoped that it then gets drained out. Drainage is also a primary function of the basecourse and subbase layers. However, it was
observed in this research under accelerated pavement testing conditions that the inflow of water into the basecourse was more than the outflow and moisture was trapped in the basecourse layer. It was found that the more compacted the basecourse the less the water penetrated into the layer and consequently the basecourse then lasted the longest. In contrast the other basecourse sections with higher permeability allowed water to penetrate deeper but still not drain through and therefore remained in the unbound layer and thereby caused an early failure. The water in the basecourse did not get enough time to drain through and the immediate loading then caused early failures in the basecourse pavement layer.

### 9.4.2 Repeated Load Triaxial Tests

The basecourse materials used in the CAPTIF experiment were subjected to laboratory testing and full scale accelerated pavement testing at high moisture conditions to examine the similarities and differences in the engineering performance of the materials. It was found that the performance ranking of two materials (MG, and FG) was the same in both the RLT test and full scale APT i.e., the MG performed better than the FG material. However, there was a clear difference in the performance of the CG material in the RLT and the full scale APT. The CG material was the least deformed material in the RLT test among the tested materials whereas in the full scale APT, it was the first material to fail.

It is inferred that the main reason for the different performance of the CG material in RLT and APT is its gradation. It seems that when the Talbots constant which is controlling the particle size distribution increased from 0.37 (FG material) to 0.43 (MG material) the performance of both materials was similar in both types of tests. However, when the Talbots constant increased to 0.50, the boundary conditions affected the performance as they were different in the RLT and the APT test results. In the RLT test, stress conditions does not allow material to flow on the sides of the specimen as the confining stress is the same throughout the height of the sample. However, in the full scale APT test, the confining stress at the top of the basecourse is less than at the bottom of the basecourse layer. This difference is believed to cause a failure at the top of basecourse layer in highly wet conditions in the APT tests but not in the RLT tests.

The density, particle size distribution, and permeability of the MG and FG materials are thought to be controlling the conditions in the APT. The materials were denser and offered higher resistance to the shear deformation as compared to CG material which had comparatively lesser density. The lesser permeability of the MG and FG materials did not
allow moisture to seep deep into the layer as compared to the CG material which also was the observed difference in engineering performance. In the RLT the materials were not allowed to deform under variable confining stress which resulted in the best ranking of CG material.

It can be inferred from the above discussion that there lies an optimum particle size distribution which controls the permeability and maximum dry density of material that causes a basecourse material to perform best at high moisture contents. From this research with limited data it is surmised that the Talbots constant (n) of 0.5 is too high for the materials to perform optimally in full APT testing hence the basecourse material Talbots constant (n) should remain between 0.4 to 0.45 to allow the basecourse material to perform best in elevated moisture conditions.

9.5 Conclusions

This research has been a combination of laboratory (RLT) and accelerated pavement testing (CAPTIF) that is very time consuming and expensive to undertake and as such it was not possible to undertake enough tests / data points that would satisfy normal statistical robustness. The conclusions made in this research are therefore undertaken on a limited data set and are therefore inferred from the data set and it is realised that the variance of the data is largely unknown and therefore statistical confidence cannot be assured. However, this is the nature of accelerated pavement tests and RLT experimental research that are both expensive to undertake and very time consuming and is the best that could be undertaken given the research constraints. The main conclusions from this research are:

The mineral composition of UGM can affect the ability to resist the permanent deformation that is induced by repeated loading in varying moisture conditions. (Objective – i and ii)

The mineral composition of unbound materials is an important parameter which can affect the performance of compacted unbound granular aggregate materials when they come in contact with moisture. The relative percentage of moisture susceptible minerals (such as smectite and chlorite) to the minerals forming the main matrix in the aggregates (quartz and feldspar) controls the performance of material in elevated levels of moisture The moisture susceptible minerals absorb moisture and expand resulting in reducing the load carrying capacity of the aggregate structure and compacted mix.
The mineral composition of the aggregate geological rock source can vary significantly with a change in location. (Objective – i)

The most commonly used aggregate for road construction in NZ, Greywacke, is a sedimentary rock that is different from one regional location to the next. The mineral composition of a specific geological type of rock changes with the change in area as in the case of greywacke rock whose mineralogy is found to be very different from the North Island and the South Island of New Zealand. Therefore the mineral composition of the aggregates is an important property to be investigated and understood before they are approved for construction of the pavement especially when considering the various climatic and drainage conditions of the area.

The strength of coarsely graded unbound material decreases suddenly when high stresses are applied in un-drained conditions. (Objective – iii)

The coarser gradation of unbound materials at lower fines levels has greater pore spaces in between the particles. At lower stress levels the traffic induced pressure is absorbed by the movement of water in these pores in un-drained conditions which prevents the development of high pore-water pressures. When higher magnitudes of traffic induced stresses are applied over CG material, the material is thought to be compressed and the particles can then slide and rotate which decreases the pore space between the particles and the pore-water pressure can then increase which reduces the effective stress and material shows sudden deformations. Moreover, in the un-drained condition, the water is not allowed to escape the specimen in comparison to the drained condition; where the water can drain out from the specimen being tested.

The increase in moisture above the optimum moisture content decreases the strength of UGM. (Objective – iii)

The increase in moisture of the UGM has been shown to cause an increase in the permanent deformation of the material which shows that the lubrication of aggregates allows them to move and compact more easily in wet conditions. Moreover, the fine particles that fill the matrix in between the coarse particles become plastic with an increase in moisture which weakens the inter-particle friction resulting in higher deformations under applied loadings. This result confirms practical experience with the only difference is that we can better understand the deterioration mechanism as a result of the research.
The new permanent deformation model for UGM in RLT test followed the deformation trend better than the existing model at higher stress states. (Objective – iv)

At higher stress states, the model developed through this research improved the prediction of existing models for the permanent deformation of UGM for the RLT test. Previous models were under predicting the permanent deformation at higher stresses for 50,000 loading cycles and were showing the trends which did not match the data while the new model developed shows a trend very similar to the trend shown by RLT test data.

A newly, well-constructed chipseal can have hairline cracks which cause the water to seep through during a surface runoff event even without traffic induced loading. (Objective – v)

It is concluded from the full accelerated pavement test that a newly constructed ‘good looking’ chipseal with minor construction faults can cause the surface runoff water to seep through it into the pavement structure. The subsequent loading during the surface water runoff event can cause immediate failures in such pavements. Moreover, the application of a prime coat before the laying of a chipseal extends the life of the pavement but the pavement fails after some time if loading continues on the pavement along with a water film over the pavement surface.

The MG unbound basecourse material lasted longer compared to CG and FG materials in both the un-primed and primed cases. The MG material had the maximum dry density and the least permeability in comparison to the others which prevented the moisture penetration into the layer in contrast to the other materials. As a result the moisture of the MG material did not increase for a while and the load bearing capacity lasted the longest in comparison to the other materials.

The boundary conditions in the RLT test categorise MG and FG materials as in the full scale CAPTIF test. However, the boundary conditions do not predict the same relative performance of the CG material in the RLT test when compared to the CAPTIF test. (Objective – v)

The boundary conditions provided in the RLT test tend to simulate the field conditions imposed on the unbound materials. The stresses applied in the RLT are equivalent to the stresses that a cylindrical element in the pavement undergoes under wheel loading. The boundary conditions were relatively good for predicting the relative performance of FG and
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MG material as the MG performed better than the FG material in the RLT test and the CAPTIF test. The Talbots constant for FG and MG were 0.37 and 0.43 respectively. However, it is conjectured from the limited data obtained from this research that the boundary conditions in the RLT test especially for CG material affect the test results and are not representative of field or APT performance. This research indicates that when the material is ‘coarse’ i.e., with a Talbots constant (n) of 0.5 or greater the compatibility of RLT results with accelerated pavement tests is significantly affected by the RLT test conditions. The CG material performs better when it has a high confining stress. However, the CG material can shear more easily in the case of a weak confining stresses. In the CAPTIF test, the CG material coarser gradation confining stress was not enough to hold the material in place and caused the material to move to the side at the top of the layer under standard axle load for elevated moisture conditions. But in the RLT test, there was no provision for material to move on the side from the top as the size of the sample and piston were equal and the confining stresses were constant throughout the depth of the sample. Hence the material lasted the longest in the RLT but failed the earliest in the CAPTIF test. The coarser gradation allowed the particles to move around more easily at higher moisture due to a dual action of lubrication of the particles and a high percentage of voids. This was not in the case of the MG and FG materials.

9.6 References

10.1 Recommendations

The recommendations out of this research are divided into two sections; recommendations related to the understanding the material behaviour, and recommendations for the experiments conducted to simulate the material under controlled conditions.

10.1.1 Understanding Material Behaviour

Recommendation 1: Mineralogy tests:

The information of mineralogy of the source rocks used as aggregated for road construction is important as it gives explains the behaviour of basecourse material at elevated moisture content. Of this the most important aspect is the mineralogy of the fines of the materials.

On the basis of this research it is recommended to include the X-Ray Diffraction test to the specifications for the material selection for the unbound layer in the pavements. Most of the material used in NZ for road construction consists of Greywacke material, which is a sedimentary rock with highly variable properties. Understanding the specific make-up of the minerals largely explains its behaviour in wet conditions.
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If the weight percentage of moisture susceptible minerals (such as smectite) is higher than the other constituent clay minerals (i.e., chlorite, illite and kaolin), the material load carrying capacity reduces considerably at elevated moisture content as the swelling minerals absorb moisture and weaken the individual aggregate as well as the whole mix. Moreover, the method that has been developed in this research to compare the relative percentage of moisture susceptible minerals (e.g., smectite, chlorite) is to be used which will help in the selection of the materials on the basis of percentage of constituent minerals.

Recommendation 2: Base Course Gradation Specifications:

Along with mineralogy, the gradation is an important factor in the design of unbound granular basecourse materials. The current New Zealand M/4 basecourse specification finer and coarser gradation Talbots constant varies from 0.3 to 0.6 respectively. This research has confirmed the appropriateness of the specification. However, it is believed that if extra caution is required due to factors either related to a sensitive material or environment, this specification could be limited further to a Talbots constant range of 0.4 to 0.45. This research has confirmed this gradation level to be the optimal for achieving the maximum dry density for a material. In return, it ensures minimum permeability of the layer, thus reducing the risks associated with moisture increase within the base-layer.

Recommendation 3: Chip Surfacing Technology and Practice

Engineer should take note of the high permeability of chip seals measured during this research. The prime coat under the chipseal increases its water proof ability and should be included in the specifications as a construction standard with chipseals. This recommendation is pertinent within the context of the first coat surfacing practices where the first chip surface layer is often in service for more than a year prior to the addition of the secondary coat.

The importance of resealing as a periodic maintenance action has been re-confirmed. It is realised under funding constraints, engineers attempt to maximise the life from surfaces. However, the relatively low cost of resurfacing should be considered against the backdrop of increased failure risk should a surface become more permeable due to cracking.
Chapter 10 Recommendations

Recommendation 4: Forecasting Permanent Deformation

The ability to forecast permanent deformation under wheel loads is an important input into the design and maintenance models for road pavements. On the basis of this research, a new statistical model was developed to predict the permanent deformation of unbound materials in RLT tests. This model is more accurate in predicting the trend of RLT test data at higher stress states. It includes number of loading cycles, stresses as input parameters. It is given by:

$$\varepsilon_p = [a_1 N^{b_1} + (m_1 N + a_2)(1 - e^{-b_2 N})] \left[ \left( \frac{L}{p_a} \right)^n \frac{1}{m_2 + \frac{s - q}{p}} \right]$$

- $\varepsilon_p$ = Permanent deformation (µm)
- $N$ = Number of loading cycles
- $q$ and $p$ = deviator and mean stresses respectively (kPa)
- $L$ = Length of stress path ($L = \sqrt{q^2 + p^2}$)
- $p_a$ = 100 kPa
- $a_1, b_1, a_2, b_2$
- $m_1, n, m_2, s$ = Regression parameters

10.1.2 Experiments Conducted

Recommendation 5: Testing Procedure for the RLT Tests

The Repeated Load Triaxial tests conducted in this research followed the Draft standard TNZ T/15 where there is a need to modify the sequence of applied stresses. It is recommended that stresses should be applied in ascending order with respect to the magnitude of deviator stresses. This is because of the fact that material is already overstressed in the previous stage and the realistic results are hard to find. The stress sequence was found good for stages 4, 5, and 6. The order should be changed according to Table 10.1.
Table 10.1 Old and suggest sequence of stress states

<table>
<thead>
<tr>
<th>Magnitude of Stresses for old sequence</th>
<th>Old Sequence of Stress State</th>
<th>Suggested Sequence of Stress State</th>
<th>Magnitude of stresses for new sequence</th>
</tr>
</thead>
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<tr>
<td>σ1</td>
<td>σ3</td>
<td>1</td>
<td>3</td>
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<td>210</td>
<td>120</td>
<td>2</td>
<td>2</td>
</tr>
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<td>6</td>
</tr>
<tr>
<td>530</td>
<td>110</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

It was learnt from the research that a stress condition was lacking in the RLT test that represented the failure in CAPTIF pavement. It may also be the case in actual pavements where there is a rain event of more than an hour in rush hour times. It is recommended that a new stress state should be included for the unbound aggregates that are to be used in high rain fall areas – which are in the sequence no 5 with σ1 value of 530 kPa and σ3 of magnitude 42 kPa. This extreme stress condition simulates the top of the basecourse which is directly under the chipseal layer, in both drained and un-drained conditions.

The RLT tests were conducted at optimum moisture content, slightly more than optimum moisture and relatively higher moisture content. There should be at least 5 moisture levels at which RLT tests should be conducted for the purpose of statistical modelling so that a factor of moisture content can be included in the model. As the RLT test specimen shows the position of an element of the pavement at different levels in the pavement by varying the vertical and confining stresses, the moisture change can enhance the understanding of material behaviour at such levels while moisture is changing.

**10.2 Further Research**

This research has investigated the performance of unbound granular basecourse materials obtained from different greywacke resources at high moisture conditions both in laboratory and full scale accelerated pavement testing. Various factors affecting the performance of UGM basecourse materials at high moisture are discussed and the research gives a
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framework for the selection of the aggregates to be used according to the environmental conditions. It is however realised that a limited range of materials and testing conditions have been investigated during the research. Recommended further work is document in the subsequent sections.

10.2.1 Quantification of Clay Minerals effects on Permanent Deformation of Unbound Aggregates in RLT testing

There is a need to quantify the effect of different proportions of swelling clay minerals on the performance of the unbound granular materials. For the purpose XRD tests to identify and quantify the minerals and RLT tests for unbound basecourse materials obtained from various resources can be conducted.

10.2.2 Repeated Load Triaxial Testing in Drying and Wetting Phases

The stress states to which an unbound aggregate sample is subjected can be proposed according to their expected use in the pavements i.e., the stresses are different when an unbound aggregate layer lies just under the thin wearing course compared to the stresses induced when the unbound basecourse lies under thick asphalt and stabilised base.

The RLT test for the UGM basecourse can be conducted in wetting and drying phase which gives an idea about the performance of these materials in the real pavements when they are subjected to wetting and drying cycles. The drying phase of the drying and wetting cycle considers the effect of suction when drying and the resistance to permanent deformation can be compared with the wetting phase.

Moreover, the size of the sample in RLT test can be increased so that full size of the materials can be used in RLT test. The increased size of the sample in length and diameter is to counter the boundary conditions that UGM face in real pavements.

10.2.3 Inclusion of More Factors in Permanent Deformation Prediction Model

Some more factors such as moisture, suction, and percentage of clay minerals can be incorporated in permanent deformation statistical modelling process to fit in RLT test data. This will increase the confidence in predicting the performance of UGM at different environmental conditions.
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A new instrumentation for suction should be included in CAPTIF pavement sections which can tell the relative suction of the basecourse while increasing or decreasing relative moisture content.

10.2.4 Life Cycle Cost for Surfacing/ Resurfacing Technology

In future, a life cycle cost analysis should also be conducted for different surfacing options along with the unbound basecourse layer. The life cycle cost analysis will help the designers and practitioners to adapt the most feasible solution in relevant conditions.
REFERENCES


References


Florida DOT 204. (2010). Standard Specifications for Road and Bridge Construction, *Division II - General Construction Operations Section 204 Graded Aggregate Base*.


References


References


References


References


References


