Faculty of Engineering and Industrial Sciences
Centre for Sustainable Infrastructure

Geotechnical Characteristics of Recycled Glass in Road Pavement Applications

M. M. Younus Ali

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Swinburne University of Technology
DECLARATION

This thesis contains no material which has been accepted for the award of any other degree or diploma in any university.

To the best of my knowledge and belief this thesis contains no material previously published by any other person except where due acknowledgement has been made.

The following publications have resulted from the work carried out for this degree.

Refereed Journal Papers:


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Current levels of natural aggregates production are under threat due to tighter restrictions on planning issues related to quarrying of virgin materials. There is now a developing emphasis on environmental management which has resulted in growing pressure to investigate the viability of reuse of all categories of waste materials. Recycled glass continues to be one of the principal waste products generated by the public and its reuse applications are urgently sought by the recycling industry. Recycled glass is a waste material produced in large quantities in municipal and industrial areas in Australia and worldwide. Currently in the state of Victoria, Australia, 186,000 tonnes of recycled glass are stockpiled annually and these stockpiles are growing. However, there is little known reuse application for recycled glass in pavement subbase applications due to limited knowledge concerning its geotechnical properties. The reuse of recycled glass in road pavement applications will provide an opportunity to reduce the unnecessary wastage of not only diminishing virgin materials stocks but also valuable land-fill space. Using recycled glass in road pavement applications would eliminate the need for expensive sorting offering an opportunity to use it as aggregate in locations where virgin aggregate sources are scarce.

The geotechnical characteristics of up to 50% recycled glass (by mass) in blends with crushed rock and recycled crushed concrete were obtained from an extensive laboratory testing including high level of tests such as static triaxial tests and repeated load triaxial tests. The research also explores the performance of recycled glass blends in field trial pavement sections with up to 30% recycled glass as an additive into crushed rock and recycled crushed concrete mixes for pavement base applications. The research also explores the performance of granular materials using finite element modelling.

The findings of this research suggest that up to 15% content of recycled glass with the maximum particle size of 4.75 mm could be safely added in blends with recycled crushed concrete and crushed rock. Test results show that the extent of breakdown occurring in the blends in excess of 15% recycled glass is on the limit of what would be acceptable for this material. It might be possible to increase the percentages of recycled glass but this depends on the results of future field trials.
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<tbody>
<tr>
<td>AGI</td>
<td>American Geological Institute</td>
</tr>
<tr>
<td>$C_c$</td>
<td>Coefficient of curvature</td>
</tr>
<tr>
<td>$C_u$</td>
<td>Coefficient of uniformity</td>
</tr>
<tr>
<td>$c_{ref}$</td>
<td>Referenced cohesion</td>
</tr>
<tr>
<td>$c'$</td>
<td>Effective cohesion</td>
</tr>
<tr>
<td>CBR</td>
<td>California Bearing Ratio</td>
</tr>
<tr>
<td>CH</td>
<td>Clegg Hammer</td>
</tr>
<tr>
<td>CHM</td>
<td>Clegg Hammer Modulus</td>
</tr>
<tr>
<td>CIV</td>
<td>Clegg Impact Value</td>
</tr>
<tr>
<td>CWC</td>
<td>Clean Washington Centre</td>
</tr>
<tr>
<td>DCP</td>
<td>Dynamic Cone Penetration</td>
</tr>
<tr>
<td>DST</td>
<td>Direct Shear Test</td>
</tr>
<tr>
<td>$D_{10}$</td>
<td>Particle diameter corresponding to 10% passing by weight</td>
</tr>
<tr>
<td>$D_{30}$</td>
<td>Particle diameter corresponding to 30% passing by weight</td>
</tr>
<tr>
<td>$D_{60}$</td>
<td>Particle diameter corresponding to 60% passing by weight</td>
</tr>
<tr>
<td>ESA</td>
<td>Equivalent Standard Axles</td>
</tr>
<tr>
<td>$E_o$</td>
<td>Composite modulus</td>
</tr>
<tr>
<td>$E_v$</td>
<td>Secant modulus</td>
</tr>
<tr>
<td>FWD</td>
<td>Falling Weight Deflectometer</td>
</tr>
<tr>
<td>$G_s$</td>
<td>Specific gravity</td>
</tr>
<tr>
<td>IMEA</td>
<td>Institute of Municipal Engineering Australia</td>
</tr>
<tr>
<td>$k_1$</td>
<td>Regression coefficient</td>
</tr>
<tr>
<td>$k_2$</td>
<td>Regression coefficient</td>
</tr>
<tr>
<td>$k_x$</td>
<td>Coefficient of permeability in horizontal direction</td>
</tr>
<tr>
<td>$k_y$</td>
<td>Coefficient of permeability in vertical direction</td>
</tr>
<tr>
<td>LA</td>
<td>Los Angeles</td>
</tr>
<tr>
<td>MDD</td>
<td>Maximum dry density</td>
</tr>
<tr>
<td>$M_R$</td>
<td>Resilient modulus</td>
</tr>
<tr>
<td>OMC</td>
<td>Optimum moisture content</td>
</tr>
<tr>
<td>$p$</td>
<td>Vertical stress</td>
</tr>
<tr>
<td>$q$</td>
<td>Deviator stress</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
</tr>
<tr>
<td>--------</td>
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</tr>
<tr>
<td>R</td>
<td>Modular ratio</td>
</tr>
<tr>
<td>RTA</td>
<td>Roads and Traffic Authority</td>
</tr>
<tr>
<td>r</td>
<td>Radius</td>
</tr>
<tr>
<td>$s'$</td>
<td>Average effective principal stress</td>
</tr>
<tr>
<td>$t'$</td>
<td>Average effective deviator stress</td>
</tr>
<tr>
<td>WMAA</td>
<td>Waste Management Association of Australia</td>
</tr>
<tr>
<td>$w_{opt}$</td>
<td>Optimum moisture content</td>
</tr>
<tr>
<td>$\rho_w$</td>
<td>Density of water at test temperature</td>
</tr>
<tr>
<td>$p'$</td>
<td>Mean effective stress</td>
</tr>
<tr>
<td>$\phi'$</td>
<td>Effective angle of internal friction</td>
</tr>
<tr>
<td>$\psi$</td>
<td>Dilatancy angle</td>
</tr>
<tr>
<td>$\sigma'_1$</td>
<td>Effective major principal stress</td>
</tr>
<tr>
<td>$\sigma'_2$</td>
<td>Effective intermediate principal stress</td>
</tr>
<tr>
<td>$\sigma'_3$</td>
<td>Effective minor principal stress</td>
</tr>
<tr>
<td>$\sigma_c$</td>
<td>Confining stress</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Bulk stress</td>
</tr>
<tr>
<td>$\gamma_{sat}$</td>
<td>Saturated unit weight</td>
</tr>
<tr>
<td>$\gamma_{unsat}$</td>
<td>Unsaturated unit weight</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson's ratio</td>
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CHAPTER ONE

1 INTRODUCTION

1.1 Background

As the world population is increasing rapidly, more goods will be produced to satisfy their consumption. As a result, more waste is also generated. Due to the increasing volume of waste and by-product materials generated in our society and the associated cost of disposal of the waste materials, there is increased pressure and incentive to recover and recycle these materials for use in secondary applications such as pavement applications (Ali et al. 2011, Arulrajah et al. 2011, 2012). From a pavement engineering point of view, recovered materials should be used without compromising the expected performance of the pavement (Arnold et al. 2008). Furthermore, the pavement road construction technology is primarily dependent on aggregates extracted from rocks (Indian Highways 2011). But, we must take no more from nature than nature can replenish. Again, the amount of energy consumed for blasting the hills for quarrying operations, crushing the rocks, transportation of this material to plants, mixing, laying etc. is doing unspeakable damage to the environment. Therefore, we have to think about finding the innovative ways for the recycling and reusing of waste materials.

Emphasis is therefore to be given for use of such materials which are easily accessible and available in sufficient quantity and can be used for road construction either directly or in combination with other materials as an alternative to conventional materials, with significant economy after studying their physical and engineering properties for their suitability in road construction. There may be situations where the existing pavements have to be dismantled. In such cases, the dismantled materials can be considered for reuse by recycling, duly supplemented with fresh materials compatible with dismantled materials.

Currently in the state of Victoria, Australia, 186,000 tonnes of recycled glass are stockpiled annually and these stockpiles are growing (Sustainability Victoria 2010). In Victoria, Australia the total amount of the recovered waste material is recorded as 6.56
million tonnes during the financial year 2008-2009 and around 64% of the solid wastes were recycled over that period (Sustainability Victoria 2009). 82% of the material received for reprocessing during the 2008–2009 was sourced from the combined industry sectors, i.e. commercial and industrial and construction and demolition (C&D) industries, with C&D material accounts for 47% of all material (by weight) recovered. Among C&D material, concrete was the major component representing 55% (by weight) of the total followed by rock/excavation stone (21%), brick/brick rubble (8%), soil/sand (5%), asphalt (7%), mixed C&D waste (3%) and plasterboard (1%) (Sustainability Victoria 2009). Recycling of C&D materials would clearly provide substantial benefits to the industry in terms of reduced material supply and waste disposal cost, increased sustainability and reduced environmental impact (Arulrajah et al. 2011, 2012; Sivakumar et al. 2004).

Although, the governments at national and/or regional levels have taken some measures to recover the C&D wastes to a certain extent, still there is plenty of room to extend the recovery in a sustainable manner. Without finding the sustainable applications of recycled C&D waste materials, it will be difficult to encourage or enforce the recovery of C&D wastes (Aatheesan 2011). Extensive research studies on C&D waste materials enable possible applications in sustainable manner. Although, crushed rock and recycled crushed concrete are being used in pavements, the use of recycled glass in pavements has been limited due to lack of standardised guidelines or performance based specifications for recycled glass.

This research has been undertaken to fully characterize the geotechnical properties of recycled glass and assess the suitability of recycled glass and its blends with crushed rock and recycled crushed concrete for pavement subbase/light duty base applications. In addition, in order to find out the deformation behaviour of recycled materials under traffic loading and to compare the deformation behaviour of recycled materials, a finite element modelling was conducted using PLAXIS 2D finite element model.

This research is significant as it investigates the sustainable reuse of recycled glass potentially as a pavement subbase/light duty base material in combination with other recycled materials such as crushed rock and recycled crushed concrete.
1.2 Objectives of the research

The main objective of this research is to characterize and evaluate the use of recycled glass in road pavement applications. Although the use of recycled glass (RG), also called “cullet” or “glass cullet” in roadway construction is increasing, many agencies are still reluctant to consider its use as a suitable substitute for materials already being used. This is mainly due to unfamiliarity with the engineering properties of recycled glass and a lack of suitable sources that supply recycled glass (e.g., lack of glass collection systems and glass processing systems; distance to markets, varying levels of contribution to recycling programs by householders). Therefore, this research will be carried out to identify the sound engineering and environmental uses of recycled glass in roadway construction and maintenance projects and to characterize glass cullet as a highway material without impairing its performance. Also, it is necessary to develop specifications for each successful use of recycled glass evaluated based on current Local State Authority’s Roads specifications.

The objectives of the research are set as follows:

➢ To review the available literature and hence document the present status of knowledge on the use of recycled glass and other construction and demolition (C&D) materials as a highway material.

➢ To perform necessary laboratory experiments to know the geotechnical properties for the projected applications of recycled glass in blends with crushed rock and recycled crushed concrete.

➢ To perform environmental assessment tests on the recycled materials.

➢ Construction of field trial pavement test sections and field testing of recycled glass blends.

➢ Field performance monitoring of recycled glass blends.

➢ To compare the test results with the Local State Authority's compaction criteria for subbase and base and submit laboratory and testing reports to the Local State Authority.
Authority for consideration for their specifications to be revised to incorporate recycled materials.

➢ To develop finite element modelling for the uses of recycled glass in road pavement applications.

1.3 Outline of the dissertation

This research describes a comprehensive study of on the characteristics of recycled glass in blends with other construction and demolition (C&D) waste materials such as crushed rock and recycled crushed concrete for road pavement applications. The research focuses on investigating the engineering properties of recycled glass in blends with crushed rock and recycled crushed concrete in terms of laboratory, field testing and PLAXIS finite element modelling.

This research has been divided into eight chapters. Chapter 1 describes an introduction of the entire research study and highlights the importance of using recycled glass in blends with crushed rock and recycled crushed concrete as road pavement applications.

Chapter 2 describes the literature review on different construction and demolition materials including different local load authority’s specifications for recycled materials.

Chapter 3 highlights the extensive laboratory testing methodologies to investigate the engineering properties of recycled glass, crushed rock, recycled crushed concrete and the recycled glass in blends with crushed rock and recycled crushed concrete.

Chapter 4 describes the engineering properties of recycled glass, crushed rock and recycled glass in blends with crushed rock.

Chapter 5 describes the engineering properties of recycled crushed concrete and recycled glass in blends with recycled crushed concrete.

Chapter 6 describes the performance of recycled glass in blends with crushed rock and recycled crushed concrete based on field testing results.
Chapter 7 describes the modelling of unbound granular pavement subbase/light duty base using a finite element program.

Finally, the important conclusions have been reported in Chapter 8.
CHAPTER TWO

2 LITERATURE REVIEW

2.1 Research background

As recycling continues to grow as a means of utilizing waste materials in today’s world, more markets must be established for products containing recycled materials. Specifically, recycled glass continues to be one of the principal waste products generated by the public and recovered by the recycling industry. This research has been undertaken to fully investigate the geotechnical characteristics of recycled glass in road pavement applications. Recycled glass is commonly derived from household waste collection as well as construction and demolition (C&D) activities. Construction wastes are produced during different phases of construction such as transportation, stocking or working and are generally heterogeneous by nature. Demolition waste materials arise from demolition activities and are generally homogenous by nature. Currently, approximately 186,000 tonnes of recycled glass per year are sent to landfill in Victoria (Sustainability Victoria 2010). Proportionately, approximately 1,000,000 tonnes per year are sent to landfill in Australia.

There is now a developing emphasis on environmental management which has resulted in a growing push to investigate the sustainable reuse of all categories of waste material such as C&D materials (Aathesan 2011). Currently there is little known reuse application for recycled glass in engineering and pavement applications. Therefore, it is the aim of this research to seek viable geotechnical engineering applications for waste glass as a road construction material principally as pavement subbase. The use of recycled C&D waste materials such as recycled glass would greatly reduce the demand for landfill sites and for virgin resource materials by reusing what would be normally regarded as a waste material. The reuse of various recycled waste materials will significantly reduce carbon footprints as compared to traditional quarried materials and ultimately will lead to a more sustainable environment (Disfani et al. 2012, Tam 2009).
This research is significant as it will investigate the geotechnical characteristics of recycled glass to determine its suitability when used in combination with other recycled materials such as recycled crushed concrete (RCC), crushed rock (CR).

2.2 Construction and demolition (C&D) materials

Construction and demolition (C&D) debris is generally produced during the construction of new structures and when existing structures are renovated or demolished (US EPA 1998). The problems associated with the definitions of construction and demolition (C&D) materials are similar to the problems with the differences in the definition of recycling by different states (MassDEP 2008). The different states of USA define the C&D materials in different ways. For example, the State of Massachusetts defines the C&D materials as the waste building materials and rubble resulting from the construction, remodeling, repair or demolition of buildings, pavements, roads or other structures. It includes but is not limited to, concrete, bricks, lumber, masonry, road paving materials, rebar and plaster (MassDEP 2008). US EPA defined the C&D materials as the debris generated during the construction, renovation, and demolition of buildings, roads, and bridges and often contain bulky, heavy materials, such as concrete, wood, metals, glass, and salvaged building components (US EPA 2011). C&D waste consists mostly of excavated material (soil and rock), concrete, bricks and masonry items, timber, plasterboard and packaging material (NSW EPA 1998). Recycling of C&D materials is an important industry in Victoria. WorkSafe Victoria defined the C&D material as the excess or waste material associated with the construction and demolition of buildings and structures, including concrete, brick, steel, timber, plastics and other building materials and products (WorkSafe Victoria 2006). Sustainability Victoria defined the C&D materials as the waste from residential, civil and commercial C&D activities excluding the construction waste from owner or occupier renovations (Sustainability Victoria 2009).

Construction wastes are produced during different phases of construction such as transportation, stocking or working. Their main characteristic is heterogeneous and their main components are as follows (Portas 2004):

- Pieces of wood
- Different kinds of plastic materials
Different kinds of metals
- Paper
- Cardboard boxes
- Synthetic materials, insulating materials
- Concrete pieces
- Bricks, tiles
- Bituminous materials
- Asbestos
- Excavation soil and rock
- Glass.

Demolition waste materials are the materials those arise from demolition activities and are generally homogeneous by nature. Homogeneity increases the possibility to reuse or recycle waste materials. Their main components are as follows (Portas 2004):
- Bricks;
- Cement, concrete;
- Asphalt.

It is possible to find other C&D materials, but the percentage is low compared with the main components mentioned above. In general, it is not easy to evaluate C&D material composition as it varies greatly with location, level of industrialization and construction techniques all over the countries (Portas 2004).

### 2.3 C&D materials in other countries

European Council of Civil Engineers (1998) reported C&D materials in various European countries. It was reported that about 24 million tonnes per years are generated as construction waste in France and possible reusable amount is 16 million tonnes and around 4-5 million tonnes can be used as effectively. In Denmark, recycling of construction wastes showed a rapid increase to 80% in 1993 compared to 10% in 1985. It was reported that the UK government has set a plan to increase the use of secondary material instead of using virgin material. For instance, the targeted use of recycled aggregate from 40 million tonnes in 2001 to 55 million tonnes by the year 2006.
In Northern Ireland, a survey by the Environmental and Heritage Service (2002) estimated that nearly 6 million tonnes of C&D waste were produced in 2001. However, only 100,000 tonnes were estimated to have been recycled or processed by C&D waste operations.

In United States, the Environmental Protection Authority estimated that around 136 million tonnes of C&D were generated in the United States (US) in 1996.

In Oman, Taha et al. (2004) stated that about 900,000 tonnes of solid wastes were generated annually and there were barriers in using recycled materials. Taha et al. (2004) has analysed the use of reclaimed asphalt pavement (RAP) aggregates in road bases and subbases.

Pappu et al. (2007) have reported that about 960 million tonnes of solid wastes are generated in India by various activities. Moreover, about 14.5 million tonnes of solid wastes are being generated annually as construction wastes.

Poon and Chan (2006) reported over 20 million tonnes of C&D wastes were generated in Hong Kong in the year 2004. They also stated that the local landfills in Hong Kong will be saturated in about 8 years and there is an important need to find the viable ways to reuse these waste materials.

### 2.4 C&D materials in Australia

In Victoria, Australia the total amount of the recovered waste material is recorded as 6.56 million tonnes during the financial year 2008-2009 and around 64% of the solid wastes were recycled over that period (Sustainability Victoria 2009). 82% of the material received for reprocessing during the 2008–2009 financial year was sourced from the combined industry sectors i.e., commercial and industrial and construction and demolition (C&D) industries, with C&D material accounts for 47% of all material (by weight) recovered. Among C&D material, concrete was the major component representing 55% (by weight) of the total followed by rock/excavation stone (21%), brick/brick rubble (8%), soil/sand (5%), asphalt (7%), mixed C&D waste (3%) and plasterboard (1%). The total quantity of glass recovered for reprocessing in Victoria
during the financial year 2008-09 was almost 186,000 tonnes indicating an increase of 7% from the previous financial year (Sustainability Victoria 2009). Recycling of the C&D waste materials would clearly provide substantial benefits to the industry in terms of reduced material supply and waste disposal cost, increased sustainability, and reduced environmental impact (Arulrajah et al. 2011, 2012; Sivakumar et al. 2004).

NSW EPA (1998) stated that the total amount of C&D wastes generated in New South Wales were around 1.56 million tonnes. The report further mentioned that around 55% of the total C&D waste was sourced from demolition work and about 40% was generated by construction activities in New South Wales. Reddrop et al. (1997), regarding housing construction waste, found bricks (52.4%) were the most waste material by weight and followed by roof tiles (23.5%).

Ritchie (2006) stated in his response to New South Wales state plan that only the C&D waste stream are on track to achieve the targets in terms of efficient recycling and reuse of materials and responsible waste management in Sydney metropolitan area. In addition, the commercial and domestic sectors will focus on recycling and proper waste management as the land fill waste is much higher than the recycled waste in both these sectors. Ritchie (2006) stated that 75% of the waste materials from C&D sector were recycled in the year 2003 and there is a significant increase in recycling since 1998.

SITA Environmental Solutions (2006) stated that in Perth, around 20% of the C&D waste was recycled in the year 2005 and it further mentioned that there is no date available on recycling of C&D wastes in previous years. Report on disposal and recycling showed that around 30% of the C&D waste was recycled in the year 2005. This data clearly indicates that there is significant room to increase the recycling process.

2.5 Regulations for C&D materials in Australia

EPA Victoria (2005) and the Environment Protection Act (1970) have identified a waste management hierarchy based on principles which advocate the following seven practices:

- Avoidance
The best method of waste management is to avoid the creation of waste. If waste generation is unavoidable, then it must be treated as a resource material. Every effort should be taken to reuse or recycle the waste and burial of waste should only be considered as a last resort.

EPA Victoria (2001) has classified demolition material along with other construction materials under the category of solid and inert waste. Solid and inert waste is defined by the State Protection Policy (Victorian Government Gazette 1991) as ‘hard waste and dry vegetative material which has negligible activity or effect on the environment’. This is the least hazardous category of waste that EPA controls. EPA Victoria (2001, 2004b) specifies that reuse and recycle options should be closely examined since in many cases solid inert waste, such as building materials, can be reused.

C&D industries generate 40% of all waste going to landfill in Victoria (Nolan-ITU 2002). Over 75% of this waste is clean excavated material, concrete, bricks and timber (Sustainability Victoria 2006). Sustainability Victoria has also provided various strategies and checklists on construction and demolition waste strategies and plans on their website which are developed specifically for the C&D industries.

NSW EPA (1997) has provided recommendation for the beneficial use of C&D waste materials. The plan advocates a range of measures aimed at encouraging the greater use of recycled materials in civil engineering activities involving local governments. The plan provides initiatives for the development of a waste exchange for C&D materials, a C&D Waste Management Manual, a Waste Audit Service, Design and Best Practice Guidelines, a C&D Waste Award Scheme and the establishment of C&D Resource Recovery Facilities.
The Recycled Materials Usage Panel of the Institute of Municipal Engineering Australia (IMEA) and the Waste Management Association of Australia (WMAA) have also recognized the need for the local government and the private sector to review relevant specifications and quality assurance processes and if feasible, remove those provisions that discriminate against the use of recycled materials (IMEA 1996).

Increasing the utilization of recycled construction and demolition waste will require the adoption of specifications and the supply of suitable recycled materials in a cost-effective manner.

2.6 Existing specifications for C&D materials in Australia

Victoria

This section describes the existing specifications relevant to C&D materials in Victoria.

Pavement subbase

In Victoria, the construction of road works is in accordance with specifications established by VicRoads. Section 820 of the VicRoads specification (VicRoads 2009) describes guidelines for the use of recycled crushed concrete for pavement subbase and light duty base. However, there is no specific guideline available for recycled glass, although it can be considered as high density foreign material within the recycled crushed concrete specifications. The required engineering properties for recycled crushed concrete and the limitations for foreign materials are summarised in Table 2.1 and Table 2.2, respectively. The grading requirements for uncompacted crushed concrete (Class 3) is tabulated in Table 2.3.
Table 2.1: Acceptable engineering properties of crushed concrete (VicRoads 2009)

<table>
<thead>
<tr>
<th>Test Value</th>
<th>Test Class 2</th>
<th>Test Class 3</th>
<th>Test Class 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid limit % (Max)</td>
<td>35</td>
<td>35</td>
<td>40</td>
</tr>
<tr>
<td>Plastic index (Max)</td>
<td>6</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td>California Bearing Ratio % (Min)</td>
<td>100</td>
<td>80</td>
<td>20</td>
</tr>
<tr>
<td>Los Angeles abrasion loss (%) (Max)</td>
<td>35</td>
<td>40</td>
<td>45</td>
</tr>
<tr>
<td>Flakiness index</td>
<td>35</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 2.2: Maximum allowable foreign material (%) in crushed concrete (VicRoads 2009)

<table>
<thead>
<tr>
<th>Foreign Material Type</th>
<th>Test Class 2</th>
<th>Test Class 3</th>
<th>Test Class 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>High density materials such as metal, glass and brick</td>
<td>2</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>Low density materials such as plastic, rubber, plaster, clay lumps and other friable material</td>
<td>0.5</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>Wood and other vegetable or decomposable matter</td>
<td>0.1</td>
<td>0.2</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 2.3 : Grading Requirements for 20 mm Class 3 Crushed Concrete (VicRoads 2009)

<table>
<thead>
<tr>
<th>Sieve Size (mm)</th>
<th>Target Grading (% Passing)</th>
<th>Limits of Grading</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Test Value before Compaction (% Passing)</td>
</tr>
<tr>
<td>26.5</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>19.0</td>
<td>100</td>
<td>95-100</td>
</tr>
<tr>
<td>13.2</td>
<td>85</td>
<td>75-95</td>
</tr>
<tr>
<td>9.5</td>
<td>75</td>
<td>60-90</td>
</tr>
<tr>
<td>4.75</td>
<td>59</td>
<td>42-76</td>
</tr>
<tr>
<td>2.36</td>
<td>44</td>
<td>28-60</td>
</tr>
<tr>
<td>0.425</td>
<td>19</td>
<td>10-28</td>
</tr>
<tr>
<td>0.075</td>
<td>6</td>
<td>2-10</td>
</tr>
</tbody>
</table>
Section 812 of the VicRoads specification describes guidelines for the use of crushed rock for base and subbase pavement. The required engineering properties for crushed rock is summarised in Table 2.4. The grading requirements for uncompacted crushed rock (Class 3) are tabulated in Table 2.5 and Table 2.6.

Table 2.4 Acceptable engineering properties of crushed rock (VicRoads 2010)

<table>
<thead>
<tr>
<th>Test</th>
<th>Test value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Class 1</td>
</tr>
<tr>
<td>Liquid limit (%) (max)</td>
<td>30</td>
</tr>
<tr>
<td>Plasticity index (range)</td>
<td>2-6 (+)</td>
</tr>
<tr>
<td>California Bearing Ratio (%) (min)</td>
<td>-</td>
</tr>
<tr>
<td>Flakiness index (%) (max)</td>
<td>35</td>
</tr>
</tbody>
</table>

Table 2.5: Grading limits for 20 mm Class 3 subbase from all rocks (except Granitic rocks) with a Los Angeles value of 25 or less (VicRoads 2010)

<table>
<thead>
<tr>
<th>Sieve size (mm)</th>
<th>Test before compaction-Limits of Grading (% passing by mass)</th>
</tr>
</thead>
<tbody>
<tr>
<td>26.5</td>
<td>100</td>
</tr>
<tr>
<td>19.0</td>
<td>95-100</td>
</tr>
<tr>
<td>13.2</td>
<td>75-95</td>
</tr>
<tr>
<td>9.5</td>
<td>60-90</td>
</tr>
<tr>
<td>4.75</td>
<td>42-76</td>
</tr>
<tr>
<td>2.36</td>
<td>28-60</td>
</tr>
<tr>
<td>0.425</td>
<td>14-28</td>
</tr>
<tr>
<td>0.075</td>
<td>6-13</td>
</tr>
</tbody>
</table>

Granular filter material

The VicRoads specification for Granular filter material is tabulated in Table 2.8. The granular filter materials are classified as (a) single and first stage filters and (b) second stage filters. The single stage filter is defined as granular filter materials that are surrounded by the pervious pipe system and placed in contact with the trench sides. Whereas, first stage filter is granular filter material or geotextile surrounded by a second stage filter and also placed in contact with the trench sides. Single and first stage filters are designated as A1, A2, A3, A4, A5 and A6 based on its grading criteria.
Similar to the first stage filter, the second stage filter is also granular filter material or geotextile, but it is placed in contact with the pervious pipe system and surrounded by a first stage filter. The second stage filters are designated as B1, B2, B3 and B4 according to its grading criteria.

**Bedding and backfill materials**

VicRoads specifications for bedding and backfill material are summarised in Table 2.9 and Table 2.10. Melbourne Water specification for the supply of crushed rock to their work sites is summarised in Table 2.11.

Table 2.6: Grading limits for 20 mm Class 3 subbase from Granitic rocks and all other rocks with a Los Angeles value of 26 or more (VicRoads 2010)

<table>
<thead>
<tr>
<th>Sieve size (mm)</th>
<th>Test before compaction-Limits of Grading (% passing by mass)</th>
</tr>
</thead>
<tbody>
<tr>
<td>26.5</td>
<td>100</td>
</tr>
<tr>
<td>19.0</td>
<td>95-100</td>
</tr>
<tr>
<td>13.2</td>
<td>75-95</td>
</tr>
<tr>
<td>9.5</td>
<td>60-90</td>
</tr>
<tr>
<td>4.75</td>
<td>42-76</td>
</tr>
<tr>
<td>2.36</td>
<td>28-60</td>
</tr>
<tr>
<td>0.425</td>
<td>10-28</td>
</tr>
<tr>
<td>0.075</td>
<td>2-10</td>
</tr>
</tbody>
</table>

The grading requirement for after compacted crushed rock (Class 3) is tabulated in Table 2.7.
Table 2.7: Grading requirements for Class 3 subbase crushed rock and crushed concrete after compaction (VicRoads 2008)

<table>
<thead>
<tr>
<th>Sieve size (mm)</th>
<th>Permitted Grading- After Compaction (% passing by mass)</th>
</tr>
</thead>
<tbody>
<tr>
<td>26.5</td>
<td>100</td>
</tr>
<tr>
<td>19.0</td>
<td>95-100</td>
</tr>
<tr>
<td>13.2</td>
<td>75-95</td>
</tr>
<tr>
<td>9.5</td>
<td>60-90</td>
</tr>
<tr>
<td>4.75</td>
<td>42-76</td>
</tr>
<tr>
<td>2.36</td>
<td>28-60</td>
</tr>
<tr>
<td>0.425</td>
<td>14-29</td>
</tr>
<tr>
<td>0.075</td>
<td>6-14</td>
</tr>
</tbody>
</table>

Table 2.8: Grading Requirements for Granular filter Material (VicRoads 2006a)

<table>
<thead>
<tr>
<th>Sieve Size AS (mm)</th>
<th>Limits of Grading (% passing) (by mass)</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Single and First Stage Filters</td>
<td>Second Stage Filters</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>A1</td>
<td>A2</td>
<td>A3</td>
<td>A4</td>
<td>A5</td>
</tr>
<tr>
<td>37.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>26.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>19.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>13.2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>9.50</td>
<td>-</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>70-100</td>
</tr>
<tr>
<td>4.75</td>
<td>-</td>
<td>90-100</td>
<td>90-100</td>
<td>70-100</td>
<td>28-100</td>
</tr>
<tr>
<td>2.36</td>
<td>100</td>
<td>75-100</td>
<td>70-100</td>
<td>0-50</td>
<td>0-28</td>
</tr>
<tr>
<td>1.18</td>
<td>95-100</td>
<td>50-98</td>
<td>40-65</td>
<td>0-10</td>
<td>0-8</td>
</tr>
<tr>
<td>0.600</td>
<td>70-98</td>
<td>30-80</td>
<td>12-40</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>0.300</td>
<td>30-60</td>
<td>10-40</td>
<td>0-16</td>
<td>0-5</td>
<td>0-5</td>
</tr>
<tr>
<td>0.150</td>
<td>0-12</td>
<td>0-7</td>
<td>0-4</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>0.075</td>
<td>0</td>
<td>0-3</td>
<td>0-3</td>
<td>0-3</td>
<td>0-3</td>
</tr>
</tbody>
</table>

16
New South Wales (NSW)

Roads and Traffic Authority (RTA) and Resource NSW have developed specifications for the use of C&D materials in their engineering road works. Roads and Traffic Authority (1998) has developed Specification 3051 on unbound and modified base and subbase materials for surfaced road pavements. It has mainly been developed for natural materials; however it permits a limited usage of recycled building materials.

Table 2.12 indicates the limits of different types of foreign materials for recycled building materials according to the traffic category.

Resource NSW also published a draft specification for supply of recycled materials for roads, drainage (Resource NSW 2003). The material classifications and acceptable limits of recycled materials for various applications are illustrated in Table 2.13 and Table 2.14, respectively.

Table 2.9: Grading Requirements for Bedding and Backfill materials (VicRoads 2006b)

<table>
<thead>
<tr>
<th>Material</th>
<th>Sieve Size-AS (mm)</th>
<th>Percentage Passing (by mass)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>75.0</td>
<td>37.5</td>
</tr>
<tr>
<td>Bedding</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Selected backfill</td>
<td>-</td>
<td>100</td>
</tr>
<tr>
<td>Ordinary Backfill</td>
<td>100</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 2.10: Plasticity Requirement for bedding and backfill materials (VicRoads 2006c)

<table>
<thead>
<tr>
<th>Test</th>
<th>Test value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plasticity index (max)</td>
<td>20</td>
</tr>
</tbody>
</table>

Western Australia

Main Roads, Western Australia has developed a specification for alternative pavement materials such as crushed recycled concrete which describes the construction materials and methods as well.
The limits for foreign materials and other acceptable engineering properties of crushed concrete for pavement sub-base are outlined in Table 2.15 and Table 2.16, respectively. The limits of foreign materials and other acceptable engineering properties of crushed concrete for pavement base are outlined in Table 2.17 and Table 2.18, respectively.

Table 2.11: Grading Limits and Other Requirements for Crushed Rock (Melbourne Water 2001)

<table>
<thead>
<tr>
<th>Sieve size</th>
<th>Graded crushed rock</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aperture (mm)</td>
<td>Limits of percentage passing sieve aperture by mass</td>
</tr>
<tr>
<td>20 mm “B” Grade</td>
<td>20 mm “A” Grade</td>
</tr>
<tr>
<td>53</td>
<td>-</td>
</tr>
<tr>
<td>37.5</td>
<td>-</td>
</tr>
<tr>
<td>26.5</td>
<td>100</td>
</tr>
<tr>
<td>19.0</td>
<td>80-100</td>
</tr>
<tr>
<td>13.2</td>
<td>-</td>
</tr>
<tr>
<td>9.5</td>
<td>58-80</td>
</tr>
<tr>
<td>6.7</td>
<td>-</td>
</tr>
<tr>
<td>4.75</td>
<td>44-65</td>
</tr>
<tr>
<td>2.36</td>
<td>32-54</td>
</tr>
<tr>
<td>1.18</td>
<td>24-45</td>
</tr>
<tr>
<td>0.600</td>
<td>18-36</td>
</tr>
<tr>
<td>0.300</td>
<td>12-30</td>
</tr>
<tr>
<td>0.150</td>
<td>8-24</td>
</tr>
<tr>
<td>0.075</td>
<td>5-20</td>
</tr>
<tr>
<td>Liquid limit (%)</td>
<td>35</td>
</tr>
<tr>
<td>Plasticity index (max)</td>
<td>16</td>
</tr>
<tr>
<td>Los Angeles abrasion loss (%) (max)</td>
<td>35</td>
</tr>
</tbody>
</table>
Table 2.12: Limits of foreign material for recycled building material (RTA 1998)

<table>
<thead>
<tr>
<th>Foreign Material Type</th>
<th>Traffic Category</th>
<th>Maximum Limit by Mass (Test method RTA T276)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Base</td>
</tr>
<tr>
<td>Type I:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Metal, glass, asphalt, stone, ceramics and slag (other than blast furnace slag)</td>
<td>1, 2(a) &amp; 2(b)</td>
<td>3%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2(c) &amp; 2(d)</td>
</tr>
<tr>
<td>Type II:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plaster, clay lumps and other friable material</td>
<td>1, 2(a) &amp; 2(b)</td>
<td>0.2%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2(c) &amp; 2(d)</td>
</tr>
<tr>
<td>Type III:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rubber, plastic, bitumen, paper, cloth, paint, wood and other vegetable matter</td>
<td>1, 2(a) &amp; 2(b)</td>
<td>0.1%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2(c) &amp; 2(d)</td>
</tr>
</tbody>
</table>

Note: Traffic category 1 = $10^7 \leq N$, 2(a) = $4 \times 10^6 < N < 10^7$, 2(b) = $10^6 < N \leq 4 \times 10^6$, 2(c) = $10^5 < N < 10^6$, 2(d) = $N \leq 10^5$, here N refers Equivalent Standard Axles
### Table 2.13: Classification of recycled material (Resource NSW 2003)

<table>
<thead>
<tr>
<th>Material</th>
<th>Class</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Road base</td>
<td>Class R1</td>
<td>Suitable for use on roads with a traffic loading of greater than $1 \times 10^6$ ESA* as either base course or sub-base. This material has similar characteristics to RTA 3051 for dense graded base course.</td>
</tr>
<tr>
<td></td>
<td>Class R2</td>
<td>Suitable for use on roads with a traffic loading of less than $1 \times 10^6$ ESA as either base course or sub-base. This material has similar characteristics to RTA 3051 for dense graded base course.</td>
</tr>
<tr>
<td>Select fill</td>
<td>Class S</td>
<td>Material placed directly on the subgrade to improve subgrade performance. Can also be used as engineered fill to raise site levels, particularly in road embankments or beneath buildings. Engineered fill should have a CBR of at least 5%.</td>
</tr>
<tr>
<td>Bedding material</td>
<td>Class B</td>
<td>A material with about a 7 mm maximum particle size used as support for paving blocks in pedestrian areas, car parks, shopping malls, footpaths, cycle ways or on lightly trafficked access ways.</td>
</tr>
<tr>
<td>Drainage medium</td>
<td>Class D10</td>
<td>Backfilling material for stormwater pipes, sewer pipes or sub-surface drainage lines (except for pipelines covered by AS3725- Loads on Buried Concrete Pipes). In some circumstances, geofabric separators may be needed.</td>
</tr>
<tr>
<td></td>
<td>Class D20</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Class D75</td>
<td></td>
</tr>
</tbody>
</table>

*ESA-Equivalent Standard Axles
Table 2.14: Acceptable limits of recycled materials (Resource NSW 2003)

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Road base</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>N &gt; $10^6$</td>
<td>N &lt; $10^6$</td>
<td>Select fill</td>
<td>Bedding material</td>
<td>Drainage medium</td>
</tr>
<tr>
<td></td>
<td>ESA</td>
<td>ESA</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Class R1</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Class R2</td>
<td>40</td>
<td>40</td>
<td>50</td>
<td>20</td>
<td>5</td>
</tr>
<tr>
<td>Class S</td>
<td>3</td>
<td>30</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Class B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Class D75</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Class D20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Class D10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Material Proportions (max % by mass)</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Reclaimed asphalt</td>
<td>40</td>
<td>40</td>
<td>50</td>
<td>20</td>
<td>5</td>
</tr>
<tr>
<td>Clay brick tile, crushed rock masonry</td>
<td>3</td>
<td>30</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Other materials (max % by mass)</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Asbestos</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Metal, glass and ceramics</td>
<td>3</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Plaster, clay lumps and other friable materials</td>
<td>0.2</td>
<td>0.2</td>
<td>1</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Rubber, plastic, bitumen, paper, cloth, paint, wood and other vegetable matter</td>
<td>0.1</td>
<td>0.1</td>
<td>0.2</td>
<td>0.5</td>
<td>0.5</td>
</tr>
</tbody>
</table>
Table 2.15: Limits of foreign material in recycled crushed concrete: Sub-base material
(Main Roads 2007)

<table>
<thead>
<tr>
<th>Material</th>
<th>Maximum % Retained by mass on 4.75 mm sieve</th>
</tr>
</thead>
<tbody>
<tr>
<td>High density material (brick, glass, etc.)</td>
<td>15</td>
</tr>
<tr>
<td>Low density materials (plastic, plaster, etc.)</td>
<td>3</td>
</tr>
<tr>
<td>Wood and other vegetable matter</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 2.16: Accepted engineering properties of recycled concrete: Subbase material
(Main Roads 2007)

<table>
<thead>
<tr>
<th>Test</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Los Angeles abrasion value</td>
<td>45 % maximum</td>
</tr>
<tr>
<td>Liquid limit (Cone penetrometer)</td>
<td>45 % maximum</td>
</tr>
<tr>
<td>Linear shrinkage</td>
<td>4% maximum</td>
</tr>
<tr>
<td>California Bearing Ratio (CBR)(Soaked 4 days) at 94 % of MDD and 100% of OMC</td>
<td>50 % minimum</td>
</tr>
</tbody>
</table>

Table 2.17: Limits of foreign material in recycled crushed concrete: Base material
(Main Roads 2007)

<table>
<thead>
<tr>
<th>Material</th>
<th>Maximum % Retained by mass on 4.75 mm sieve</th>
</tr>
</thead>
<tbody>
<tr>
<td>High density materials (brick, glass, etc.)</td>
<td>5.0</td>
</tr>
<tr>
<td>Low density materials (plastic, plaster, etc.)</td>
<td>2.0</td>
</tr>
<tr>
<td>Wood and other vegetable matter</td>
<td>0.5</td>
</tr>
<tr>
<td>Asbestos</td>
<td>0.0</td>
</tr>
</tbody>
</table>
Table 2.18: Accepted engineering properties of recycled concrete: Base material (Main Roads 2007)

<table>
<thead>
<tr>
<th>Test</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid limit</td>
<td>35 % maximum</td>
</tr>
<tr>
<td>Linear shrinkage</td>
<td>3 % maximum</td>
</tr>
<tr>
<td>LA abrasion value</td>
<td>40 % maximum</td>
</tr>
<tr>
<td>Maximum dry compressive strength</td>
<td>1.7 MPa minimum</td>
</tr>
<tr>
<td>California Bearing Ratio (Soaked 4 days)</td>
<td>100 % minimum</td>
</tr>
<tr>
<td>% of MDD and 100 % of OMC</td>
<td></td>
</tr>
<tr>
<td>Unconfined compressive strength (UCS-7 days cured and 4 hours immersed)</td>
<td>0.6 MPa to 1.0 MPa</td>
</tr>
</tbody>
</table>

Note: Suitable for 20-Year design traffic <5x10^6 ESA, ESA-Equivalent Standard Axles

2.7 Existing specifications allowing glass in aggregate in different countries

As crushed glass is of similar strength to crushed rock in base course, several road controlling authorities have allowed percentages of crushed glass to be added to the base course aggregate (Arnold et al. 2008). At present, the Transit NZ (2006) specification for base course aggregate (TNZ M/4) allows 5% by mass of crushed glass (<9.5 mm) to be added to the source aggregate with no adjustment in grading necessary. This change was based on international use where generally quantities of up to 15% are considered acceptable. A conservative 5% was chosen, as base courses in New Zealand are only covered by a chipseal compared with the Northern Hemisphere where the base courses are protected by at least 100 mm of structural asphalt (Arnold et al. 2008).

Minnesota Department of Transport has developed an engineering specification for using recycled glass waste in base course aggregate (Hyman and Johnson 2004). The specification allows the addition of this material to granular aggregate at no more than 10% by mass. In addition, the specification restricts debris (e.g. bottle caps, paper and plastic) to 5% of the total glass volume.

Shin and Sonntag (1994) in Washington DC undertook a comprehensive study of varying percentages of crushed glass (0, 15, 50 and 100%) and aggregate for a range of
construction applications. A range of tests were conducted including: compaction tests, gradation, California bearing ratio (CBR) and elastic RLT tests. The study showed that glass, as an aggregate, is strong, safe, clean and economical to use from an engineering standpoint provided percentages of glass and debris (contaminants in glass) are below the limits in Table 2.19, Table 2.20, and Table 2.21.

Table 2.19: Application specification for structural uses of aggregate and glass mixtures (Shin and Sonntag 1994).

<table>
<thead>
<tr>
<th>Gradation of glass fraction</th>
<th>Sieve size (mm)</th>
<th>% passing by weight</th>
<th>Use</th>
<th>Max glass content (%)</th>
<th>Max debris level (%)</th>
<th>Min compaction level (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>19.0</td>
<td>100</td>
<td>Base course</td>
<td>15</td>
<td>5</td>
<td>95</td>
</tr>
<tr>
<td></td>
<td>6.3</td>
<td>10–100</td>
<td>Subbase</td>
<td>30</td>
<td>5</td>
<td>95</td>
</tr>
<tr>
<td></td>
<td>1.9</td>
<td>0–50</td>
<td>Embankments</td>
<td>30</td>
<td>5</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td>0.425</td>
<td>0–25</td>
<td>Nonstructural fill</td>
<td>100</td>
<td>10</td>
<td>85</td>
</tr>
<tr>
<td></td>
<td>0.075</td>
<td>0–5</td>
<td>Utility bedding and backfill</td>
<td>100</td>
<td>5</td>
<td>90</td>
</tr>
</tbody>
</table>

Table 2.20: Application specification for drainage fills using aggregate and glass mixtures (Shin and Sonntag 1994).

<table>
<thead>
<tr>
<th>Gradation of glass fraction</th>
<th>Sieve size (mm)</th>
<th>% passing by weight</th>
<th>Use</th>
<th>Max glass content (%)</th>
<th>Max debris level (%)</th>
<th>Min compaction level (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>19.0</td>
<td>100</td>
<td>Retaining</td>
<td>100</td>
<td>5</td>
<td>95</td>
</tr>
<tr>
<td></td>
<td>6.3</td>
<td>10–100</td>
<td>Foundation drainage</td>
<td>100</td>
<td>5</td>
<td>95</td>
</tr>
<tr>
<td></td>
<td>1.9</td>
<td>0–100</td>
<td>Drainage blankets</td>
<td>100</td>
<td>5</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td>0.425</td>
<td>0–50</td>
<td>French drains</td>
<td>100</td>
<td>5</td>
<td>90</td>
</tr>
</tbody>
</table>
Table 2.21: Glass grading requirements for drainage and bedding material (Siddiki et al. 2004)

<table>
<thead>
<tr>
<th>Sieve size (mm)</th>
<th>% Fines</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.5</td>
<td>85 to 100</td>
</tr>
<tr>
<td>4.75</td>
<td>45 to 85</td>
</tr>
<tr>
<td>2.0</td>
<td>25 to 70</td>
</tr>
<tr>
<td>0.425</td>
<td>10 to 30</td>
</tr>
<tr>
<td>0.075</td>
<td>0 to 10</td>
</tr>
</tbody>
</table>

Glass cullet appears to be an excellent supplement or replacement for natural aggregates in many construction applications. Finkle et al. (2007) studied the strength and moisture/density characteristics of different glass and aggregate blends to examine the effects of blending glass cullet into base course aggregate. Two sources of natural aggregates were tested, one being crusher run and very angular in nature and the other being pit run and rounded in nature. The glass was introduced into the aggregate at replacement rates of 10%, 20% and 30%. Four different maximum glass cullet sizes were also tested, with maximum sizes ranging from 19 mm to 9.5 mm. The strength of the glass-aggregate blends was evaluated using the American Association of State Highway and Transportation Officials (AASHTO) T 190–Resistance R-value test (AASHTO T 190-09). Analysis of the data showed that glass cullet, mixed with the more angular crusher run aggregate, performed more consistently than when the cullet was combined with the rounded natural aggregate for all sizes and replacement rates. The crusher run aggregate glass-aggregate blends had average strength values above or slightly lower than the control mix across all replacement rates and maximum cullet sizes. The rounded natural aggregate blends exhibited a decrease in strength as both cullet size and replacement rate increased. The moisture-density relationships were determined in accordance with the AASHTO T 99 standard Proctor test. The maximum size of glass cullet used was shown to be insignificant in determining the optimum moisture content and maximum density of the blends. The replacement rate had a significant effect on both compaction properties. As the cullet content increased the optimum moisture content increased and the maximum density decreased.
Embankments in the transportation infrastructure use large quantities of good and marginal quality aggregate materials and are, therefore, good applications for possible use of waste materials such as glass. Senadheera et al. (2007) investigated the resilient characteristics of a blend of caliche, a marginal quality granular material, which is typically used in low traffic volume pavement subbase layers and glass cullet. The authors conducted resilient modulus tests on a series of specimens containing different caliche-glass cullet blends. Resilient modulus of granular material depends on factors such as aggregate mineralogy, particle characteristics, density, moisture content and gradation. However, when a material blend is used in addition to the factors indicated above, the material response appears to depend also on relative strengths and compatibility of constituent materials in the blend. The general acceptance has been that by blending glass cullet with conventional materials, its engineering properties, particularly the strength, decreases. However, results from resilient modulus tests indicated that for marginal quality granular materials such as caliche, the introduction of glass cullet increased the strength of the material blend. However, the gain in strength appeared to be accompanied by a likelihood of the material failing by dilation at higher stress levels. As long as the caliche-glass cullet blend was not subjected to excessive stress levels, the presence of cullet in the blend appeared to strengthen the resilient properties of the granular material.

The Washington State Department of Transportation allows up to 15% recycled glass as an additive to unbound aggregate used for 17 specific applications, including a number of fill and ballast uses (CWC 1996). Less than 10% of the glass should be retained on a 6.4 mm sieve. They also provides specifications for construction aggregates composed entirely of cullet to be used for wall backfill, rigid and flexible pipe bedding, drainage backfill, drainage blanket, and gravel borrow. The cullet must be smaller than 19 mm and should contain no more than 5% by weight of material finer than 0.075 mm sieve. The maximum debris content, including all non-glass constituents, is 10% as identified by visual methods. Moreover, the glass supplier must test the total lead content of the cullet according to Environmental Protection Agency methods 3010/6010 on a quarterly basis. The mean of these tests should not exceed 80 parts per million lead (CWC 1996).

The Oregon Department of Transportation issued special provisions with bid specifications allowing the use up to 100% recycled glass in a number of applications,
such as drainage blanket, utility bedding and backfill and subsurface drains. 100% of the glass must pass a 12.7 mm sieve, with a maximum of 5% by weight finer than 0.075 mm sieve. Maximum debris content is 5% or 10%, as specified per application, determined by visual classification (CWC 1996).

The California Department of Transportation accepted 100% cullet, or a mixture of cullet and other reclaimed materials, such as asphalt concrete, cement concrete, lean concrete base and cement treated base for Classes 1, 2, 3 and 4 bases and Classes 2 and 3 subbase roadways aggregate for the support of flexible and rigid pavements. The different classes of base and subbase aggregate are distinguished by gradations. The size of the cullet used must follow the size criteria specified for those aggregate applications by California Department of Transportation. Material used in these base and subbase aggregates must be free of organic material and other deleterious substances. Surfacing material must be placed over all aggregate bases and subbases containing glass cullet (CWC 1996).

The State of Connecticut specifies that aggregate used for roadway embankments may contain up to 25% by weight of cullet smaller than 25.4 mm (CWC 1996). Aggregate containing cullet should not be placed within five feet from the face of any slope.

The New York Department of Transportation permits aggregates containing up to 30% by volume of glass cullet for embankments (CWC 1996). In addition, up to 30% by weight of glass cullet may be used for roadway subbase material. Cullet used for these applications must be smaller than 9.5 mm, and should contain less than 5% by volume of ceramics and non-glass materials, based on visual inspection. Waste glass cannot be placed in contact with any synthetic liners, geogrids or geotextile material.

The New Hampshire Department of Transportation allows 5% glass cullet by weight of the dry aggregate used for roadway base course material and specifies that the material used to produce this cullet should consist primarily of recycled food and beverage glass containers (CWC 1996). Small amounts of ceramics and plate glass are also permitted, although glass containing hazardous or toxic materials is not allowed. The cullet must be smaller than 12.5 mm in size, and not more than 1.5% of the material smaller than a 4.75 mm sieve should be smaller than a 0.075 mm sieve. The New Hampshire
Department of Transportation requires that all base courses be tested for compliance with this gradation prior to placement. Post-placement visual inspection of the base course is also required. Base course containing cullet must be capped with non-cullet aggregate before the public is allowed to drive over the material (CWC 1996).

In December 2000, the American Association of State Highway and Transportation Officials (AASHTO) adopted a new national specification, M 318-01 Glass cullet use for soil aggregate base course (AASHTO 2000) for recycling glass in soil aggregate base courses. Up to 20% glass cullet is routinely allowed to be mixed with aggregate base course. However, the glass cullet is required to be at least 95% container/beverage glass to limit the use of other glass-like ceramics. There is a requirement to check that the resilient modulus from RLT testing and CBR is not affected/reduced due to addition of glass. The engineer may also allow higher percentages of glass cullet provided the CBR and resilient modulus does not reduce. For safety requirements AASHTO requires 99% of the glass cullet to pass the 4.75 mm sieve.

2.8 Sustainability and feasibility of recycling

The use of recycling materials in various civil engineering applications have been investigated, tested and implemented to varying degrees. Many barriers exist preventing maximum utilization, particularly the recognition or acceptance of these materials in specifications and the need for research to demonstrate suitability of recycled materials. Recycling provides direct environmental, economical and social benefits.

The price of the recycled materials is almost similar to the price of the virgin materials. But, the recycled materials and the blends of recycled materials can contribute significantly by reducing greenhouse gas emissions, and delivering significant energy and water savings, as well as preserving non-renewable virgin resources. Sustainability Victoria (2009) stated that the recycling industry is contributing to the environment by reducing greenhouse gas emissions, and delivering significant energy and water savings, as well as preserving non-renewable virgin resources. It further mentioned that life cycle analysis modelling showed Victoria saved more than 75 million gigajoules of energy in 2008-2009 by substituting recycled materials for virgin materials. In addition, 50,000 megalitres of water were saved which is enough to fill almost 20,000 Olympic-
sized swimming pools. Moreover, recycling in Victoria prevented more than 3.5 million tonnes of greenhouse gases being emitted into the atmosphere, which is equivalent to removing 588,000 cars from the road (Sustainability Victoria 2009).

Sustainability Victoria (2009) stated that Victoria’s recycling industries added approximately $144 million to the state’s economy during the 2008-2009 financial year which was a significant increase compared to the previous year.

Sustainability Victoria (2005) indicated that Victorian reprocessing industries employ over 1600 peoples associated with the recycling of secondary use materials during 2008-2005, a 19% increase over the previous year.

The Minnesota Department of transportation determined the potential net benefits of using glass in base course for Ramsay County with a population of 500,000 producing 15,000 tonnes of waste glass per year (Hyman and Johnson 2004). Currently in the Ramsay County the waste glass is taken to a landfill 130 km out of town. Discounted costs at a 7% discount rate over a 20-year period were determined for the incremental change in highway costs ($1,154,530); and the avoided costs of not disposing the waste material (i.e. mixed broken glass) in the landfill including transportation costs ($2,210,441). It was found that reusing the mixed, broken glass as a 10% additive to granular base material resulted in a savings over a 20-year period with a discounted present value of $1,055,911 ($2,210,441 minus $1,154,530) for the Ramsay County. Over a 20-year period this would equate to 300,000 tonnes of waste glass being used in base course aggregate at a net present worth benefit of $3.52 per tonne of waste glass.

2.9 Experimental studies on C&D material

The application of industrial waste in earth construction is not a recent development. The Romans were already using brick rubble and slag from forges in roads and buildings (Lidelow 2004). Portas (2004) reported that an experimental road has been built in Sardinia, Italy with a sub grade consisting of demolition waste material. The case study investigated the mechanical behaviour and reliability of sub grade layer built with construction and demolition materials. The performance of the road was monitored in both the short term and long term. The study concluded the importance of the further
studies and research in view of chemical, physical and mechanical behaviour of C&D waste materials.

Raboïotti and Caprez (2004) reported on the Swiss experience of studying the compaction characteristics and bearing capacity of recycled materials for road construction. The report investigated the compaction properties and developed an empirical relationship between bearing capacity and the degree of compaction. The bearing characteristics of the recycled materials were measured using standard proctor and modified proctor with an AASHTO and CBR mould. The research reported that the grain size and the content of residual chemical substances could affect the determination of water content resulting in unknown compaction properties and inadequate bearing capacity (Raboïotti and Caprez 2004). The study concluded that there was no easily definable relationship between compaction and bearing capacity.

Sivakumar et al. (2004) reported the performance of construction waste such as recycled concrete and brickwork sizing of 20-40 mm under repeated loading in a large (305 mm × 305 mm) direct shear apparatus. Performance of these construction wastes were compared with crushed basalt rock. The samples were tested under initial vertical stresses of 60-300 kPa and subjected to repeated shear loading for up to eight cycles. The internal friction angle reduction of basalt is reported as from 47° to 45° after eight loading cycles. The internal friction angle reduction from 43° to 38° was reported to recycled crushed concrete and a reduction from 43° to 39° was reported to brickwork under repeated loading. The reductions in the frictional resistance of recycled material were mainly due to particle crushing under repeated loading.

Phillip et al. (2008) studied a comparison of the California bearing ratio (CBR) for recycled glass (RG) with other recycled materials and a basaltic virgin aggregate, all having similar gradations. They revealed that the CBR of RG is superior to that of recycled asphalt pavement but less than that of recycled concrete and virgin aggregate. Direct shear tests were then run on the as-received gradation to derive strength parameters for RG prepared at very high and very low relative densities. For “dense” RG, the peak failure envelope was nonlinear with secant friction angles varying from 50° to 61°. For “loose” RG, the peak and critical state failure envelopes were linear with
friction angles of 41° and 38°, respectively. Bolton’s postulate that the peak friction angle is approximately equal to the critical friction angle plus 0.8 times the maximum angle of dilation works well for the RG tested in direct shear. A friction angle of 38° at critical state is significant, implying that RG has the potential to be used in even more foundation and ground improvement applications that are so often associated with transportation infrastructure construction. With increased use of recycled materials, civil engineers can help comply with the demand for sustainable development, a major theme in society today.

Wartman et al. (2004) studied the engineering properties of crushed glass-soil blends. The study revealed that the cohesive strength of the CG was increased by 50% to 100% with the addition of the fine-grained soils. The increase in cohesive strength was accompanied by a 20% to 45% decrease in frictional strength but still remained above 35 degrees. The addition of CG to kaolin and quarry fines markedly improved their frictional strength characteristics. These findings suggest that the engineering strength properties of marginal materials (e.g. dredged material, mining and quarry spoils) may be significantly enhanced through the addition of CG. This suggests that CG and CG-soil blends may be used in a number of civil, construction and geotechnical engineering applications, including embankments, compacted fill, trench backfill, retaining wall or mechanically stabilized earth (MSE) wall backfill, and roadway subbase among others.

Wartman et al. (2004) also experimentally evaluated the select engineering characteristics of crushed glass produced using two processing techniques (crushing versus screening) to an American Society of State Highway and Transportation Officials No. 10 gradation. The crushed glass samples were classified as well graded sands with gravel (SW) and exhibited excellent strength and workability characteristics. The low specific gravity (2.49) contributed to crushed glass having compacted maximum dry densities on the order of 1.69-1.71 and 1.78-187 Mg/m³ by the standard and modified Proctor compaction tests, respectively. Direct shear friction angles were measured between 47 and 62° at normal stresses ranging from 0 to 200 kPa. Friction angles obtained by drained triaxial shear were on the order of 48° for similar stress ranges. Measured hydraulic conductivities were on the order of 1-6×10⁻⁴ cm/s. The results indicate that crushed glass is a readily available, freely draining, environmentally
clean, relatively low cost material whose engineering performance properties generally equal or exceed those of most natural aggregates.

Henry et al. (1997) studied the frost susceptibility of crushed glass used as construction aggregate. They determined the frost susceptibility for 100% glass cullet specimens and 30% by weight glass cullet-aggregate specimens using ASTM D 5918. They reported that the cullet has negligible to very low frost susceptibility, and it did not increase the frost susceptibility of the aggregate.

A comprehensive laboratory evaluation of blending 9.5 mm minus curbside-collected crushed glass (CG) with dredged material (DM) was conducted by Grubb et al. (2006) to evaluate their potential for beneficial use as fill materials for urban applications. Tests (i.e. moisture content test, compaction test, specific gravity test, hydraulic conductivity test, loss on ignition test, grain size distribution, Atterberg limits, Direct Shear, Unconsolidated Undrained Triaxial Strength Test, Isotropically consolidated undrained triaxial shear tests with pore pressure measurements and consolidation) were performed on 100% CG (USCS classification SP) and 100% DM (OH) specimens and 20/80, 40/60, 50/50, 60/40, and 80/20 CG/DM blends (dry weight percent CG content reported first). They proved that the range of properties obtainable by the CG/DM blends offers a versatility that allows for the design of fills that can be potentially optimized to meet multiple design parameters (e.g. strength, settlement, drainage, or higher CG or DM content) (Grubb et al. 2006).

Grubb et al. (2006) also evaluated three crushed glass-dredged material (CG/DM) blends in the field to explore the feasibility of using CG/DM blends for embankment and structural fill applications. It was revealed that CG/DM blending offers substantial beneficial use opportunities and savings over conventional disposal (Grubb et al. 2006).

McKelvy et al. (2002) reported the shear strength of cycled construction waste material sizing of 20-40 mm under direct shear test in a large (305 mm x 305 mm) shear box. The internal friction angle of the both dry and wet condition crushed concrete material was 39° and it was reduced to 32° and 30° when it was mixed with 10% and 20% of kaolin slurry respectively. There was cohesion of about 13 kN/m² determined for crushed concrete with 20% kaolin slurry. Building debris (at least 95% of bricks by
weight) showed an internal friction angle of $37^\circ$ and $35^\circ$ at dry and wet conditions respectively. Considerable amount of particle breakage also observed during shearing process.

Serridge (2005) reported the use of recycled crushed concrete in vibro stone columns. Between 4000 ton and 4500 ton of recycled crushed concrete aggregates were used in Coatbridge, Scotland for ground improvement work as stone columns. The size of aggregates was selected between 40-75 mm as common practices in stone columns. The lengths of the stone columns were varied from 2.5 m to 6.0 m according to the soil conditions. It was further noted that the stone columns were constructed with satisfactory performance. From 600 mm diameter plate load test, an average stone column deformation modulus of 48 MN/m$^2$ was determined.

Poon and Chan (2006) have investigated the possibility of using recycled concrete aggregates and crushed clay bricks as aggregates in unbound sub base materials in Hong Kong. The use of 100% recycled concrete aggregate increased the optimum moisture content and decreased the maximum dry density of the sub base materials compared to those of natural sub base materials. In addition, the replacement of recycled concrete aggregates by crushed clay bricks further increased the optimum moisture content and decreased the maximum dry density. It was reported that the CBR value decreased with increasing coarse clay brick content. However, it was feasible to blend recycled concrete aggregates and crushed clay brick to produce a sub base with a soaked CBR value of at least 35, which is a minimum requirement in Hong Kong (Poon and Chan 2006).

Poon and Chan (2006) carried out the laboratory tests in Hong Kong using different blending percentages of recycled concrete aggregate and crushed brick. The blend ratios for the subbase materials were set out in two series as summarized in Table 2.22. Some of the mechanical properties are summarized in Table 2.23.

2.10 Geotechnical characteristics of recycled glass as construction aggregates

Several studies have been conducted regarding the use of recycled crushed glass (or glass cullet) as construction aggregate. This section evaluates the suitability of glass
cullet as a construction aggregate in terms of its properties (CWC 1993), engineering suitability (CWC 1993) and environmental suitability (CWC Study, Environmental Suitability, June 1993). The study conducted by Dames and Moore, Inc. for the Clean Washington Center was based on three independent variables. They are the maximum size of cullet, debris level in the cullet, and cullet content as a percentage of total aggregate. The maximum cullet sizes considered were 19 mm and 6.3 mm. The measurement of properties of the glass cullet was based on debris levels of 5 percent for 19 mm cullet and 2 to 3 percent for 6.3 mm cullet. The cullet contents used as a percentage of total aggregate used were 100 percent, 50 percent and 15 percent by weight.

2.10.1 Properties of glass cullet

Waste glass is being recycled for a number of uses including container glass and fiber glass manufacture. Both these uses involve color sorting and cleaning the waste glass to meet applicable standards. However, if waste glass were to be used in the highway construction industry, it would not have to be sorted by color and therefore, could be crushed immediately. This would drive the cost of glass cullet down, but it would also result in the inclusion of a number of impurities within the glass cullet mix. These impurities would include paper from labels, plastics from bottle caps, bottle corks, metal from bottle caps and any other non-glass materials. If the glass were to be cleaned of these impurities, the cost of waste glass would be very high. Therefore, any study on the feasibility of glass cullet use in construction would have to incorporate the allowable debris level as a study parameter. Debris may be defined as any materials that may impact the performance of the engineered fill if present in sufficient quantities. Organic materials may decay and result in volume reduction. Metals, ceramics, and plastic, if present in large enough quantities, can affect the engineering properties. A maximum debris level can be specified for each application of glass cullet, and when the debris level exceeds this amount, it could have a negative impact on the performance of the glass cullet in the application.

Several methods for quantifying physical contamination levels were identified in this study (CWC 1993). However, a specification should always indicate that no hazardous materials are allowed. The semi-quantitative visual classification method to obtain a
percentage or index of contamination is based on the method proposed by the American Geological Institute (AGI Data Sheet 23.1 and 23.2). The debris level obtained using this visual procedure is much higher than the debris content measured by weight or volume.

Table 2.22: Blend ratios for subbase material (Modified from Poon and Chan 2006)

<table>
<thead>
<tr>
<th>Blend</th>
<th>40 mm</th>
<th>20 mm</th>
<th>10 mm</th>
<th>&lt; 5 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>20</td>
<td>10</td>
<td>40</td>
<td>30</td>
</tr>
</tbody>
</table>

Series I (with recycled concrete aggregate as fine aggregate)

<table>
<thead>
<tr>
<th>Blend</th>
<th>RCA</th>
<th>RCA</th>
<th>CB</th>
<th>RCA</th>
<th>CB</th>
<th>RCA</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 RCA</td>
<td>20</td>
<td>10</td>
<td>0</td>
<td>40</td>
<td>0</td>
<td>30</td>
</tr>
<tr>
<td>87.5 RCA</td>
<td>20</td>
<td>7.5</td>
<td>2.5</td>
<td>30</td>
<td>10</td>
<td>30</td>
</tr>
<tr>
<td>75 RCA</td>
<td>20</td>
<td>5</td>
<td>5</td>
<td>20</td>
<td>20</td>
<td>30</td>
</tr>
</tbody>
</table>

Series II (with crushed clay brick as fine aggregate)

<table>
<thead>
<tr>
<th>Blend</th>
<th>RCA</th>
<th>RCA</th>
<th>CB</th>
<th>RCA</th>
<th>CB</th>
</tr>
</thead>
<tbody>
<tr>
<td>70 RCA</td>
<td>20</td>
<td>10</td>
<td>0</td>
<td>40</td>
<td>0</td>
</tr>
<tr>
<td>57.5 RCA</td>
<td>20</td>
<td>7.5</td>
<td>2.5</td>
<td>30</td>
<td>10</td>
</tr>
<tr>
<td>45 RCA</td>
<td>20</td>
<td>5</td>
<td>5</td>
<td>20</td>
<td>20</td>
</tr>
</tbody>
</table>

This is because paper residue, which appears to represent 10% by two-dimensional classification, may actually be less than 2% by volume or weight. In the Clean Washington Center study, the debris contamination level of glass cullet was measured by weight, by volume and by physical separation (CWC 1993). These measurements were then correlated with results from the visual method which was recommended as being satisfactory in a construction industry environment.
Table 2.23: Summary of optimum moisture content, maximum dry density and CBR
(Modified from Poon and Chan 2006)

<table>
<thead>
<tr>
<th>Modified Blend</th>
<th>Blend</th>
<th>Optimum moisture content (%)</th>
<th>Maximum dry density (Mg/m$^3$)</th>
<th>CBR (%) - 4 day soaked</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td></td>
<td>8.6</td>
<td>2.15</td>
<td>82</td>
</tr>
<tr>
<td>Series I</td>
<td>100 RCA</td>
<td>11.8</td>
<td>2.02</td>
<td>66</td>
</tr>
<tr>
<td>87.5 RCA</td>
<td>75 RCA</td>
<td>12.7</td>
<td>1.97</td>
<td>62</td>
</tr>
<tr>
<td>75 RCA</td>
<td>50 RCA</td>
<td>16.0</td>
<td>1.82</td>
<td>47</td>
</tr>
<tr>
<td>Series II</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>70 RCA</td>
<td>100 RCA</td>
<td>14.9</td>
<td>1.92</td>
<td>36</td>
</tr>
<tr>
<td>57.5 RCA</td>
<td>75 RCA</td>
<td>15.3</td>
<td>1.87</td>
<td>42</td>
</tr>
<tr>
<td>45 RCA</td>
<td>50 RCA</td>
<td>19.0</td>
<td>1.74</td>
<td>35</td>
</tr>
</tbody>
</table>

2.10.2 Cullet classification properties

The specific gravity tests conducted using ASTM C 127 (ASTM C 127-07) for coarse aggregate and ASTM D 854 (ASTM D 854-10) for fine aggregate reveal specific gravity of glass cullet range from 1.96 to 2.41 for coarse cullet and 2.49 to 2.52 for fine cullet. The specific gravity of crushed rock and gravelly sand ranges from 2.60 to 2.83 (CWC 1998). The tests also show that the higher the level of debris, the lower the specific gravity. The difference in the specific gravities of the cullet and natural aggregate affect the relative density and the unit weight of the compacted samples. The relative density test indicates that cullet content has a large effect on the density of the cullet sample. The greater the cullet content, the lesser the value of relative density. The size of cullet has a minor effect on relative density.

This is because paper residue, which appears to represent 10% by two-dimensional classification, may actually be less than 2% by volume or weight. In the Clean Washington Center study, the debris contamination level of glass cullet was measured by weight, by volume and by physical separation (CWC 1993). These measurements were then correlated with results from the visual method which was recommended as being satisfactory in a construction industry environment.
Table 2.23 indicates the results from compaction tests. Gradation tests made on the compacted samples indicate that significant changes in gradation occur only when 100 percent cullet samples are subjected to heavy compaction. Degree of gradation change decreases with decreasing glass cullet content. Further, most of the gradation changes occur in the coarse and medium sizes. The degrees of gradation change are higher for higher levels of debris. The results of the gradation tests indicate the feasibility of using compaction methods for the field control of fill materials comprised of cullet. There is minimum breakage of cullet under normal working loads. This implies that cullet particles, like crushed rock, have adequate strength to behave like an elastic body which deforms under hydrostatic loads and displaces or rotates near shear planes.

The results of LA Abrasion tests conducted using ASTM C 131 (ASTM C 131-06) indicate that cullet is not as mechanically sound as crushed rock. The debris level also plays a part in the percent loss, with higher losses at higher debris levels.

### 2.10.3 Engineering properties

Table 2.24 and Table 2.25 show results on how the compaction related properties of glass cullet aggregate mixes are influenced by the maximum size of glass cullet, cullet content in the mix, compaction method, and the type of conventional aggregate. It was observed from these results that the compacted density of cullet is affected largely by the cullet content and that density increases with decreasing cullet content. Also, the optimum moisture content increases slightly with decreasing cullet content. Higher densities are encountered at lower debris levels. The Proctor compaction curves are relatively flat, i.e., the compacted density is not sensitive to moisture content. This means that the material can be placed and effectively compacted during wet weather, keeping construction down time to a minimum.

The results from thermal conductivity tests done in accordance with the ASTM C 518 (Table 2.28) indicate that the thermal conductivities of cullet are lower than natural aggregate. Further, conductivity value decreases with increasing cullet content.
The results from the permeability tests done in accordance to the ASTM D 2434 (ASTM D 2434-06) (Table 2.27) show that permeability of cullet increases with increasing cullet content, cullet size and debris level but decreases with increasing degree of compaction. Indirect assessment of results of specific gravity, gradation and durability indicate that cullet has intermediate filtration capacity.

Direct shear and triaxial shear test results indicate that the strength of cullet is the same as natural aggregate. Also, cullet content and debris level do not have appreciable effect within the test range. The addition of smaller size cullet to a natural aggregate reduces the potential to have plastic volumetric strain and also tends to increase the bulk modulus (CWC 1993).

The CBR value is a common parameter used in flexible pavement design. Typical values of a compacted granular material range from 40 to 80 (NYDOT 1995). CBR tests were conducted using the ASTM D 1883 (ASTM D 1883-07e2) test procedure. The test results indicate that the CBR values of all the cullet-added samples are within this typical range. The test data also indicate that adding 15% cullet to the crushed rock does not produce a noticeable difference in the CBR value. However, as the cullet content increases to 50%, an obvious reduction in the CBR value is shown. For those samples prepared using the impact compactor, this reduction was about 25% when the cullet content increases from 15% to 50%. A much higher reduction was noted for samples prepared using the vibratory compactor. The reduction in this case was about 50%. This discrepancy implies the importance of choosing the correct specimen preparation method for materials with cullet content over 15% (CWC 1993).
Table 2.24: Change in Gradation with Different Compaction Procedures (CWC 1993).

<table>
<thead>
<tr>
<th>Max. Cullet Content (mm)</th>
<th>Cullet Content (%)</th>
<th>Size Fraction</th>
<th>Std. Proctor ASTM D 698-07e</th>
<th>Mod. Proctor ASTM D 1557-09</th>
<th>Vibratory WSDOT 606-11</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>B.C.</td>
<td>A.C.</td>
<td>B.C.</td>
<td>A.C.</td>
</tr>
<tr>
<td>19</td>
<td>100</td>
<td>gravel</td>
<td>86.4</td>
<td>84.5</td>
<td>83.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>sand</td>
<td>13.4</td>
<td>14.9</td>
<td>16.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>fines</td>
<td>0.2</td>
<td>0.6</td>
<td>0.4</td>
</tr>
<tr>
<td>15</td>
<td>gravel</td>
<td>33.1</td>
<td>34.1</td>
<td>57.2</td>
<td>51.1</td>
</tr>
<tr>
<td></td>
<td>sand</td>
<td>66.2</td>
<td>65.0</td>
<td>40.9</td>
<td>44.3</td>
</tr>
<tr>
<td></td>
<td>fines</td>
<td>0.7</td>
<td>0.9</td>
<td>1.9</td>
<td>4.6</td>
</tr>
<tr>
<td>6.3</td>
<td>100</td>
<td>gravel</td>
<td>0.0</td>
<td>0.0</td>
<td>8.2</td>
</tr>
<tr>
<td></td>
<td>sand</td>
<td>99.4</td>
<td>98.7</td>
<td>90.8</td>
<td>91.7</td>
</tr>
<tr>
<td></td>
<td>fines</td>
<td>0.6</td>
<td>1.3</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>15</td>
<td>gravel</td>
<td>21.6</td>
<td>21.9</td>
<td>---</td>
<td>41.3</td>
</tr>
<tr>
<td></td>
<td>sand</td>
<td>77.6</td>
<td>76.9</td>
<td>---</td>
<td>53.7</td>
</tr>
<tr>
<td></td>
<td>fines</td>
<td>0.8</td>
<td>1.2</td>
<td>---</td>
<td>5.0</td>
</tr>
</tbody>
</table>

Note: B. C.: Before Compaction; A. C.: After Compaction
Table 2.25: Compaction Test Parameters for Gravely Sand and Crushed Rock when Combined with Glass Cullet at Different Percentages (CWC 1993).

<table>
<thead>
<tr>
<th>Max. Cullet Size (mm)</th>
<th>Cullet (%</th>
<th>Proctor Test Method</th>
<th>Optimum Moisture Content (%)</th>
<th>Max. Dry Density (Mg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>19 (Gravely Sand)</td>
<td>15</td>
<td>Standard</td>
<td>5.7</td>
<td>2.09</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>Standard</td>
<td>6.0</td>
<td>1.99</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>Standard</td>
<td>5.5</td>
<td>1.59</td>
</tr>
<tr>
<td>6.3 (Gravely Sand)</td>
<td>15</td>
<td>Standard</td>
<td>6.5</td>
<td>2.02</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>Standard</td>
<td>6.5</td>
<td>1.91</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>Standard</td>
<td>4.7</td>
<td>1.67</td>
</tr>
<tr>
<td>19 (Crushed Rock)</td>
<td>15</td>
<td>Modified</td>
<td>6.0</td>
<td>2.22</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>Modified</td>
<td>6.2</td>
<td>2.00</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>Modified</td>
<td>7.5</td>
<td>1.78</td>
</tr>
<tr>
<td>6.3 (Crushed Rock)</td>
<td>15</td>
<td>Modified</td>
<td>5.5</td>
<td>2.22</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>Modified</td>
<td>9.2</td>
<td>2.02</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>Modified</td>
<td>5.6</td>
<td>1.78</td>
</tr>
</tbody>
</table>

Table 2.26: Compaction Test Parameters for Gravely Sand and Crushed Rock without Glass Cullet (CWC 1993).

<table>
<thead>
<tr>
<th>Description</th>
<th>Test Method</th>
<th>Optimum Moisture Content (%)</th>
<th>Max. Dry Density (Mg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushed Rock</td>
<td>Standard Proctor</td>
<td>7.2</td>
<td>2.27</td>
</tr>
<tr>
<td>Gravely Sand</td>
<td>Modified Proctor</td>
<td>8.3</td>
<td>2.15</td>
</tr>
</tbody>
</table>

Table 2.27: Results from the Constant Head Permeability Test (CWC 1993).

<table>
<thead>
<tr>
<th>Cullet Size (mm)</th>
<th>Cullet Content (%)</th>
<th>Dry Density (Mg/m³)</th>
<th>Permeability (10⁻² cm/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>19</td>
<td>100</td>
<td>1.44</td>
<td>26.0</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>1.81</td>
<td>4.8</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>1.90</td>
<td>3.1</td>
</tr>
<tr>
<td>6.3</td>
<td>100</td>
<td>1.52</td>
<td>6.0</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>1.73</td>
<td>4.4</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>1.85</td>
<td>2.6</td>
</tr>
</tbody>
</table>
Note: Approximate relative compaction is 90 percent of ASTM D698

Table 2.28: Results from the Thermal Conductivity Test (CWC 1993).

<table>
<thead>
<tr>
<th>Max. Cullet Size (mm)</th>
<th>Type of Natural Aggregate</th>
<th>Cullet Content (%)</th>
<th>Moisture Content (%)</th>
<th>Sample Density (Mg/m³)</th>
<th>Apparent Thermal Conductivity (W/m-K)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.3</td>
<td>-</td>
<td>100</td>
<td>10.7</td>
<td>1.32</td>
<td>0.315</td>
</tr>
<tr>
<td>6.3</td>
<td>-</td>
<td>50</td>
<td>7.4</td>
<td>1.52</td>
<td>0.463</td>
</tr>
<tr>
<td>-</td>
<td>Natural Sand</td>
<td>0</td>
<td>6.6</td>
<td>1.71</td>
<td>0.638</td>
</tr>
</tbody>
</table>

The Resistance Value (R-value) test is a material stiffness tests and relates indirectly to the strength of the material (AASHTO T 190-09). The value is commonly used to specify base or subbase aggregate. Generally, the required R-value is higher for the base than for the subbase materials. Resistance R-Value tests were performed using the WSDOT T 611 (WSDOT T 611-11) test procedure. This test procedure is a modification of the AASHTO T 190 test method. The modification involves using 15 to 25 blows of kneading compaction at 689 and 1724 kPa pressure, respectively, which are lower than those specified in the AASHTO T 190 (AASHTO T 190-09) method. The test results indicate that adding cullet to crushed rock will reduce the R-Value slightly, and this reduction will increase with increasing cullet content. From the test results of the cullet samples, it is clear that the cullet added crushed rock, with cullet content up to 50%, processes adequate strength for both base and subbase aggregate (CWC 1993).

The resilient modulus (cyclic triaxial) is a measure of a material’s stiffness and can be used for pavement design. The resilient modulus of natural aggregate is typically about 207 MPa at a bulk stress of 173 kPa. The resilient modulus tests were performed using a modified AASHTO T 294 (AASHTO T 294-94) test procedure where an internal load cell was used instead of an external load cell. Addition of cullet to crushed rock will reduce the resilient modulus and this reduction will increase with increasing cullet content. In order to address the concern regarding the use of cullet in roadway construction, the change of resilient modulus of the cullet samples over the first 1000 cycles is compared with that of crushed rock. It was found that cullet, like crushed rock, does not show appreciable change in the modulus value. Note that the samples were
subjected to a confining pressure of 28 kPa and deviator stress of 56 kPa in the first 1000 cycles. This stress level is typical of a subbase material under medium to heavy traffic loads. For the crushed rock material, this stress level is much lower than the level at which crushing or breaking of the crushed rock particles would occur. This implies that the cullet samples, like the crushed rock, did not experience any appreciable breaking or crushing of particles (CWC 1993).

2.10.4 Engineering suitability of glass cullet

This section deals with the suitability of glass cullet in various engineering applications (CWC 1993).

Roadways

Roadway applications include the use of cullet aggregate in base course, subbase, subgrade and embankments. Cullet is appropriate for use in all these applications depending on cullet percentages. Table 2.29 lists recommended percentages, debris levels, and compaction levels for cullet use in roadway applications.

Embankments

Disfani et al. (2011, 2012) carried out an extensive research works on the feasibility of using recycled glass in embankment applications.

Drainage

Drainage applications include retaining wall backfill, footing drains, drainage blankets, and French drains. Recommended specifications on cullet content, debris content and compaction level are indicated in Table 2.30.

General backfill

General Backfill applications include fills that support heavy stationary loads such as fills beneath footings and slabs, fluctuating loads such as those supporting reciprocating
pumps and compressors, as well as non-loaded fills such as landscaping fill or fill placed beneath pedestrian sidewalks. The recommendations for the use of glass cullet in these applications are shown in Table 2.31.

**Utilities**

Utility applications involve the use of cullet aggregate for trench bedding and backfill, and the recommendations are shown in Table 2.32. The specifications listed in Table 2.29 apply to backfill, which is not subjected to surcharge loading such as from a roadway or slab. If the trench backfill lies within five feet of a loading area, then the specifications provided in *General Backfill* in Table 2.31 would apply.

**Miscellaneous**

Miscellaneous uses of cullet aggregate include landfill cover and underground storage tank (UST) backfill. Model specifications for these applications are presented in Table 2.33.

### 2.10.5 Environmental suitability

The environmental test program included three components (CWC 1993). They are the organic and inorganic chemical characterization of feedstock and an assessment of the potential for bacteria growth, an assessment of contaminant leachability over time, and an assessment of the incidence of lead and leachable lead contamination in different feedstock.

The total lead tests were performed on samples received from all the sources and a subset of the samples were analyzed for leachable lead. Chemical and contaminant leaching over time were conducted on three representative "high debris level" and "low debris level" samples. The chemical characterization tests assessed pH value, priority pollutant total metals, semi-volatile organics and total organic carbon. Contaminant leaching over time was assessed by performing a sequential batch extraction using aqueous solution followed by pH analysis, biological oxygen demand, chemical oxygen demand, total organic carbon and total metals. No appreciable environmental impact
was detected. All the cullet sources but one exhibited total and leachable lead concentrations below acceptable regulatory limits. The lead concentrations found in the anomalous sample was attributed to lead foil wrappers. Limited organic compounds were found in the high and low debris cullet samples and included samples of plastics, low concentration of food residues, and organics that occur naturally in nature. The evaluation of the contaminant leaching over time indicated that little or no potential exists for supporting bacteria growth and that metal concentrations did not appear to pose a risk to the environment.

Table 2.29: Recommended Cullet Levels for Roadway Applications (CWC 1993).

<table>
<thead>
<tr>
<th>Roadway Applications</th>
<th>Maximum Cullet Content (%)</th>
<th>Maximum Debris Content (%)</th>
<th>Minimum Compaction Level (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Course</td>
<td>15</td>
<td>5</td>
<td>95</td>
</tr>
<tr>
<td>Subbase</td>
<td>30</td>
<td>5</td>
<td>95</td>
</tr>
<tr>
<td>Embankments</td>
<td>30</td>
<td>5</td>
<td>90</td>
</tr>
</tbody>
</table>

Table 2.30: Recommended Use of Glass Cullet in Drainage Applications (CWC 1993).

<table>
<thead>
<tr>
<th>Drainage Fill Applications</th>
<th>Max Cullet Content (%)</th>
<th>Max Debris Content (%)</th>
<th>Min Compaction Level (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Retaining Wall</td>
<td>100</td>
<td>5</td>
<td>95</td>
</tr>
<tr>
<td>Foundation drain</td>
<td>100</td>
<td>5</td>
<td>95</td>
</tr>
<tr>
<td>Drainage Blanket</td>
<td>100</td>
<td>5</td>
<td>90</td>
</tr>
<tr>
<td>French Drain</td>
<td>100</td>
<td>5</td>
<td>90</td>
</tr>
</tbody>
</table>

Table 2.31: Recommended Use of Glass Cullet in General Backfill Applications (CWC 1993).

<table>
<thead>
<tr>
<th>General Backfill Applications</th>
<th>Max Cullet Content (%)</th>
<th>Max Debris Content (%)</th>
<th>Min Compaction Level (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stationary Loads</td>
<td>30</td>
<td>5</td>
<td>95</td>
</tr>
<tr>
<td>Fluctuating Loads</td>
<td>15</td>
<td>5</td>
<td>95</td>
</tr>
<tr>
<td>Non-Loading General fill</td>
<td>100</td>
<td>10</td>
<td>85</td>
</tr>
</tbody>
</table>
Table 2.32: Recommended Use of Glass Cullet in Utilities Applications (CWC 1993).

<table>
<thead>
<tr>
<th>Utility Trench Bedding and Backfill Applications</th>
<th>Max Cullet Content (%)</th>
<th>Max Debris Content (%)</th>
<th>Min Compaction Level (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water &amp; Sewer Pipes</td>
<td>100</td>
<td>5</td>
<td>90</td>
</tr>
<tr>
<td>Electrical Conduits</td>
<td>100</td>
<td>5</td>
<td>90</td>
</tr>
<tr>
<td>Fiber Optic Lines</td>
<td>100</td>
<td>5</td>
<td>90</td>
</tr>
</tbody>
</table>

Table 2.33: Recommended Use of Glass Cullet in Miscellaneous Applications (CWC 1993).

<table>
<thead>
<tr>
<th>Miscellaneous Applications</th>
<th>Max Cullet Content (%)</th>
<th>Max Debris Content (%)</th>
<th>Min Compaction Level (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Landfill Cover</td>
<td>100</td>
<td>5</td>
<td>90</td>
</tr>
<tr>
<td>UST Backfill</td>
<td>100</td>
<td>5</td>
<td>90</td>
</tr>
</tbody>
</table>

2.11 Alternative uses of glass

In Minnesota some landfills accept broken glass as daily cover at a discounted cost because broken glass is rather innocuous and resembles aggregates (Hyman and Johnson 2004).

Recycled glass as a backfill material around drainage pipes was used in a demonstration project in Bruceville, Indiana (Siddiki et al. 2004). The type of recycled glass used was comparable to INDOT B borrow (granular fill) as in INDOT standard specifications. For 100% recycled glass to be used as bedding material, the followings are required: (a) the source of the crushed glass should be glass beverage and food containers that have been processed by equipment specifically designed to crush glass into aggregate; (b) the resultant material must be relatively free of bottle caps, labelling paper, clay balls and other unsuitable materials; and (c) 100% crushed glass must have particle size gradations as indicated in Table 2.21. On the basis of both construction data and recent observations of the site, the embankment and pavement continue to perform well.

Foamed glass has also been used as an insulating layer over the subgrade in frost prone areas subject to freeze and thaw cycles (Kestler 2004). The United Kingdom has
approval to use up to 10% glass in their ‘Foamix’ bound subbase product (ENVIROS 2003). These applications for glass are being developed by the United Kingdom’s largest companies, Tarmac and Lafarge.

2.12 Glass with cemented aggregates

A 1974 American Society for Testing and Materials (ASTM) study proved that glass with cement and aggregate causes an alkali-silica reaction (ASR) (Ahmed 1993). The reaction between glass and cement causes an expansion of glass which reduces the strength of the concrete. The workability of the concrete mix is also affected due to the elongated particles typical of glass cullet.

Due to the reaction between cement and glass, California Department of Transportation (Ahmed 1993) prohibits the use of glass as an aggregate substitute in Portland cement concrete, cement-treated base, lean concrete base and cement-treated permeable base.

However, the major drawback of glass is that the alkali in the cement can react with the silica in the glass. ASR produces a gel on the aggregate surface which swells and can cause the concrete to crack. In the United Kingdom the concrete industry has avoided using glass as an aggregate, particularly as the ASR may take several years to manifest itself. However, recent work in the United Kingdom has demonstrated that ASR can be avoided by either using a fine-sized glass aggregate, less than about 1 mm, or by suppressing the reaction with admixtures or using a low-alkali cement. Other work has shown that with additions of very fine glass powder (<45 microns) the glass undergoes pozzolanic reactions, which have the potential to increase the concrete strength. The uses of glass aggregates in concrete are being investigated by the two Waste and Resources Action programme (WRAP)-funded projects, one at the University of Dundee and one at the University of Sheffield (ENVIROS 2003).

2.13 Foamed waste glass

Foamed waste glass (FWG) newly developed lightweight, opaque glass material having a closed-cell structure (Lu and Onitsuka 2004).
Foamed glass grain can be used wherever a finely grained, free-flowing bulk material is required. It is especially suitable for light-weight embankment fill (density of around 0.3 Mg/m³) and thin-walled thermal insulations, such as for window frames, cement bricks and insulating plasters (Arnold et al. 2008). Other uses are:

- road construction: lightweight bulk material with excellent drainage properties
- pile foundation
- building insulation
- trench filling material with high compression strength
- stabilizing of slipping soil
- insulation of sports grounds
- insulation of swimming pools
- roof insulation.

Research in Japan (Onitsuka and Shen 1999) evaluated lime-stabilised clay with foamed waste glass as a pavement material. They found the CBR of the coarser foamed waste glass (2.00 mm to 4.75 mm particle size) was 31% and the unconfined compressive strength (UCS) was 3.5 MPa. Mixtures of coarse foamed waste glass (10, 15, 20 and 25%) and lime (5, 10, 15 and 20%) stabilized clays at 7 and 28-day curing periods were tested in the laboratory. The highest values recorded were a UCS of 1.8 MPa and CBR of 72%, these were for the clay mixture with the greatest quantities of foamed glass and lime used in this study.

2.14 Safety measures

Cullet is an abrasive material and irritation can be prevented with the use of protective clothing. Safety clothing used when working with natural aggregate can be worn while working with cullet (CWC 1993).

Recycled glass aggregate is a relatively new alternative construction material. To promote its use, studies have been conducted to determine its engineering performance and environmental impact. The studies suggest that cullet aggregate is strong, clean, and safe to use. However, because cullet aggregate remains a relatively unfamiliar material, safety concerns surrounding the use of “crushed glass” may discourage its use. These concerns generally involve the potential for skin cuts and punctures, and exposure to
glass dust during transportation, handling, and placement. In order to ease concerns and to maintain a safe working environment, information about necessary precautions is needed.

The most common health concern regarding the use of cullet aggregates is the potential for skin cuts or penetration. Workers may come into physical contact with cullet particles during transportation or placement of the cullet. Experience has shown that cullet 19 mm or smaller presents no greater cut or penetration hazard than fractured natural aggregates such as crushed rock. Therefore, the same safety precautions for working with natural aggregates should also be followed when working with cullet.

Exposure to glass dust is another health concern with cullet aggregate. Studies have been conducted to determine whether glass dust contains crystalline silica, a known carcinogen. These studies have found that glass aggregate dust typically contains less than 1% crystalline silica by weight and is not considered hazardous by federal standards. This places cullet in the category of “nuisance dust” with a permissible exposure limit of 10 mg/m³ (CWC 1993).

Glass cullet dust can be a skin and eye irritant. Cullet dust is abrasive due to the high angularity of its particle shapes, and appears to be more irritating than dust from natural aggregates or soils. However, experience from construction sites indicates that cullet dust, and the irritations associated with the dust, can be easily prevented using simple measures. For information regarding construction site dust control measures, refer to the Best Practices Dust Control with Glass Aggregate at Construction Sites and Analysis of Glass Dusts.

The following safety precautions are based on the field experience of construction site personnel (CWC 1996):

Before every construction project, the use of cullet aggregate should be discussed with the owner, engineers, general contractor, contractor’s earthwork sub-contractor, labor foremen and laborers. The discussion should include but not be limited to the following items:
The advantages and disadvantages of using cullet as a construction aggregate and the merit of cullet fill. The awareness of the rationale for using a new construction material at all levels of the crew tends to mitigate concern, and to facilitate the cost-effective use of the material.

Proper procedures for handling, placing and compacting glass. This discussion will allow project team members and workers to realize that glass aggregate will behave similarly to a natural aggregate; and, that the cullet aggregate has good workability in terms of handling and compaction.

All personnel should know that direct skin contact with glass cullet should be avoided. To protect against possible cuts or penetration injuries, site personnel working with cullet should wear long sleeves, pants, gloves, work boots, hard-hats, ear protection, and eye protection. Shirt sleeves and pant legs can be taped for additional protection. Site personnel should also be instructed not to sit, kneel, or lay on cullet surfaces, or work surfaces containing cullet. Furthermore, working surfaces should be kept clean of cullet particles by sweeping.

Construction personnel should be made aware of the potential inhalation hazard and skin and eye irritation from cullet dust. To minimize exposure of glass dust to skin, ears, and eyes, site personnel should use the same protective gear listed above for protection against cuts and penetrative wounds. To protect against dust inhalation, workers can also wear disposable nuisance dust masks. Samples of the glass should be brought to the meeting so personnel know what to expect.

Although all personnel should have knowledge of dust control measures, responsibilities should be clearly assigned. Minimizing cullet dust hazards should begin with a dust control program. As with any aggregate, the need for dust control is most obvious during dry weather. Since glass has a specific gravity less than that of natural aggregate, the fines from cullet aggregate may be more prone to becoming airborne. On construction sites, cullet dust can be generated when the cullet is delivered and end-dumped from trucks. Handling and stockpiling of cullet aggregate on-site can also create a dust cloud. Site personnel involved in handling or stockpiling cullet should monitor for potential cullet dusting, and be prepared to implement dust control
measures. Wet suppression using a garden hose is the most common and effective measure of dust control. Since cullet aggregates are generally free-draining, the application of water to cullet generally does not adversely impact its compaction characteristics.

Cullet may draw the attention of curious onlookers or passers-by. For maximum safety, measures should be taken to minimize public access to areas where cullet is being used or stockpiled. These areas should be surrounded by cautionary tape, and cullet stockpiles should be placed in low visibility or minimum access areas.
CHAPTER THREE

3 SAMPLING AND LABORATORY TESTING METHODOLOGY

3.1 Introduction

This chapter describes the sampling and the test methods used to determine the geotechnical characteristics of recycled glass, crushed rock, recycled crushed concrete and recycled crushed glass blended with various proportions of recycled crushed concrete (Class 3) and crushed rock (Class 3). The laboratory testing program comprised of the following tests:

- Static Triaxial
- Repeated Load Triaxial
- Direct Shear test
- Particle Size Distribution
- Particle Density
- Water Absorption
- Modified Compaction
- California Bearing Ratio (CBR)
- pH value
- Fines Content
- Plasticity Index
- Organic Content
- Los Angeles Abrasion Loss
- Wet and Dry Strength Variation
- Flakiness Index and
- Permeability

The blended mixtures comprised recycled glass with Class 3 recycled crushed concrete (RCC) and recycled glass with Class 3 crushed rock (CR). A suite of blended mixtures of 10%, 15%, 20%, 30%, 40% and 50% recycled glass with recycled crushed concrete and crushed rock were tested. The blend mixtures were prepared using the splitter to the
required percentages by mass. The details of the recycled crushed glass blended with recycled crushed concrete and crushed rock for each of the test are presented in Table 3.1.

Table 3.1: Recycled Crushed Glass Blends

<table>
<thead>
<tr>
<th>Sample description</th>
<th>Blending percentages</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Crushed Rock (CR) Class 3</td>
</tr>
<tr>
<td>RG10/CR90</td>
<td>90</td>
</tr>
<tr>
<td>RG15/CR85</td>
<td>85</td>
</tr>
<tr>
<td>RG20/CR80</td>
<td>80</td>
</tr>
<tr>
<td>RG30/CR70</td>
<td>70</td>
</tr>
<tr>
<td>RG40/CR60</td>
<td>60</td>
</tr>
<tr>
<td>RG50/CR50</td>
<td>50</td>
</tr>
<tr>
<td>RG10/RCC90</td>
<td>90</td>
</tr>
<tr>
<td>RG15/RCC85</td>
<td>85</td>
</tr>
<tr>
<td>RG20/RCC80</td>
<td>80</td>
</tr>
<tr>
<td>RG30/RCC70</td>
<td>70</td>
</tr>
<tr>
<td>RG40/RCC60</td>
<td>60</td>
</tr>
<tr>
<td>RG50/RCC50</td>
<td>50</td>
</tr>
</tbody>
</table>

3.2 Sampling locations and sources of recycled materials

Samples of recycled glass (5 mm nominal size), Class 3 crushed rock and recycled crushed concrete (20 mm nominal size) were collected from the Alex Fraser Group recycling site in Victoria, located approximately 20 km to the west of Melbourne, Victoria, Australia. Following standard procedures and taking all necessary precautions, representative samples were collected from the stockpiles and transported to Geomechanics laboratory at Swinburne University of Technology in sealed plastic bags.

Recycled glass used in this research was derived from household waste collection as well as C&D activities. Recycled glass in stockpiles characteristically becomes segregated by size and contaminants such as closures, labels, or miscellaneous debris.
(CWC 1996). Therefore, all necessary precautions were taken to capture a sample containing representative particle sizes and representative amounts of debris.

Recycled glass processed from containers comes mainly from empty soft drink, beer, food, wine, and liquor containers. It is generally collected from residential curbside, drop boxes, trash barrels, deposit stations, or recycling stations, and is either source-separated or co-mingled with plastics, aluminium cans, ceramics, or colored glass containers (Landris 2007). Figure 3.1 shows the stockpiles of processed 4.75 mm minus RG at Alex Fraser, Laverton site, Figure 3.2 shows the close up view of the 4.75 mm minus RG, and Figure 3.3 shows the as-received 4.75 mm minus RG (scale in mm).

Figure 3.1: Stockpiles of processed 4.75 mm minus Recycled Glass at Alex Fraser, Laverton site

The crushed rock used in this study was originated from basalt floaters or surface excavation rock, which commonly occurs near the surface to the west and north of Melbourne. Traditionally this material would have been discarded as waste (often into landfill). However, because this rock is generally hard and durable (LA<25) VicRoads has allowed (under controlled conditions) its use for pavement subbase (Class 3) and other uses. The rock is often encountered in sub-divisional excavation for residential
properties and in excavation works for drainage lines as well as other sub-surface infrastructure. Figure 3.4 shows the stockpiles of crushed rock at Alex Fraser Recycling, Laverton.

Figure 3.2: Close up view of the 4.75 mm minus RG

Figure 3.3: As-received 4.75 mm minus RG (scale in mm)
The RCC used in this research was mainly obtained from C&D materials. Figure 3.5 shows the stockpiles of recycled crushed concrete at Alex Fraser Recycling, Laverton.
The particle size distribution tests are also referred to as particle size analysis, sizing tests or mechanical analysis tests. Two separate and quite different procedures are used, in order to span the very wide range of particle sizes which are encountered. These are the sieving and the sedimentation procedures. Sieving is used for gravel and sand size (coarse) particles, which can be separated into different size ranges with a series of sieves of standard aperture openings. Sieving cannot be used for the very much smaller silt and clay size (fine) particles, so a sedimentation procedure is used instead for particles passing 75 μm sieve. For soils containing both coarse and fine particles, composite tests using both sieving and sedimentation methods have to be carried out if a full particle size analysis is required (Head 1994).

There are two different procedures for sieving. They are dry sieving and wet sieving. According to the British Standard (BS 1377-1990), dry sieving may be carried out only on materials for which this procedure gives the same results as the wet-sieving procedure. This means that it is applicable only to clean granular materials, which usually implies clean sandy or gravely soils—that is, soils containing negligible amounts of particles of silt or clay size. Normally the wet-sieving procedure should be followed for all soils.

In this research, particle size distribution tests by sieving were performed in accordance with AS 1141.11 “Particle size distribution by sieving” (AS 1141.11-1996). The Australian Standard sieves were used with the aperture sizes of 26.5 mm, 19 mm, 13.2 mm, 9.5 mm, 6.7 mm, 4.75 mm, 2.36 mm, 1.18 mm, 600 μm, 425 μm, 300 μm, 150 μm and 75 μm. As the nominal size of the aggregates was 20 mm, the minimum amount of 3 kilograms of blends was washed through the 75 μm sieve with a guard sieve of 1.12 mm as shown in Figure 3.6. As the nominal size of the recycled glass was 5 mm, a minimum amount of 300 gm was taken for the sieve analysis of recycled glass. The soil is placed a little at a time on the 1.12 mm sieve, and washed over a sink with a jet or spray of clean water, as shown in Figure 3.7. The silt and clay passing the 75 μm sieve was allowed to run to waste. When the material on the 1.12 mm guard sieve had been washed free of fines, washing on the 75 μm sieve was continued until the waste water is seen to run clear. The whole of the material retained on each sieve was allowed to drain, and was carefully transferred to trays. These were placed in an oven to dry at 105°C,
preferably overnight. Later, the oven dried blends were cooled down to room temperature. After cooling, the whole of the dried material was put together and weighed to an accuracy of 0.01%. The dry soil was sieved for approximately 10 minutes through the above mentioned sieves which were mounted with mechanical sieve shaker as shown in Figure 3.8. The retaining mass of blends on each sieve was recorded and grading curves were plotted for each blend. The coefficient of uniformity ($C_u$) and coefficient of curvature ($C_c$) was calculated from the particle size distribution curve using the following equations.

$$C_u = \frac{D_{60}}{D_{10}}$$  
Equation 3.1

$$C_u = \frac{(D_{10})^2}{(D_{60}) \times (D_{10})}$$  
Equation 3.2

where,

$D_{10}$ = particle diameter (mm) corresponding to 10% passing by weight

$D_{30}$ = particle diameter (mm) corresponding to 30% passing by weight

$D_{60}$ = particle diameter (mm) corresponding to 60% passing by weight

$C_u$ = coefficient of uniformity

$C_c$ = coefficient of curvature

Very poorly graded soils (e.g. beach sands) have $C_u$ value of 2 or 3, whereas very well graded soils may have a $C_u$ 14 or greater (Holtz and Kovacs 1981). A well graded soil has a coefficient of curvature between 1 and 3 as long as the value of $C_u$ is also greater than 4 for gravels and 6 for sands (Holtz and Kovacs 1981).

For the materials passing the 75 μm sieve, sedimentation process/hydrometer analysis were performed for grain size distribution. Hydrometer analysis tests were performed according to ASTM D 422-63 (2002) “Standard test method for particle size analysis of soils”. Dispersing agent was prepared with sodium hexametaphosphate and distilled water, at the rate of 40 g of sodium hexametaphosphate/litre of solution. The 152H type hydrometer was used for hydrometer analysis. 50 g of sample passing 75 μm sieve was placed into the 250 ml beaker and covered with the 125 ml sodium hexametaphosphate solution (40g/l). The mixture was stirred well and allowed to soak for around 16 hours.
After soaking, the mixture was poured into the stirring apparatus and carefully washed the remaining from the beaker to the stirring cup with distilled water. More distilled water was added to make the cup more than half full and stirred for about 1 minute. Immediately after dispersion, the slurry was transferred into the glass sedimentation cylinder and distilled water was added to make total volume of 1000 ml. The cylinder was agitated by turning the cylinder upside down and backed for a period of 1 minute by using the palm of the hand over the open end of the cylinder.
After that, the hydrometer was carefully inserted into the cylinder and the reading was recorded. The hydrometer was instantly taken out from the cylinder smoothly after the reading. The hydrometer reading was taken at specified time intervals. Percentages of soil in suspension were calculated with correction factors and hydrometer readings. By
using stroke’s law the particle diameter was determined. Figure 3.9 shows the arrangement for hydrometer analysis used in this research.

3.2.1 Particle density

The particle density is defined as the average mass per unit volume of the solid particles in a sample of soil, where the volume includes any sealed voids contained within solid particles. It is denoted by the symbol \( \rho_s \). It is also numerically equal to the specific gravity \( G_s \).

The particle density is rarely used as an index for soil classification. But knowledge of the particle density is essential in relation to some other soil tests, especially for calculation of porosity and voids ratio, and is particularly important when compaction and consolidation properties are considered. The particle density must be known for the computation of particle size analysis from a sedimentation procedure.

Particle density tests for coarse material retained on 4.75 mm sieve were performed in accordance with AS 1141.6.1 “Particle density and water absorption of coarse aggregate -Weighing-in-Water Method” (AS 1141.6.1-2000). Particle density tests for fine material passing 4.75 mm sieve were performed in accordance with AS 1141.6.1 “Particle density and water absorption of fine aggregates” (AS 1141.6.1-2000). As the amount of blends passing the 4.75 mm sieve was more than 10%, the particle density and water absorption tests were performed in coarse fraction and fine fraction separately and the weighted average particle density was calculated as specified in the standards.

Coarse aggregate

A minimum amount of 2 kg of blends were prepared by sieving the material over 19 mm sieve and retaining on 4.75 mm sieve. The sample was thoroughly washed to remove dust or other coatings from the surface of the particles. The material was then immersed in water at room temperature for a period of at least 24 hours. The material was covered at least 20 mm of water above the top of the material. Afterwards, the material was transferred into a wire basket and immersed in a water bath below the balance. The basket was jiggled to remove the air bubbles and then hanged to the
balance. The weight of the basket with the material under water was recorded. Then the basket was removed and the material was allowed to drain. As both the particle density and water absorption had to be determined, the surfaces of the particles were dried by using dry cloth. The material was surface dried until all visible films of water had been removed but the surfaces of particles still appeared damp. The weight of the surface dry material was recorded. After that, the material was dried at 105°C to constant weight and the weight was recorded. Figure 3.10 shows the apparatus used for the determination of particle density of coarse aggregate. The particle density of the coarse aggregate was determined as follows:

\[
\text{Apparent particle density (Mg/m}^3\text{)} = \frac{m_2 \times \rho_w}{m_2 - (w_1 - w_2)}
\]

Equation 3.3

where,
\[
\rho_w = \text{density of water at test temperature (Mg/m}^3\text{)}
\]
\[
w_1 = \text{weight of basket and materials under water (g)}
\]
\[
w_2 = \text{weight of basket under water (g)}
\]
\[
m_2 = \text{mass of dry material (g)}
\]

Figure 3.10: Apparatus used for particle density of coarse aggregate
Fine aggregate

A minimum amount of 500 g of blends were prepared by sieving the material through 4.75 mm sieve. The material was immersed in water at room temperature for a period of at least 24 hours. Afterwards, the material was drained and spread on flat impervious surface. The material was surface dried by using gently rotating small fan. Then the part of the material was loosely filled into the conical mould (Figure 3.11a) and tamped by a tamping rod (Figure 3.11b) for 25 times with a fall of 10 mm above the surface of aggregate. The conical mould was lifted vertically and checked for slumping of the aggregates to occur. The test was performed until a slumping had been observed by continuing drying and repeating the above steps. As the slumping of aggregates indicates the saturated surface dry condition, the weight of the material at slumping stage was recorded. The material was then placed into the volumetric flask and filled with distilled water to the 500 ml mark. The flask was jiggled to remove the air bubbles. The weight of the flask and its contents were recorded. Then the material was placed in a tray and oven dried at 105°C to constant weight and recorded the weight. The flask was filled with distilled water to 500 ml mark and the weight of flask and water was recorded. The particle density of the fine aggregate was determined as follows:

Apparent particle density (Mg/m$^3$) = $\frac{m_4 \times \rho_w}{m_3 + m_4 - m_2}$

where,

$\rho_w$ = density of water at test temperature (Mg/m$^3$)

$m_2$ = mass of the flask filled with water and fine materials (g)

$m_3$ = mass of the flask filled with water (g)

$m_4$ = mass of dry material (g)
3.2.2 Water absorption

The water absorption is defined as the increase in mass of aggregate due to water penetration into the pores of the particles during a prescribed period of time, but not including water adhering to the outside surface of the particles, expressed as a percentage of the dry mass.

Water absorption tests for coarse aggregates were performed in accordance with AS 1141.6.1 “Particle density and water absorption of coarse aggregate – Weighing-in-Water Method” (AS 1141.6.1-2000). Water absorption tests for fine aggregates were performed in accordance with AS 1141.5 “Particle density and water absorption of fine aggregates” (AS 1141.6.1-2000). The water absorption of coarse aggregate was calculated using the following equation:

\[ W_A(\%) = \left( \frac{m_2 - m_1}{m_1} \right) \times 100 \]  

Equation 3.5
where,

\[ W_A = \text{the water absorption of the coarse aggregate (\%)} \]
\[ m_1 = \text{the dry mass of the aggregate (g)} \]
\[ m_2 = \text{the mass of the saturated surf-dry aggregate (g)} \]

The water absorption \((W_a)\) of the fine aggregate was calculated from the following equation:

\[
W_a(\%) = \left(\frac{m_2 - m_1}{m_1}\right) \times 100 \quad \text{Equation 3.6}
\]

where

\[ W_a = \text{the water absorption of the fine aggregate (\%)} \]
\[ m_1 = \text{the dry mass of the fine aggregate (g)} \]
\[ m_2 = \text{the mass of the saturated surface dry fine aggregate (g)} \]

3.2.3 **Compaction**

Compaction is a process by which the soil particles are artificially rearranged and packed together into a closer state of contact by mechanical means in order to decrease the porosity (or void ratio) of the soil and thus increase its dry density. The compaction process may be accomplished by rolling, tamping, or vibration.

When aggregate is used as road subbase material or fill, one of the most important influences on its behaviour is density. Since best compaction can be achieved at the optimum moisture content (OMC) its determination will help to carry out compaction in the field by adding that much amount of water to the soil during compaction. Moreover, the compaction effort will continue in the field till maximum dry density is obtained in the compacted soil. Hence, the laboratory determination of OMC and maximum dry density \((\gamma_{d,max})\) are helpful in specifying compaction specifications for field compaction of soils.

Increasing the density of a granular material is achieved by compaction which involves reducing the air voids, without reducing the moisture content (Head 1994). Air voids cannot be eliminated altogether by compaction but they can generally be reduced to 5%.
The two factors which have the greatest effect on density are moisture content and the compactive effort exerted on the material. To examine the effect of moisture content, a series of tests is normally conducted using a standardised test, i.e. with a constant compactive effort, for a range of moisture content.

As the moisture content of a fine grained soil is increased, the dry density ($\rho_d$) also increases until it reaches a point of peak density ($\rho_{d, \text{peak}}$). This moisture content at which this occurs is termed as the optimum moisture content (OMC). When the OMC has been reached and the moisture content is increased further, the excess water begins to push the particles apart so that $\rho_d$ is reduced (Head 1994). Cohesionless soils do not respond to variations in moisture content in the same manner as fine grained soils. A peak density is reached but on the dry side of OMC the curve is quite flat, particularly for well graded materials, and it is not uncommon for a second peak to be recorded at low moisture content (Lee, White and Ingles 1983). The two peaks on the dry density/moisture content curve are normally separated by a point of low density. Lambe and Whitman (1979) concluded that this point of low density, obtained at a low moisture content, is due to capillary forces resisting rearrangement of the particles.

Lambe and Whitman (1979) noted that the term dry density is usually used as another expression for dry unit weight. They are, however, not equal because density is actually $1/g$ times unit weight where $g$ is the gravitational constant.

In the compaction process, loose fills are placed in small lifts. Water is then added to the soil to serve as a lubricating agent on the soil particles. With the application of compacting effort, the soil particles slip over each other and move into a densely packed position. As the moisture content is increased under the same compactive effort, the mass of soil solids in a unit volume gradually increases till a maximum dry density is achieved. If the moisture content is increased further, the soil becomes more workable. However, the moisture takes up the space that might have been occupied by soil solids; consequently, the moist unit mass decreases. The test can be repeated several times at various moisture contents of soil. By plotting dry density ($\rho_d$) against the corresponding moisture content, the optimum moisture content and the maximum dry density can be obtained.
Compaction of soil samples in the laboratory is carried out in standard cylinders. Energy is applied by a hammer of standard size and mass dropping freely from a standard height on a layer of the sample inside the cylinder. Each specimen is made in 3 or 5 layers with a specified number of blows depending on the codes and method of testing. In the field, the energy is applied by means of different types of rollers. The term compactive effort is used to describe the energy given to a unit volume of soil. Compaction increases the number of particles per unit volume and dry density is used to indicate the degree of compaction. Compactive effort is most effective if a uniform mixture of soil and water is used.

In the laboratory, two methods are normally used for the compaction of soils. They are standard compaction and modified compaction.

In this research, Australian Standard on Methods of testing soils for engineering purposes, “Method 5.2.1: Soil compaction and density tests-Determination of the dry density/moisture content relation of a soil using modified compactive effort” (AS 1289.5.2.1-2003) was used to determine the maximum dry density and optimum moisture content. As pavement subbase is being subjected to considerably heavy loading, modified compactive effort is used to find the maximum dry density and optimum moisture content.

The aggregates used in this study were maximum size of 20 mm and almost all particles passed 19 mm sieve. For modified compaction tests around 14 kg of materials were sieved on 19 mm sieve (the material retained on 19 mm sieve was rejected) and by riffling five representative portions of the sieved soil was taken, each of sufficient quantity to produce a compacted volume in excess of the volume of the mould. Then, each portion was thoroughly mixed and different percentages of water were added so that the optimum moisture content is judged to be straddled. The test specimens were cured in a sealed container for 24 hours to allow the water to become more uniformly distributed through the sample before compaction. The mass of the mould with baseplate was determined. Then the mould, collar and baseplate was assembled and the assembly was placed on the compaction machine. According to the specification, a cylindrical mould having an internal diameter of 105 mm and effective height of 115.5 mm was used for compaction. The blends were compacted into the mould in five layers,
so that the compacted height of soil in the mould is 23 mm to 28 mm in the first layer, 47 mm to 52 mm in the second layer, 70 mm to 75 mm in the third layer, 93 mm to 98 mm in the fourth layer and 116 mm to 120 mm in the fifth layer. Specimens that do not meet these tolerances were discarded. Each layer was compacted by 25 uniformly distributed blows of 4.9 kg rammer falling freely from a height of 450 mm. The compactive effort used in the compaction method was 2703 KJ/m$^3$. Then, the material from around the inside of the collar was freed and the collar was carefully removed. Afterwards the surface of the specimen was trimmed by means of the straightedge while the mould was still attached to the baseplate. The smaller size material was used to patch any holes developed in the surface from removal of coarse material during trimming. Then the mass of the mould and soil, with was determined, the soil specimen was immediately removed from the mould and the full height of the specimen was kept in the oven to determine the moisture content of this sample in accordance with AS 1289.2.1.1-2000. The compaction process was repeated until 5 runs have been made ensuring that at least two of which shall be dryer, and two wetter, than optimum moisture content to satisfactorily define the dry density/moisture content relationship. Figure 3.12 shows the mechanical compactor used for compaction and Figure 3.13 shows the other accessories required for compaction tests.
3.2.4 California Bearing Ratio (CBR)

The California Bearing Ratio (CBR) test is an empirical test which was first developed in California, USA, for estimating the bearing value of highway subbases and subgrades, hence its name (Head 1994). The test follows a standardised procedure.

This test method is used to evaluate the potential strength of subgrade, subbase, and base course material, including recycled materials for use in road and airfield pavements. The CBR value obtained in this test forms an integral part of several flexible pavement design methods (ASTM D 1883-2007).

The test is performed by pushing a standard plunger into the soil at a fixed rate of penetration, and measuring the force required to maintain that rate. From the resulting load-penetration relationship the CBR can be derived for the soil in the condition at which it was tested.

Figure 3.13: Other accessories required for compaction testing

In this investigation, CBR tests were performed in accordance with Australian Standards on Methods of testing soils for engineering purposes, “Method 6.1.1: Soil...
strength and consolidation tests-Determination of the California Bearing Ratio of a soil-
Standard laboratory method for a remoulded specimen” (AS 1289.6.1.1-2000). The samples were compacted in a cylindrical mould having an internal diameter of 152 ± 1 mm, height 178 ± 1 mm and wall thickness of at least 5 mm, provided with a metal extension collar and a perforated metal baseplate. The blends were compacted into the mould in five layers to an effective height of 117 mm by using a spacer disc (150 ± 0.5 mm diameter and 61 ± 0.25 mm high, fitted with a removable handle for lifting the disc from the mould) inserted into the mould before compaction. According to the standard the materials passing the 19 mm sieve was taken. Modified compactive effort was also used here as modified CBR values as being used for pavement subbase.

By riffling the sieved material passing the 19 mm sieve, representative test portions was obtained for determining maximum dry density, optimum moisture content and CBR. The tests portions were thoroughly mixed and allowed to cure for 24 hours. Then the mass of the mould ($m_1$) was determined with base plate. The spacer disc was inserted, the mould (with the extension collar attached) was clamped to the baseplate and a filter paper was placed on top of the spacer disc. Immediately prior to compaction, the cured soil was thoroughly mixed and a representative fraction of the test portion was taken to determine the moisture content ($w_1$) of in accordance with AS 1289.2.1.1 (AS 1289.2.1.1-2000). Then the specimen was compacted uniformly in the mould to a laboratory density ratio of 100 percent and a laboratory moisture ratio of 100 percent with 53 blows per layer of the 4.9 kg rammer falling freely from a height of 450 mm.

Afterwards, the material from around the inside of the collar was freed and the collar was removed carefully. While the baseplate is still attached, the surface of the compacted specimen was trimmed and levelled with the top of the mould by means of a straightedge using smaller size material to patch any holes developed in the surface from removal of coarse material during trimming. Then, the perforated baseplate, spacer disc and filter paper, was removed. Again, a filter paper was placed on the perforated baseplate, the mould plus the compacted soil was inverted and was placed it on the baseplate. The baseplate was clamped to the mould with the compacted soil in contact with the filter paper. The mass of the baseplate plus mould plus specimen including the filter pares was measured. Afterwards, the stem and perforated plate was placed on the
compacted soil specimen in the mould, surcharges of 4.5 kg were applied and the
surcharged specimen was immersed in water allowing free access of water to the top
and the bottom of the specimen. The specimen was allowed to soak for 4 days
maintaining the water level above the mould during this period. The swelling of the
specimen was measured. After measuring the swelling and the soaking periods, the
specimen was tilted to remove the surface water. Then the mould was returned to
vertical position and the specimen was allowed to drain downward for 15 min without
disturbing the surface of the specimen during the removal of water. Afterwards, the
surcharges stem and perforated plate was removed and the mass of the baseplate plus
mould plus specimen was measured including the filter papers.

Then the 4.5 kg surcharge was placed on the soil surface and the mould plus specimen
plus baseplate was placed in the loading machine. As the expected CBR was greater
than 30, a seating load of 250 N was applied with the penetration piston of 49.5 mm
diameter. Then the force-measuring device and the displacement measuring device was
set to zero and the plunger was made to penetrate the sample with a constant rate of
penetration of 1 mm/min, the readings of force on the plunger were noted at
penetrations of 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 7.5, 10.0 and 12.5 mm. Then the soil
from the mould was extruded and the moisture content of the top 30 mm layer and that
of the remaining specimen was measured in accordance with AS 1289.2.1.1.

The force on the plunger was plotted against penetration for each test and the forces
applied at penetrations of 2.5 mm and 5 mm were noted by the GeoComp software. AS
1289.6.1.1-1998 states that if the initial portion of the curve is concave upwards, the
following correction should be made. A tangent should be drawn at the point of
maximum slope, and extended to intersect the horizontal axis. The whole curve should
then be moved to the left until this intersection coincides with the origin. The forces for
2.5 mm and 5 mm penetration should then be read from this corrected curve. The CBR
is calculated using the formula:

\[
CBR = \frac{\text{Measured Force} \times 100}{\text{Standard Force}}
\]

Equation 3.7
Where the standard force is the force required for the same penetration into a sample of compacted crushed rock (BS 1377-1990). The standard forces for 2.5 mm and 5 mm penetration are 13.2 kN and 19.8 kN respectively and CBR is calculated for both levels of penetration. AS 1289.6.1.1 (2000) states that the higher of the two values should be taken as the final result. Figure 3.14 shows the accessories required for the preparation of CBR specimen.

Figure 3.15 shows the soaking of the compacted CBR specimen and Figure 3.16 shows the testing set up of soaked sample with an automated testing machine.

![Figure 3.14: The accessories required for the preparation of CBR specimen](image)

### 3.2.5 pH value

pH value is a measure of the acidity or alkalinity of a solution. It is the logarithm to base 10 of the reciprocal of the concentration of hydrogen ions in an aqueous solution. Solutions with a pH value less than seven are considered acidic, while those with a pH value greater than seven are considered basic (alkaline). pH 7 is considered neutral because it is the accepted pH of pure water at 25°C. pH value is normally dependent upon the activity of hydrogen (H\(^+\)). Practically every phase of water supply and wastewater treatment such as acid-base neutrality, water softening, precipitation,
coagulation, disinfection and corrosion are pH dependent. pH value suitable for existence of most biological life is typically in the range of 6 to 9.

Figure 3.15: Soaking of the compacted CBR specimen

Figure 3.16: Testing set up of soaked sample with an automated testing machine
Excessive acidity or alkalinity of the groundwater in soils can have detrimental effects on concrete buried in the ground. Even moderate degree of acidity can cause corrosion of metals. Measurement of the pH value of the groundwater reveals these potential dangers so that remedial measures can be taken (Head 1994). In the stabilisation of soils for roads, some resinous materials are unsuitable for alkaline soils, yet may be satisfactory with neutral or slightly acid soils (Head 1994).

In this research, pH tests were performed in accordance with AS 1289.4.3.1-2000 “Soil chemical tests - Determination of the pH value of a soil - Electrometric method”. For pH test, the material was sieved on the 2.36 mm sieve and about 35 g of sample was prepared by riffling from the material passing the sieve. Exactly 30 g of the sample was transferred into a 100 mL beaker and 75 mL of distilled water was added to the sample. The mixture was stirred well and kept for 4 hours. Before the test, the mixture was stirred again and pH was measured by electrometric pH meter. The pH meter was calibrated by standard buffer solutions. The mixture was briefly stirred while taking the reading and the pH was recorded when the reading was stable. Figure 3.17 shows the pH meter with the sample during testing.
The fines content for aggregates refers to the determination of the amount of fines (materials finer than 75 μm in aggregate) by wash sieving. Clay particles and other aggregate particles which are dispersed by the wash water and water soluble materials will be removed from the aggregate during the test.

Fines content (materials finer than 75 μm in aggregates) tests were performed to determine the fines content of samples in accordance with AS 1141.12-1996 “Methods for sampling and testing aggregates-Method 12: Materials finer than 75 μm in aggregates (by washing)”.

Depending on the nominal size of the materials, the amount specified in the standards (3 kg for 20 mm nominal size and 300 g for 5 mm nominal size graded aggregate) was washed through the 75 μm sieve with a guard sieve of 1.12 mm as shown in Figure 3.6. The soil is placed a little at a time on the 1.12 mm sieve, and washed over a sink with a jet or spray of clean water, as shown in Figure 3.7. The silt and clay passing the 75 μm sieve was allowed to run to waste. When the material on the 1.12 mm guard sieve had been washed free of fines, washing on the 75 μm sieve was continued until the waste water is seen to run clear. The whole of the material retained on each sieve was allowed to drain, and was carefully transferred to trays. These were placed in an oven to dry at 105°C, preferably overnight. Later, the oven dried blends were cooled down to room temperature. After cooling, the whole of the dried material was put together and weighed to an accuracy of 0.01%. The difference between the ovens dried mass is taken as the fines content and is calculated as follows:

\[
\text{Percentage passing the 75 μm sieve} = \frac{(m_2 - m_1)}{m_1} \quad \text{Equation 3.8}
\]

where,

- \( m_1 \) = the mass of the dried test portion before washing, in grams
- \( m_2 \) = the mass of the dried washed test portion, in grams
3.2.7 Plasticity index

Plastic limit, liquid limit and plasticity index tests were performed in accordance with AS 1289.3.1.1 “Soil classification tests–Determination of the liquid limit of a soil – Four point Casagrande method” (AS 1289.3.1.1-2000) for liquid limit and AS 1289.3.2.1 “Soil classification tests-Determination of the plastic limit of a soil-Standard method” (AS 1289.3.2.1-2000) for plastic limit.

Liquid limit

The sample of about 250 g was obtained from the material passing through 425 μm sieve. The sample was thoroughly mixed with water in the mixing bowl. Water was added incrementally and mixed well until the sample became a thick homogeneous paste. The paste was covered and allowed to cure for at least 12 hours at room temperature. The paste was remixed thoroughly before starting the test. The paste was placed in the cup and levelled off parallel to the base. The depth of the soil in the cup was not allowed to exceed 10 mm. The soil patch was divided by drawing the grooving tool along the diameter through the center line of the hinge. The crank was rotated at the rate of 2 rev/sec until the two soil parts became touched along the bottom of the groove for about 10 mm. Number of blows to reach this state was recorded. Number of blows was targeted between 15 and 40 to get four points and the test was started from drier to the wetter condition of the soils. Small amount of soil paste was taken from each trial for moisture determination. The moisture content and corresponding number of blows was plotted in semi logarithmic scale. Moisture contents were plotted as ordinates on the linear scale and number of blows was plotted as abscissa on the logarithmic scale. The liquid limit was considered as the moisture content corresponding to the intersection of the best fit line with the point corresponding to 25 blows on the abscissa.

Plastic limit

The sample of about 40 g was obtained from the material passing through 425 μm sieve. The sample was thoroughly mixed with water in the mixing bowl. Water was added incrementally and mixed well until paste became homogeneous and plastic enough to be shaped into a ball. The paste was covered and allowed to cure for at least 12 hours at
room temperature. About 8 g of soil mixture was taken and moulded between fingers. The soil mixture ball was rolled into about 3 mm diameter thread until it crumbled. If the soil thread crumbled before the diameter reached 3 mm, more water was added, remixed and the thread was rolled again. If the soil thread rolled down to 3 mm diameter without crumbling, small amount of soil was added, remixed and the thread was rolled again. The procedure was performed until the thread crumbled when it became 3 mm in diameter. More threads were rolled until it crumbled at 3 mm in diameter and the moisture content of these threads was determined. Plastic limit was considered as the moisture content at crumbled stage with a diameter of about 3 mm.

3.2.8 Organic Content

Organic matter in soil is derived from a wide variety of animal and plant remains, so there can be a great variety of organic compounds. They all can have undesirable effects on the engineering behaviour of soils (Head 1994).

The organic content is used as criteria for acceptability of materials for construction, such as granular materials for road bases and structural fill. Organic materials degrade over time and therefore cause production of gases and result in settlement.

Organic content tests were performed in accordance with ASTM D 2974-07 “Standard Test Methods for Moisture, Ash and Organic Matter of Peat and Other Organic Soils”. The ignition method was used to determine the organic content of the aggregates. The material was sieved on the 2.36 mm sieve and about 200 g of sample was prepared by riffling from the material passing the sieve. The sample was oven dried at 105°C to remove moisture. The mass of the oven dried sample was recorded and the sample was kept in the furnace at 440°C and held until the specimen is completely ashed (no change of mass occurs after a further period of heating). The sample was cooled down to room temperature and the weight of the sample was recorded.

Figure 3.18, Figure 3.19 and Figure 3.20 show the Muffle furnace, Muffle furnace with samples and High-silica porcelain dish with samples, respectively. The organic content was calculated as follows:
Organic content (%) = 100 – $\frac{C \times 100}{B}$

where,

$C = \text{mass of ash (g)}$

$B = \text{oven dried test portions (g)}$

Figure 3.18: Muffle furnace

Figure 3.19: Muffle furnace with samples
3.2.9 Los Angeles Abrasion Loss

This test is a measure of degradation of aggregates of standard gradings resulting from a combination of actions including abrasion or attrition, impact and grinding in a rotating steel drum containing a specified number of steel spheres, the number depending upon the grading of the test sample. As the drum rotates, a shelf plate picks up the sample and the steel spheres, carrying them around until they are dropped to the opposite side of the drum, creating an impact crushing effect. The contents then roll within the drum with an abrading and grinding action until the shelf plate picks up the sample and the steel spheres, and the cycle is repeated. After the prescribed number of revolutions, the contents are removed from the drum and the aggregate portion is sieved to measure the degradation as percent loss (ASTM C 131-2006).

This test has been widely used as an indicator of the relative quality or competence of various sources of aggregate having similar mineral compositions. The results do not automatically permit valid comparisons to be made between sources distinctly different in origin, composition, or structure (ASTM C 131-2006).
Los Angeles abrasion loss tests were performed in accordance with AS 1141.23 “Methods for sampling and testing aggregates – Los Angeles Value” (AS 1141.23-1995) and ASTM C 131-2006 “Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine” (ASTM C 131-2006). From the samples around 2650 g of unwashed coarse fraction between 19.0 mm to 13.2 mm and another 2650 g of coarse fraction between 13.2 mm to 9.5 mm were prepared by sieving. Each coarse fraction was washed thoroughly to remove dust or other coatings. The samples were oven dried at 105°C for a period of 15 to 16 hours and allowed to cool by standing in laboratory ambient conditions for at least 4 hours. An amount of 2500 g was obtained from each fraction and combined to a total amount of 5000 g according to the test grading, 11 standard steel balls were used in the test. For pure recycled glass, 5000 g of washed materials passing 4.75 mm and retained on 2.36 mm sieve was taken (Grading D-ASTM C 131-2006). Both the samples and the steel balls were transferred into the drum of the Los Angeles machine and 500 revolutions were completed. The tested samples were sieved and the weight of material retained on 1.70 mm sieve and material passing 1.70 mm sieves were recorded. The material retained on 1.70 mm sieve was washed through 1.70 mm sieve and the washed material was oven dried at 105°C for a minimum period of 16 hours and allowed to cool for around 4 hours. The material was sieved again and the weight of the material retained on 1.70 mm sieve was recorded. Figure 3.21 shows the LA Abrasion machine and steel ball charges.

![Figure 3.21](image1.jpg)

(a) LA Abrasion test machine, (b) LA Abrasion steel ball charges.
The Los Angeles value shall be calculated from the following equation:

\[
\text{Los Angeles value} = \frac{m_T - m_w}{m_T} \times 100
\]

Equation 3.10

where,

- \(m_T\) = total mass of the washed and dried material before abrasion (g)
- \(m_w\) = mass of material retained on the 1.70 mm sieve after abrasion, washing, drying and re-sieving (g)

### 3.2.10 Flakiness Index

Aggregate particles are classified as flaky when they have a thickness (smallest dimension) of less than 60% of their mean sieve size, this size being taken as the mean of the limiting sieve apertures used for determining the size fraction in which the particle occurs. The flakiness index of an aggregate sample is found by separating the flaky particles and expressing their mass as a percentage of the mass of the sample tested. The test is not applicable to material passing a 6.30 mm test sieve or retained on a 63.0 mm test sieve (BS 812-105.1-2000).

Flakiness is a measure of the particle shape and is defined according to BS 812-105.1-2000. A high flakiness index indicates a ‘plate-like’ shape and generally, cuboid or rounded particle shapes are preferred.

Flakiness index is a measure of the thin and flat particles in the sample. This influences the material strength in two ways. Firstly, reduced strength is yielded when load is applied to the flat side or across shortest dimension of the aggregate. Secondly, flaky materials are prone to segregation and breakdown during compaction which is an important consideration while using for road pavement.

In this research BS 812-105.1-2000 was used to determine the flakiness index of the samples. The samples were prepared by riffling to produce a test portion that complies with Table 3.2 and the test portion was dried by heating at a temperature of 105 \(\pm\) 5 °C to achieve a dry mass which is constant to within 0.1 %. Then the sample was allowed to cool and weighed. After that a sieve analysis was carried out by using the set of sieves given in Table 3.3. Each of the individual size-fractions retained on the sieves
(other than the 63.0 mm test sieve) was weighed and stored them in separate trays with their size marked on the trays. Where the mass of any size-fraction was considered to be excessive, the fraction was subdivided provided that the mass of the subdivided fraction was not less than half the appropriate mass given in Table 3.4. From the sums of the masses of the fractions in the trays ($M_1$), the individual percentage retained on each of the various sieves was calculated. Any fraction whose mass was 5 % or less of mass $M_1$ was discarded. The remaining mass ($M_2$) was recorded. Then, each fraction was gauged by using the special sieves: The special sieve was selected appropriate to the size-fraction under test. The whole of the size-fraction was placed into the sieve and the sieve was shaked until the majority of the flaky particles have passed through the slots. Then the particles passing each of the special sieves were combined and weighed ($M_3$). Figure 3.22 shows the flakiness sieves with flakiness gauge. The flakiness index was calculated as follows.

\[
\text{Flakiness index} = \frac{M_2}{M_3} \times 100
\]

**Equation 3.11**

where,

$M_2$ = mass of test portions (g)

$M_3$ = mass of particles passing the special sieves (g)

Figure 3.22: Flakiness sieves with flakiness gauge
Table 3.2: Minimum mass of test portion (BS 812-105.1-2000)

<table>
<thead>
<tr>
<th>Nominal size of material</th>
<th>Minimum mass of test portion after rejection of oversize and undersize particles</th>
</tr>
</thead>
<tbody>
<tr>
<td>mm</td>
<td>kg</td>
</tr>
<tr>
<td>50</td>
<td>35</td>
</tr>
<tr>
<td>40</td>
<td>15</td>
</tr>
<tr>
<td>28</td>
<td>5</td>
</tr>
<tr>
<td>20</td>
<td>2</td>
</tr>
<tr>
<td>14</td>
<td>1</td>
</tr>
<tr>
<td>10</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 3.3: Particulars of sieves (BS 812-105.1-2000)

(Nominal aperture sizes
Square hole perforated plate 450 mm or 300 mm diameter)

<table>
<thead>
<tr>
<th>Mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>63.0</td>
</tr>
<tr>
<td>50.0</td>
</tr>
<tr>
<td>37.5</td>
</tr>
<tr>
<td>28.0</td>
</tr>
<tr>
<td>20.0</td>
</tr>
<tr>
<td>14.0</td>
</tr>
<tr>
<td>10.0</td>
</tr>
<tr>
<td>6.30</td>
</tr>
</tbody>
</table>

3.2.11 Permeability Test

The permeability (also called “hydraulic conductivity”) of a soil is a measure of its capacity to allow the flow of a fluid through it. The fluid may be either a liquid or a gas, but soils engineers are concerned only with liquid (mainly water) permeability (Head 1994).
Table 3.4: Data for determination of flakiness index (BS 812-105.1-2000)

<table>
<thead>
<tr>
<th>Aggregate size-fraction</th>
<th>Width of slot in thickness gauge or special sieve</th>
<th>Minimum mass for subdivision</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS test sieve nominal aperture size</td>
<td></td>
<td></td>
</tr>
<tr>
<td>100 % passing</td>
<td>100 % retained</td>
<td>Mm</td>
</tr>
<tr>
<td>mm</td>
<td>Mm</td>
<td></td>
</tr>
<tr>
<td>63.0</td>
<td>50.0</td>
<td>33.9 ± 0.3</td>
</tr>
<tr>
<td>50.0</td>
<td>37.5</td>
<td>26.3 ± 0.3</td>
</tr>
<tr>
<td>37.5</td>
<td>28.0</td>
<td>19.7 ± 0.3</td>
</tr>
<tr>
<td>28.0</td>
<td>20.0</td>
<td>14.4 ± 0.15</td>
</tr>
<tr>
<td>20.0</td>
<td>14.0</td>
<td>10.2 ± 0.15</td>
</tr>
<tr>
<td>14.0</td>
<td>10.0</td>
<td>7.2 ± 0.1</td>
</tr>
<tr>
<td>10.0</td>
<td>6.30</td>
<td>4.9 ± 0.1</td>
</tr>
</tbody>
</table>

Soils are permeable (water may flow through them) because they consist not only of solid particles, but also a network of interconnected pores through which water can flow from points of high energy to points of low energy. But, permeability is not a fundamental property of soil. The degree to which soils are permeable depends upon a number of factors, such as soil type, type of flow, fluid viscosity, pore-size distribution, particle size distribution, particle shape and texture, voids ratio, roughness of mineral particles, soil history, and degree of saturation (Head 1994). In clayey soils, structure plays an important role in hydraulic conductivity. Other major factors that affect the permeability of clays are the ionic concentration and thickness of layers of water held to the clay particles. The value of hydraulic conductivity \( k \) varies widely for different soils. Some typical values for saturated soils are given in Table 3.5. The hydraulic conductivity of unsaturated soils is lower and increases rapidly with the degree of saturation. This degree of permeability is characterised by the coefficient of permeability. The permeability is generally expressed in cm/sec or m/sec in SI units.
Table 3.5: Typical Values of Hydraulic Conductivity of Saturated Soils (Das 2006)

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Hydraulic conductivity, k cm/sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clean sand</td>
<td>100-1.0</td>
</tr>
<tr>
<td>Coarse sand</td>
<td>1.0-0.01</td>
</tr>
<tr>
<td>Fine sand</td>
<td>0.01-0.001</td>
</tr>
<tr>
<td>Silty clay</td>
<td>0.001-0.0001</td>
</tr>
<tr>
<td>Clay</td>
<td>&lt;0.000001</td>
</tr>
</tbody>
</table>

The study of the flow of water through permeable soil media is important in soil mechanics. It is necessary for estimating the quantity of underground seepage under various hydraulic conditions, for investigating problems involving the pumping of water for underground construction, and for making stability analysis of earth dams and earth-retaining structures that are subject to seepage forces.

The permeability of a fill material also plays a decisive role in drainage applications. The rate of fluid flowing through a soil mass relates directly to its permeability. In hydrogeologic studies of natural and processed materials, permeability is usually the most important property. In engineering practice, the permeability of a fill material often plays a decisive role in material selection, particularly for applications related to drainage. For granular fill material, high permeability is usually more beneficial than low. The exception to that may be for leachate treatment where a specific range of permeability may be required. The permeability of a granular material depends on its gradation and density. Generally, a well-graded material is less permeable due to its lower void ratio. It is believed that permeability is also a function of surface texture, which affects drag or friction between the fluid and particle surface. As a result, a mix of aggregate and "smooth" cullet may have a higher permeability than that of "rough" natural sand and gravel.

A number of different methods for determining the coefficient of permeability for soils exist, including in-situ methods and laboratory methods. There are two typical laboratory tests available for the determination of permeability. These two tests are the constant head permeability test and falling head permeability test. All methods assume
the validity of Darcy’s law, which states that the coefficient of permeability is the ratio of the flow rate to the hydraulic head gradient (Fredlund and Rahardjo 1993).

For soils of high permeability (sands and gravels) a constant head permeability is used. For soils of intermediate to low permeability, a falling head test is used. The former is used principally for coarse-grained soils (clean sands and gravels) with permeability greater than $1 \times 10^{-4}$ cm/s and the latter is used primarily for fine-grained soils (silt and clay) with permeability less than $1 \times 10^{-4}$ cm/s.

The selection of hydraulic conductivity method is mainly based on the permeability classification of the material which can be roughly predicted by looking at the gradation curve and soil classification. If the selected test method found to be inappropriate, the other test methods should be used. The classification of soils on the basis of permeability and also their drainage characteristics is shown in Figure 3.23. Figure 3.23 also shows the test method appropriate for each permeability classification and the indirect methods of measuring the hydraulic conductivity.

In the design of engineering projects, one of the most important soil properties of interest to the soil engineer is permeability. To some degree, permeability will play a role in the design of almost any structure. For example, the durability of concrete is related to its permeability. In designs that make use of earthen materials (soils and rock, etc) the permeability of these materials will usually be of great importance.

**Principles of permeability through soils**

Darcy’s law ($v = ki$) is valid i.e. flow through soils is assumed laminar. Figure 3.24 shows the zones of laminar and turbulent flows.

In this research, the constant head permeability test method was used for recycled glass and falling head method was used for other materials. The modified compaction effort was used to prepare the samples.
**Constant head permeability tests**

The constant head procedure is used for the measurement of the permeability of sands and gravels containing little or no silt (Head 1994). British standard for determination of permeability by the constant head method specifies that this method is only suitable for soils having the coefficient of permeability in the range of $10^{-2}$ to $10^{-5}$ m/s (BS 1377-5-1990). In Britain, the most common permeability cell (permeameter) is 75 mm diameter and is intended for sands containing particles up to about 5 mm. A larger cell, 114 mm diameter, can be used for testing sands containing particles up to about 10 mm, i.e. medium gravel size. As a general rule, the ratio of the cell diameter to the diameter of the largest size of particles in significant quantity should be at least 12 (Head 1994).

![Diagram](image)

**Figure 3.23:** Range of hydraulic conductivity values based on soil type (Holtz and Kovacs 1981)
In this research, recycled glass with particle size less than 4.75 mm was tested using the ELE large cell with the internal diameter of 114 mm due to the unavailability of 75 mm diameter cell in the Geomechanics laboratory of Swinburne University of Technology.

In the constant head test, water is made to flow through a column of soil under the application of a pressure difference which remains constant for fulfilling the condition of laminar flow during the test. The amount of water passing through the soil in a known time is measured, and the permeability of the sample is calculated by using the specified formula.

Ideally, the water used for a permeability test should be the ground water found in situ. But this is seldom practicable, and some treatment of available water is usually desirable. The main objections to the use of untreated tap-water are the presence of dissolved air, dissolved solids and possibly bacteria.
As the water flows through the voids between the soil particles, air contained in the water tends to form bubbles and these bubbles of air in the voids impede the flow of water, thereby giving an erroneously low measurement of permeability (Head 1994). To eliminate this problem, the air should be removed from water, so de-aired water was used in this research. For the same reason the sample itself must be de-aired and fully saturated before starting the test (Head 1994). Figure 3.25 shows the automatic de-airing apparatus used in this research for supplying a continuous flow of de-aired water during the constant head permeability test.

The British Standard for “Determination of permeability by constant head method” (BS 1377-5-1990) was followed to measure the hydraulic conductivity of the recycled glass. The standard requires of placement of a filter material with the suitable grading next to the perforated plates at both ends of the permeameter. The gradation curve of the filter material depends on the particle size distribution of the test sample (BS 1377-5-1990). The filter material was prepared by mixing different sizes of clean washed sand and gravel. Figure 3.26 shows the filter material prepared for using to determine the hydraulic conductivity of the recycled glass.

Figure 3.25: Automatic de-airing apparatus
The test specimen was prepared by mixing potable water to achieve the optimum moisture content and then it was left in a sealed container for 24 hours before placing the specimen inside the cell. A representative sample was taken and dried in the oven to measure the moisture content of the test specimen to ensure its difference with the optimum moisture content is less than 1%. Then the filter material was placed at the bottom of the cell with the height of around 5 to 6 cm. The wire gauze was placed on top of the filter material and afterwards the calculated mass of the test specimen was placed on top of the wire gauze and was compacted using the tamping rod to reach the desired height. The height of the compacted layer was measured using the vernier to make sure that it has been compacted to the desired density. Before placing the next layer, the surface of the layer was scarified with the spatula. The test specimen was placed in 5 layers inside the permeability cell making sure that the final height of the specimen was always bigger than twice of cell internal diameter. On top of the last layer, the upper wire gauze was placed and then another layer of filter material with the height of 5 to 6 cm was placed (BS 1377-5-1990).

![Filter material used](image)

Figure 3.26: Filter material used

After this the top plate was fitted and then the piston was lowered carefully and was held down firmly and then the locker collar around the piston rod was tightened. This is to keep the filter layers and the test specimen in their position during saturation and the
test period and to prevent any movement or dilation of the test specimen. Figure 3.27 shows the filter layers placed at the top and bottom of the recycled glass material inside the permeability cell.

To saturate the test specimen and the filter layers, the valve at the bottom of the cell was connected to the de-aired water supply tank and the air bleeding valve at the top of the cell lead was left open. A small hydraulic gradient (around 0.2) was applied during the saturation in order to avoid any disturbance to the sample and was held for at least 24 hours to make sure the sample is fully saturated.

In the next step, the direction of water was changed to downward direction to start the permeability test. During the test, the surface of the discharge reservoir was held slightly above the outlet end of the sample to make sure the water inside the sample is constantly under a small amount of pressure and also the bottom of the sample is always saturated (Head 1994). An initial hydraulic gradient of 0.2 was applied to the test specimen to allow the water rise inside the manometers and reach a stable condition. The hydraulic gradient during the test should be small enough to avoid any disturbance of the sample. The hydraulic gradient in the sample was determined by means of piezometer tubes inserted at three different levels, as shown in Figure 3.28. The advantage of measuring the hydraulic gradient from piezometer level readings rather than from the difference between the permeameter inlet and outlet levels, is that the difference in head between the inlet and the outlet levels include small pressure losses which occur within the filter layers, and possibly in the pipeline connections (BS 1377-5-1990). The room temperature was also maintained around 20°C to increase the accuracy of the results. Figure 3.29 shows the constant head permeability test arrangement during a test on recycled glass.

The amount of water passing thorough the soil in a known time was measured and knowing the water head and also the length of the test specimen between the manometers, the permeability of the sample was calculated. This stage was repeated at least 5 times to obtain consistent results. Then the head of water was changed and after reaching stable water flow, the readings were repeated and compared with the previous series of test data to make sure that the results are consistent. During all these steps the
hydraulic gradient was small enough to make sure that a laminar flow of water is passing through the sample.

Figure 3.27: Filter layers placed at the top and bottom of the recycled glass material

Figure 3.28: Piezometer tubes inserted at three different levels
For measuring the permeability of soils of intermediate and low permeability (less than $10^{-4}$ m/s), i.e. silts and clays, the falling head procedure is used. In the falling head test a relatively short sample is connected to a standpipe which provides both the head of water and the means of measuring the quantity of water flowing through the sample. Several standpipes of different diameters are normally available from which the diameter that is most suitable for the type of material being tested can be selected.

In falling head permeability tests, generally much higher hydraulic gradients are used in comparison to the hydraulic gradients normally used in the constant head permeability tests to induce any measurable flow due to its low hydraulic permeability. The cohesion of clays provides resistance to failure by piping at gradients of up to several hundred, even under quite low confining or surcharge pressures (Zaslavsky and Kassiff 1965).

The diameter of the standpipe determines the duration of the test, which should not be so short that timing becomes inaccurate, nor inconveniently long. If the testing period ranges from say half a minute to about 8 hours, by selecting suitable standpipe diameter
permeability in the range from $10^{-4}$ to $10^{-8}$ m/s can be measured by this method. In this research, a standpipe with the diameter of 2.90 mm was found to be suitable for conducting the falling head permeability test.

In this research, Australian Standard on Methods of testing soils for engineering purposes, “Method 6.7.2: Soil strength and consolidation tests-Determination of permeability of a soil-Falling head method for a remoulded specimen” (AS 1289.6.7.2-2001) was used to determine the hydraulic conductivity of the materials. The method is suitable for soils with coefficient of permeability of about $10^{-7}$ to $10^{-9}$ m/s.

Before starting the test, the cell and all the other connections were checked to make sure that there is no leakage in the system. Sealing rings were lightly coated with silicone grease to empower sealing. The test specimen was prepared by mixing potable water with the dried material to achieve the optimum moisture content. The test specimen was cured in a sealed container for 24 hours to allow the water to become more uniformly distributed through the sample before compaction. The porous discs were also saturated in water under vacuum for at least 1 hour before starting the test. A thin layer of wax was applied to the side of the mould to prevent piping of water between the mould and the specimen. The amount of wax on the side of the mould was taken into account when calculating the volume of the specimen. Immediately prior to compaction, the cured sample was thoroughly mixed and a representative fraction of the test portion was taken to determine the moisture content of the specimen to ensure that the soil shall be compacted within ±5% of the laboratory moisture ratio specified. The specimen was compacted in the mould in equal 3 layers (within 5 mm) to achieve the required laboratory density ratio within ±1% using the appropriate compaction rammer and layers depending on the compactive effort specified ensuring that material is not segregated and that each layer was scarified about 2 to 5 mm prior to the compaction of the next layer. Then the material from around the inside of the collar was freed and the collar was removed carefully. While the baseplate is still attached, it was ensured that the surface of the compacted specimen was level with the top of the mould by means of a straightedge. Smaller size material was used to patch any holes developed in the surface by the removal of coarse material during trimming. Then, the baseplate was removed and the dry density of the compacted specimen was calculated.
The height of the specimen was determined to the nearest 1 mm by taking measurement at least three separate points distributed over the specimen and the mean height was calculated. A porous plate and filter paper disc was placed on the permeameter baseplate and it was placed on the mould containing the specimen. The baseplate was clamped to the mould with the compacted soil in contact with the filter paper. The assembly was inverted and another filter paper and a porous disc were placed on top of the specimen. The permeameter was gently rocked to expel air from the base. The inlets and outlets were closed and the permeameter was removed from the container. Air from the specimen was evacuated in a vacuum container of not less than 35 kPa, and the water was allowed to be drawn into the specimen at a rate not exceeding 0.5% of the specimen volume per minute to totally cover the specimen. Evacuation was continued, increasing the vacuum if required until the specimen is saturated ensuring that the upper porous disc is held firmly against the specimen to avoid unravelling. On completion of saturation the inlets were closed.

The reservoir of the falling head apparatus was filled with water and the falling head apparatus was attached to the permeameter ensuring no air is trapped in the system. A typical arrangement is shown in Figure 3.30. The height of the permeameter was adjusted in such a way that the zero mark on the standpipe measure is at the same level as the overflow level in the permeameter. Water was allowed to flow freely for about 5 min through the overflow outlet and the time was recorded and the standpipe water height was taken to the nearest 1 mm. The reservoir of water was isolated from the standpipe. The water height in the standpipe and the time interval between readings were recorded at regular intervals over a period of at least three days. When the standpipe is refilled, the initial height and time were recorded for the next reading ensuring the reservoir of water is isolated from the standpipe. During the test, the water flow through the specimen was uninterrupted until the test is completed. The readings were continued until the permeability becomes constant and the temperature of the outflow water at the time of permeability readings was recorded over the last 24 hours of the test. The permeability is achieved when the difference in measured permeabilities over at least a 24 h period does not exceed 20% of the lowest measured permeability in that period. The average permeability over the last 24 hours of the test was calculated.

The height of the mould shall be sufficient to include the test specimen of height greater than 5 times the maximum particle size for testing, porous plates and surcharges.
height of the specimen shall be not less than 50 mm. Table 3.6 shows the permeameter cylinder diameter required for different particle sizes.

Table 3.6: Permeameter Cylinder Diameter (AS 1289.6.7.2-2001)

<table>
<thead>
<tr>
<th>Maximum particle size for testing mm</th>
<th>Minimum cylinder diameter mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>50</td>
</tr>
<tr>
<td>10</td>
<td>100</td>
</tr>
<tr>
<td>20</td>
<td>150</td>
</tr>
<tr>
<td>40</td>
<td>200</td>
</tr>
</tbody>
</table>

Figure 3.30: Falling head permeability test arrangement
Although the horizontal permeability will be significantly higher than the vertical permeability, but for simplification it was not considered in this research.

### 3.2.12 Debris Content

Debris is defined as any nonglass material, such as bottle caps, labels, paper, foil, plastics, metal, corks, wood debris, food residue, grass, or other deleterious materials.

The amount of debris in glass cullet can affect its engineering properties. It can negatively impact the performance of the engineered fill. No hazardous materials should be allowed in the glass cullet. Specifications should place a limit on the percentage of debris allowed in the cullet. Generally, debris levels should not exceed a maximum of 10 percent and in many applications 5 percent (CWC 1998).

The debris content is generally determined by visual method, by weight method and by volume method. Generally, the visual classification method produced a greater quantitative variation between the high and low debris levels than the volume and weight testing method. This is because most of the debris is platey in nature (labels, plastic and caps). A platey material (one which has a length and width but a very small thickness), will be readily measured using the visual method, which quantifies the cullet debris in a two-dimensional view. The volume and weight methods however, are affected by all three dimensions of the debris in equal proportion. As a result, the platey nature of the debris is reflected in the lower percentage results of these two methods. The visual inspection and classification test should be used for sub samples retrieved from various portions of the glass storage. The number of tests should be based on the quantity and homogeneity of the bulk material. In general, at least one test should be conducted for every 38.23 m$^3$ of material. The test results for all sub samples should be reported. In this investigation, debris content was determined by visual method and by weight method.

**Test method for determining debris content by visual method**

Visual inspection is a common procedure for the initial assessment of the acceptability of construction aggregate. The inspection is usually performed at storage sites prior to
any laboratory testing of the material. Sometimes, visual inspection is performed as a field screening procedure. In some cases, the acceptability of the material for a particular application may be based solely on the results of the field visual inspection. There is little background for standardized visual inspection procedures for recycled glass. A simple method has been used to obtain a percentage level of debris content of a glass cullet sample. The method is based on the Percent Composition Charts developed by American Geological Institute (AGI Data Sheets 23.1 and 23.2). These charts, shown in Figure 3.31 show the estimated percentage of composition of debris in a sample from 1 to 50%. The method uses a test pan of 203.2 to 254 mm in diameter and 25.4 to 50.8 mm in depth. 453.6 to 1360.8 gm of glass cullet is placed and levelled in the test pan. The test pan is then placed next to the standard charts and an estimated percentage is selected based on the comparison of the composition shown on the charts and the debris present on the test pan. It is important to disregard the aggregate and compare only the contaminants with the charts. The results can be recorded quantitatively using percentages, or qualitatively using terms such as low for 1 to 3%, medium for 3 to 15%, and high for over 15% (CWC 1998). Inter-medium terms such as low to medium, and medium to high can also be considered. It should be clearly understood that the percentages produced by the visual classification method are neither mass nor volume percentages. Rather, they are parameter-less indicators of the relative level of contamination in a glass sample.

![Figure 3.31: Estimating percentage contamination (CWC 1998)](image_url)
The purpose of performing the physical debris testing was to visually classify the recycled glass to identify the type and percentage of debris present. The collected recycled glass was evaluated using a visual classification method (American Geological Institute). This visual classification is a standard field method used by geologists to estimate the percentage composition of constituents in soil and rock (CWC 1993).

Test method for determining debris content by weight method

Debris Content by weight methods were performed on the oven-dried materials. Debris (material other than glass, e.g. bottle caps and labels, plastic tops, paper, foil, wood debris, food residue, etc.) was manually removed by hand and the sample was reweighed. The debris content by weight, $DC_w$, was determined by simply comparing the weight of the oven-dried debris, $W_d$, to the weight of the total oven-dried sample, $W_t$, i.e., the debris content was computed as the weight of oven-dried debris divided by the weight of total oven-dried materials.

$$ DC_w = \frac{W_d}{W_t} \times 100(\%) $$

Equation 3.12

where,

- $DC_w$ = the debris content by weight (%)
- $W_d$ = the weight of the oven-dried debris (g)
- $W_t$ = the total oven-dried sample (%)

3.2.13 Static Triaxial Test

The static triaxial tests are normally carried out by two methods. They are: (i) Consolidated undrained (CU) triaxial compression test and (ii) Consolidated drained (CD) triaxial compression test.

The CU triaxial compression test with measurement of pore pressure gives the undrained shear strength of a specimen subjected to a known initial effective stress and the pore pressure changes during shear from which the pore pressure coefficient $A$ can
be derived and the effective shear strength parameters at failure, $c'$ and $\phi'$, can be obtained from a set of tests.

During the CU compression stage, the cell pressure is maintained constant while the specimen is sheared at a constant rate of axial deformation with strain-controlled compression until failure. No drainage is permitted and therefore the moisture content remains constant during compression. The resulting changes in pore pressure are usually measured at the base of the specimen and the rate of axial deformation is applied slowly enough to ensure adequate equalization of excess pore pressures (Head 1994).

The CD triaxial compression test with measurement of volume change gives the drained shear strength and volume change characteristics during shear of a specimen from which the pore water is allowed to drain freely. The drained effective shear strength parameters at failure, $c'$ and $\phi'$, can be obtained from a set of tests.

During the CD compression stage, the cell pressure is maintained constant while the specimen is sheared at a constant rate of strain-controlled axial deformation until failure. Free drainage of pore water from the specimen is allowed. The test is run slowly enough to ensure that pore pressure changes due to shearing are negligible. The required rate of strain can be much slower than that for a CU test on a similar specimen under similar conditions. Since the pore pressure remains virtually constant, the effective confining pressure does not vary. The volume of pore fluid draining out of or into the specimen is measured by means of the volume change indicator in the back pressure line, and is equal to the change in volume of the specimen during shear.

For many soils other than heavily-overconsolidated clays, the parameters $c'$ and $\phi'$, determined from the two types of test, can be considered to be identical for most practical purposes (Head 1994).

Both types of test are usually carried out in three stages:

1) saturation
2) consolidation
3) compression
The first two stages saturate the specimen and bring it to the desired state of effective stress for the compression test, and are common to both types of test.

The procedures described relate to strain-controlled apparatus for compression in a mechanical load frame, and a detachable triaxial cell. Alternatively, hydraulic triaxial cells may be used, provided that the essential principles are maintained.

In this research, the static triaxial compression tests were performed in an automated triaxial testing system in accordance with ASTM D 4767 (ASTM D 4767-04), “Standard Test Method for Consolidated Undrained Triaxial compression Test for Cohesive soils”. According to the standard, the specimens shall have a height equal to about twice the diameter, with plane ends normal to the axis and the diameter of the largest particle shall not be greater than one-sixth of the specimen diameter (ASTM D 4767-04). Therefore, RG samples with a maximum particle size of 4.75 mm were tested using 50 mm by 100 mm specimens. As the maximum aggregate size of the other specimens was 20 mm, the samples were prepared in split mould of 100 mm in diameter and 200 mm in height. All aggregates were oven dried for 24 hours before preparing the sample and being mixed with the appropriate optimum moisture content. For blend samples, the aggregates were mixed as required percentage at dry weight by hand mixing with the use of aggregate splitter as well. The mixture was kept for 24 hours in a closed container to make moisture uniformity throughout the sample. The mixture was compacted to 98% of the maximum dry density in a split mould in eight layers. The compaction was done by mechanical compactor with around 18 blows of modified compactive effort for each layer.

After compaction, compacted specimens were sealed and left to mature for at least 24 hours to allow equalisation of pore pressure set up during compaction. Then, the sample was carefully separated from the split mould and the height, diameter and mass of the specimen was measured. A saturated porous disc was placed by sliding on to a layer of water on the triaxial base pedestal without entrapping air and any surplus water was removed. The specimen was placed on the disc without delay and without entrapping air. The second saturated disc was placed, with excess water removed, on top of the specimen. No side drains or filter papers were used in this research. Before setting the
rubber membrane with the specimen, the membrane was soaked in water and checked for any puncture. Using the membrane stretcher, the soaked rubber membrane was placed around the specimen, after allowing surplus water to drain off. The membrane was sealed to the base pedestal using two rubber O-rings removing air pockets from between the membrane and the specimen by light stroking upwards. Then another two O-rings were placed around the drainage lines connected to the top loadings cap. The followings equations were used to account for the corrections of membrane. The correction for the membrane stiffness was calculated as follows:

\[
\text{MSCOR} = \frac{\pi \times D}{A_0} \times \varepsilon_v \times (1 - \varepsilon_v) \times \text{CMEM}
\]

Where, CMEM is the membrane stiffness factor. The Figure 3.32 shows the arrangement for determining the membrane stiffness factor and Figure 3.33 shows the load versus displacement curve for calculating membrane stiffness factor. The calculated values of CMEM were 0.5141616716 N/mm and 0.2287423005 N/mm for a standard 100 mm diameter membrane of 0.635 mm thickness and 50 mm diameter membrane of 0.3025 mm thickness respectively.

If filter strips are present, an additional correction for filter strips is made as follows:

\[
\text{FXCOR} = \frac{\text{FSC} \times \varepsilon_v}{0.02} \quad : \quad 0 < \varepsilon_v < 0.02
\]

Where, FSC is the filter strip correction. Bishop and Henkel (1957) suggest a value of 13.8 kPa for FSC on 38.1 mm diameter samples with vertical filter strips covering half of the sample's circumference and 3.45 kPa on 101.6 mm diameter samples. As no strips were used in this research, correction for filter strips was regarded as zero.

Finally, the corrected deviator stress on the sample was calculated as follows:

\[
\sigma_{d_{corr}} = \sigma_d - \text{MSCOR} - \text{FXCOR}
\]

The compacted specimen for triaxial test is shown in Figure 3.34. The sample with membrane and O-ring set up is shown in Figure 3.35. The sample pressure and drainage
lines were also connected to the top loading cap. After that the triaxial cell body with the loading piston well clear of the specimen top cap was assembled, the cell was filled with the de-aired water ensuring that all the air is displaced through the air bleed plug. After that the triaxial cell was placed on the base of an automated loading frame called “Load Trac” which was connected to the computer to control the deviator stress.

The cell and sample pressure of the triaxial system was controlled by the automated flow pump units called “Flow Trac”. It contained embedded control systems and the components to generate pressures on a specimen and control and measure pressure and volume changes. The triaxial cell was connected to both the cell and sample pressure flow pump units. The “Flow Trac” is shown in Figure 3.36.

![Figure 3.32: Arrangement for determining membrane stiffness factor](image)
As the capacity of the sample Flow Trac cylinder and cell Flow Trac cylinder is only 250 cc and it shows empty or full limit problem, first the soil specimen was flushed with de-aired water using Jog menu form the sample Flow Trac. Then the initialization process was carried out before the saturation phase for the following reasons:
➢ It allows the user to check the proper functioning of the entire system (Load Trac-II and two Flow Trac-II units) while it is applying only a small effective stress of about 7 kPa on the sample.

➢ It allows the user to detect early leaks through the rubber membrane, the fittings, or the triaxial cell (acrylic chamber not tightened, O-rings out of alignment, etc.).

➢ It applies a small effective stress on the sample so that the sample state of stresses is moved away from the failure line on a stress path diagram. This will prevent any possible early set-up failure of the sample, especially for very soft soils or very loose sands.

![The sample with membrane and O-rings set up](image)

Figure 3.35: The sample with membrane and O-rings set up

Then, the back pressure saturation stage was initiated by increasing the back pressure and cell pressure at an effective stress of 13 kPa up to a minimum back pressure of 560 kPa. If the Skempton’s B-value of 0.95 was not achieved with these minimum back pressure of 560 kPa, cyclic process of increasing and decreasing cell pressure was continued until Skempton’s B-value of 0.95 or more was achieved in each case. Immediately after completion of back pressure saturation, isotropic consolidation stage was carried out to bring the specimen to the state of effective stress required for
carrying out the compression test. In this investigation, the isotropic consolidation stage was carried out at effective confining stresses of 50 kPa, 100 kPa and 200 kPa. The soil specimen was kept for consolidation for a period of 8 to 12 hours.

Figure 3.36: The “Flow Tracs”

Triaxial compression (shearing) was executed on the saturated and previously consolidated specimens. The samples were compressed at the given confining pressures under drained conditions (CD test). The shearing was performed under strain-controlled condition at the selected strain rate of 0.01 %/min. In these tests, the cell pressure was controlled by the cell pressure “Flow Trac-II” flow pumps to the water in the triaxial chamber and the back pressure was controlled by the sample pressure “Flow Trac-II” flow pumps to the sample in terms of water pressure. All the phases of the tests were automatically controlled by the specialized software program called “TRIAXIAL”. The software allowed the user to define the conditions for running the test. The program was featured with real time data and status information displayed on the monitor in numeric form or graphical form. The Load Trac was used to take the reading of the applied load, an external LVDT was used to calculate axial deformation of the specimen and Flow Track-II was used to take the reading of volume change. The fully automated triaxial system is shown in Figure 3.37. The test was continued until either of the following conditions has been clearly identified:

- Maximum deviator stress;
- Shear deformation continuing at constant volume and constant shear stress.
If neither of the required failure conditions is evident, the test was terminated at an axial strain of 15%.

When the test has ended, a window indicating that the test has finished will appear on the screen. Then clicking on the OK on this window, the window will close but the load on the sample will be maintained until we are ready to dismantle it.

Immediately after completing the compression stage and releasing all stresses on it, the specimen was removed from the triaxial cell pedestal so that the absorption of water from the porous discs is kept to a minimum and to get an accurate water content determination for the specimen at the end of the test.

![The fully automated triaxial system](image)

Figure 3.37: The fully automated triaxial system

Before lowering the platen, the piston was locked, the sample-pressure valve and the cell-pressure valve were closed and the sample-pressure tubing and the cell-pressure tubing were removed from the quick-connect couplings.

The platen was lowered utilizing the Load Trac-II LCD Position menu and the keypad and the triaxial cell was removed from the load frame. The cell was placed in a sink, a
nipple was inserted into the quick-connect coupling on the cell top to release the pressure and the cell water was discarded into the sink. The knobs at the top of the posts were loosened and the posts were removed. The piston lock was loosened and the piston was removed from the cell. The top was lifted from the plastic chamber and the chamber was removed from the base. The two tubes were removed from the specimen cap. The specimen was pulled off of the base pedestal and carried it to a work area. The O-rings at the top and bottom of the specimen were removed, unfolding the membranes as needed to expose the O-rings under the fold and the membranes and porous stones were removed from the specimen.

The diameter of specimen was measured near the top, at the middle and near the bottom and the average diameter was recorded. Three measurements of the specimen's height, each about a third of the way around the specimen from the other was made to determine the average height. The whole of the specimen was weighed to the nearest 0.01 gm and a representative amount of trimmings was also taken for determining the after-test water content of the specimen.

3.2.14 Wet and Dry strength

Wet and Dry strength tests were performed in accordance with AS 1141.22 “Methods for sampling and testing aggregates-Wet/Dry strength variation” (AS 1141.22-2008).

The selected coarse fraction of the material was taken from material passing through 13.2 mm sieve and retaining on 9.5 mm sieve. The selected material was then washed with water to remove the adhering fines and oven dried to constant mass. Then, the measure was filled slowly to approximately one third capacity, pouring the aggregate from about 100 mm above the measure. The measure was tapped with 25 strokes of the mallet around its surface to ensure that the settling effect has taken place. The procedure was repeated for another two layers by the same method and top of the measure was gently levelled using the straightedge. The mass of the material filled the measure was recorded \(m_A\). At least five test portions of aggregate of mass \(m_A\) were determined as above. The samples were kept in sealed and labelled containers and store until ready for testing.
Two test portions were tested for dry strength. A portion of sample was placed in the cylinder into three layers and the cylinder was tapped with 25 strokes as previously done to the measure. The plunger was placed horizontally on top of the aggregates and fixed with compression machine. The force was applied at a uniform rate for 10 minutes and the maximum applied load at the end of the test was recorded. The tested sample was sieved through 2.36 mm sieve and the remaining mass was weighed \((m_B)\). The passing percentage of fines should fall between 7.5% and 10%, the force for the second portion was selected to provide a passing value between 10% and 12.5 % and vice versa.

The percentage loss of fines was calculated from the following equation:

\[
\text{Percentage loss} = \left( \frac{m_A - m_B}{m_A} \right) \times 100 \quad \text{Equation 3.16}
\]

where,

\[
m_A = \text{the debris content by weight (\%)}
\]
\[
m_B = \text{the weight of the oven-dried debris (g)}
\]

Another two test portions were tested for saturated surface dry condition. The test portion was immersed in the container at room temperature for a minimum of 24 hours and a maximum of 72 hours with a cover of at least 50 mm of water above the top of the aggregate. After that the sample was transferred to the separation sieve of 2.36 mm and washed under running water until the water passing the sieve runs clear. The sample was drained for few minutes and transferred to a dry absorbent cloth. The material was surface dried until all the visible films of water had been removed but the surfaces of particles still appeared damp. Then all the procedures were followed same as for dry strength.

For each dry and saturated surface dry condition, the compression force (kN) versus percentage of loss were plotted and a straight line was drawn through the points lying between 7.5 % and 12.5 %. The compression forces corresponding to 10 % loss of fines were interpolated and recorded for each condition. The dry strength \((D)\) and wet strength \((W)\) were recorded and the wet and dry strength variation was calculated using the following equation:

\[
\text{Wet/dry strength variation} = \left( \frac{D - W}{D} \right) \times 100 \quad \text{Equation 3.17}
\]
where,
\[ D = \text{dry strength in kN} \]
\[ W = \text{wet strength in kN} \]

### 3.2.15 Repeated Load Triaxial (RLT) Test

The Repeated Load Triaxial (RLT) tests were performed in accordance with the Austroads (2000) Repeat Load Triaxial Test Method AG:PT/T053 “Determination of Permanent Deformation and Resilient Modulus Characteristics of Unbound Granular Materials Under Drained Conditions”, the testing procedure of which is similar to the test methods specified by AASHTO T 307-99 (2003). In general, the RLT testing consists of two phases of testing, permanent strain testing followed by resilient modulus testing. Permanent strain testing consists of three or four stages, each performed at different deviator stresses and a constant confining stress. A constant confining stress of 50 kPa and deviator stresses of 150 kPa, 250 kPa and 350 kPa were applied at each stage respectively. Each loading consisted of 10,000 repetitions. The resilient modulus testing phase consists of sixty six (66) loading stages with 50 repetitions, where confining stress varies between 20 kPa and 150 kPa and deviator stress varies between 100 kPa to 600 kPa. 3 samples were prepared and air dried back to target moisture contents of 70 %, 80 % and 90 % of the optimum moisture content (OMC), to simulate the possible field moisture contents of sub-bases.

**Specimen preparation**

Repeat load triaxial (RLT) test specimens were prepared for RLT testing using the dynamic compaction method (AS 1289.5.2.1-2003). The automatic (mechanical) compaction apparatus, which permits a continuous and even compaction mode, was used to produce uniform specimens to specified density and moisture condition. All the specimens were compacted to the target density of 98% MDD and target moisture content of 100% OMC. They were then allowed to dry back to target moisture ratio. Generally, it was able to prepare the specimens within the tolerance of 0.5% for density ratio using the dynamic compaction method at 100% OMC. However, it was more difficult to achieve moisture contents within the tolerance of 0.5% for moisture ratio.
using the dry back method. The mechanical compactor used to prepare the RLT sample is shown in Figure 3.38. Figure 3.39 shows the compacted sample of RCC blended with RG after drying to the targeted moisture content and ready for testing. Figure 3.40 shows the sample which was kept in the room temperature for dry back. Although the dry back procedure applied in the laboratory will create moisture gradient from inside to outside of the sample and differ from field dry back procedure, but is the standard procedure which is normally followed by most of the laboratory.

Figure 3.38: The mechanical compactor used to prepare the RLT sample

Figure 3.39: The compacted sample of RCC blended with RG after drying to the targeted moisture content
**RLT test equipment**

The RLT testing equipment used in the laboratory testing program complied with the standard equipment specifications. A brief description of the equipment is given below:

The loading system comprises a Universal Material Testing Apparatus (UMATTA)-Open Loop. It has the maximum vertical dynamic force capacity of 5 kN with loading cycle having a period of 3 s with a load pulse width of 1 s and rise and fall times up to 0.3 s and maximum static pressure capacity of 500 kPa. Seating stress due to friction in the loading actuator is less than 5 kPa. Standard triaxial cell for 100 mm diameter samples with a working pressure of at least 500 kPa is used here. Bellofram seal is used to seal loading shaft-top cell cover assembly. Pressure transducer meeting the requirements for AS 2193 for industrial gauges and load transducer meeting the requirements for AS 2193 for Grade A load transducers are used here. Output of the internal load transducer is independent of confining pressure. 20 mm range external displacement transducer for measuring permanent strain and meeting the requirements for AS 1545 for a Grade B extensometer is used here. 2 x 5 mm range internal
displacement transducers meeting the requirements for AS 1545 for a Grade B extensometer is used for measuring resilient strain. Latest version of UTM Computer software which is capable of filtering electrical and mechanical noise levels in the recorded data (i) < 2 kPa for vertical stress, (ii) < 2 kPa for confining stress, (iii) < 25 micro-strain for resilient strain, and (iv) < 50 micro-strain for permanent strain is used for recording the data.

Figure 3.41 shows the automated repeat load triaxial system used in this study. Once the sample was ready for testing, it was assembled in the triaxial cell. Air was used as the confining fluid instead of water. Load, pressure and displacement were measure by the respective transducers and the whole system was controlled by the computer software. After entering the input parameters into the controlling software, the test was started by a single click. All the readings were automatically saved for analysis purposes.

![Figure 3.41: The repeated load triaxial system](image)

**RLT test procedure**

The RLT testing procedure consists of a permanent strain test followed by a resilient modulus test. The permanent deformation determination characterises the vertical permanent strain with multiple loading stages (at different stress conditions) to enable
quantification of the effects of vertical stress on permanent strain in a single test. For the crushed rock blends, which appeared to fail quickly under the stress level of 450 kPa deviator stress and 50 kPa confining stress, three different loading stages (at specified deviator stresses of 150 kPa, 250 kPa, 350 kPa, 450 kPa and 550 kPa, respectively) were used, each loading stage involved 10,000 repetitions. A confining stress of 50 kPa was applied for all loading stages. For the crushed concrete blends, which could carry the stress level of 450 kPa deviator stress and 50 kPa confining stress, four different loading stages (at specified deviator stresses of 250 kPa, 350 kPa, 450 kPa and 550 kPa, respectively) were used, each loading stage involved 10,000 repetitions.

The resilient modulus determination characteristics the vertical resilient strain response over sixty six (66) stress conditions using combinations of applied dynamic vertical and static lateral stresses in the ranges from 100 to 600 kPa and from 20 to 150 kPa, respectively. Each stress condition involved 50 load repetitions. The stress paths for a typical resilient modulus testing are shown in Figure 3.42. The stresses and stress ratio were increased in small sizes to avoid early failure, which can occur at high stress ratios.

![Figure 3.42: The stress paths for a typical resilient modulus testing](image-url)
3.2.16 Direct Shear Test (DST)

The Direct Shear test was conducted according to the British Standard (BS 1377-7-1990) on specimens compacted inside the shear box with the optimum water content to achieve the maximum dry density obtained in the modified compaction tests. It is recommended that the size of the largest particle of the test specimen should not exceed one-tenth of the specimen height (BS 1377-7-1990). Therefore, with the 10 cm by 10 cm shear box having the effective depth of 4 cm, only the RG samples of particle size smaller than 4.75 mm were tested. The tests were performed on RG specimens compacted to 95% (± 2%) of their maximum dry density ($\gamma_{dmax}$) values based on the modified Proctor compaction results. Figure 3.43 shows the accessories required for compaction small DST samples in the small shear box. Figure 3.44 shows the fully automated small DST system.

In order to assess the degree of nonlinearity in the Mohr-Coulomb failure envelope, direct shear tests were conducted on 5 specimens over a wide range of normal stress ranging from 25 to 400 kPa. The RG samples were placed in the direct shear box in 5 layers and were compacted with a rubber tipped pestle. The RG specimens were sheared at a rate of 0.3 mm/min. Figure 3.45 shows the small shear boxes sample after shearing and dismantling from the system.

The CR and RCC samples had a nominal size of 20 mm. Therefore, 30.5 cm by 30.5 cm by 20.3 cm deep large shear box was used for the direct shear tests of as-received CR and RCC specimens. All the CR and RCC samples were oven dried for at least 24 hours at 105°C and allowed to cool down before adding with the appropriate optimum water content. The mixture was kept for 24 hours in a closed container to make moisture uniformity throughout the sample. The mixture was compacted to 98% of the maximum dry density in a split mould in three layers.

The upper and lower boxes of the large direct shear apparatus were fixed together using the appropriate screws. Filter paper was placed on the inside of the bottom shear box. The sample was compacted to 98% of maximum dry density into the shear box in three layers by using vibrator compactor as shown in Figure 3.46. The compacted surface was scarified to around 5 mm before the placement of the next layer for better bonding.
between the layers. The height of the compacted sample was targeted as 192 mm and the top of the sample surface was levelled. The top plate was placed on top of the levelled sample. Figure 3.47 shows the final compacted sample in the large shear box.

Figure 3.43: Accessories required for preparing small DST Sample

Figure 3.44: The fully automated small DST system
Figure 3.45: The small shear boxes sample after shearing and dismantling from the system

Figure 3.46: Vibrator hammer used for compacting sample into the shear box
The shear box was shifted to the automated apparatus and all the fittings and LVDTs were connected. Figure 3.48 shows the final set up of the fully automated large shear box with compacted sample. The normal load was applied at the centre of the large shear box using the load cell. The upper box was fixed and the lower box was moveable. During the consolidation phase, vertical normal force was applied and all the displacements and loadings were recorded via LVDTs and load cells. At the beginning of the consolidation phase, water was poured around shear box up to top of the lower box for consolidation purposes and to prevent loss of water due to evaporation. After the consolidation stage, connections between the lower and upper boxes were released and around 2 mm shear gap was provided between the upper and lower boxes to avoid the friction between the edge surfaces of the boxes. Shearing was allowed at a prescribed rate based on the consolidation of the samples. All the readings were automatically recorded via LVDTs and load cells.

![Figure 3.47: Final compacted sample in the large shear box](image)

All the phases of the tests were automatically controlled by the specialized software program called “SHEAR” which allowed the user to define the conditions for running the test. The program was featured with real time data and status information displayed on the monitor in numeric form or graphical form. Figure 3.49 shows the shear box and the sheared sample after dismantling form the testing system.
Figure 3.48: Final set up of the fully automated large shear box with compacted sample

Figure 3.49: Shear boxes and sample after shearing and dismantling from the system
CHAPTER FOUR

4 GEOTECHNICAL PROPERTIES OF RECYCLED GLASS AND CRUSHED ROCK BLENDS

4.1 Introduction

The laboratory study on RG/CR blends were conducted on mixtures comprised of 10%, 15%, 20%, 30%, 40% and 50% RG (by mass) mixed with Class 3 CR. The blends are represented by RG10/CR90, RG15/CR85, RG20/CR80, RG30/CR70, RG40/CR60 and RG50/CR50, respectively. Here, RG blended with CR is denoted by RGXX/CRYY. The suffix XX after RG represents the percentages of RG and the suffix YY after CR represents the percentages of CR in the mixtures by mass, respectively. The tests were also conducted on 100% RG and 100% CR for comparison and regarded as RG100 and CR100, respectively.

4.2 Test results

4.2.1 Particle size distribution

Particle size distribution tests were conducted on “as-received RG and CR” samples to determine their grain size distribution curves and soil classification according to the Australian Soil Classification System (ASCS). The main difference between the Unified Soil Classification System (USCS) and the ASCS is the boundary between sand and gravel size particles. The USCS (ASTM D 2487-2011) assumes that 4.75 mm (No. 4 sieve) is the border line between sand and gravel, whereas in the ASCS 2.36 mm has been selected as the border line between sand and gravel particles. This means that particles larger than 2.36 mm are considered gravel according to the ASCS. Table 4.1 presents the physical properties of as-received RG and CR samples based on their particle size distribution test results. RG is classified as well graded sand mixture with little amount of silt size particles (SW) and CR is classified as poorly graded gravel with silt and sand mixture (GP-GM) (Standards Australia 1993).
Additional sieve analyses were also conducted on compacted samples of RG and CR from the Modified Proctor test to assess the compaction induced particle breakage that may shift the grain-size distribution of the materials. These samples are referred to as “after compacted”. Under modified compactive effort, a small portion of the gravel and sand size particles were crushed and reduced to fine size particles (Table 4.1). The percentages of gravel size particles in the mixture decreased from 26 for the as-received sample to 25.4 for the after compacted sample. The percentages of sand size particles in the mixture decreased from 71.2 for the as-received sample to 70.4 for the after compacted sample. The change in particle size distribution of recycled glass after modified compaction test is due to the crushing of gravel and sand size particles under compaction energy and subsequent decrease in their size to mainly finer particles. Overall, the results showed negligible change in the particle size distribution curve of the RG samples before and after modified compaction. This indicates that the RG materials are stable mixtures in engineering operations including handling, spreading and especially compaction.

Figure 4.1 presents the grain size distribution curves for the as-received and after compacted recycled glass samples. In the as-received condition, samples were predominately granular, with less than 3% passing the No. 200 sieve (i.e., less than 75 μm). The small amount of material passing the number 200 sieve was non-plastic. As expected, the samples extracted from the compaction mould were a little bit finer-grained because of particle breakage; however, as indicated in Table 4.1, both the as-received and after compacted samples were classified as SW based on the ASCS. Materials meeting this classification (SW) typically have high strengths and low compressibility, making them good candidates for compacted fill applications (PennDOT 2001).

Again, in case of CR, under modified compaction energy, a small portion of the gravel size particles were crushed and reduced to sand and fine size particles as shown in Table 4.1. The post compaction sieve analysis results on CR source showed a little change in gradation curve after modified compaction as compared with the as-received sample. The sand content of the as-received CR sample increased from 28% to 30.2% after modified compaction effort. The percentage of gravel size particles in the mixture decreased from 60% for the as-received sample to 56.6% after modified compaction
The change in particle size distribution of CR after modified compaction test is due to the crushing of gravel size particles under modified compaction energy and subsequent decrease in their size to mainly to sand size and finer particles. Overall, the small changes in the particle size distribution of CR sources after modified compaction indicate that the CR sources are stable mixtures during the engineering operations including handling, spreading and especially compaction. Figure 4.2 presents the grain size distribution curves for the as-received and after compacted crushed rock. Table 4.2 demonstrates the results of other tests performed on as-received RG and CR samples.

Table 4.1: Basic properties of as-received samples based on AS 1726-1993 Standard

<table>
<thead>
<tr>
<th>Material Identification</th>
<th>Recycled Glass (RG)</th>
<th>RG-After Compacted</th>
<th>Crushed Rock (CR)</th>
<th>CR-After Compacted</th>
</tr>
</thead>
<tbody>
<tr>
<td>Classification</td>
<td>SW</td>
<td>SW</td>
<td>GP-GM</td>
<td>GP-GM</td>
</tr>
<tr>
<td>Maximum particle size (mm)</td>
<td>4.75</td>
<td>4.75</td>
<td>20.0</td>
<td>20.0</td>
</tr>
<tr>
<td>Coefficient of uniformity (Cu)</td>
<td>6.2</td>
<td>8.4</td>
<td>88</td>
<td>95.5</td>
</tr>
<tr>
<td>Coefficient of curvature (Cc)</td>
<td>1.5</td>
<td>1.6</td>
<td>4.5</td>
<td>5.0</td>
</tr>
<tr>
<td>Fines content (&lt;0.075 mm) (%)</td>
<td>2.8</td>
<td>4.2</td>
<td>12.0</td>
<td>13.2</td>
</tr>
<tr>
<td>Gravel (&gt; 2.36 mm) (%)</td>
<td>26.0</td>
<td>25.4</td>
<td>60.0</td>
<td>56.6</td>
</tr>
<tr>
<td>Sand (0.075-2.36 mm) (%)</td>
<td>71.2</td>
<td>70.4</td>
<td>28.0</td>
<td>30.2</td>
</tr>
</tbody>
</table>

Sieve analyses were conducted on the RG/CR blends to determine their grain size distribution curves. Additional sieve analyses were also conducted on compacted samples of RG/CR blends from the Modified Proctor test to assess the compaction induced particle breakage that may shift the grain-size distribution of the materials. These samples are referred to as “after compacted”. Table 4.3 and Table 4.4 summarise the grain size distribution of before compaction and after compaction, respectively for RG blended with CR (Class 3) sourced from Alex Fraser Recycling, Laverton. The gradation curves of RG/CR blends before compaction as compared with local state authority’s requirements are shown in Figure 4.3. Figure 4.4 shows the grain size distribution curves of RG/CR blends after modified compaction along with local state authority’s requirements. Table 4.5 summarise the engineering properties of RG blended with CR (Class 3).
Figure 4.1: Grain size distribution curves for the as-received and after compacted RG

Figure 4.2: Grain size distribution curves for the as-received and after compacted CR
Table 4.2: Geotechnical properties of 100% RG and 100% CR Samples

<table>
<thead>
<tr>
<th>Test</th>
<th>Standard method</th>
<th>RG</th>
<th>CR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity (G_s)</td>
<td>AS 1141.5 and AS 1141.6.1</td>
<td>2.49</td>
<td>2.84</td>
</tr>
<tr>
<td>Organic content (%)</td>
<td>ASTM D 2947-00</td>
<td>0.5</td>
<td>0.8</td>
</tr>
<tr>
<td>Debris level (visual method) (%)</td>
<td>AGI1 23.1 &amp; 23.2</td>
<td>2</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>Debris level (weight method) (%)</td>
<td>CWC2 Method</td>
<td>0.2</td>
<td>&lt; 0.1</td>
</tr>
<tr>
<td>pH value</td>
<td>AS 1289.4.3.1</td>
<td>9.6</td>
<td>9.4</td>
</tr>
<tr>
<td>LA abrasion loss (%)</td>
<td>ASTM C 131-06</td>
<td>27</td>
<td>24</td>
</tr>
<tr>
<td>Flakiness index</td>
<td>BS 812-105.1</td>
<td>NA3</td>
<td>16</td>
</tr>
<tr>
<td>Standard proctor</td>
<td>AS 1289.5.1.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>γ_d max (Mg/m^3)</td>
<td>AS 1289.5.2.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>w_opt (%)</td>
<td>AS 1289.5.2.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Modified proctor</td>
<td>AS 1289.5.2.1</td>
<td>1.73</td>
<td>2.10</td>
</tr>
<tr>
<td>Hydrulic conductivity (m/s)</td>
<td>BS 1377-5</td>
<td>3.3E-5</td>
<td>6.1E-8</td>
</tr>
<tr>
<td>California Bearing Ratio (CBR)</td>
<td>AS 1289.6.1.1</td>
<td>44</td>
<td>181</td>
</tr>
<tr>
<td>Direct Shear Test (DST)</td>
<td>BS 1377-7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cohesion (kPa)</td>
<td></td>
<td>37.7</td>
<td>286.4</td>
</tr>
<tr>
<td>Internal friction angle (degree)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>σ_n (50-200 kPa)</td>
<td></td>
<td>49.0</td>
<td>48.3</td>
</tr>
</tbody>
</table>

1 American Geological Institute  
2 Clean Washington Centre 1998  
3 Not applicable

The results of before compaction grading curves indicated that CR blends with greater than 40% & 50% RG additive would potentially produce grading exceeding the local state authority’s upper limits. The difference in the trends of the curves would be due to slight variations in the constitution of the samples.
The results of after compaction grading curves indicated that CR blends with greater than 50% RG additive would potentially produce grading exceeding the local state authority’s upper limits.

Overall, the before and after compacted grain size distribution curves of the samples of RG/CR blends indicate that the materials appear to be remaining reasonably well graded through the compaction process and this will generally aid the compaction process.

Table 4.3: Particle size distribution (before compaction)

<table>
<thead>
<tr>
<th>Sample Description</th>
<th>RG50/CR50</th>
<th>RG40/CR60</th>
<th>RG30/CR70</th>
<th>RG20/CR80</th>
<th>RG15/CR85</th>
<th>RG10/CR90</th>
</tr>
</thead>
<tbody>
<tr>
<td>RG (%) by mass</td>
<td>50</td>
<td>40</td>
<td>30</td>
<td>20</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>Particle size (mm)</td>
<td>26.5</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td></td>
<td>19.0</td>
<td>98.8</td>
<td>99.3</td>
<td>99.3</td>
<td>99.0</td>
<td>98.1</td>
</tr>
<tr>
<td></td>
<td>13.2</td>
<td>93.2</td>
<td>92.7</td>
<td>88.4</td>
<td>86.8</td>
<td>86.4</td>
</tr>
<tr>
<td></td>
<td>9.5</td>
<td>89.4</td>
<td>88.0</td>
<td>80.8</td>
<td>79.4</td>
<td>79.2</td>
</tr>
<tr>
<td></td>
<td>4.75</td>
<td>80.1</td>
<td>75.4</td>
<td>67.9</td>
<td>63.8</td>
<td>63.4</td>
</tr>
<tr>
<td></td>
<td>2.36</td>
<td>56.6</td>
<td>52.1</td>
<td>49.0</td>
<td>44.2</td>
<td>45.7</td>
</tr>
<tr>
<td></td>
<td>0.075</td>
<td>8.1</td>
<td>8.8</td>
<td>9.2</td>
<td>9.9</td>
<td>10.9</td>
</tr>
</tbody>
</table>

4.2.2 Particle Density

Particle density, a measure of material’s specific gravity, affects the dry, partially-saturated, and saturated unit weights of porous media and is a widely used parameter in establishing the density-volume relationship of a soil mass. Typical values of specific gravity for natural aggregates are 2.65 to 2.68 (Bowles 1988), and typical values for commercial glass are 2.49 to 2.51 (CWC 1998). The specific gravity of most soils lies between 2.60 and 2.80, but the presence of particles consisting of other minerals will
result in different value (Head 1994). The specific gravities of the crushed rock and gravelly sand ranged from 2.60 to 2.83 (CWC 1998).

Table 4.4: Particle size distribution (after compaction)

<table>
<thead>
<tr>
<th>Sample Description</th>
<th>RG50/CR50</th>
<th>RG40/CR60</th>
<th>RG30/CR70</th>
<th>RG20/CR80</th>
<th>RG15/CR85</th>
<th>RG10/CR90</th>
</tr>
</thead>
<tbody>
<tr>
<td>RG (%) by mass</td>
<td>50</td>
<td>40</td>
<td>30</td>
<td>20</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>Particle size (mm)</td>
<td>Percentage of total passing (%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>26.5</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>19.0</td>
<td>99.5</td>
<td>100.0</td>
<td>99.7</td>
<td>99.3</td>
<td>99.7</td>
<td>99.8</td>
</tr>
<tr>
<td>13.2</td>
<td>92.2</td>
<td>93.4</td>
<td>87.3</td>
<td>91.5</td>
<td>89.3</td>
<td>87.5</td>
</tr>
<tr>
<td>9.5</td>
<td>86.5</td>
<td>87.9</td>
<td>81.2</td>
<td>83.8</td>
<td>81.1</td>
<td>79.9</td>
</tr>
<tr>
<td>4.75</td>
<td>77.5</td>
<td>76.8</td>
<td>69.5</td>
<td>69.0</td>
<td>64.5</td>
<td>63.0</td>
</tr>
<tr>
<td>2.36</td>
<td>60.0</td>
<td>58.5</td>
<td>53.4</td>
<td>52.4</td>
<td>48.6</td>
<td>47.5</td>
</tr>
<tr>
<td>0.425</td>
<td>20.6</td>
<td>20.7</td>
<td>20.8</td>
<td>21.1</td>
<td>21.0</td>
<td>21.4</td>
</tr>
<tr>
<td>0.075</td>
<td>9.1</td>
<td>8.4</td>
<td>10.8</td>
<td>11.8</td>
<td>11.9</td>
<td>12.6</td>
</tr>
</tbody>
</table>

Figure 4.3. Gradation curves of RG/CR blends (before compaction)
Table 4.5: Engineering properties of recycled glass blended with crushed rock (Class 3)

<table>
<thead>
<tr>
<th>Sample Description</th>
<th>RG50/CR50</th>
<th>RG40/CR60</th>
<th>RG30/CR70</th>
<th>RG20/CR80</th>
<th>RG15/CR85</th>
<th>RG10/CR90</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glass Content (%) by weight</td>
<td>50</td>
<td>40</td>
<td>30</td>
<td>20</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>Test description</td>
<td>Test results</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Particle density (Coarse) (Mg/m$^3$)</td>
<td>2.76</td>
<td>2.76</td>
<td>2.77</td>
<td>2.77</td>
<td>2.78</td>
<td>2.78</td>
</tr>
<tr>
<td>Particle density (Fine) (Mg/m$^3$)</td>
<td>2.61</td>
<td>2.65</td>
<td>2.71</td>
<td>2.79</td>
<td>2.75</td>
<td>2.80</td>
</tr>
<tr>
<td>Water absorption (Coarse) (%)</td>
<td>2.00</td>
<td>2.50</td>
<td>2.60</td>
<td>3.00</td>
<td>3.10</td>
<td>3.15</td>
</tr>
<tr>
<td>Water absorption (Fine) (%)</td>
<td>2.00</td>
<td>2.00</td>
<td>2.50</td>
<td>2.45</td>
<td>2.40</td>
<td>2.30</td>
</tr>
<tr>
<td>CBR (%)</td>
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<td>137</td>
<td>152</td>
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<tr>
<td>Los Angeles abrasion loss (%)</td>
<td>25</td>
<td>25</td>
<td>24</td>
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<td>Organic content (%)</td>
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<td>0.6</td>
<td>0.7</td>
<td>0.7</td>
<td>0.8</td>
</tr>
<tr>
<td>pH value</td>
<td>9.6</td>
<td>9.7</td>
<td>9.7</td>
<td>9.7</td>
<td>9.7</td>
<td>9.7</td>
</tr>
<tr>
<td>Modified Compaction</td>
<td>MDD (Mg/m$^3$)</td>
<td>2.13</td>
<td>2.14</td>
<td>2.18</td>
<td>2.21</td>
<td>2.24</td>
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<td></td>
<td>OMC (%)</td>
<td>8.80</td>
<td>9.00</td>
<td>9.30</td>
<td>9.20</td>
<td>8.50</td>
</tr>
<tr>
<td>Fines content (%)</td>
<td>8.1</td>
<td>8.8</td>
<td>9.2</td>
<td>9.9</td>
<td>10.9</td>
<td>10.5</td>
</tr>
</tbody>
</table>
Since density relates directly to engineering properties such as compaction and shear strength, specific gravity is an important baseline property. Particle density test was performed in accordance with AS 1141.6.1 and AS 1141.5. The RG had specific gravity value of 2.49 as shown in Table 4.2. The value is about 10% lower than those of most natural aggregates, suggesting that glass cullet will be associated with lower surcharge and backfill pressures than natural aggregates (PennDOT 2001). The specific gravity of RG test result shows that at the same weight, 10% to 15% more volume of RG aggregates can be shipped compared with natural aggregates, resulting in lower shipping costs (CWC 1998).

The CR had specific gravity value of 2.84 as shown in Table 4.2. The specific gravity test results suggest that CR possess a specific gravity value similar to those of natural aggregates (Craig 1997).

The particle densities of coarse blended aggregates passing 19 mm and retained on 4.75 mm sieve varies from 2.76 to 2.78 Mg/m$^3$ and that of fine blended aggregates passing 4.75 mm sieve varies from 2.61 to 2.80 Mg/m$^3$ as shown in Figure 4.5 and Figure 4.6, respectively. The weighted average particle densities of the RG/CR blends vary from 2.64 to 2.79 Mg/m$^3$ as shown in Figure 4.7. The specific gravity test results suggest that RG/CR blends possess a specific gravity value similar to those of natural aggregates (Craig 1997).

### 4.2.3 Water Absorption

The water absorption of coarse blended aggregates passing 19 mm and retained on 4.75 mm sieve varies from 2.00 to 3.15% and that of fine blended aggregates passing 4.75 mm sieve varies from 2.00 to 2.50% as shown in Figure 4.8 and Figure 4.9, respectively. The weighted average water absorption of the RG/CR blends vary from 2.00 to 2.70% as shown in Figure 4.10.
Figure 4.5: Particle density of coarse RG/CR blends

Figure 4.6: Particle density of fine RG/CR blends
Figure 4.7: Weighed average particle density of RG/CR blends

Figure 4.8: Water absorption of coarse RG/CR blends
4.2.4 Debris content

Debris content tests were performed in accordance with the visual method and weight method on the recycled glass in its as-received condition. Debris (material other than
glass, e.g. woods, metals, bottle caps and labels, plastic tops, etc.) was manually removed and the sample was reweighed. The debris content was computed as the weight of debris divided by the weight of recycled glass.

The visual method is based on the Percent Composition Charts developed by American Geological Institute (Comparison Chart for Estimating Percentage Composition, AGI Data Sheets 23.1 and 23.2).

The debris levels of RG were 2% and 0.2% for visual method weight method, respectively. The debris content determined by the visual method was 10 times higher than that by weight method. The debris content determined by the visual inspection method will generally produce results higher than the debris content measured by physical tests such as the measurement of percent debris by weight as the debris content lays flat and the visual method is based on the two-dimensional view of debris (CWC 1998). As the debris contents determined by both the methods are less than 3%, it can be said that recycled glass sources have low debris values obtained through visual and weight methods (CWC 1998).

The debris levels of CR were less than 1% & 0.1%, respectively for visual method and for weight method. The debris content determined by the visual method was about 10 times higher than that by weight method. The debris content determined by the visual method is based on the two-dimensional view of debris as the debris lays flat, the debris content determined by the visual inspection method will generally produce results higher than the debris content measured by physical tests such as the measurement of percent debris by weight (CWC 1998). As the debris content determined by both the methods is less than 3%, it can be said that both CR sources have low debris values obtained through visual and weight methods (CWC 1998).

4.2.5 Standard and Modified Compaction

Standard and Modified Proctor compaction tests were performed on the as-received recycled glass samples. Figure 4.11 shows the moisture-density relationships for the as-received recycled glass. For comparison purposes, the zero-air-void curve is also shown on this figure. Table 4.2 summarizes the maximum dry density and optimum water
content for the compaction tests. Table 4.2 shows the value of maximum dry density obtained for RG sample is about 10% lower than the values found for the natural aggregate with the same soil classification (Craig 1997). Similar to many natural aggregates, the Modified Proctor data shows maximum dry densities that exceeded those of the Standard Proctor values by approximately 6%. The moisture-density curves show relatively flat compaction curves as shown in Figure 4.11. The relatively flat compaction curves suggest that, all of the RG samples were relatively insensitive to water content, thus indicating stable compaction characteristics over a wide range of moisture contents. This insensitivity to moisture content also indicates that RG can be placed in the field during wet weather keeping the construction downtime a minimum (CWC 1998).

Figure 4.11 shows that for RG similar to sands, the dry density is first decreasing to a certain value as moisture content is increasing and then the dry density is increasing to a maximum value with further increase of moisture content. The decrease of dry density obtained at lower moisture contents is a result of the effect of capillary tension in the pore water. The capillary tension resists the movement of soil particles at lower moisture contents and thus prevents the soil from becoming densely packed (Das 2006).

Standard and Modified Proctor compaction tests were also performed on the as-received crushed rock samples. Figure 4.12 shows the moisture-density relationships for the as-received CR samples. For comparison purposes, the zero-air-void curve is also shown on this figure. Table 4.2 summarizes the maximum dry density and optimum water content for the compaction tests. Similar to many natural aggregates, the Modified Proctor data show maximum dry densities that exceeded those of the Standard Proctor values by approximately 10% for CR. The moisture-density curves exhibited relatively flat compaction curves as shown in Figure 4.12. The relatively flat compaction curves suggest a stable compaction characteristic over a wide range of moisture contents.

Standard and Modified Proctor compaction tests were also performed on the RG/CR blends. Figure 4.13 shows the moisture-density relationships for the RG/CR blends. For comparison purposes, the zero-air-void curves are also shown on this figure. Table 4.5 summarizes the maximum dry density and optimum water content for the compaction tests. Figure 4.14 shows the value of maximum dry densities obtained for RG/CR
blends vary from 2.13 to 2.26 Mg/m$^3$, which is similar to the values found for the natural aggregate with the same soil classification (Craig 1997). The moisture-density curves show relatively flat compaction curves as shown in Figure 4.13. The relatively flat compaction curves suggest that, all of the RG/CR blends were relatively insensitive to water content, thus indicating stable compaction characteristics over a wide range of moisture contents. This insensitivity to moisture content also indicates that RG/CR blends can be placed in the field during wet weather keeping the construction downtime a minimum (CWC 1998). The optimum moisture contents of the RG/CR blends vary from 8 to 9.5% as shown in Figure 4.15.

Figure 4.11: Moisture-density relationships for the as-received RG
Figure 4.12: Moisture-density relationships for the as-received CR

Figure 4.13: Moisture-density relationships of RG/CR blends
4.2.6 California Bearing Ratio (CBR)

California bearing ratio is a common bearing capacity test for pavement materials. The samples were compacted in a cylindrical mould having an internal diameter of 152 mm. The blends were compacted into the mould in five layers with 53 blows per layer,
totalling an effective height of 117 mm by using a spacer disc inserted into the mould before compaction. Modified compactive effort was used here as modified CBR values as being used for pavement subbase. The blends were compacted at their optimum moisture content to achieve 98% to 100% maximum dry density. The prepared samples were tested in a compression loading machine after four-day soaked period with 4.5 kg surcharge to simulate the worst scenario.

The CBR value is a common parameter used in the design of flexible pavement. Typical CBR values of a compacted granular material vary from 40 to 80 (NYDOT 1995). The CBR value of the RG was 44. The test result indicates that the CBR value of the RG was within this typical range, but it alone does not satisfy the local state authority’s specification of CBR 80 for class 3 subbase materials.

VicRoads (2009) specifies the minimum CBR values for crushed 3 aggregates as 80. The CBR values for the class 3 CR was 181. So, it can be said the class 3 CR is much better than the local state authority’s specifications.

The CBR value of the RG/CR blends vary from 121 to 200% as shown in Figure 4.16. The CBR values of all the RG/CR blends were much higher than the Local state authority’s requirements of 80%. Therefore in can be concluded that the RG/CR blends will perform much better than the Class 3 materials.

![Figure 4.16: CBR of RG/CR blends](image-url)
4.2.7 pH value

pH value plays a significant role to characterize pore fluids of all geomaterials. The pH value of the RG was 9.6, which was similar to levels that are found naturally in geologic materials. According to the federal regulatory limit, a solid material is designated as hazardous waste when it contains a pH less than or equal to 2.0 or greater than or equal to 12.5 (CWC 1998). As the pH value of the RG was outside the limit designated as hazardous, it can be said that it will not produce any potential harm to anything.

The pH value of the CR was 9.4 and greater than 7 which shows that the CR is alkaline by nature. The pH value of the CR was also similar to levels that are found naturally in geomaterials. As the pH value of the CR was outside hazardous limit, it can be said that it is safe to use the crushed rock in road pavement applications.

The pH values of the RG/CR blends vary from 9.6 to 9.7 as shown in Figure 4.17. As the pH values of the RG/CR blends were outside the limit designated as hazardous, it can be said that it will not produce any potential harm to anything.

Figure 4.17: pH value of RG/CR blends
4.2.8 Fines Content

The fines content of the RG was 2.8% in its as-received condition and 4.2% after modified compactive effort. The fines were non-plastic with silty materials.

The fines content of the CR was 12% in its as-received condition and 13.2% after modified compactive effort. The fines content of the RG/CR blends vary from 8 to 11%. The fine contents of the RG/CR blends are presented in Figure 4.18. The fines were non-plastic with silty materials.

According to VicRoads (1995), the permitted fines content of Class 3 subbase produced from all rocks for before and after compaction should be 6-13% and 6-14%, respectively. The fines content of the crushed rock are within that typical range.

![Figure 4.18: Fine content of RG/CR blends](image)

4.2.9 Plasticity Index

The fines content in the RG were very low and the fines were non-plastic made out of silt size particles. Therefore, the plastic limit and liquid limit could not be obtained. The primary reason for this is the fact that the Atterberg limit is directly related to the clay mineralogy and as such, very low fines content with silty materials results in
immeasurable plasticity. This aspect suggest that some difficulties may be experienced with the workability of the RG as cohesion of particles and a “tight” prepared surface are usually sought after characteristics. The addition of small quantities of clayey sand or plastic crusher fines may also be a good solution to overcome this potential problem.

Similar to RG, the fines content in the CR were non-plastic made out of silt size particles, the liquid limit and plastic limit could not be determined.

The fine contents in the RG/CR blends were also non-plastic made out of silt size particles. Therefore, the plastic limit and liquid limit could not be obtained.

**4.2.10 Organic Content**

The organic content is also used as criteria for acceptability of materials for road bases and structural fill and degrades over time causing production of gases and result in settlement. The organic content of the as received recycled glass was 0.5% and it can be considered as negligible for road construction materials.

Soils with high percentages of organic content are often encountered during construction work which tends to decrease the maximum dry density of compaction and increase the compressibility of the soil. These tendencies are not desirable in the construction of foundations, embankments, etc. (Das 2006). The decaying of organic materials may cause potential settlement of the engineered fill (CWC 1997). The results of the organic content test indicate that the organic content value of CR was 0.8% which can be considered as negligible.

The organic content is also used as criteria for acceptability of materials for road bases and structural fill and degrades over time causing production of gases and result in settlement. The organic content of the RG/CR blends vary from 0.60 to 0.80% as shown in Figure 4.19, which can be considered negligible in case of road base materials.
4.2.11 Los Angeles Abrasion Loss

Various tests are generally undertaken to assess the durability and resistance of aggregate materials. The Los Angeles abrasion test (LA) is commonly used in highway and materials engineering to assess the abrasion resistance of aggregate materials (Wartman et al. 2004). The RG in its as-received condition had wear values of 27%.

The CR in its as-received condition had wear values of 24% and the RG/CR blends had wear values in the range of 23 to 25% as shown in Figure 4.20. Natural aggregates typically have wear values in the range of 10% to 35% (PennDOT 2001) and VicRoads (1997) specifies the maximum limit of LA as 35% for crushed aggregates. Therefore, the Los Angeles abrasion values of RG, CR and RG/CR blends are well within this typical range.
**4.2.12 Flakiness Index**

Aggregate particles are classified as flaky when they have the thickness (smaller dimension) of less than about 0.6 of their mean sieve size (BS 812-105.1-2000). The thin and flaky particles influence the strength of any materials in two ways. Firstly, they can reduce the strength of any structure when load is applied to the flat side of the aggregate or across its shortest dimension and secondly, they are also prone to segregation and breakdown during compaction, creating additional fines (Tam and Tam 2007). The VicRoads (2009) specifies the maximum limit of flakiness index as 35% for Class 1 and Class 2 crushed aggregates, but they didn’t mention their limit for Class 3 crushed aggregates. The flakiness index of the CR was 16% and was much lower than the acceptable limits even for Class 1 and Class 2 crushed aggregates (VicRoads 2009).

**4.2.13 Permeability**

The permeability of natural and processed materials is usually the most important property in hydrogeologic studies. In engineering practice, the permeability of a fill material often plays a significant role in material selection, particularly for applications related to drainage. For granular fill material, high permeability is usually more
beneficial than low. The exception to that may be for leachate treatment where a specific range of permeability may be required (CWC 1998).

Constant head permeability tests were performed on the as-received recycled glass sample. Each of the test specimens were compacted in a rigid wall permeameter to a dry density equal to 95% (±1%) of the maximum dry density based on the Modified Proctor compaction test. The specimens were compacted in to five layers using a rubber-tipped pestle.

The average permeability of triplicate samples was 3.3 x 10⁻⁵ m/s as shown in Table 4.2. All measured hydraulic conductivity values are within the typical range for compacted natural aggregates and soils with SW and GP designations (CWC 1998). The measured permeability values suggest that the recycled glass is a relatively free-draining material with good filtration and drainage characteristics.

4.2.14 Direct Shear Test

Direct shear tests (DST) were conducted according to the British Standard (BS 1377-7-1990) on specimens compacted inside the shear box with optimum water content to achieve the maximum dry density obtained in modified compaction tests. It is recommended that the size of the largest particle of the test specimen should not exceed one-tenth of the specimen height and consequently with the 10 cm by 10 cm shear box having the effective depth of 4 cm, only the RG samples of particle size smaller than 4.75 mm were tested. Five different normal stress levels were applied to the test samples. The internal friction angle of recycled glass source declined from 55º to 46º with normal stress increasing from 25 kPa to 400 kPa. The internal friction angle of recycled glass is found to be similar to that of dense sand with angular grains (Das 1983). This suggests that the recycled glass source exhibits the satisfactory friction characteristics for usage in some geotechnical engineering applications. The direct shear test results also indicate that the recycled glass possesses lack of cohesion resistance between particles. This is most probably the result of smooth surface of the glass particles and little amount of fine particles in the mixture. Figure 4.21 and Figure 4.22 show the stress and volume change behavior versus axial strain of compacted crushed glass in direct shear test. From Figure 4.21, it is observed that with increase in normal
stress, the shear stress also increase as we would expect from our knowledge of sliding friction (a concurrent increase in the shear stress on the failure plane at failure with the increase of the normal stress $\sigma_n$). From Figure 4.22, it is seen that at first there is a slight reduction in height or volume of the soil specimen, followed by a dilation or increase in height or volume. As the normal stress $\sigma_n$ increases, the harder it is for the soil to dilate during shear, which seems reasonable.

![Shear Stress vs Axial Strain](image)

Figure 4.21: Stress versus axial strain of compacted crushed glass in direct shear test

The large shear box with 30.5 cm by 30.5 cm having the effective depth of 20.3 cm was used for the direct shear tests of CR. Three different normal stress levels were applied to the test samples. The internal friction angle of the crushed rock was found to be 48.34° with normal stress ranging from 50 kPa to 200 kPa and the cohesion was 286.43 kPa. Figure 4.24 and Figure 4.25 show the stress and volume change behavior versus axial strain of compacted crushed glass in direct shear test. From Figure 4.24, it is observed that with increase in normal stress, the shear stress also increase as we would expect from our knowledge of sliding friction (a concurrent increase in the shear stress on the failure plane at failure with the increase of the normal stress $\sigma_n$). From Figure 4.25, it is seen that at first there is a slight reduction in height or volume of the soil specimen,
followed by a dilation or increase in height or volume. As the normal stress $\sigma_n$ increases, the harder it is for the soil to dilate during shear, which seems reasonable.

Figure 4.22: Volume change behavior versus axial strain of compacted RG in DST

Figure 4.23: Mohr-Coulomb failure envelope for RG in direct shear test
It was noted that the shear stresses for all three normal stresses increased with the horizontal displacement until the stresses reached the peak strengths. It was also observed that at higher normal stresses, the peak shear stresses also became higher and the horizontal displacement corresponding to the peak stresses became higher as well. All the three curves for the three selected normal stresses in Figure 4.24 revealed that after the peak stresses, shear stresses decrease with increase in horizontal displacement. These decreases of shear stresses after the peak stresses are normally regarded as strain softening which would be a good characteristic for road pavement materials. From Figure 4.25, it is seen that at first there is a slight reduction in height or volume of the soil specimen, followed by a dilation or increase in height or volume. As the normal stress $\sigma_n$ increases, the harder it is for the soil to dilate during shear, which seems reasonable. Figure 4.26 shows the Mohr-Coulomb failure envelope for compacted CR100 sample. From the Mohr-Coulomb failure envelope an internal friction angle of 48.34° and an apparent cohesion of 286.43 kPa were obtained as mentioned before.

Figure 4.24: Stress versus axial strain of compacted CR100 in direct shear test
4.2.15 Consolidated Drained (CD) Triaxial Compression Test

Consolidated drained (CD) triaxial compression tests were carried out for RG100, CR100 and RG/CR blends for determining the effective shear strength parameters (i.e.,
cohesion, \(c'\) and angle of internal friction, \(\phi'\). The selected effective confining stresses were 50 kPa, 100 kPa and 200 kPa on the specimens in each test.

**Deviator stress versus axial strain relationship**

Figures 4.27 to 4.34 show the relationship between the deviator stress (\(q\)) and the axial strain (\(\varepsilon_a\)) at the three different effective confining stress levels for CR100, RG10/CR90, RG15/CR85, RG20/CR80, RG30/CR70, RG40/CR60, RG50/CR50 and RG100 specimens, respectively. It was noticed that the deviator stress increases with axial strain until it reaches the peak strength. It was also noticed that at higher confining pressures, the peak strength became higher and the axial strain corresponding to the peak strength also became higher. All three curves reveal that after the peak strength, the deviator stress decreases with axial strain. This characteristic is normally described as strain softening.

![Deviator stress versus axial strain relationship for CR100](image)

Figure 4.27: Deviator stress versus axial strain relationship for CR100
Figure 4.28: Deviator stress versus axial strain relationship for RG10/CR90

Figure 4.29: Deviator stress versus axial strain relationship for RG15/CR85
Figure 4.30: Deviator stress versus axial strain relationship for RG20/CR80

Figure 4.31: Deviator stress versus axial strain relationship for RG30/CR70
Figure 4.32: Deviator stress versus axial strain relationship for RG40/CR60

Figure 4.33: Deviator stress versus axial strain relationship for RG50/CR50
Volume change versus axial strain relationship

The variation of the volumetric strain with axial strain from a series of CD triaxial compression tests under different confining stresses for CR100, RG10/CR90, RG15/CR85, RG20/CR80, RG30/CR70, RG40/CR60, RG50/CR50 and RG100 specimens, respectively are shown in Figures 4.35 to 4.42. The positive volumetric strain indicates the contractive behaviour and the negative volumetric strain indicates the dilative behaviour of the samples. All the samples show similar volumetric strain characteristics. The samples compressed in the initial stage of the shearing and then started to dilate. Generally, the dilation of samples during shearing increases with increased axial strain. It can be also seen that the higher the confining stresses, lower the volumetric strain at the same axial strain. The volumetric strain characteristics of the specimens are similar to the dense cohesionless soil subjected to similar test conditions.
Figure 4.35: Volumetric strain versus axial strain relationship for CR100

Figure 4.36: Volumetric strain versus axial strain relationship for RG10/CR90
Figure 4.37: Volumetric strain versus axial strain relationship for RG15/CR85

Figure 4.38: Volumetric strain versus axial strain relationship for RG20/CR80
Figure 4.39: Volumetric strain versus axial strain relationship for RG30/CR70

Figure 4.40: Volumetric strain versus axial strain relationship for RG40/CR60
Figure 4.41: Volumetric strain versus axial strain relationship for RG50/CR50

Figure 4.42: Volumetric strain versus axial strain relationship for RG100
MIT stress field

Figures 4.43 to 4.50 show the effective stress path envelopes for CR100, RG10/CR90, RG15/CR85, RG20/CR80, RG30/CR70, RG40/CR60, RG50/CR50 and RG100 specimens, respectively on MIT stress field under three different effective confining stresses. The stress paths of the samples are drawn on s’-t space. In the MIT stress field, s’ and t are defined in terms of the principal stresses as follows (Head 1994):

\[ s' = \frac{\sigma'_1 + \sigma'_3}{2} \]  
\[ t = \frac{\sigma_1 - \sigma_3}{2} \]

Equation 4.1  
Equation 4.2

In the consolidated drained triaxial compression tests, the effective confining stress, \( \sigma'_3 \) remains constant and the effective principal stress, \( \sigma'_1 \) increases to peak failure stress and then drops as the test progress. The stress path follows a straight line with an angle of 45° with the horizontal, i.e., the slope of the stress path is 1 on 1. The results show that the failure envelope corresponding to the peak stress is linear for the tested stress ranges of 50 kPa, 100 kPa and 200 kPa.

Figure 4.43: Stress path in s’-t space for CR100
Figure 4.44: Stress path in $s'$-t space for RG10/CR90

Figure 4.45: Stress path in $s'$-t space for RG15/CR85
Figure 4.46: Stress path in $s'$-$t$ space for RG20/CR80

Figure 4.47: Stress path in $s'$-$t$ space for RG30/CR70
Figure 4.48: Stress path in $s'-t$ space for RG40/CR60

Figure 4.49: Stress path in $s'-t$ space for RG50/CR50
Cambridge stress field

Figures 4.51 to 4.58 show the effective stress path envelopes for CR100, RG10/CR90, RG15/CR85, RG20/CR80, RG30/CR70, RG40/CR60, RG50/CR50 and RG100 specimens, respectively on Cambridge stress field under different effective confining stresses. In the Cambridge stress field, the deviator stress, $q$, is plotted against the mean effective applied stress, $p'$, in which the parameter $p'$ is defined in terms of effective stress as follows (Head 1994):

$$ p' = \frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3} \quad \text{Equation 4.3} $$

The parameter $q$ is defined as follows as being equal to the deviator stress:

$$ q = \sigma'_1 - \sigma'_3 = \sigma_1 - \sigma_3 \quad \text{Equation 4.4} $$

In the triaxial test two of the principal effective stresses are equal to the horizontal effective stress, and Equation 4.3 is expressed as follows:

$$ p' = \frac{\sigma'_1 + 2\sigma'_3}{3} \quad \text{Equation 4.5} $$
As mentioned earlier, in the consolidated drained triaxial compression tests, the effective confining stress, $\sigma_3'$ remains constant and the effective principal stress, $\sigma_1'$ increases to peak failure stress and then drops as the test progress. Here, in the Cambridge stress field, the stress path also follows a straight line inclined at a slope of $\frac{3}{1}$. The results show that the failure envelope corresponding to the peak stress is linear for the tested stress ranges.

![Stress path in p'q space for CR100](image1.png)

**Figure 4.51: Stress path in p'-q space for CR100**

![Stress path in p'q space for RG10/CR90](image2.png)

**Figure 4.52: Stress path in p'-q space for RG10/CR90**
Figure 4.53: Stress path in $p'$-q space for RG15/CR85

Figure 4.54: Stress path in $p'$-q space for RG20/CR80
Figure 4.55: Stress path in \( p' - q \) space for RG30/CR70

Figure 4.56: Stress path in \( p' - q \) space for RG40/CR60
Figure 4.57: Stress path in p’-q space for RG50/CR50

Figure 4.58: Stress path in p’-q space for RG100

Shear strength parameters (For Mohr’s circles)

The Mohr’s circles and Mohr-Coulomb failure envelope under consolidated drained (CD) triaxial compression tests for CR100, RG10/CR90, RG15/CR85, RG20/CR80,
RG30/CR70, RG40/CR60, RG50/CR50 and RG100 specimens, respectively are shown in Figures 4.59 to 4.66. Table 4.6 shows the shear strength parameters of RG/CR blends.

Table 4.6: The shear strength parameters of RG/CR blends

<table>
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<tr>
<th>Blends</th>
<th>Effective cohesion, c’ (kPa)</th>
<th>Effective angle of internal friction, φ’ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CR100</td>
<td>69.6</td>
<td>48.3</td>
</tr>
<tr>
<td>RG10/CR90</td>
<td>32.8</td>
<td>50.3</td>
</tr>
<tr>
<td>RG15/CR85</td>
<td>59.3</td>
<td>47.2</td>
</tr>
<tr>
<td>RG20/CR80</td>
<td>42.6</td>
<td>49.3</td>
</tr>
<tr>
<td>RG30/CR70</td>
<td>42.6</td>
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</tr>
<tr>
<td>RG40/CR60</td>
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</tr>
<tr>
<td>RG50/CR50</td>
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</tr>
<tr>
<td>RG100</td>
<td>5.0</td>
<td>41.7</td>
</tr>
</tbody>
</table>

As presented in Table 4.6, it has been noted that shear strength parameters of RG/CR blends are similar to the cohesionless materials. The internal friction angle varies from 41.7˚ to 50.3˚ and the cohesion ranges between 5 kPa to 69.6 kPa. Generally, all blends attain high shear strength parameters. Although the CR100 and RG/CR blends are considered as cohesionless frictional material, it deviates from purely frictional behaviour. It might be due to the effect of confining stress. At higher confining stresses, particle became flattened at their contact points, sharp corners are crushed and the interlocking are also reduced (Aathee san 2011). In addition, actual curvature of Mohr-Coulomb envelope is highest for dense granular soils (Lambe and Whitman 1979) and generally considered as straight line. The minor cohesion value for RG as shown in Table 4.6 might be due to particle adhesion resulting from label glues, residual sugars or other cohesive substances present in the samples (Ali and Arulrajah 2012, Wartman et al. 2004).
Figure 4.59: Mohr’s circles and Mohr-Coulomb failure envelope for CR100

Figure 4.60: Mohr’s circles and Mohr-Coulomb failure envelope for RG10/CR90
Figure 4.61: Mohr’s circles and Mohr-Coulomb failure envelope for RG15/CR85

Figure 4.62: Mohr’s circles and Mohr-Coulomb failure envelope for RG20/CR80
Figure 4.63: Mohr’s circles and Mohr-Coulomb failure envelope for RG30/CR70

Figure 4.64: Mohr’s circles and Mohr-Coulomb failure envelope for RG40/CR60
4.2.16 Repeated Load Triaxial (RLT) Test

In the repeated load triaxial test, the permanent strain parameter characterizes the vertical permanent strain with multiple loading stages at different stress conditions. In
this research, three different loading stages at specified deviator stresses of 250 kPa, 350 kPa and 450 kPa were used with each loading stage involving 10,000 repetitions. The resilient modulus characterizes the vertical resilient strain response over sixty six stress conditions using combinations of applied dynamic vertical and static confining stresses in the ranges of 100-600 kPa and 20-150 kPa, respectively. Each stress condition involved at least 50 load repetitions at the stress condition of specified repeated deviator stress and static confining stress. Table 4.7 shows the permanent strain and resilient modulus values obtained for 3 moisture levels for RG/CR blends. The results of the RLT tests presented in Table 4.7 indicate that permanent strains are sensitive to moisture content between 60% and 90% modified optimum moisture content. The lower moisture content results in a smaller permanent deformation value. From Table 4.7, it can be concluded that generally a higher RG content could potentially produce higher permanent strains. However, this trend is not consistent through the results especially for stage 1 and this might be due to the variation of moisture ratio which makes it difficult to establish a trend for permanent strain (Ali et al. 2011). On the other hand, the resilient modulus was not sensitive to changes in either moisture ratio or RG content. The performance of the RG/CR blends in RLT tests in this research is comparable to those of natural granular subbase reported in Table 4.7.

Figure 4.67 shows the variation of permanent strain and resilient modulus characteristics with different moisture ratio and deviator stress of RG10/CR90 blends. As shown in Figure 4.67, it was noted that the permanent strain increases with increase in moisture ratio, but the resilient modulus was not sensitive to changes in moisture ratio, the RG10/CR90 blends did not show any significant increase or decrease of resilient modulus with increase in moisture ratio.
Figure 4.67: Results of permanent strain and resilient modulus testing for RG10/CR90

Figure 4.68 shows the variation of permanent strain and resilient modulus characteristics with different moisture ratio and deviator stress of RG15/CR85 blends. As shown in Figure 4.68, it was noted that the permanent strain increases with increase in moisture ratio, but the resilient modulus was not sensitive to changes in moisture ratio as compared to permanent strain, the RG15/CR85 blends did not show any significant increase or decrease of resilient modulus with increase in moisture ratio. The
specimen at 70% OMC was loaded from 350 kPa deviator stress, therefore, it experienced two loading stages and was presented from loading stage 2.

![Graph showing permanent strain and resilient modulus testing for RG15/CR85 blends](image)

Figure 4.68: Results of permanent strain and resilient modulus testing for RG15/CR85

Figure 4.69 shows the variation of permanent strain and resilient modulus characteristics with different moisture ratio and deviator stress of RG20/CR80 blends.
As shown in Figure 4.69, it was noted that the permanent strain increases with increase in moisture ratio, but the resilient modulus shows opposite trend with respect to moisture ratio for 67% and 57% moisture ratio although the resilient modulus for 67% moisture ratio was higher than that of 72% moisture ratio. The resilient modulus for these two specimens (57% and 67% moisture ratio) might be due to the fact that the initial modulus was not to the desired level as expected after the permanent strain test.

Figure 4.69: Results of permanent strain and resilient modulus testing for RG20/CR80
Figure 4.70 shows the variation of permanent strain and resilient modulus characteristics with different moisture ratio and deviator stress of RG30/CR70 blends. As shown in Figure 4.70, it was noted that the permanent strain increases with increase in moisture content, but the resilient modulus was not sensitive to changes in moisture ratio, the RG30/CR70 blends did not show any significant increase or decrease of resilient modulus with increase in moisture ratio.
Table 4.7: RLT results for RG/CR blends

<table>
<thead>
<tr>
<th>Blend</th>
<th>% MDD</th>
<th>% W&lt;sub&gt;opt&lt;/sub&gt;</th>
<th>Permanent strain (×10&lt;sup&gt;3&lt;/sup&gt; με)</th>
<th>Resilient modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Stage 1</td>
<td>Stage 2</td>
</tr>
<tr>
<td>RG10/CR90</td>
<td>98.1</td>
<td>86.5</td>
<td>22.0</td>
<td>F&lt;sup&gt;1&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td>97.7</td>
<td>81.6</td>
<td>7.5</td>
<td>17.2</td>
</tr>
<tr>
<td></td>
<td>97.5</td>
<td>67.0</td>
<td>4.9</td>
<td>6.2</td>
</tr>
<tr>
<td>RG15/CR85</td>
<td>99.2</td>
<td>72.5</td>
<td>18.1</td>
<td>F</td>
</tr>
<tr>
<td></td>
<td>98.3</td>
<td>70.5</td>
<td>8.6</td>
<td>18.0</td>
</tr>
<tr>
<td></td>
<td>97.9</td>
<td>60.9</td>
<td>4.5</td>
<td>8.3</td>
</tr>
<tr>
<td>RG20/CR80</td>
<td>98.9</td>
<td>72.4</td>
<td>14.1</td>
<td>F</td>
</tr>
<tr>
<td></td>
<td>98.4</td>
<td>67.0</td>
<td>10.6</td>
<td>16.5</td>
</tr>
<tr>
<td></td>
<td>98.2</td>
<td>57.3</td>
<td>7.2</td>
<td>12.6</td>
</tr>
<tr>
<td>RG30/CR70</td>
<td>98.6</td>
<td>76.3</td>
<td>9.5</td>
<td>F</td>
</tr>
<tr>
<td></td>
<td>98.5</td>
<td>67.0</td>
<td>8.8</td>
<td>17.4</td>
</tr>
<tr>
<td></td>
<td>98.0</td>
<td>58.6</td>
<td>7.4</td>
<td>12.6</td>
</tr>
<tr>
<td>Natural granular subbase&lt;sup&gt;3&lt;/sup&gt;</td>
<td>98.0</td>
<td>70.0</td>
<td>3-10</td>
<td>4-15</td>
</tr>
<tr>
<td></td>
<td>98.0</td>
<td>80.0</td>
<td>5-10</td>
<td>7-15</td>
</tr>
<tr>
<td></td>
<td>98.0</td>
<td>90.0</td>
<td>7-15</td>
<td>10-F</td>
</tr>
</tbody>
</table>

<sup>1</sup> The sample failed during the test before reaching next stage  
<sup>2</sup> Not recorded as the sample failed before this stage  
<sup>3</sup> ARRB 2010

Finally, it can be concluded that the results of the laboratory RLT testing on the RG/CR blends indicated that permanent strains are sensitive to moisture content (in the range of 60-90% modified OMC) and a higher content of RG could potentially produce higher permanent strains. However, resilient modulus was not sensitive to both moisture and glass additive content. The performances (in terms of both permanent deformation and resilient modulus) of the RG/CR blends tested are comparable to those of natural CR subbases.
4.3 Findings of the geotechnical properties of recycled glass in blends with crushed rock

The following conclusions can be drawn from the results of RG/CR blends:

- The results of the laboratory tests show that blending up to 50% RG results in low to minimal effect on the physical and mechanical properties of CR, the original material. As such, the RG/CR blends were demonstrated to meet the current VicRoads requirements.

- The research indicates that up to 15% of 4.75 mm RG can be safely added to Class 3 CR. The degree of breakdown occurring in the RG/CR blend is on the limit of what is acceptable for Class 3 pavement subbase material.

- Depending on the results of future field trials, it may be possible to increase the percentage of RG in the mixture.

- All RG/CR blends tested for CBR and LA abrasion values passed VicRoads requirements for Class 3 pavement subbase material.

- The RG/CR blends showed good shear strength in CD triaxial compression tests.

- RLT testing on the blends indicate that permanent strains are sensitive to moisture content and a higher content of glass additive could potentially produce higher permanent strains. However, the resilient modulus was not sensitive to either changes in moisture or glass additive content.

- The performance in terms of both the permanent deformation and the resilient modulus of the RG/CR blend, with up to 15% RG content, is therefore comparable to those of natural granular subbases.
CHAPTER FIVE

5 GEOTECHNICAL PROPERTIES OF RECYCLED GLASS AND RECYCLED CRUSHED CONCRETE BLENDS

5.1 Introduction

The geotechnical properties of recycled glass (RG) blended with recycled crushed concrete (RCC) were evaluated using different geotechnical tests. The laboratory tests on RG/RCC blends were conducted on mixtures with 10%, 15%, 20%, 30%, 40% and 50% RG (by mass) with Class 3 RCC. These blends are referred to as RG10/RCC90, RG15/RCC85, RG20/RCC80, RG30/RCC70, RG40/RCC60 and RG50/RCC50, respectively. Here, RG blended with RCC is denoted by RGXX/RCCYY. The suffix XX after RG represents the percentages of RG and the suffix YY after RCC represents the percentages of RCC in the mixtures by mass, respectively. The tests were also conducted on 100% RCC for comparison and regarded as RCC100.

5.2 Test results

5.2.1 Particle size distribution

Sieve analyses were conducted on the as-received RCC and RG/RCC blends to determine their grain size distribution curve. Additional sieve analyses were also conducted on compacted samples of as-received RCC and RG/RCC blends from the Modified Proctor test to assess the compaction induced particle breakage that may shift the grain-size distribution of the materials. These samples are referred to as “after compacted”. Table 5.1 shows the basic properties of as-received RCC samples. Under modified compactive effort, a small portion of the gravel and sand size particles were crushed and reduced to fine size particles as shown in Table 5.1. The percentages of gravel size particles in the mixture decreased from 58.0 for the as-received sample to 55.7 for the after compacted sample. The percentages of sand size particles in the mixture increased from 35.0 for the as-received sample to 35.9 for the after compacted sample. The change in particle size distribution of RCC after modified compaction test
is due to the crushing of gravel and sand size particles under compaction energy and subsequent decrease in their size to mainly finer particles. Overall, the results showed negligible change in the particle size distribution curve of the RCC samples before and after modified compaction. This indicates that the RCC materials are stable mixtures in engineering operations including handling, spreading and especially compaction.

Figure 5.1 presents the grain size distribution curves for the as-received and after compacted RCC samples. Table 5.2 demonstrates the results of other tests performed on as-received RCC samples.

Table 5.3 and Table 5.4 summarise the grain size distribution of before compaction and after compaction, respectively for recycled glass blended with RCC (Class 3) sourced from Alex Fraser Recycling, Laverton. The gradation curves of RG/RCC blends before compaction as compared with local state authority’s requirements are shown in Figure 5.2. Figure 5.3 shows the grain size distribution curves of RG/RCC blends after modified compaction along with local state authority’s requirements. Table 5.5 summarise the engineering properties of RG blended with RCC (Class 3).

The results of before compaction grading curves indicated that RCC blends with greater than 40% and 50% recycled glass additive would potentially produce grading exceeding the local state authority’s upper limits. The difference in the trends of the curves would be due to slight variations in the constitution of the samples.

The results of after compaction grading curves indicated that RCC blends with greater than 50% recycled glass additive would potentially produce grading exceeding the local state authority’s upper limits.

Overall, the before and after compacted grain size distribution curves of the samples of RG/RCC blends indicate that the materials appear to be remaining reasonably well graded through the compaction process and this will generally aid the compaction process.
### Table 5.1: Basic properties of as-received samples based on AS 1726-1993 Standard

<table>
<thead>
<tr>
<th>Resource</th>
<th>Classification</th>
<th>Maximum particle size</th>
<th>Coefficient of uniformity (Cu)</th>
<th>Coefficient of curvature (Cc)</th>
<th>Fine Content (&lt; 0.075 mm)</th>
<th>Gravel Content (&gt;2.36 mm) (%)</th>
<th>Sand Content (0.075-2.36 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RCC</td>
<td>GW-GM</td>
<td>20.0</td>
<td>47</td>
<td>1.0</td>
<td>7.0</td>
<td>58.0</td>
<td>35.0</td>
</tr>
<tr>
<td>RCC-After Compaction</td>
<td>GW-GM</td>
<td>20.0</td>
<td>54</td>
<td>1.0</td>
<td>8.4</td>
<td>55.7</td>
<td>35.9</td>
</tr>
</tbody>
</table>

### Table 5.2: Geotechnical properties of 100% RCC

<table>
<thead>
<tr>
<th>Test</th>
<th>Standard method</th>
<th>RCC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity (G_s)</td>
<td>AS 1141.5 and AS 1141.6.1</td>
<td>2.72</td>
</tr>
<tr>
<td>Organic content (%)</td>
<td>ASTM D 2947-00</td>
<td>3.0</td>
</tr>
<tr>
<td>Debris level (visual method) (%)</td>
<td>AGI 23.1 &amp; 23.2</td>
<td>&lt; 2</td>
</tr>
<tr>
<td>Debris level (weight method) (%)</td>
<td>CWC Method</td>
<td>&lt; 0.2</td>
</tr>
<tr>
<td>pH value</td>
<td>AS 1289.4.3.1</td>
<td>11.3</td>
</tr>
<tr>
<td>Flakiness index</td>
<td>BS 812-105.1</td>
<td>11</td>
</tr>
<tr>
<td>LA abrasion loss (%)</td>
<td>ASTM C 131-06</td>
<td>28</td>
</tr>
<tr>
<td>Standard proctor</td>
<td>AS 1289.5.1.1</td>
<td></td>
</tr>
<tr>
<td>$\gamma_d$ max (Mg/m$^3$)</td>
<td></td>
<td>1.90</td>
</tr>
<tr>
<td>$w_{opt}$ (%)</td>
<td></td>
<td>14.3</td>
</tr>
<tr>
<td>Modified proctor</td>
<td>AS 1289.5.2.1</td>
<td></td>
</tr>
<tr>
<td>$\gamma_d$ max (Mg/m$^3$)</td>
<td></td>
<td>1.98</td>
</tr>
<tr>
<td>$w_{opt}$ (%)</td>
<td></td>
<td>13.8</td>
</tr>
<tr>
<td>California Bearing Ratio (CBR)</td>
<td>AS 1289.6.1.1</td>
<td>211</td>
</tr>
<tr>
<td>Hydraulic conductivity (m/s)</td>
<td>AS 1289.6.7.2</td>
<td>7.0E-08</td>
</tr>
<tr>
<td>Direct Shear Test (DST)</td>
<td>BS 1377-7</td>
<td></td>
</tr>
<tr>
<td>Internal friction angle (°)</td>
<td></td>
<td>44.63</td>
</tr>
<tr>
<td>Cohesion (kPa)</td>
<td></td>
<td>154.30</td>
</tr>
<tr>
<td>CD Triaxial Test</td>
<td>ASTM D 4767-04</td>
<td></td>
</tr>
<tr>
<td>Effec. internal friction angle (°)</td>
<td></td>
<td>51.0</td>
</tr>
<tr>
<td>Effec. cohesion (kPa)</td>
<td></td>
<td>75.4</td>
</tr>
</tbody>
</table>
Figure 5.1: Grain size distribution curves for the as-received and after compacted RCC samples

Table 5.3: Particle size distribution (before compaction)

<table>
<thead>
<tr>
<th>Sample Description</th>
<th>RG50/ RCC50</th>
<th>RG40/ RCC60</th>
<th>RG30/ RCC70</th>
<th>RG20/ RCC80</th>
<th>RG15/ RCC85</th>
<th>RG10/ RCC90</th>
</tr>
</thead>
<tbody>
<tr>
<td>RG (%) by mass</td>
<td>50</td>
<td>40</td>
<td>30</td>
<td>20</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>Particle size (mm)</td>
<td>Percentage of total passing (%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>26.5</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>19.0</td>
<td>99.7</td>
<td>99.7</td>
<td>99.4</td>
<td>100.0</td>
<td>100.0</td>
<td>99.4</td>
</tr>
<tr>
<td>13.2</td>
<td>92.3</td>
<td>92.0</td>
<td>91.1</td>
<td>90.0</td>
<td>88.2</td>
<td>92.2</td>
</tr>
<tr>
<td>9.5</td>
<td>87.1</td>
<td>85.8</td>
<td>82.6</td>
<td>78.4</td>
<td>76.2</td>
<td>83.2</td>
</tr>
<tr>
<td>4.75</td>
<td>76.0</td>
<td>73.7</td>
<td>67.4</td>
<td>59.7</td>
<td>58.2</td>
<td>64.6</td>
</tr>
<tr>
<td>2.36</td>
<td>56.8</td>
<td>54.6</td>
<td>50.6</td>
<td>44.4</td>
<td>43.7</td>
<td>50.1</td>
</tr>
<tr>
<td>0.075</td>
<td>5.8</td>
<td>6.6</td>
<td>6.5</td>
<td>6.9</td>
<td>6.8</td>
<td>8.6</td>
</tr>
</tbody>
</table>
Table 5.4: Particle size distribution (after compaction)

<table>
<thead>
<tr>
<th>Sample Description</th>
<th>RG50/RCC50</th>
<th>RG40/RCC60</th>
<th>RG30/RCC70</th>
<th>RG20/RCC80</th>
<th>RG15/RCC85</th>
<th>RG10/RCC90</th>
</tr>
</thead>
<tbody>
<tr>
<td>RG (%) by mass</td>
<td>50</td>
<td>40</td>
<td>30</td>
<td>20</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>Particle size (mm)</td>
<td>Percentage of total passing (%)</td>
<td>26.5</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td></td>
<td>19.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td></td>
<td>13.2</td>
<td>94.8</td>
<td>95.4</td>
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<td>8.2</td>
<td>8.5</td>
<td>8.7</td>
<td>8.9</td>
<td>10.0</td>
</tr>
</tbody>
</table>

Figure 5.2. Gradation curves of RG/RCC blends (before compaction)
Figure 5.3. Gradation curves of RG/RCC blends (after compaction)

Table 5.5: Engineering properties of RG blended with RCC (Class 3)

<table>
<thead>
<tr>
<th>Sample Description</th>
<th>RG50/RCC50</th>
<th>RG40/RCC60</th>
<th>RG30/RCC70</th>
<th>RG20/RCC80</th>
<th>RG15/RCC85</th>
<th>RG10/RCC90</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glass Content (%) by weight</td>
<td>50</td>
<td>40</td>
<td>30</td>
<td>20</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>Test description</td>
<td>Test results</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Particle density (Coarse) (Mg/m$^3$)</td>
<td>2.71</td>
<td>2.72</td>
<td>2.71</td>
<td>2.72</td>
<td>2.72</td>
<td>2.72</td>
</tr>
<tr>
<td>Particle density (Fine) (Mg/m$^3$)</td>
<td>2.50</td>
<td>2.60</td>
<td>2.61</td>
<td>2.61</td>
<td>2.63</td>
<td>2.65</td>
</tr>
<tr>
<td>Water absorption (Coarse) (%)</td>
<td>4.30</td>
<td>4.40</td>
<td>4.50</td>
<td>4.50</td>
<td>4.45</td>
<td>4.50</td>
</tr>
<tr>
<td>Water absorption (Fine) (%)</td>
<td>4.40</td>
<td>4.50</td>
<td>5.00</td>
<td>6.00</td>
<td>7.00</td>
<td>8.20</td>
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<tr>
<td>CBR (%)</td>
<td>98</td>
<td>110</td>
<td>120</td>
<td>144</td>
<td>176</td>
<td>203</td>
</tr>
<tr>
<td>Los Angeles abrasion loss (%)</td>
<td>30</td>
<td>28</td>
<td>30</td>
<td>31</td>
<td>32</td>
<td>32</td>
</tr>
<tr>
<td>Organic content (%)</td>
<td>1.4</td>
<td>1.5</td>
<td>1.7</td>
<td>2.0</td>
<td>2.7</td>
<td>2.8</td>
</tr>
<tr>
<td>pH value</td>
<td>11.2</td>
<td>11.2</td>
<td>11.3</td>
<td>11.2</td>
<td>11.1</td>
<td>11.2</td>
</tr>
<tr>
<td>Modified Compaction MDD (Mg/m$^3$)</td>
<td>1.90</td>
<td>1.97</td>
<td>1.95</td>
<td>1.98</td>
<td>1.97</td>
<td>1.98</td>
</tr>
<tr>
<td>Modified Compaction OMC (%)</td>
<td>11.2</td>
<td>11.7</td>
<td>10.5</td>
<td>11.5</td>
<td>11.3</td>
<td>12.1</td>
</tr>
<tr>
<td>Fines content (%)</td>
<td>5.8</td>
<td>6.6</td>
<td>6.5</td>
<td>6.9</td>
<td>6.8</td>
<td>8.6</td>
</tr>
</tbody>
</table>
5.2.2 Particle Density

Particle density, a measure of material’s specific gravity, affects the dry, partially-saturated, and saturated unit weights of porous media and is a widely used parameter in establishing the density-volume relationship of a soil mass. The specific gravity of most soils lies between 2.60 and 2.80, but the presence of particles consisting of other minerals will result in different value (Head 1994). The specific gravities of the crushed rock and gravelly sand ranged from 2.60 to 2.83 (CWC 1998).

Since density relates directly to engineering properties such as compaction and shear strength, specific gravity is an important baseline property. Particle density test was performed in accordance with AS 1141.6.1 and AS 1141.5. The particle density of fine blended aggregates passing 4.75 mm sieve varies from 2.50 to 2.65 Mg/m$^3$ and that of coarse blended aggregates passing 19 mm and retained on 4.75 mm sieve varies from 2.71 to 2.72 Mg/m$^3$ as shown in Figure 5.4 and Figure 5.5, respectively. The weighted average particle densities of the RG/RCC blends vary from 2.55 to 2.65 Mg/m$^3$ as shown in Figure 5.6. The specific gravity test results suggest that RG/RCC blends possess a specific gravity value similar to those of natural aggregates (Craig 1997).

![Figure 5.4: Particle density of fine RG/RCC blends](image)

Table 5.4: Particle density of fine RG/RCC blends

<table>
<thead>
<tr>
<th>Sample description</th>
<th>Particle density (Mg/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RG50/RCC50</td>
<td>2.50</td>
</tr>
<tr>
<td>RG40/RCC60</td>
<td>2.55</td>
</tr>
<tr>
<td>RG30/RCC70</td>
<td>2.60</td>
</tr>
<tr>
<td>RG20/RCC80</td>
<td>2.65</td>
</tr>
<tr>
<td>RG15/RCC85</td>
<td>2.71</td>
</tr>
<tr>
<td>RG10/RCC90</td>
<td>2.72</td>
</tr>
</tbody>
</table>
5.2.3 Water Absorption

Water absorption test was performed in accordance with AS 1141.6.1 and AS 1141.5. The water absorption of coarse blended aggregates passing 19 mm and retained on 4.75 mm sieve varies from 4.30 to 4.50% and that of fine blended aggregates passing 4.75 mm sieve varies between 4.40 and 8.20% as shown in Figure 5.7 and Figure 5.8,
respectively. The weighted average water absorption of the RG/RCC blends varies between 4 and 7% as shown in Figure 5.9.

Figure 5.7: Water absorption of coarse RG/RCC blends

Figure 5.8: Water absorption of fine RG/RCC blends
5.2.4 Standard and Modified Compaction

Standard and Modified Proctor compaction tests were performed on the as received RCC and modified compaction tests were performed on RG/RCC blends.

Figure 5.10 shows the moisture-density relationships for the as-received RCC. For comparison purposes, the zero-air-void curve is also shown in this figure. Table 5.2 summarizes the maximum dry density and optimum water content for the compaction tests. Table 5.2 shows that the value of maximum dry density obtained for RCC sample is similar to the values found for the natural aggregate with the same soil classification (Craig 1997). Similar to many natural aggregates, the Modified Proctor data shows maximum dry densities that exceeded those of the Standard Proctor values by approximately 10%. The moisture-density curves show relatively flat compaction curves as shown in Figure 5.10. The relatively flat compaction curves suggest that, all of the RCC samples were relatively insensitive to water content, thus indicating stable compaction characteristics over a wide range of moisture contents. This insensitivity to moisture content also indicates that RCC can be placed in the field during wet weather keeping the construction downtime a minimum (CWC 1998).
Figure 5.11 shows the moisture-density relationships for the RG/RCC blends. For comparison purposes, the zero-air-void curves are also shown on this figure. Table 5.5 summarizes the maximum dry density and optimum water content for the compaction tests. Figure 5.12 shows the value of maximum dry densities obtained for RG/RCC blends vary from 1.90 to 1.98 Mg/m$^3$, which is similar to the values found for the natural aggregate with the same soil classification (Craig 1997). The moisture-density curves show relatively flat compaction curves as shown in Figure 5.11. The relatively flat compaction curves suggest that, all of the RG/RCC blends were relatively insensitive to water content, thus indicating stable compaction characteristics over a wide range of moisture contents. This insensitivity to moisture content also indicates that RG/RCC blends can also be placed in the field during wet weather keeping the construction downtime a minimum (CWC 1998). The optimum moisture contents of the RG/RCC blends vary from 10 to 12.5% as shown in Figure 5.13.

![Figure 5.10: Moisture-density relationships for the as-received RCC](image-url)
Figure 5.11: Moisture-density relationships of RG/RCC blends

Figure 5.12: Maximum dry density of RG/RCC blends
CBR tests were conducted on the as-received RCC and RG/RCC blends. The samples were compacted in a cylindrical mould having an internal diameter of 152 mm. The blends were compacted into the mould in five layers with 53 blows per layer, totalling an effective height of 117 mm by using a spacer disc inserted into the mould before compaction. Modified compactive effort was used here as modified CBR values as being used for pavement subbase. The blends were compacted at their optimum moisture content to achieve 98% to 100% maximum dry density. The prepared samples were tested in a compression loading machine after four-day soaked period with 4.5 kg surcharge to simulate the worst scenario.

The CBR value is a common parameter used in the design of flexible pavement. Typical CBR values of a compacted granular material vary from 40 to 80 (NYDOT 1995). VicRoads (1997) specifies the minimum CBR values for Class 3 aggregates as 80. The CBR value for the as-received Class 3 RCC was 211. So, it can be said the Class 3 RCC are much better than the VicRoads specifications.
The CBR values of the RG/RCC blends vary from 98 to 203% as shown in Figure 5.14. The CBR values of all the RG/RCC blends were much higher than the VicRoads requirements of 80%. Therefore, it can be concluded that the RG/RCC blends will perform much better than the Class 3 materials.

5.2.6 Los Angeles (LA) abrasion value

The RCC in their as-received condition had wear values of 28%. The RG/RCC blends had wear values in the range of 28 to 32% as shown in Figure 5.15. Natural aggregates typically have wear values in the range of 10% to 35% (PennDOT 2001), and VicRoads (1997) specifies the maximum limit of LA as 35% for crushed aggregates. Therefore, the as-received RCC and RG/RCC blends have wear values well within this typical range.

![Figure 5.14: CBR of RG/RCC blends](image)

5.2.7 pH

pH plays a significant role to characterize pore fluids of all geomaterials. The pH value of the as-received RCC was 11.3 and the pH values of the RG/RCC blends vary from 11.1 to 11.3 as shown in Figure 5.16. The pH value of the as-received RCC and RG/RCC blends are greater than 7 which indicate that they are alkaline by nature. The
pH value of the as-received RCC and RG/RCC blends were similar to levels that are found naturally in geologic materials. According to the federal regulatory limit, a solid material is designated as hazardous waste when it contains a pH value less than or equal to 2.0 or greater than or equal to 12.5 (CWC 1998). As the pH value of the as-received RCC and RG/RCC blends were outside the hazardous limit, it can be said that it is safe to use them in road pavement applications and it will not produce any potential harm to any water bodies.

Figure 5.15: Los Angeles abrasion value of RG/RCC blends
5.2.8 Organic Content

The results of the organic content test indicate that the organic content value of the as-received RCC was 3%. The organic content of the RCC was more than that of CR, most probably due to increased amount of debris in the RCC samples. The organic content of the RG/RCC blends vary from 1 to 3% as shown in Figure 5.17, which can be considered negligible for road base materials.

5.2.9 Fines Content

The fines content of the as-received RCC was 7% and 8.4% after modified compactive effort. The fines content of the RG/RCC blends vary from 5.8 to 8.6%. The fine contents of the RG/RCC blends are presented in Figure 5.18. The fines were non-plastic with silt size materials. Local State Authority specifies that the fines content of Class 3 RCC should be 2-10% before compaction (VicRoads 2007) and 6-14% after compaction (VicRoads 1995). The fines content of the as-received RCC and RG/RCC blends are within that typical range.
Similar to RG/CR blends, the fine contents in the RG/RCC blends were also non-plastic made out of silt size particles. Therefore, the plastic limit and liquid limit could not be obtained. The primary reason for this is the fact that the Atterberg limit is directly related to the clay mineralogy and as such, very low fines content with silty materials

5.2.10 Plasticity Index
results in immeasurable plasticity. This aspect suggest that some difficulties may be experienced with the workability of the RG/RCC blends as cohesion of particles and a “tight” prepared surface are usually sought after characteristics. The addition of small quantities of clayey sand or plastic crusher fines may also be a good solution to overcome this potential problem.

5.2.11 Direct Shear test

Direct shear tests (DSTs) were conducted according to the British Standard (BS 1377-7-1990) on specimens compacted inside the shear box with optimum water content to achieve the maximum dry density obtained in modified compaction tests. It is recommended that the size of the largest particle of the test specimen should not exceed one-tenth of the specimen height. Therefore, large shear box with 30.5 cm by 30.5 cm having the effective depth of 20.3 cm was used for the direct shear tests of as-received RCC. As the amount of materials needed for 20 mm nominal size particles is too much and as it was very difficult to get the vast amount of materials, DST was carried out on as-received RCC only. Three different normal stress levels were applied to the test samples. The internal friction angle of the RCC was found to be 43.8° with normal stress ranging from 50 kPa to 200 kPa and the cohesion was 158 kPa. Figure 5.19 and Figure 5.20 show the stress and the volume change behavior versus axial strain of compacted as-received RCC in direct shear test, respectively. From Figure 5.19, it is observed that with increase in normal stress, the shear stress also increase as we would expect from our knowledge of sliding friction (a concurrent increase in the shear stress on the failure plane at failure with the increase of the normal stress, $\sigma_n$). It was noted that the shear stresses for all three normal stresses increase with the horizontal displacement until the stresses reach the peak strengths. It was also observed that at higher normal stresses, the peak shear stresses also become higher. All the three curves for the three selected normal stresses in Figure 5.19 revealed that after the peak stresses, shear stresses decrease with increase in horizontal displacement. These decreases of shear stresses after the peak stresses are normally regarded as strain softening which would be a good characteristic for road pavement materials. From Figure 5.20, it is seen that at first there is a slight reduction in height or volume of the soil specimen, followed by a dilation or increase in height or volume. As the normal stress ($\sigma_n$) increases, the harder it is for the soil to dilate during shear, which seems
reasonable. Figure 5.21 shows the Mohr-Coulomb failure envelope for compacted RCC100 sample. From the Mohr-Coulomb failure envelope an internal friction angle of 43.8° and an apparent cohesion of 158 kPa were obtained as mentioned before.

![Figure 5.19: Stress versus axial strain of compacted RCC100 in direct shear test](image1)

![Figure 5.20: Volume change versus axial strain of compacted RCC100 in direct shear test](image2)
Figure 5.21: Mohr-Coulomb failure envelope for RCC100 in direct shear test

5.2.12 Consolidated drained triaxial compression test

In order to find out the effective shear strength parameters (i.e., cohesion, $c'$ and angle of internal friction, $\phi'$), consolidated drained triaxial compression tests were carried out for the RG100, RCC100 and RG/RCC blends. The selected effective confining stresses were 50 kPa, 100 kPa and 200 kPa on the specimens in each test.

**Deviator stress versus axial strain relationship**

Figure 5.22 to Figure 5.28 show the relationship between the deviator stress ($q$) and the axial strain ($\varepsilon_a$) at the three different effective confining stress levels for RCC100, RG10/RCC90, RG15/RCC85, RG20/RCC80, RG30/RCC70, RG40/RCC60 and RG50/RCC50 specimens, respectively. It was noticed that the deviator stress increases with axial strain until it reaches the peak strength. It was also noticed that at higher confining pressures, the peak strength became higher and the axial strain corresponding to the peak strength also became higher. All three curves reveal that after the peak strength, the deviator stress decreases with axial strain. This characteristic is normally described as strain softening which would be a good characteristic for road subbase and base material.
Figure 5.22: Deviator stress versus axial strain relationship for RCC100

Figure 5.23: Deviator stress versus axial strain relationship for RG10/RCC90
Figure 5.24: Deviator stress versus axial strain relationship for RG15/RCC85

Figure 5.25: Deviator stress versus axial strain relationship for RG20/RCC80
Figure 5.26: Deviator stress versus axial strain relationship for RG30/RCC70

Figure 5.27: Deviator stress versus axial strain relationship for RG40/RCC60
The peak deviator stresses from the above stress – strain relationship are summarised in Table 5.6.

Table 5.6: The peak deviator stresses and effective confining stresses of RG/RCC blends in consolidated drained triaxial compression test

<table>
<thead>
<tr>
<th>Effective Confining Stress (kPa)</th>
<th>50</th>
<th>100</th>
<th>200</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blends</td>
<td>Peak Deviator Stress (kPa)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RCC100</td>
<td>785.50</td>
<td>1150.33</td>
<td>1882.80</td>
</tr>
<tr>
<td>RG10/RCC90</td>
<td>573.24</td>
<td>959.49</td>
<td>1491.55</td>
</tr>
<tr>
<td>RG15/RCC85</td>
<td>577.46</td>
<td>881.50</td>
<td>1509.12</td>
</tr>
<tr>
<td>RG20/RCC80</td>
<td>534.80</td>
<td>784.85</td>
<td>1310.62</td>
</tr>
<tr>
<td>RG30/RCC70</td>
<td>572.67</td>
<td>925.63</td>
<td>1414.30</td>
</tr>
<tr>
<td>RG40/RCC60</td>
<td>441.70</td>
<td>812.30</td>
<td>1324.78</td>
</tr>
<tr>
<td>RG50/RCC50</td>
<td>399.20</td>
<td>659.43</td>
<td>1130.29</td>
</tr>
<tr>
<td>RG100</td>
<td>399.20</td>
<td>659.43</td>
<td>1130.29</td>
</tr>
</tbody>
</table>
Volume change versus axial strain relationship

The variation of the volumetric strain with axial strain from a series of consolidated drained triaxial compression tests under different confining pressures for RCC100, RG10/RCC90, RG15/RCC85, RG20/RCC80, RG30/RCC70, RG40/RCC60 and RG50/RCC50 specimens, respectively are shown in Figure 5.29 to Figure 5.35. The positive volumetric strain indicates the contractive behaviour and the negative volumetric strain indicates the dilative behaviour of the samples. All the samples show similar volumetric strain characteristics. The samples compress in the initial stage of the shearing and then start to dilate. Generally, the dilation of samples during shearing increases with increased axial strain. It can be also seen that the higher the confining pressures, lower the volumetric strain at the same axial strain. The volumetric strain characteristics of samples are similar to the dense cohesion less soil subjected to similar test conditions.

![Graph showing volumetric strain versus axial strain relationship for RCC100](image)

Figure 5.29: Volumetric strain versus axial strain relationship for RCC100
Figure 5.30: Volumetric strain versus axial strain relationship for RG10/RCC90

Figure 5.31: Volumetric strain versus axial strain relationship for RG15/RCC85
Figure 5.32: Volumetric strain versus axial strain relationship for RG20/RCC80

Figure 5.33: Volumetric strain versus axial strain relationship for RG30/RCC70
Figure 5.34: Volumetric strain versus axial strain relationship for RG40/RCC60

Figure 5.35: Volumetric strain versus axial strain relationship for RG50/RCC50

**MIT Stress Field**

Figure 5.36 to Figure 5.42 show the effective stress paths envelope for RCC100, RG10/RCC90, RG15/RCC85, RG20/RCC80, RG30/RCC70, RG40/RCC60 and RG50/RCC50 specimens, respectively on MIT stress field under three different
effective confining stresses. The stress paths of the samples are drawn on \( s'-t \) space. In the MIT stress field, \( s' \) and \( t \) are defined in terms of the principal stresses as follows (Head 1994):

\[
s' = \frac{\sigma_1' + \sigma_3'}{2}
\]

Equation 5.1

\[
t = \frac{\sigma_1' - \sigma_3'}{2}
\]

Equation 5.2

In these consolidated drained triaxial compression tests, the effective confining stress, \( \sigma_3' \), remains constant and the effective principal stress, \( \sigma_1' \), increases to peak failure stress and then drops as the test progress. The stress path follows a straight line with an angle of 45° with the horizontal, i. e., the slope of the stress path is 1 on 1. The results show that the failure envelope corresponding to the peak stress is linear for the tested stress ranges of 50 kPa, 100 kPa and 200 kPa.

Figure 5.36: Stress path in \( s'-t \) space for RCC100
Figure 5.37: Stress path in s'-t space for RG10/RCC90

Figure 5.38: Stress path in s'-t space for RG15/RCC85
Figure 5.39: Stress path in $s'$-$t$ space for RG20/RCC80

Figure 5.40: Stress path in $s'$-$t$ space for RG30/RCC70
Figure 5.41: Stress path in $s'$-t space for RG40/RCC60

Figure 5.42: Stress path in $s'$-t space for RG50/RCC50
Cambridge Stress Field

Figure 5.43 to Figure 5.49 show the effective stress paths envelope for RCC100, RG10/RCC90, RG15/RCC85, RG20/RCC80, RG30/RCC70, RG40/RCC60 and RG50/RCC50 specimens, respectively on Cambridge stress field under different effective confining stresses. In the Cambridge stress field, the deviator stress, $q$ is plotted against the mean effective applied stress, $p'$, in which the parameter $p'$ is defined in terms of effective stress as follows (Head 1994):

$$p' = \frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3} \quad \text{Equation 5.3}$$

The parameter $q$ is defined as follows as being equal to the deviator stress:

$$q = \sigma'_1 - \sigma'_3 = \sigma_1 - \sigma_3 \quad \text{Equation 5.4}$$

In the triaxial test two of the principal effective stresses are equal to the horizontal effective stress, and Equation 5.3 is expressed as follows:

$$p' = \frac{\sigma'_1 + 2\sigma'_3}{3} \quad \text{Equation 5.5}$$

As mentioned earlier, in the consolidated drained triaxial compression tests, the effective confining stress, $\sigma'_3$ remains constant and the effective principal stress, $\sigma'_1$ increases to peak failure stress and then drops as the test progress. Here, in the Cambridge stress field, the stress path also follows a straight line inclined at a slope of 3 on 1. The results show that the failure envelope corresponding to the peak stress is linear for the tested stress ranges.
Figure 5.43: Stress path in $p'$-$q$ space for RCC100

Figure 5.44: Stress path in $p'$-$q$ space for RG10/RCC90
Figure 5.45: Stress path in p'-q space for RG15/RCC85

Figure 5.46: Stress path in p'-q space for RG20/RCC80
Figure 5.47: Stress path in p'-q space for RG30/RCC70

Figure 5.48: Stress path in p'-q space for RG40/RCC60
Shear Strength Parameters (For Mohr’s Circles)

The Mohr’s circles and Mohr-Coulomb failure envelope under consolidated drained triaxial compression tests for RCC100, RG10/RCC90, RG15/RCC85, RG20/RCC80, RG30/RCC70, RG40/RCC60 and RG50/RCC50 specimens, respectively are shown in Figure 5.50 to Figure 5.56. Table 5.7 shows the shear strength parameters of RG/RCC blends.

Table 5.7: The shear strength parameters of RG/RCC blends

<table>
<thead>
<tr>
<th>Blends</th>
<th>Effective cohesion, c’ (kPa)</th>
<th>Effective angle of internal friction, $\phi’$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RCC100</td>
<td>75.4</td>
<td>51.0</td>
</tr>
<tr>
<td>RG10/RCC90</td>
<td>59.1</td>
<td>47.7</td>
</tr>
<tr>
<td>RG15/RCC85</td>
<td>51.2</td>
<td>48.1</td>
</tr>
<tr>
<td>RG20/RCC80</td>
<td>52.9</td>
<td>45.4</td>
</tr>
<tr>
<td>RG30/RCC70</td>
<td>63.2</td>
<td>46.4</td>
</tr>
<tr>
<td>RG40/RCC60</td>
<td>34.0</td>
<td>47.1</td>
</tr>
<tr>
<td>RG50/RCC50</td>
<td>33.4</td>
<td>44.7</td>
</tr>
<tr>
<td>RG100</td>
<td>5</td>
<td>41.7</td>
</tr>
</tbody>
</table>
As presented in the above table, it has been noted that shear strength parameters of RG/RCC blends are similar to the cohesionless materials. The internal friction angles vary from 44.7° to 51° and the cohesion ranges between 33.4 kPa to 91.2 kPa. Generally, all blends attain high shear strength parameters. Although the RCC100 and RG/RCC blends are considered as cohesionless frictional material, it deviates from purely frictional behaviour. It might be due to the effect of confining stress. At higher confining stresses, particle became flattened at their contact points, sharp corners are crushed and the interlocking are also reduced (Aatheesan 2011). In addition, actual curvature of Mohr-Coulomb envelope is highest for dense granular soils (Lambe and Whitman 1979) and generally considered as straight line.

![Mohr's circles and Mohr-Coulomb failure envelope for RCC100](image)

Figure 5.50: Mohr’s circles and Mohr-Coulomb failure envelope for RCC100
Mohr - Coulomb failure envelope

\[ \tau' = \sigma'_n \tan (47.7^\circ) + 59.1 \]

\[ \sigma'_n = 200 \text{ kPa} \]

\[ \sigma'_n = 100 \text{ kPa} \]

\[ \sigma'_n = 50 \text{ kPa} \]

Figure 5.51: Mohr’s circles and Mohr-Coulomb failure envelope for RG10/RCC90

Mohr - Coulomb failure envelope

\[ \tau' = \sigma'_n \tan (48.1^\circ) + 51.2 \]

\[ \sigma'_n = 200 \text{ kPa} \]

\[ \sigma'_n = 100 \text{ kPa} \]

\[ \sigma'_n = 50 \text{ kPa} \]

Figure 5.52: Mohr’s circles and Mohr-Coulomb failure envelope for RG15/RCC85
Shear stress (kPa) vs. Effective normal stress (kPa) for different pressures:
- 50 kPa effective confining pressure
- 100 kPa effective confining pressure
- 200 kPa effective confining pressure

Mohr - Coulomb failure envelope:

\[ \tau' = \sigma_n' \tan(\theta) + \phi \]

- \( \tau' = \sigma_3' \) for 50 kPa
- \( \tau' = 100 kPa \) for 100 kPa
- \( \tau' = 200 kPa \) for 200 kPa

Figure 5.53: Mohr’s circles and Mohr-Coulomb failure envelope for RG20/RCC80

Figure 5.54: Mohr’s circles and Mohr-Coulomb failure envelope for RG30/RCC70
Figure 5.55: Mohr’s circles and Mohr-Coulomb failure envelope for RG40/RCC60

5.2.13 Repeated load Triaxial (RLT) test

Results of permanent strain and resilient modulus for the RG/RCC blends are given in Table 5.8. The results in Table 5.8 indicate that, for the compaction standard of 98% modified MDD and moisture contents in the range of 65%-90% modified OMC, the
materials produced much smaller permanent strain and much higher resilient moduli value than those of natural granular subbases. This indicates that the performances (in terms of both permanent deformation and resilient modulus) of the recycled crushed concrete blends are superior as compared to those of natural granular subbases.

Table 5.8: RLT results for RG/RCC blends

<table>
<thead>
<tr>
<th>Blend</th>
<th>% MDD</th>
<th>% W_{opt}</th>
<th>Permanent strain ($\times 10^3 \mu$e)</th>
<th>Resilient modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Stage 1</td>
<td>Stage 2</td>
</tr>
<tr>
<td>RG10/RCC90</td>
<td>96.5</td>
<td>80</td>
<td>3.84</td>
<td>3.94</td>
</tr>
<tr>
<td></td>
<td>97.7</td>
<td>82</td>
<td>3.19</td>
<td>3.20</td>
</tr>
<tr>
<td></td>
<td>98.0</td>
<td>98</td>
<td>3.94</td>
<td>4.00</td>
</tr>
<tr>
<td>RG15/RCC85</td>
<td>97.0</td>
<td>68</td>
<td>3.38</td>
<td>3.42</td>
</tr>
<tr>
<td></td>
<td>97.8</td>
<td>74</td>
<td>3.02</td>
<td>3.07</td>
</tr>
<tr>
<td></td>
<td>98.9</td>
<td>81</td>
<td>4.17</td>
<td>4.49</td>
</tr>
<tr>
<td>RG20/RCC80</td>
<td>97.4</td>
<td>62</td>
<td>4.01</td>
<td>4.20</td>
</tr>
<tr>
<td></td>
<td>97.7</td>
<td>76</td>
<td>5.07</td>
<td>5.22</td>
</tr>
<tr>
<td></td>
<td>98.0</td>
<td>89</td>
<td>4.85</td>
<td>5.45</td>
</tr>
<tr>
<td>RG30/RCC70</td>
<td>98.3</td>
<td>43</td>
<td>3.99</td>
<td>4.07</td>
</tr>
<tr>
<td></td>
<td>97.5</td>
<td>82</td>
<td>2.98</td>
<td>3.05</td>
</tr>
<tr>
<td></td>
<td>97.8</td>
<td>93</td>
<td>4.15</td>
<td>4.52</td>
</tr>
<tr>
<td>Natural</td>
<td>98.0</td>
<td>70.0</td>
<td>3-10</td>
<td>4-15</td>
</tr>
<tr>
<td>granular</td>
<td>98.0</td>
<td>80.0</td>
<td>5-10</td>
<td>7-15</td>
</tr>
<tr>
<td>subbase (^1)</td>
<td>98.0</td>
<td>90.0</td>
<td>7-15</td>
<td>10-F</td>
</tr>
</tbody>
</table>

\(^1\) ARRB 2010

Figure 5.57 shows the variation of permanent strain and resilient modulus characteristics with different moisture ratio and deviator stress of RG10/RCC90 blends. As shown in Figure 5.57 it was noted that the permanent strain does not show any significant variation with dynamic deviator stress although it increases with increase in moisture ratio. But the resilient modulus decreases with increase in moisture ratio. The results also indicate that for all three moisture ratio, the samples of the RG10/RCC90 blends gained stiffness during the permanent strain testing. It might be due to both densification under repeated loading and binding action from the active cement content in the blends.
Figure 5.57: Results of permanent strain and resilient modulus testing for RG10/RCC90

Figure 5.58 shows the variation of permanent strain and resilient modulus characteristics with different moisture ratio and deviator stress of RG15/RCC85 blends. As shown in Figure 5.58, it was noted that similar to RG10/RCC90 blends, the permanent strain also does not show any significant variation with dynamic deviator
stress although it increases with increase in moisture ratio. The permanent strain at 74% OMC is less than those of the 68% OMC, most probably due to the heterogeneous nature of the recycled materials. But the resilient modulus decreases with increase in moisture ratio. The results also indicate that for all three moisture ratio, the samples also gained stiffness during the permanent strain testing. It might be due to both densification under repeated loading and binding action from the active cement content in the blends.

![Graph showing permanent strain and resilient modulus](image)

Figure 5.58: Results of permanent strain and resilient modulus testing for RG15/RCC85
Figure 5.59 shows the variation of permanent strain and resilient modulus characteristics with different moisture ratio and deviator stress of RG20/RCC80 blends. As shown in Figure 5.59, it was noted that the permanent strain shows a slight variation with dynamic deviator stress although it increases significantly with increase in moisture ratio, but the resilient modulus decreases with increase in moisture ratio. Similar to the other blends, the results also indicate that for all three moisture ratio, the specimens also gained stiffness during the permanent strain testing. It might be due to the densification under repeated loading and binding action from the active cement content in the blends.

(a) Permanent strain testing for RG20/RCC80
Figure 5.59: Results of permanent strain and resilient modulus testing for RG20/RCC80

Figure 5.60 shows the variation of permanent strain and resilient modulus characteristics with different moisture ratio and deviator stress of RG30/RCC70 blends. As shown in Figure 5.60, it was noted that the permanent strain shows a slight variation with dynamic deviator stress although it increases significantly with increase in moisture ratio. The permanent strain of 82% OMC is less than those of the 43% OMC. It might be due to the heterogeneous nature of the recycled materials. The resilient modulus decreases with increase in moisture ratio. Similar to the other blends, the results also indicate that for all three moisture ratio, the samples of the RG30/RCC70 blends gained stiffness during the permanent strain testing. Similar to the other blends, it might be due to the densification under repeated loading and binding action from the active cement content in the blends.
Figure 5.60: Results of permanent strain and resilient modulus testing for RG30/RCC70

The results of the resilient modulus testing of the RG10/RCC90 with stress stages are shown in Figure 5.61. As mentioned earlier that for all three moisture ratio, the samples of the RG/RCC blends gained stiffness during the permanent strain testing, most
probably due to the densification under repeated loading and binding action from the active cement content present in the blends. Therefore, the resilient modulus results obtained after the permanent strain testing may not be representative of the original sample condition.

Figure 5.62 shows the variation of resilient modulus of RG15/RCC85 blends with different stress stages. As shown in Figure 5.62, it is clear that the lower moisture ratio produced higher resilient modulus. Overall, all three specimens performed satisfactorily over the sixty six stress stages. However, for all the 66 stress stages, the resilient modulus does not show any significant decreases although high deviator stresses and high stress ratios were applied in later part of stress stages.

Figure 5.61: Results of resilient modulus testing with stress stages for RG10/RCC90
Figure 5.62: Results of resilient modulus testing with stress stages for RG15/RCC85

Figure 5.63 shows the variation of resilient modulus of RG20/RCC80 blends with different stress stages. As shown in Figure 5.63, it is clear that the lower moisture ratio produced higher resilient modulus. Specimen at 82% moisture ratio shows much lower resilient modulus than those of the specimens at 76% OMC and 62% OMC. Overall, all three specimens performed satisfactorily over the sixty six stress stages. The specimens at 76% OMC and 62% OMC show similar resilient modulus with stress stages. It can be noted that after the 18 stress stages, the resilient modulus decreases as high deviator stresses and high stress ratios were applied in later part of stress stages.
Figure 5.63: Results of resilient modulus testing with stress stages for RG20/RCC80

Figure 5.64 shows the variation of resilient modulus of RG30/RCC70 blends with different stress stages. As shown in Figure 5.64, it is clear that the lower moisture ratio produced higher resilient modulus. Specimen at 93% moisture ratio shows much lower resilient modulus than those of the specimens at 82% moisture ratio and 43% moisture ratio. Overall all three specimens performed satisfactorily over the stated stress stages.

In general, it can be concluded that the permanent strain was not sensitive to both moisture content and RG content. However, resilient modulus was sensitive to both moisture content and RG content and a higher content of RG could potentially produce lower resilient modulus, most probably due to the reduction of active cement content in the blends with the addition of RG. Overall, the performance of the RG/RCC blends in RLT tests in this research is superior to those of natural granular subbases as reported in Table 5.8.
It should be noted that RG/RCC blends with active cement content could develop very high stiffness (as close to a cement treated layer), but would have a very low tensile strength and may easily develop cracks due to heavily loaded vehicle and/or lateral movements in subgrade (shrinkage cracks). Consequently, they may not be used in a base layer, which is in direct contact with the thin bituminous surfacing, to avoid problems associated with reflective cracking in the base layer. Therefore, the use of RG in RCC would reduce potential high stiffness and cracking problems by reducing the active cement content.

Two-Parameter Theta or Bulk Stress model (K-θ model)

According to the recommendation of the AASHTO test procedures (AASHTO 1999), the resilient modulus tests results can be analysed by using different regression models such as bulk stress model and deviatoric stress model (Puppala et al. 2011). In the two parameter model, the resilient modulus can be described as follows (Hicks and Monismith 1971):

\[ M_R = k_1 \theta^{k_2} \]  

Equation 5.6

Equation 5.6 can also be expressed in the logarithmic form as follows:
\[ \log M_R = \log k_1 + k_2 \times \log \theta \]  

Equation 5.7

where,

\[ M_R = \text{Resilient modulus (MPa)} \]
\[ k_1, k_2 = \text{Regression coefficients of theta model} \]
\[ \theta = (\sigma_1 + \sigma_2 + \sigma_3) = \text{Bulk stress (kPa)} \]

Figure 5.65 to Figure 5.68 show the resilient modulus characteristics with bulk stress in K-\( \theta \) model of RG10/RCC90, RG15/RCC85, RG20/RCC80 and RG30/RCC70 blends, respectively. As shown in the figures, it was observed that the RG/RCC blends showed higher resilient modulus at higher bulk stress. It was also observed that generally the blends at lower moisture ratio showed higher resilient modulus at the same bulk stress as expected. At higher moisture ratios, the resilient modulus of the blends showed a significant drop as compared to lower moisture ratio.

The K-Theta model constants \( k_1, k_2 \) and the coefficient of determination values of RG/RCC blends are presented in Table 5.9. It was noticed that the specimens at lower moisture ratio performed satisfactorily in terms of resilient modulus testing. The coefficient of determination values of all RG/RCC blends were greater than 0.75, suggesting that a reasonably good fit was obtained using the K-Theta model for RG/RCC blends. Generally, \( k_1 \) values are higher than the \( k_2 \) values as shown in Table 5.9. In addition, the \( k_1 \) value of the same blend is higher at lower moisture ratio as compared to the higher moisture ratio. This also signifies that the higher \( k_1 \) value is an indicator of higher resilient modulus. The constant parameter \( k_1 \), which is an indicator of resilient moduli magnitudes varied from 9.85 to 629.31 and the parameter \( k_2 \), which represents the nonlinear nature of the stress dependency, varied from 0.19 to 0.76.
Table 5.9: Regression coefficients of K-Theta model of RG/RCC Blends

<table>
<thead>
<tr>
<th>Blends</th>
<th>Moisture ratio (%)</th>
<th>$k_1$</th>
<th>$k_2$</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>RG10/RCC90</td>
<td>98</td>
<td>13.235</td>
<td>0.7623</td>
<td>0.8846</td>
</tr>
<tr>
<td></td>
<td>82</td>
<td>264.660</td>
<td>0.3515</td>
<td>0.9048</td>
</tr>
<tr>
<td></td>
<td>80</td>
<td>15.078</td>
<td>0.7434</td>
<td>0.8508</td>
</tr>
<tr>
<td>RG15/RCC85</td>
<td>81</td>
<td>157.010</td>
<td>0.3344</td>
<td>0.7916</td>
</tr>
<tr>
<td></td>
<td>74</td>
<td>110.340</td>
<td>0.4591</td>
<td>0.8624</td>
</tr>
<tr>
<td></td>
<td>68</td>
<td>39.790</td>
<td>0.2900</td>
<td>0.8017</td>
</tr>
<tr>
<td>RG20/RCC80</td>
<td>89</td>
<td>9.852</td>
<td>0.6359</td>
<td>0.8323</td>
</tr>
<tr>
<td></td>
<td>76</td>
<td>268.960</td>
<td>0.2829</td>
<td>0.7723</td>
</tr>
<tr>
<td></td>
<td>62</td>
<td>272.480</td>
<td>0.2666</td>
<td>0.7621</td>
</tr>
<tr>
<td>RG30/RCC70</td>
<td>93</td>
<td>12.161</td>
<td>0.6364</td>
<td>0.8951</td>
</tr>
<tr>
<td></td>
<td>82</td>
<td>63.342</td>
<td>0.5075</td>
<td>0.9015</td>
</tr>
<tr>
<td></td>
<td>43</td>
<td>629.310</td>
<td>0.1928</td>
<td>0.7539</td>
</tr>
</tbody>
</table>

$M_r (82\% \text{ OMC}) = 264.66 \times \theta^{0.3515}$
$R^2 = 0.9048$

$M_r (80\% \text{ OMC}) = 15.078 \times \theta^{0.7434}$
$R^2 = 0.8508$

Figure 5.65: Variation of resilient modulus with bulk stress for RG10/RCC90
MR (74% OMC) = 110.34 \times \theta^{0.4591} \quad R^2 = 0.8624

MR (68% OMC) = 390.79 \times \theta^{0.29} \quad R^2 = 0.8017

MR (81% OMC) = 157.01 \times \theta^{0.3344} \quad R^2 = 0.7916

MR (74% OMC) = 110.34 \times \theta^{0.4591} \quad R^2 = 0.8624

MR (74% OMC) = 110.34 \times \theta^{0.4591} \quad R^2 = 0.8624

MR (68% OMC) = 390.79 \times \theta^{0.29} \quad R^2 = 0.8017

MR (81% OMC) = 157.01 \times \theta^{0.3344} \quad R^2 = 0.7916

Figure 5.66: Variation of resilient modulus with bulk stress for RG15/RCC85

MR (62% OMC) = 272.48 \times \theta^{0.2666} \quad R^2 = 0.7621

MR (89% OMC) = 9.8522 \times \theta^{0.6359} \quad R^2 = 0.8323

MR (76% OMC) = 268.96 \times \theta^{0.2829} \quad R^2 = 0.7723

Figure 5.67: Variation of resilient modulus with bulk stress for RG20/RCC80
Rada and Witczak (1981) mentioned that in the K-Theta model, higher quality granular materials exhibit higher $k_1$ values and smaller $k_2$ values. All the RG/RCC blends indicate higher $k_1$ values as compared to $k_2$ values. In general, higher moisture content affects the strength of the materials and results in lower resilient modulus.

Figure 5.69 shows the variation of $\log k_1$ with $k_2$ for the RG/RCC blends. It was observed that reasonable good correlation with a regression coefficient ($R^2$) of 0.79 existed between $\log k_1$ and $k_2$. Due to the slight variation of the physical properties (e.g., particle density, water absorption, maximum dry density, etc.) of the blends, the $\log k_1$ and $k_2$ showed reasonable good correlation with a regression coefficient ($R^2$) of 0.79.
The main drawback of the K-Theta model is that, it neglects the important effect of shear stress on the resilient modulus (May and Witczak 1981, Uzan 1985). Uzan (1985) compared the measured resilient moduli with those predicted using the model for a dense-graded aggregate as shown in Figure 5.70. The discrepancy between the measured and the predicted values of moduli as shown in Figure 5.70 is mainly due to neglecting the effects of shear stress and shear strain when calculating the response using the K-Theta model (Thomson 1998). The k-Theta model can only represent a very limited range of stress paths and leads to erroneous results (Brown and Pappin 1981). It also does not describe the descending behaviour of the resilient modulus with axial strain (Thomson et al. 1998).
Figure 5.70: Measured and the predicted behaviour using K-Theta Model for a dense graded aggregate (Uzan 1985)

5.3 Findings of the geotechnical properties of recycled glass in blends with recycled crushed concrete

The following conclusions can be drawn from the results of RG/RCC blends:

- The laboratory studies carried out in this research work have shown overall that the incorporation of up to 50% RG into RCC has low to minimal effect on the physical and mechanical properties of the original material. As such, the RG/RCC blends with the maximum percentage of 30% of RG were found to satisfactorily meet the current VicRoads requirements.
• The grading limits of most of the RG blends were well within the upper bounds of the grading envelope specified by the VicRoads upper and lower bounds for crushed aggregates. The before and after compaction grading curves for RG50/RCC50 and RG40/RCC60 were noted to be on the VicRoads upper limits.

• CBR and LA abrasion values of all the mixtures were found to be satisfactory according to VicRoads requirements for Class 3 subbase material.

• The RG/RCC blends show good shear strength in CD triaxial compression tests.

• The results obtained from RLT tests indicated that all blends tested are suitable as subbase materials. Further field trials have also been conducted to assess the constructability of the crushed glass blends and to determine their long-term field performance for comparison with the laboratory performance.

• The research indicates that initially up to 15% “4.75 mm RG” could be safely added to Class 3 RCC. The degree of breakdown occurring in the RG blends with in excess of 15% is on the limit of what would be acceptable for this material. Depending on the results of future field trials, it may be possible to increase the percentage of RG.
CHAPTER SIX

6 FIELD TESTING

6.1 Introduction

The research involves the construction of field trial pavement sections with up to 30% RG as an additive into CR and RCC mixes for pavement base applications at Alex Fraser’s recycling site. This was followed by field testing to assess the constructability of the RG blends and field monitoring programs to determine the field performance of the test pavements.

This chapter details the construction of the field trial pavements, field testing and field monitoring. Nine sections of unbound granular base pavements, comprising of up to 30% Recycled Glass (RG) in blends with Recycled Crushed Concrete (RCC) and Crushed Rock (CR) in the pavement base were constructed between October and December 2009, on the main haul road at the Alex Fraser’s recycling site in Melbourne, Australia. Each of the pavement sections was 80 m in length and 4.75 m in width. The design of these granular base pavements was based on the outcomes of the initial laboratory testing phase of this research. RG/RCC and RG/CR blends were found to satisfy the requirements of a pavement subbase material in the laboratory testing phase of this research, however it was decided to use these materials in the pavement base and assess their performances as a higher quality pavement base material in the field trials. Each pavement comprised an unbound granular base of nominal 200 mm thickness, overlying a 200 mm thick subbase. The 200 mm thick base layer was subsequently overlaid by a 50 mm thick glassphalt cover. Seven sections of the base materials comprised of 10% to 30% of RG/RCC or RG/CR. Four sections were constructed with RCC with 10%, 15%, 20% and 30% of RG by mass. Another three sections were constructed with CR in blends with 10%, 20% and 30% of RG by mass.

Two control sections comprising 100% of RCC and 100%CR were placed. These aggregates are commonly accepted for usage in pavement base applications in Australia
and both of these control materials have known performance characteristics upon which the other pavement sections can be assessed.

Some significant issues were encountered during the construction of the pavement trial sections and those issues are summarised below:

- There was a significant variation in the subgrade CBR within each pavement section.

- Significant variation of subgrade CBR was also noticed between pavement sections. This might be due to the nature of the in situ and imported materials used for the subgrade.

- There was a considerable variation in the placed subbase and base thicknesses, most probably due to the variability of the subgrade and subbase materials.

- Small variations were evident in base course thickness within each of 9 pavement sections. This might be due to the difficulties related to underlying subbase materials.

The results confirmed that some difficulties were experienced in constructing base layers using the RG/RCC and RG/CR blends. While these problems are not desirable, they probably represent what happens in the real world with the construction of non-homogenous pavements on arterial and local roads (equivalent to VicRoads Scale C roads). These types of roads would typically be the target use for the RG and other recycled materials.

In comparing with the Local State Authority’s compaction criteria for base, it was found that:

- Most base pavement sections containing less than 20% RG content achieved the target mean value of density ratio of 98% for subbase layer; whereas all base pavement sections struggled to meet the minimum required compaction criteria
for base layer, i.e. less than the target mean value of density ratio of 100%. This suggested that the RG blends only meet VicRoads specifications on Class 3 subbase materials, but would not meet the VicRoads specifications on Class 2 base materials for failing to meet the compaction criteria. The field trial also indicated that blends containing a RG content greater than 20% would likely result in a lower field dry density.

In comparing with the VicRoads drying back specifications of base prior to bituminous surfacing, it appears that all pavement sections complied with the specifications concerned, possibly due to the timing as the construction was taken during the summer, which was expected to aid the dry back.

As a result of good drying back, all pavements developed quite high stiffness, with Falling Weight Deflectometer (FWD) mean maximum deflection ($D_0$) normalised to 700 kPa applied stress varying in the range of 365-585 $\mu$m. The RG/RCC blends also produced higher stiffness than the RG/CR blends. This was consistent with the results of resilient modulus determined with the laboratory RLT test method.

It is recommended that field performance monitoring (FWD and roughness) of the pavement sections will enable better interpretation of the long-term performance results. Although the nine test pavements are not as uniform as desirable, there is sufficient detailed thickness, density and deflection data collected during the construction at exact locations along each 80 m long pavement section for each pavement section to act as a realistic trial of RG as an additive.

6.2 Material selection, pavement design and construction

The nine unbound pavement sections were designed and constructed to most likely represent what occurs in reality during the construction of arterial and local roads (equivalent to VicRoads Scale C roads). These types of roads would typically be the target use for RG and other recycled products.
6.2.1 Selected Recycled Material Blends with Recycled Glass

The material selection was based on the material sources available in Victoria and previous results obtained from laboratory testing for material specification and performance as given below.

6.2.2 Material Sources

The CR, RCC and RG for this research were obtained from Alex Fraser Recycling site at Laverton, Victoria, located approximately 20 km to the west of Melbourne, Australia.

- The CR was manufactured from recycled basalt surface excavation rock (basalt floaters), which commonly occurs near the surface to the west and north of Melbourne. Traditionally, this material would have been discarded as waste (often into landfill). However, because this rock is generally hard and durable (LA<25), it can be used under controlled conditions for pavement subbase (Class 3) and other uses.

- The RCC (Class 3) mainly comes from demolished concrete and comprised graded aggregates up to 20 mm nominal size.

- The RG comes mainly from empty soft drink, beer, food, wine and liquor containers and comprised with the maximum particle size of 4.75 mm.

Laboratory tests were carried out and the results of material such as Plasticity Index, LA Abrasion, Wet and Dry strength, Flakiness index (Table 6.1) indicated both sourced materials had all physical properties complying with VicRoads specification for Class 3 CR and Class 3 RCC. However, it was noted that the RCC had lower wet and dry strength values (110 kN and 147 kN, respectively) compared to the CR (145 kN and 178 kN, respectively). The wet and dry strength variations were 19% and 25% for CR and RCC, respectively. Therefore, the RCC may have some potential crushing and disintegration at high field compaction efforts. However, according to Lay (1998) material with a wet and dry strength variation of less than 35% are considered durable.
Table 6.1: Physical properties of recycled materials (VicRoads 2009)

<table>
<thead>
<tr>
<th>Test method</th>
<th>Parameter</th>
<th>Recycled material</th>
<th>VicRoads Class 3 CR</th>
<th>VicRoads Class 3 RCC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>CR</td>
<td>RCC</td>
<td>CR</td>
</tr>
<tr>
<td>Atterberg Limit</td>
<td>Plasticity Index</td>
<td>0</td>
<td>0</td>
<td>0-12</td>
</tr>
<tr>
<td>Los Angeles abrasion (AS 1141.23)</td>
<td>LA abrasion (%)</td>
<td>24</td>
<td>28</td>
<td>&lt;35</td>
</tr>
<tr>
<td>Wet and Dry strength (AS 1141.22)</td>
<td>Dry strength (kN)</td>
<td>178</td>
<td>147</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Wet strength (kN)</td>
<td>145</td>
<td>110</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Wet/Dry strength variation (%)</td>
<td>19</td>
<td>25</td>
<td>-</td>
</tr>
<tr>
<td>Flakiness test (AS1141.15)</td>
<td>Flakiness index (%)</td>
<td>16</td>
<td>11</td>
<td>35</td>
</tr>
</tbody>
</table>

6.2.3 Characteristics of Recycled Material Blends with RG Additive

The CR and RCC blends with up to 50% RG additive were tested using laboratory modified compaction test, grain size distribution and CBR test. The following test results were noted:

- The results of laboratory modified compaction tests indicated that CR and RCC blends with a higher RG additive content would potentially produce a lower maximum dry density.

- The results of after compaction grading curves indicated that RCC blends with greater than 40% RG additive would potentially produce grading exceeding the Local State Authority’s upper limits; whereas CR blends with greater than 50% RG additive would potentially produce grading exceeding the Local State Authority’s upper limits.

- The results of CBR testing indicated that CR and RCC blends with a higher RG additive content would potentially reduce their soaked CBR values. For the RCC blend, an increase of 50% RG content would reduce the soaked CBR value by 50% (i.e. from CBR 200% to CBR 100%); whereas an increase of 50% RG
content in the CR would reduce the soaked CBR value by 30% (i.e. from CBR 180% to CBR 120 %). However, all the blends had soaked CBR values greater than 80%. This satisfies the Local State Authority’s requirements on CBR for Class 3 CR and Class 3 RCC subbase.

The CR and RCC blends with up to 30% RG additive were also tested using advanced laboratory repeated load test methods for mechanical properties including permanent strain and resilient modulus. The following results were noted:

- The laboratory performances (in terms of both permanent deformation and resilient modulus) of the CR blends tested are comparable to those of natural CR subbase; whereas the laboratory performances of the RCC blends tested are superior compared to those of natural CR subbase.

- For the CR blends, higher RG content would potentially produce higher permanent strain. However, this trend is not consistent through the results especially for stage 1 and this might be due to the variation of moisture content which makes it difficult to establish a trend for permanent strain. On the other hand, the resilient modulus was not sensitive to changes in either moisture content or RG content.

- For the RCC blends, permanent strain was not sensitive to RG content. However, higher RG content would potentially produce lower resilient modulus.

Based on the above test results obtained from laboratory compaction, after compaction grading and performance tests (including CBR test, RLT permanent strain and resilient modulus tests), it was decided to select CR and RCC blends with up to 30% RG additive for further field testing to assess the constructability of the RG blends and to determine their long-term field performance for comparison with the laboratory performance.
6.3 Site location and layout

The test pavements were constructed at a site situated within the Alex Fraser Recycling Centre in Laverton, Melbourne, Australia. Altogether nine trial pavement sections were constructed as mentioned earlier. They were named sections 1 to 9, corresponding with the selected RG blends used in the base layer as given in Table 6.2.

A layout of the pavement is shown in Figure 6.1, providing chainages (running along the pavement length measured relative to a standard benchmark of Chainage 0 m at the eastern end) and offset distances (running across the pavement width measured relative to a standard benchmark of offset 0 m. Table 6.3 also lists the chainages and offsets of each experiment site. It was anticipated that subsequent quarry truck trafficking would travel within the centre of each pavement.

Table 6.2: Selected base materials

<table>
<thead>
<tr>
<th>Pavement section</th>
<th>Base type</th>
<th>Blends</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100% 20 mm Class 3 RCC</td>
<td>RCC100</td>
</tr>
<tr>
<td>2</td>
<td>80% 20 mm Class 3 RCC and 20% 5 mm RG</td>
<td>RG20/RCC80</td>
</tr>
<tr>
<td>3</td>
<td>90% 20 mm class 3 RCC and 10% 5 mm RG</td>
<td>RG10/RCC90</td>
</tr>
<tr>
<td>4</td>
<td>85% 20 mm class 3 RCC and 15% 5 mm RG</td>
<td>RG15/RCC85</td>
</tr>
<tr>
<td>5</td>
<td>70% 20 mm Class 3 RCC and 30% 5 mm RG</td>
<td>RG30/RCC70</td>
</tr>
<tr>
<td>6</td>
<td>80% 20 mm Class 3 CR and 20% 5 mm RG</td>
<td>RG20/CR80</td>
</tr>
<tr>
<td>7</td>
<td>100% 20 mm Class 3 CR</td>
<td>CR100</td>
</tr>
<tr>
<td>8</td>
<td>90% 20 mm Class 3 CR and 10% 5 mm RG</td>
<td>RG10/CR90</td>
</tr>
<tr>
<td>9</td>
<td>70% 20 mm Class 3 CR and 30% 5 mm RG</td>
<td>RG30/CR70</td>
</tr>
</tbody>
</table>
Figure 6.1: Layout of test pavements (Courtesy of Alex Fraser Recycling)
Table 6.3: Location of sites with directions, chainages and offsets

<table>
<thead>
<tr>
<th>Pavement section</th>
<th>Base type</th>
<th>Direction</th>
<th>Chainage (m)</th>
<th>Centreline Offset (m)</th>
<th>Wheel path 1</th>
<th>Wheel path 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>RCC100</td>
<td>South</td>
<td>0 – 80</td>
<td>3.25</td>
<td>2.05</td>
<td>4.45</td>
</tr>
<tr>
<td>2</td>
<td>RG20/RCC80</td>
<td>South</td>
<td>0 – 80</td>
<td>7.75</td>
<td>6.55</td>
<td>8.95</td>
</tr>
<tr>
<td>3</td>
<td>RG10/RCC90</td>
<td>South</td>
<td>80-160</td>
<td>3.25</td>
<td>2.05</td>
<td>4.45</td>
</tr>
<tr>
<td>4</td>
<td>RG15/RCC85</td>
<td>South</td>
<td>80-160</td>
<td>7.75</td>
<td>6.55</td>
<td>8.95</td>
</tr>
<tr>
<td>5</td>
<td>RG30/RCC70</td>
<td>South</td>
<td>80-160</td>
<td>12.25</td>
<td>11.05</td>
<td>13.45</td>
</tr>
<tr>
<td>6</td>
<td>RG20/CR80</td>
<td>South</td>
<td>160-240</td>
<td>3.25</td>
<td>2.05</td>
<td>4.45</td>
</tr>
<tr>
<td>7</td>
<td>CR100</td>
<td>South</td>
<td>160-240</td>
<td>7.75</td>
<td>6.55</td>
<td>8.95</td>
</tr>
<tr>
<td>8</td>
<td>RG10/CR90</td>
<td>South</td>
<td>160-240</td>
<td>12.25</td>
<td>11.05</td>
<td>13.45</td>
</tr>
<tr>
<td>9</td>
<td>RG30/CR70</td>
<td>West</td>
<td>0-80</td>
<td>3.25</td>
<td>2.05</td>
<td>4.45</td>
</tr>
</tbody>
</table>

6.4 Pavement thickness design

Figure 6.2 shows the pavement configuration selected for the nine test pavement sections. Each pavement comprised a glass in asphalt (also called “glassphalt”) thickness of 50 mm, a recycled unbound granular base of nominal 200 mm thickness, an underlying 200 mm thick subbase, overlying a design subgrade CBR of 10%. The purposes of the different layers are as follows:

- A 200 mm thick subbase was selected to provide a stiff layer against which construction compaction equipment could compact the granular base material, so that uniform base densities can be achieved both with pavement depth and also longitudinally along the test pavements.

- A base course thickness of 200 mm was selected for all nine pavement materials to ensure that the only mode of failure for the constructed pavements would be the permanent deformation and this deformation would be restricted to the base material, and would not occur in any subbase or subgrade materials.
A glassphalt thickness of 50 mm was selected for surfacing the test pavements to reduce a risk that the asphalt might deform or rut in its own weight, and that pavement deformation would not be strictly limited to the unbound granular base materials.

According to the Austroads design guide (Austroads 2008), this pavement configuration, if it contains standard granular subbase/base materials and standard asphalt, would have an estimated design asphalt fatigue life of about $2 \times 10^5$ ESAs and an estimated deformation life of $10^8$ ESAs.

![Diagram of pavement structures]

Figure 6.2: New pavement structures

### 6.5 Pavement construction schedule

The initial planning of pavement construction only focussed on field testing for the assessment of the as-constructed properties (e.g., thickness, density, moisture content and layer stiffness) of the bases in all nine test pavements using available field testing equipments such as automatic level, Nuclear Density Gauge, Clegg Hammer and/or Falling Weight Deflectometer. However, due to changes in the construction schedule and some problems that arose during the construction, the following sequence of
construction was followed. The field testing program was adjusted to match the construction activities.

At first the sites clearances were done and the subgrade was removed by profiler from all pavement sections. Then, the reworking on subgrade on pavement sections was done. After that the subbases were constructed for all pavement sections. After constructing the subbases, the levels of the subbases were checked and the Nuclear Density testing, Clegg Hammer testing and Falling Weight Deflectometer testing were done on subbases to check the different parameters (e.g., density, moisture content and stiffness) on the subbases. After checking the different parameters of the subbases, the bases were constructed in two stages of 100 mm thickness. Each granular base material was mixed to the appropriate optimum moisture content in the pugmill at the recycling site and immediately transported by truck to the site, a haulage time of approximately 1-2 minutes. The material was placed in two equal lifts so as to achieve high density and to ensure that the density was uniform with depth within the base. For each lift, after placement and rough spreading, the surface was graded to a uniform level using the controlled grader, followed by compaction rolling with a 12 tonnes vibratory steel drum roller. The final surface was trimmed to an even level and was followed by the finishing of the surface with the multi-tyred roller. A minimum 3 days dry-back period was applied to each lift. During the dry-back periods, Nuclear Density testing was conducted for each lift to check density and moisture content and the stiffness of the bases was checked by Clegg Hammer testing. Nuclear Density tests were also conducted to measure the final compaction levels of the combined 200 mm base. Final levels of the base surface were also taken to confirm base thicknesses.

The RG additive component of the blends being essentially non-plastic was expected to detract from the workability and cohesion characteristics of the CR and in particular the RCC blends. The pugmilling of the products prior to placement obviously aided the uniformity of the placed material, with respect to both moisture content and grading, and therefore, segregated or bony patches were rarely observed. With regard to the final prepared surface, it was noticed that the RCC blended with RG was not as tight showing a tendency to ravel on the prepared surface in some areas as the CR with RG additive. This is probably due to the extra cohesion imparted by some clay component present in
the CR materials. Overall, the CR and RCC blends with 15% RG additive appeared to be the easiest one to place.

After checking the different parameters of the bases, the 50 mm glassphalt surface layer was placed 1.5 days after the placement of the prime coat. A specialized truck was used to apply asphalt in a single pass, followed by an automatic controlled grader and a 12 tonnes vibrating steel drum roller followed by the finishing of the surface with the multi-tyred roller. Automatic levelling tests were performed 4 days after the construction of asphalt surfacing to determine the final thickness of the asphalt layer.

6.6 Equipment used and field testing program for construction evaluation

For the assessment of the geotechnical performance of the recycled materials and their impact on the strength and stiffness, field testing was conducted at various locations after the placement of the pavement base layers using the following equipments 3 days after the placement of the base layers. It was therefore expected that the field moisture conditions at the time of testing would be lower than the optimum moisture conditions at the time of compaction, as the materials were delivered within the recycling site and haulage time was 1-2 minutes.

6.6.1 Level

An automatic level, Leica digital level DNA10, was used to measure the reduced levels of the field trial site. The equipment can read the staff height through a bar-coded staff and calculate the level instantly, estimate the target distance, and automatically store the level in its built-in memory. All level information contained in this research was recorded using this system.

For the assessment of the final outputs of the granular base and asphalt surfacing thicknesses, levels were taken on the surfaces of subbase, base and wearing course. The level readings were taken nominally at 10 m interval along the two wheel paths and the centreline of each pavement section. Figure 6.3 shows the level and staff during taking reading.
6.6.2 Nuclear Density Gauge (NDG)

The Nuclear Density Gauge (NDG) is a current standard device for the quality control (Q/C) of soil compaction in road construction. The gauge operates by producing small doses of backscattered gamma waves. The radiation reflected from the soil is detected at the base of the gauge and converted to soil density when the gauge is calibrated to the specific soil. The gauge also has a neutron source to determine the moisture content by detecting the hydrogen in a soil sphere around the gauge. The specifications for the calibration and use of the gauge for moisture and density measurements of soil and asphalt surfaces are listed in several ASTM standard procedures [ASTM D-2922, D-2950, D-3017, 2002].

The use of the NDG requires training and operation by a licensed technician and it is governed by regulations for its storage, transmission, and disposal. These requirements do not make the gauge a practical tool for user in repair jobs done routinely by the utilities in urban areas. Furthermore, NDG gauges are operated in very mobile
conditions in the field and the potential loss or damage to the gauges may result in harmful radiation exposure to the public.

NDG/moisture meters were used during construction to monitor the compaction of the placed layers. Two different NDGs were used provided by Civil Geotechnical Service (CGS) in this trial. Both were calibrated by CGS to ensure that they produced similar results.

For the assessment of the construction variability of the granular base, and their impact on pavement performance, direct transmission method of nuclear density and moisture testing was conducted on the granular bases after the construction of each layer at 10 metre intervals along 2 wheel paths for each of the pavement sections. Figure 6.4 shows the NDG used in this research.

In this research, a nuclear density gauge was used to obtain the in-place density and water content of the compacted layers following the ASTM D 6938-08a test method (ASTM 2008). Equations 6.1 and 6.2 were implemented to obtain the moisture ratio and density ratio of the compacted base layer using the values measured by NDG and the values obtained from laboratory modified Proctor compaction tests.

\[
\text{Moisture Ratio (\%)} = \frac{\text{Field Moisture Content} \times 100}{\text{Optimum Moisture Content}} \quad \text{Equation 6.1}
\]

\[
\text{Density Ratio (\%)} = \frac{\text{Field Dry Density} \times 100}{\text{Maximum Dry Density}} \quad \text{Equation 6.2}
\]
6.6.3 Dynamic Cone Penetration Test (DCPT)

In geotechnical and foundation engineering, the in situ penetration tests have been widely used for site investigation in support of analysis and design. The standard penetration test (SPT) and the cone penetration test (CPT) are two typical in situ penetration tests. While the SPT is performed by driving a sampler into the soil with hammer blow, the CPT is a quasi-static procedure. The Dynamic Cone Penetrometer (DCP) is an instrument which can be used for the rapid measurement of the in situ strength of existing flexible pavements constructed with unbound materials. The dynamic cone penetration test (DCPT) was developed in Australia by Scala (1956). The current model was developed by the Transvaal Roads Department in South Africa (Luo 1998). The mechanics of the DCPT shows features of both the CPT and the SPT. The DCPT is similar to the SPT in test. It is performed by dropping a hammer from a certain fall height (usually 510 mm) and measuring a penetration depth per blow for each tested depth. The shape of the dynamic cone is similar to that of the penetrometer used in the CPT.
The DCPT is performed by dropping a hammer of 9 kg mass from a height of 510 mm and measuring penetration depth per blow for a certain depth (Standards Australia 1997). Therefore, it is quite similar to the procedure of obtaining the blow count \( N \) using the soil sampler in the SPT. In the DCPT, however, a cone (a steel cone of 30 degrees angle and 20 ± 0.2 mm diameter) is used to obtain the penetration depth instead of using the split spoon soil sampler. In this respect, there is some resemblance with the CPT in the fact that both tests create a cavity during penetration and generate a cavity expansion resistance. In road construction, there is a need to assess the adequacy of a subgrade to behave satisfactorily beneath a pavement. Proper pavement performance requires a satisfactorily performing subgrade.

The dynamic cone penetrometer was used to assess the subgrade strength at random locations and in smaller number of tests only. DCP testing was also conducted on the base pavement material for the purpose of comparison with other equipments such as the FWD and Clegg Hammer (CH). Figure 6.5 shows the dynamic cone penetrometer used in this research.

### 6.6.4 Clegg Hammer (CH)

The Clegg Hammer (also called the Clegg Impact Soil Tester) consists of a compaction hammer operating within a vertical guide tube. When the hammer strikes the soil surface, a precision accelerometer mounted on the hammer feeds its output to a digital readout unit. The unit registers the deceleration in units of Impact Value (IV). The IV relates to soil strength and correlates with California Bearing Ratio (CBR) values. An ASTM standard covers the determination of the Impact Value (IV) of the soil [ASTM 2007].

The first version of the Clegg Hammer was developed by Dr. Baden Clegg in Australia and was named ‘The Clegg Impact Soil Tester’. It was first introduced at the 8th Australian Road Research Conference in 1976 [Clegg B 1976]. Since then, it has been widely used in Australia and Europe. It is currently manufactured in the United States and is being used by consultants and contractors in several compaction control applications and particularly in the compaction testing of sports fields.
It is used to confirm uniform compaction of over wide areas of ground, identifying poorly compacted areas and ineffective rolling of materials. It also provides a means for measuring and controlling soil strength and consolidation levels during trench reinstatement.

Many different versions of the Clegg instrument are used for different purposes. These include 0.5 kg and 2.25 kg types suitable for sports turfs and a special Golf Course Tester for testing fairways. The larger 10 kg and 20 kg Testers produced are supplied for testing harder materials and road works. The most popular Clegg instrument is one using a 4.5 kg hammer.

The Tester consists of a compaction hammer operating within a vertical guide tube. When the hammer is released from a fixed height it falls through the tube and strikes the surface under test, decelerating at a rate determined by the stiffness of the material within the region of impact. A precision accelerometer mounted on the hammer feeds its output to a hand held digital readout unit which registers the deceleration in units of
Impact Value (IV). The IV indicates soil strength and shows good correlation with CBR test results.

Operating Principle: The Tester consists of a 4.5 kg compaction hammer operating within a vertical guide tube. The Hammer falls through the tube when released and strikes the surface under test, decelerating at a rate determined by the stiffness of the material within the region of impact. The readout registers the deceleration in units of Impact Value (IV). The IV is an indication of soil strength.

The Clegg Hammer was used during construction to monitor the base stiffness.

For the assessment of a granular base stiffness before placing the next layer, CH testing was done at six random locations along the length of each test pavement, and the readings were used to ascertain whether additional dry-back period was required to achieve adequate layer stiffness.

For the assessment of the construction variability of the granular base and their impact on pavement performance, CH tests were conducted after each pavement layer was completed. In this case, a full set of CH readings was taken every 10 m along the two wheel paths of the pavement. Figure 6.6 shows the Clegg Hammer used in this research.

In this research, the standard Clegg hammer consisting of a 4.5 kg compaction hammer using a 457.2 mm drop height was used which is equipped with an accelerometer as described in ASTM D 5874-07 (ASTM 2007). The Impact Value (IV or CIV) is a dynamic force penetration property which relates to soil strength and may be used to set a strength parameter (ASTM 2007). Equation 6.3 was used to convert the Clegg Impact Value (CIV) to a field CBR value (Clegg 1986).

\[
\text{CBR Field} \, (\%) = 0.06 \text{CIV}^2 + 0.52 \text{CIV} + 1
\]

Equation 6.3

To obtain a strength ratio which is defined as the ratio of CBR obtained in field (from Clegg hammer test) to the required CBR value (100% for base application); Equation 6.4 was used to calculate the strength ratio.
Strength Ratio (%) = \frac{CBR_{Field} \times 100}{CBR_{Required}} \quad \text{Equation 6.4}

6.6.5 Falling Weight Deflectometer (FWD)

The falling weight deflectometer (FWD) is a non-destructive testing (NDT) and non-intrusive device. The FWD has been widely used in Australia and New Zealand in pavement engineering to evaluate pavement structural condition. The FWD plays a crucial role in selecting optimum pavement maintenance and rehabilitation strategies. The FWD is a tool used to achieve rapid and repeatable in situ characterization of the pavement layer stiffness.

The FWD applies dynamic loads to a pavement surface, simulating the magnitude and duration of a single heavy moving wheel load. The FWD loading system delivers a transient impulse load to the pavement surface. The pavement response (vertical deformation or deflection) at various distances from the loading plate are measured by a series (usually seven) of geophone sensors (Figure 6.7). The FWD normally uses a
contact stress of 566 kPa applied to a 300 mm diameter plate and a load pulse duration of about 25 ms. If the FWD contact stress during measurement differs from 566 kPa (40 kN load), deflections need to be normalized to a stress of 566 kPa, usually assuming deflections are linearly related to stress level. Therefore, to minimize errors of adjustment, the contact stress during testing should be as close as practicable to the standard stress, and differ from 566 kPa by less than 15% (Austroads 2004).

In FWD testing, the spacing of individual test sites in a given section of road is arranged so that the general pattern of deflections over the whole section of road can be identified and sub-sections with consistent deflections can be defined. For the purposes of statistical analysis, an adequate number of results for each sub-section, at least 10 and preferably upward of 30. Again, wheel path positions should be selected, keeping in mind any proposed changes to the road alignment, if practical (Austroads 2004).

The deflection sensors can be adjusted to variable distances from the load plate according to user’s requirement. A typical FWD test applies four different load levels at discrete locations; this test is completed in less than two minutes.

![Figure 6.7: Schematic of FWD load and deflection measurement (Texas Department of Transportation 2008)](image)

In forensic studies of the pavement layers, the test pattern should also include points where the pavement is in relatively good condition, and points where distress (the cause of which the engineers are trying to isolate) is present.

During the FWD testing, the air and pavement surface temperatures are measured; these factors can be taken into account later in the analysis.

The major factors affecting pavement deflection when using the FWD are as follows:
The pavement layer thickness,
Types of material used for the pavement layer,
Quality of material for pavement layer,
Support of the subgrade,
Environmental factors,
Discontinuities of the pavement and
Variability within the pavement structure.

The FWD is used to evaluate flexible pavement to determine overall structural strength as well as individual layer stiffness of the pavement. It is also used on rigid pavement to evaluate the load transfer across slabs and can be used to detect large voids when significant erosion of the base material has occurred under the slab joints. The FWD is commonly used to determine the variability of overall deflection along the roadway alignment.

For the assessment of the construction variability of all pavement layers and subgrade and their impact on pavement performance, a FWD survey was conducted after the pavement was completed. In this case, a full set of FWD readings was taken every 10 m along the two wheel paths of the pavement. Figure 6.8 shows the arrangement for falling weight deflectometer testing.
6.7 Field testing activities

6.7.1 Testing of Subgrade

Figure 6.9 shows the finished surface of subgrade after the removal of the existing pavement.

In the pavement design, it was assumed that the subgrade would be uniform with an expected design CBR of 10%. Therefore, initial planning did not focus on subgrade testing. However, field inspection of subgrade condition after the removal of the existing pavement indicated that the subgrade significantly varied between pavement sections as well as within each pavement section, and that the subgrade levels after trimming were uneven. As this could potentially promote variability in the construction of the granular base, it was decided to include more detailed testing of all pavement layers to fully enable an assessment of their constructability as well as their impact on
the long-term pavement performance. Given the short timeframe allowed for subgrade testing, the following additional testings were undertaken:

- Levels of exposed subgrade were measured every 10 m along the centreline of all pavements using the automatic level, and the results were used to ascertain whether there was significant variation in thickness of the subbase.

- DCP testing of subgrade was conducted on the day at random locations along the length of each test pavement, and the subgrades CBR were used to ascertain whether additional field testing on other pavement layers was required. Figure 6.10 and Figure 6.11 show the subgrade CBR with respect to depth conducted by DCP tests. The figures show significant variation in subgrade CBR with respect to depth.

From the figures, it is noted that the CBR varies from 18% to 61%, 11% to 22%, 13% to 61%, 39% to 288%, 22% to 133%, 18% to 133% and 22% to 805% for sections 1, 2, 3, 4, 5, 6 and 8, respectively. For sections 4 and 8, the maximum CBR was 288% and 805%, respectively. These unexpected higher CBR values found in sections 4 and 8 might be due to the presence of bedrock or stone in those particular locations. These large variations of CBR values among the subgrade sections indicate that the subgrades are comprised with different heterogeneous materials with varying nature and quality (e.g., density, moisture content and stiffness) of the in situ and imported materials used for the subgrades.
Figure 6.9: The finish surface of subgrade (Courtesy of ARRB Group)

Figure 6.10: Subgrade CBR of sections 1 to 8
6.7.2 Testing of Bases

Figure 6.12 shows the base finish surface after the construction of this layer.

Initial planning included a seven-day curing period for the bases for all pavement sections, during which the following tests were performed to check the base properties:

- Nuclear Density testing, after the placement of the 200 mm base was completed, at every 10 m along the two wheel paths of all pavements, and the results were used to determine the compaction of the base in each pavement section for the comparison of compaction between bases.

- Automatic levelling was performed to check the base levels at every 10 m along the two wheel paths and centreline of all pavements and, hence, the final base thickness.

However, given the tight schedules for the construction of the bases, the following field testing schedules were carried out:
Automatic levelling, Clegg Hammer and NDG testing on the 200 mm base were performed one day after the placement of the entire 200 mm base in all pavement sections.

Figure 6.12: Base finish surface (Courtesy of Alex Fraser Recycling)

### 6.7.3 Testing of Sealed Pavements

Figure 6.13 shows the finish surface of asphalt after the construction of this layer. The following field testings were carried according to the initial plan:

- Automatic levelling was performed four days after the construction of glassphalt surfacing. Levels of asphalt pavement surface was measured at every 10 m along the two wheel paths and centreline of all pavements, and the results were used to determine the final thickness of the asphalt layer in each pavement section.
Falling Weight Deflectometer testing was performed four days after the construction of asphalt surfacing. The testing was performed at every 10 m along the two wheel paths of all pavements, and the results were used to check the variability in pavement stiffness in each pavement section as well as between pavement sections. Figure 6.14 to Figure 6.17 show the FWD maximum deflection measured on the asphalt surface for different sections.

Figure 6.13: The finished surface of asphalt (Courtesy of ARRB)

It was observed that the maximum deflection values in the asphalt surfaces showed a large variation with chainages along the longitudinal direction for different sections of bases composed of RG/RCC or RG/CR blends along with two control sections of RCC100 (100% RCC) and CR100 (100% CR). These large variations of maximum deflection values along the longitudinal sections and within the wheel paths indicate that there were significant variations in subgrade as well as subbase and base modulus within each pavement section and between pavement sections most probable due to the nature (e.g., density, moisture content and modulus) of the in situ and imported materials used for the subgrade.
Figure 6.14: FWD maximum deflection measured on the asphalt surface (Sections 1, 3 & 6)

Figure 6.15: FWD maximum deflection measured on the asphalt surface (Sections 2, 4 & 7)
Figure 6.16: FWD maximum deflection measured on the asphalt surface (Sections 5 & 8)

Figure 6.17: FWD maximum deflection measured on the asphalt surface (Section 9)
6.8 Assessment of field testing results

A detailed assessment of the as-constructed properties of the test pavement sections was undertaken. The assessment included:

- Level thickness data using levels measured on the asphalt surface and on granular bases.
- Density and moisture content data using the results obtained from the NDG testing on the bases.
- Wearing surface deflection data using a FWD after the construction of the asphalt layer.

The testing was focussed in the pavement locations with tests conducted at similar chainages (10 m intervals) along the typical wheel paths of trucks to be trafficked on the pavements.

6.8.1 Assessment of Level Results

All level readings are based on a temporary benchmark with an assumed level of 100 m above the sea level datum and are not related to the design levels (which were not available).

Level results for each surface layer by pavement section are also presented in Figure 6.18 to Figure 6.21. Note that surface levels plotted are those collected along the centreline of the lane. From the figures, it is observed that sections 1 and 2 show nearly constant surface level with chainage with respect to the assumed benchmark level, whereas sections 3, 4, 5, 6, 7 and 8 show that the surface level of those sections are decreasing with chainage with respect to the assumed benchmark level. For section 9, it is observed that the surface level is reducing from west to east direction. These discrepancies might be due to the fact that the natural surface level was not in the same level in all sections along the longitudinal sections. Due to the lack of design level data,
with which the actual level data should be compared, it is not possible to assess the quality of work achieved on each layer surface.

Figure 6.18: Surface levels along Pavement sections 1, 3, and 6

Figure 6.19: Surface levels along pavement sections 2, 4, and 7
Figure 6.20: Surface levels along Pavement sections 5 and 8

Figure 6.21: Surface levels along Pavement section 9

6.8.2 Assessment of Thickness Results

From the available level data, it is possible to determine the as-constructed pavement thicknesses and check them against the target (specified) thicknesses. Thicknesses of
subbase, base and surfacing layers were derived from the level data taken on each surface at nominally 10 m intervals.

Thickness results for each surface layer by pavement section are also presented in Figure 6.22 to Figure 6.25. Here the layer thicknesses plotted are average thicknesses in Left Wheel Path (LWP), centreline and Right Wheel Path (RWP) of individual chainages.

Figure 6.22: Thicknesses along Pavement sections 1, 3, and 6

Figure 6.23: Thicknesses along Pavement sections 2, 4, and 7
Table 6.4 summarises the overall mean and standard deviation thickness values for individual pavement sections. The results showed that:

- Mean thicknesses of all base thicknesses, except for pavement sections 1, were slightly short of target (design) thickness of 200 mm and all, except for pavement section 4, have a standard deviation between 20-35 mm.
Table 6.4: Pavement layer thicknesses (mm)

<table>
<thead>
<tr>
<th>Base Section</th>
<th>Base type</th>
<th>Base thickness (mm)</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>RCC100</td>
<td></td>
<td>214</td>
<td>35</td>
<td>24</td>
</tr>
<tr>
<td>2</td>
<td>RG20/RCC80</td>
<td></td>
<td>192</td>
<td>22</td>
<td>24</td>
</tr>
<tr>
<td>3</td>
<td>RG10/RCC90</td>
<td></td>
<td>191</td>
<td>19</td>
<td>24</td>
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<tr>
<td>4</td>
<td>RG15/RCC85</td>
<td></td>
<td>184</td>
<td>13</td>
<td>24</td>
</tr>
<tr>
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<td>RG30/RCC70</td>
<td></td>
<td>187</td>
<td>24</td>
<td>24</td>
</tr>
<tr>
<td>6</td>
<td>RG20/CR80</td>
<td></td>
<td>183</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
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<td>CR100</td>
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<td>197</td>
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<tr>
<td>8</td>
<td>RG10/CR90</td>
<td></td>
<td>182</td>
<td>25</td>
<td>24</td>
</tr>
<tr>
<td>9</td>
<td>RG30/CR70</td>
<td></td>
<td>171</td>
<td>20</td>
<td>24</td>
</tr>
</tbody>
</table>

6.8.3 Comparisons with Local State Authority’s Thickness Compliance

Section 304.06(c) of VicRoads Standard Specifications (VicRoads 2008) states the following, regarding the thickness of Pavement Layers:

- “The base course shall be not less than the specified thickness by more than 10 mm and the average thickness of base over every 100 m section, over the full carriageway width, shall be not less than the specified thickness”.

Based on the results of pavement thickness data in Table 6.4, it would appear that:

- Five out of nine pavement sections (i.e. Section 4, Section 5, Section 6, Section 8 and Section 9) did not comply with the minimum required thickness for base layer (i.e. more than 10 mm short of the target value of 200 mm).

The results indicated that some difficulties were experienced in the construction of the base layers using the RG/RCC blends and the RG/CR blends. While these problems are not desirable, they probably represent what happens in the real world with the construction of non-homogenous pavements on arterial and local roads. These types of roads would typically be the target use for RG and other recycled products.
6.9 **Assessment of density and moisture results**

Direct transmission method of nuclear density and moisture testing was conducted on the granular bases after the construction of each layer. The moisture content tests during the construction were performed with the help of Civil Geotechnical Services (CGS), a contractor who operated the nuclear density gauge. It should also be noted that field density values were calibrated by using oven moisture tests obtained from the same locations as moisture contents attained by using the nuclear gauge.

6.9.1 **Dry Density and Moisture Content of Bases**

Table 6.5 and Table 6.6 show the values of maximum dry density and optimum moisture contents, determined from samples of the materials collected from each base layer and each pavement section, respectively.

<table>
<thead>
<tr>
<th>Sampling location</th>
<th>RCC100</th>
<th>RG10/ RCC90</th>
<th>RG15/ RCC85</th>
<th>RG20/ RCC80</th>
<th>RG30/ RCC70</th>
<th>CR100</th>
<th>RG10/ CR90</th>
<th>RG20/ CR80</th>
<th>RR30/ CR70</th>
</tr>
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<tr>
<td>Site 1</td>
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<td>Site 4</td>
<td>Site 2</td>
<td>Site 5</td>
<td>Site 7</td>
<td>Site 8</td>
<td>Site 6</td>
<td>Site 9</td>
<td></td>
</tr>
<tr>
<td>Bottom 100 mm</td>
<td>1.99</td>
<td>1.95</td>
<td>1.98</td>
<td>2.01</td>
<td>2.01</td>
<td>2.22</td>
<td>2.19</td>
<td>2.19</td>
<td>2.19</td>
</tr>
<tr>
<td>Top 100 mm</td>
<td>2.00</td>
<td>1.99</td>
<td>2.00</td>
<td>1.98</td>
<td>1.99</td>
<td>2.25</td>
<td>2.20</td>
<td>2.23</td>
<td>2.20</td>
</tr>
<tr>
<td>Average</td>
<td>2.00</td>
<td>1.97</td>
<td>1.99</td>
<td>2.00</td>
<td>2.00</td>
<td>2.24</td>
<td>2.20</td>
<td>2.21</td>
<td>2.20</td>
</tr>
</tbody>
</table>


Table 6.6: Results of Optimum Moisture Content (%)

<table>
<thead>
<tr>
<th>Sampling location</th>
<th>RCC100</th>
<th>RG10/RCC90</th>
<th>RG15/RCC85</th>
<th>RG20/RCC80</th>
<th>RG30/RCC70</th>
<th>CR100</th>
<th>RG10/CR90</th>
<th>RG20/CR80</th>
<th>RG30/CR70</th>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bottom 100 mm</td>
<td>13</td>
<td>12</td>
<td>12</td>
<td>11</td>
<td>10</td>
<td>7.5</td>
<td>8.5</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>Top 100 mm</td>
<td>11.5</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>8</td>
<td>8</td>
<td>8.5</td>
<td>8</td>
</tr>
<tr>
<td>Average</td>
<td>12.3</td>
<td>11.5</td>
<td>11.5</td>
<td>11.0</td>
<td>10.5</td>
<td>7.8</td>
<td>8.3</td>
<td>8.3</td>
<td>8.0</td>
</tr>
</tbody>
</table>

A NDG survey was conducted at every 10 m along the two wheel paths of all pavements after their placement to determine the compaction of the base in each pavement section for the comparison of compaction between bases. Table 6.7 provides a summary of results (mean and standard deviation) of field dry density, moisture content and density ratio for bases in individual pavement sections, respectively.

Figure 6.26 compares the average field dry densities between pavement base sections. The results indicated that for the recycled crushed concrete blends, the average field densities varied in the small range of 1.92 to 2.0 (Mg/m³). They are obviously lower than the average field densities of the crushed rock blends, which also varied in the small range of 2.12 to 2.19 (Mg/m³).

Figure 6.27 compares the average field moisture contents between pavement trial sections (base course) at the time adequate dry back was deemed to have occurred. The results indicated that for the recycled crushed concrete blends, the average moisture contents varied in the large range of 6 to 8.2% due to the significant variation in their respective OMC’s and differing dry back times. They were higher overall than the average moisture contents of the crushed rock blends, which varied within a much smaller range of 4.9 to 5.2%.
Table 6.7: Results of Field Dry Density and Moisture Content

<table>
<thead>
<tr>
<th>Pavement section</th>
<th>n</th>
<th>Dry density (Mg/m³)</th>
<th>Relative density (%)</th>
<th>Moisture content (%)</th>
<th>Relative moisture content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean Std Dev Mean Std Dev Mean Std Dev Mean Std Dev</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>12</td>
<td>1.979 0.019 99.0 1.0</td>
<td>6.9 0.67</td>
<td>56 5</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>12</td>
<td>1.948 0.043 97.4 2.1</td>
<td>6.2 0.45</td>
<td>56 4</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>12</td>
<td>1.923 0.028 97.6 1.4</td>
<td>8.3 0.58</td>
<td>72 5</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>12</td>
<td>2.000 0.027 100.5 1.4</td>
<td>6.5 0.33</td>
<td>57 3</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>12</td>
<td>1.945 0.029 97.3 1.5</td>
<td>6.0 0.30</td>
<td>63 3</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>12</td>
<td>2.163 0.041 97.9 1.8</td>
<td>4.9 0.23</td>
<td>53 2</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>12</td>
<td>2.188 0.030 97.7 1.3</td>
<td>5.0 0.33</td>
<td>65 4</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>12</td>
<td>2.173 0.031 98.8 1.4</td>
<td>5.2 0.40</td>
<td>63 5</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>8</td>
<td>2.120 0.028 96.4 1.3</td>
<td>5.1 0.50</td>
<td>64 6</td>
<td></td>
</tr>
</tbody>
</table>

Figure 6.26: Results of average dry density for the combined 200 mm base
Figure 6.27: Results of moisture content for the combined 200 mm base

All results (mean, maximum and minimum) of density ratio are summarised in the form of a histogram in Figure 6.28. It was noted that the CR blend containing 30% RG content (RG30/CR70) produced the lowest relative density compared to other CR blends containing a RG content less than or equal to 20% and the RCC blend containing 20% RG content (RG20/RCC80) produced the lower relative density. Therefore, a blend containing a RG content greater than 20% would likely result in a lower field dry density being achieved.
A number of density and moisture content tests were conducted on various base layers and the readings were recorded based on the assumption that all base materials had their optimum moisture content when delivered at the construction site.

Figure 6.29 shows the rate of moisture loss graphically over the dry back period for the bases. It was found that all bases dried back quickly during the dry back period and the moisture content before priming was well below the targeted moisture of 70% (in fact it only took three and a half days on average to reach this level).

Whilst the timing construction to take place during the summer, which was expected to aid dry back, it was anticipated that a drying period of at least three days was likely.
Table 6.8 presents the mean values of density and moisture content for the base layers for the various pavement sections. The average field densities of the RG/CR sections are noted to be higher than those of the RG/RCC blends. The RG/CR blends also had higher field density and laboratory density results than corresponding RG/RCC blends with the same RG contents. The results indicate that average density ratios in individual sites varied in the range of 95% to 100% MDD. The Control Sections (RCC, CR) as expected achieved the highest density results in the field and laboratory density tests as compared to the RG/RCC and RG/CR sections. The field results indicate that CR is a higher quality material than RCC.

It was noted that the RG30/CR70 blend containing 30% RG content produced the lowest density ratio compared to other RG/CR blends with less RG content. From this finding, it can be concluded that, a blend containing a recycled glass additive content of greater than 20%, would likely result in a lower field dry density being achieved.

Field moisture contents between pavement trial sections was taken at the time adequate dry back was deemed to have occurred. It was also assumed that all base materials had their optimum moisture content when delivered at the construction site. It was found that all bases dried back quickly during the dry back period and the moisture content
before priming was well below the targeted moisture of 70%. Whilst the timing
construction to take place during the summer, which was expected to aid dry back, it
was anticipated that a drying period of at least three days was likely. The field moisture
content results indicated that for the RG/RCC blends, the average moisture contents
varied in the large range of 6 to 8.3% which is due to variation in their respective OMCs
and differing dry back times. They were higher overall than the average moisture
contents of the RG/CR blends, which varied within a much smaller range of 4.9 to
5.2%. The control sections (RCC, CR) had field moisture contents of 6.9% and 5%
respectively, which were fairly consistent with the RG/RCC and RG/CR blends.
Table 6.8: Base layer: Mean values of density and moisture content results from Nuclear Density Gauge and field samples

<table>
<thead>
<tr>
<th>Pavement Section</th>
<th>Blends</th>
<th>Field Dry Density (Mg/m³)</th>
<th>MDD- Lab (Mg/m³)</th>
<th>Density Ratio (%)</th>
<th>Field Moisture Content (%)</th>
<th>OMC-Lab</th>
<th>Moisture Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>RCC</td>
<td>1.98</td>
<td>1.97</td>
<td>100.0</td>
<td>6.9</td>
<td>13.81</td>
<td>50</td>
</tr>
<tr>
<td>2</td>
<td>RG20/RCC80</td>
<td>1.95</td>
<td>1.98</td>
<td>98.5</td>
<td>6.2</td>
<td>11.54</td>
<td>53.7</td>
</tr>
<tr>
<td>3</td>
<td>RG10/RCC90</td>
<td>1.92</td>
<td>1.98</td>
<td>96.9</td>
<td>8.3</td>
<td>12.10</td>
<td>68.6</td>
</tr>
<tr>
<td>4</td>
<td>RG15/RCC85</td>
<td>2.00</td>
<td>1.97</td>
<td>101.5</td>
<td>6.5</td>
<td>11.30</td>
<td>57.5</td>
</tr>
<tr>
<td>5</td>
<td>RG30/RCC70</td>
<td>1.94</td>
<td>1.95</td>
<td>99.5</td>
<td>6.0</td>
<td>10.50</td>
<td>57.1</td>
</tr>
<tr>
<td>6</td>
<td>RG20/CR80</td>
<td>2.16</td>
<td>2.21</td>
<td>97.7</td>
<td>4.9</td>
<td>9.14</td>
<td>53.6</td>
</tr>
<tr>
<td>7</td>
<td>CR</td>
<td>2.19</td>
<td>2.30</td>
<td>95.2</td>
<td>5.0</td>
<td>8.67</td>
<td>57.7</td>
</tr>
<tr>
<td>8</td>
<td>RG10/CR90</td>
<td>2.17</td>
<td>2.26</td>
<td>96.0</td>
<td>5.2</td>
<td>8.09</td>
<td>64.3</td>
</tr>
<tr>
<td>9</td>
<td>RG30/CR70</td>
<td>2.12</td>
<td>2.18</td>
<td>97.2</td>
<td>5.1</td>
<td>9.31</td>
<td>54.8</td>
</tr>
</tbody>
</table>
6.9.3 Comparison between Achieved Results and VicRoads Criteria

As specified in Clause 304.08 of VicRoads Standard Specification Section 304 (VicRoads 2008), road authorities require material during compaction to have a moisture content of not less than 85% of optimum during compaction and, after completion of compaction of a layer. The moisture content of the material in the layer shall be maintained at a moisture content of not less than 85% of optimum until test rolling has been completed. Based on the results in Figure 6.29, construction of the base for the pavement trial complied with the target minimum moisture of 85% OMC (using modified compactive effort).

Table 6.9 shows Scale C requirements for testing and acceptance of compaction (as specified in Clause 304.08 and as provided in Table 304.082 of VicRoads standard specification Section 304 (VicRoads 2008).

<table>
<thead>
<tr>
<th>Compaction Scale</th>
<th>Mean Value of Density Ratio % (three tests)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Layers</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>Not less than 100.0</td>
</tr>
</tbody>
</table>

As shown in Table 6.9, road authorities require a material to have minimum mean values of density ratio of 100% for base materials for light duty pavements. In comparing with the Scale C Standard of Compaction, it would appear that:

- All base layers were also found to be marginally below these requirements except for the RCC and RG15/RCC85 sections.

6.10 Assessment of base stiffness using Clegg Impact values

6.10.1 Base modulus

There has been a considerable interest in correlating the Clegg Impact Value (CIV) with the Clegg Hammer Modulus (CHM) for application to such problems as multi-layer analysis of soil structures, particularly in the road pavement context (Clegg Impact soil
The CHM was calculated from the CIV obtained from the Clegg Hammer (CH) Test using the following equation suggested by Garcia et al. 2004 (for 4.5 kg, 45 cm drop, 5 cm diameter, standard hammer):

\[
CHM \ (MPa) = 0.088 \times (CIV)^2
\]

Equation 6.5

where,
\[
CHM = \text{Clegg Hammer Modulus (MPa), and}
\]
\[
CIV = \text{Clegg Impact Value}
\]

Figure 6.30 to Figure 6.38 show the variation of Clegg Hammer Modulus (CHM) with chainage along the longitudinal direction in different sections (Sections 1 to 9) of bases comprised of RG/RCC and RG/CR blends with two control sections of RCC100 (100% RCC) and CR100 (100% CR).

From the figures, it is observed that the CHM shows a large variation with values ranging from 55 MPa to 286 MPa for the base sections. This range of CHM along the longitudinal sections indicates that there were significant variations in subgrade as well as subbase modulus within each pavement section and between pavement trial sections. This might be due to the heterogeneous nature of the subgrade and subbase materials.
Figure 6.31: Clegg Hammer Modulus (CHM) along section 2 (RG20/RCC80)

Figure 6.32: Clegg Hammer Modulus (CHM) along section 3 (RG10/RCC90)
Figure 6.33: Clegg Hammer Modulus (CHM) along section 4 (RG15/RCC85)

Figure 6.34: Clegg Hammer Modulus (CHM) along section 5 (RG20/CR80)
Figure 6.35: Clegg Hammer Modulus (CHM) along section 6 (RG30/RCC70)

Figure 6.36: Clegg Hammer Modulus (CHM) along section 7 (CR100)
Figure 6.37: Clegg Hammer Modulus (CHM) along section 8 (RG10/CR90)

Figure 6.38: Clegg Hammer Modulus (CHM) along section 9 (RG30/CR70)

Figure 6.39 shows the summary of the CHM of different base sections. As seen in Figure 6.39, the CHM varies from 141 to 286 MPa, 69 to 203 MPa, 90 to 238 MPa, 74 to 276 MPa, 55 to 170 MPa, 85 to 220 MPa, 108 to 187 MPa, 74 to 141 MPa and 85 to 220 MPa for sections 1, 2, 3, 4, 5, 6, 7, 8 and 9, respectively. The higher values of the
base modulus indicate that the construction of 200 mm base sections with different proportion of RG/RCC and RG/CR blends over 200 mm subbase sections has significantly increased the CHM values of bases. As mentioned earlier, the large variations of CHM along the longitudinal sections and within the pavement base sections indicate that there were significant variations in subgrade as well as subbase modulus within each pavement section and between pavement trial base sections. This might be due to the heterogeneous nature of the subgrade, subbase as well as pavement base materials.

Figure 6.39: Summary of the Clegg Hammer Modulus (CHM) with different blends of RG/RCC or RG/CR of base section

Figure 6.40 shows the variation of CHM with respect to RG content in RG/RCC blends. From Figure 6.40, it can be seen that CHM decreases as expected with increase in RG contents which might be due to the brittle nature of the RG as compared to RCC. Figure 6.41 shows the variation of CHM with respect to RG content in RG/CR blends. From Figure 6.41, it can be seen that maximum CHM increases with increase in RG contents which might be due to the good quality in situ and imported materials used for the subgrade and subbase section for the sections composed of RG/CR blends. On the other hand, the average and minimum CHM do not show any significant variation of CHM for the sections composed of RG/CR blends.
In general, the results were too scattered most probable due to the variations of the nature of the in situ and imported materials used for the subgrade and subbase. Consequently, it was difficult to isolate the effects of glass content on the CHM of the RG/RCC or RG/CR blends.

Figure 6.40: Variation of CHM with respect to RG content in RG/RCC blends

Figure 6.41: Variation of CHM with respect to RG content in RG/CR blends
6.10.2 Base CBR and strength ratio

The results from the Clegg Hammer tests results were also analysed to determine CBR values of the various pavement sections as well as to determine the strength ratios after field compaction.

Figure 6.42 to Figure 6.50 present the Clegg Hammer results for CBR for the various pavement base sections with chainage along the longitudinal directions in different sections (Sections 1 to 9). From the figures, it can be concluded that the Clegg Hammer test results meet the minimum soaked field CBR of 100% for a base material in all sections except in RG10/CR90, RG10/RCC90, RG20/RCC80 and RG30/RCC70. The Clegg Hammer test results meet the specified minimum soaked field CBR of 80% for a subbase material in all sections except over short stretches of RG10/CR90, RG20/RCC80 and RG30/RCC70, for 10 to 20 m in which it was marginally below the specified requirements but is still deemed acceptable for haul roads.

The results seem to indicate that RG should be limited to pavement subbase applications and may not meet requirements for a pavement base material. The Clegg Hammer results seem to also indicate variation in the recycled blends within each pavement section and between pavement sections. Of the two control sections, RCC performed well as a pavement base material while CR section was on the border line at some short stretches while still performing satisfactorily. Both RCC and CR satisfied the requirements as a subbase material.

This can be attributed to the nature of the mixing of the recycled blends used. Limited blends of 20% RG with coarse sized recycled concrete aggregates (RG20/RCC80) and crushed rock aggregates (RG20/CR80) appears to be the optimum limits of glass additives with recycled aggregates based on the field testing results.

From the figures, it is observed that the CH CBR shows a large variation with values ranging from 52% to 226% for the base sections. This range of CBR along the longitudinal sections indicates that there were significant variations in subgrade as well as subbase modulus within each pavement section and between pavement trial sections.
Figure 6.42: Clegg Hammer CBR along section 1 (RCC100)

Figure 6.43: Clegg Hammer CBR along section 2 (RG20/RCC80)
Figure 6.44: Clegg Hammer CBR along section 3 (RG10/RCC90)

Figure 6.45: Clegg Hammer CBR along section 4 (RG15/RCC85)
Figure 6.46: Clegg Hammer CBR along section 5 (RG20/CR80)

Figure 6.47: Clegg Hammer CBR along section 6 (RG30/RCC70)
Figure 6.48: Clegg Hammer CBR along section 7 (CR100)

Figure 6.49: Clegg Hammer CBR along section 8 (RG10/CR90)
Figure 6.50: Clegg Hammer CBR along section 9 (RG30/CR70)

Figure 6.51 to Figure 6.59 present the Clegg Hammer strength ratio assessment for the various pavement base sections with chainage along the longitudinal directions in different sections (Sections 1 to 9). From the figures, it can be concluded that the Clegg Hammer strength ratio meet the minimum strength ratio of 100% for a base material in all sections except in RG10/CR90, RG10/RCC90, RG20/RCC80 and RG30/RCC70. The Clegg Hammer strength ratio meet the specified minimum strength ratio of 80% for a subbase material in all sections except over short stretches of RG10/CR90, RG20/RCC80 and RG30/RCC70, for 10 to 20 m in which it was marginally below the specified requirements but is still deemed acceptable for haul roads.

The results seem to indicate that RG should be limited to pavement subbase applications and may not meet requirements for a pavement base material. The Clegg Hammer strength ratio seems to also indicate variation in the recycled blends within each pavement section and between pavement sections. Of the two control sections, RCC performed well as a pavement base material while CR section was on the border line at some short stretches while still performing satisfactorily. Both RCC and CR satisfied the requirements as a subbase material.
This can be attributed to the nature of the mixing of the recycled blends used. Limited blends of 20% RG with coarse sized recycled concrete aggregates (RG20/RCC80) and crushed rock aggregates (RG20/CR80) appears to be the optimum limits of glass additives with recycled aggregates based on the field testing results.

From the figures, it is observed that the CH strength ratio shows a large variation with values ranging from 52% to 226% for the base sections. This range of strength ratio along the longitudinal sections indicates that there were significant variations in subgrade as well as subbase modulus within each pavement section and between pavement trial sections.

![Figure 6.51: Clegg Hammer strength ratio along section 1 (RCC100)](image-url)
Figure 6.52: Clegg Hammer strength ratio along section 2 (RG20/RCC80)

Figure 6.53: Clegg Hammer strength ratio along section 3 (RG10/RCC90)
Figure 6.54: Clegg Hammer strength ratio along section 4 (RG15/RCC85)

Figure 6.55: Clegg Hammer strength ratio along section 5 (RG20/CR80)
Figure 6.56: Clegg Hammer strength ratio along section 6 (RG30/RCC70)

Figure 6.57: Clegg Hammer strength ratio along section 7 (CR100)
Figure 6.58: Clegg Hammer strength ratio along section 8 (RG10/CR90)

Figure 6.59: Clegg Hammer strength ratio along section 9 (RG30/CR70)

Figure 6.60 shows the summary of the CH CBR of different base sections. As seen in Figure 6.60, the CH CBR varies from 118 to 226%, 63 to 164%, 79 to 190%, 67 to 218%, 52 to 140%, 75 to 177%, 93 to 152%, 67 to 118% and 75 to 177% for sections 1, 2, 3, 4, 5, 6, 7, 8 and 9, respectively. These large variations of CH CBR values along the
longitudinal sections and within the pavement base sections indicate that there were significant variations in subgrade as well as subbase CBR values within each pavement section and between pavement trial base sections. This might be due to the heterogeneous nature of the subgrade, subbase as well as pavement base materials.

Figure 6.61 shows the variation of CH CBR with respect to RG content in RG/RCC blends. From Figure 6.61, it can be seen that CHM decreases as expected with increase in RG contents which might be due to the brittle nature of the RG as compared to RCC.

Figure 6.62 shows the variation of CH CBR with respect to RG content in RG/CR blends. From Figure 6.62, it can be seen that maximum CH CBR increases with increase in RG contents which might be due to the good quality in situ and imported materials used for the subgrade and subbase section for the sections composed of RG/CR blends. On the other hand, the average and minimum CHM do not show any significant variation of CHM for the sections composed of RG/CR blends.

In general, the results were too scattered most probable due to the variations of the nature of the in situ and imported materials used for the subgrade. Consequently, it was difficult to isolate the effects of glass content on the CH CBR of the RG/RCC or RG/CR blends.
Figure 6.60: Summary of the Clegg Hammer CBR (%) with different blends of RG/RCC or RG/CR of base section

Figure 6.61: Variation of CBR with respect to RG content in RG/RCC blends
Figure 6.62: Variation of CBR with respect to RG content in RG/CR blends

6.11 Assessment of base stiffness using FWD deflection results

Information on the stiffness of the component pavement layers can be derived from the shape of the deflection bowl because different parts of this profile are influenced by different pavement layers. In principle, the maximum deflection, $D_0$, gives an indication of the overall pavement stiffness; whereas deflection curvature, $D_0 - D_{200}$, is mainly dependent on the stiffness of the combined 200 mm upper layers.

6.11.1 Asphalt Surface Deflection Results

FWD surveys were conducted on the asphalt surface for all of the pavements in order to provide a reference for the overall pavement stiffness after construction.

Table 6.10 summarises the FWD deflections (mean, standard deviations of maximum deflection, $D_0$, and deflection curvature, $D_0-D_{200}$) normalised to 700 kPa applied stress. The results show that:

- The maximum pavement deflections varied in the range of 230 to 370 μm. This indicated that there were significant variations in overall pavement stiffness between pavement trial sections.
The deflection curvatures also varied in the large ranges of 185 to 279 \( \mu \text{m} \). This also indicated that there were significant variations in base modulus between pavement trial sections.

Table 6.10: FWD initial mean \( D_0 \) (micron) and curvature (normalised to 700 kPa applied stress)

<table>
<thead>
<tr>
<th>Pavement section</th>
<th>Base type</th>
<th>Maximum deflection</th>
<th>Deflection curvature</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( D_0 ) (micron)</td>
<td>( D_0 - D_{200} ) (micron)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Mean</td>
<td>Std Dev</td>
</tr>
<tr>
<td>1</td>
<td>RCC100</td>
<td>258</td>
<td>38</td>
</tr>
<tr>
<td>3</td>
<td>RG20/RCC80</td>
<td>300</td>
<td>42</td>
</tr>
<tr>
<td>4</td>
<td>RG10/RCC90</td>
<td>234</td>
<td>37</td>
</tr>
<tr>
<td>2</td>
<td>RG15/RCC85</td>
<td>237</td>
<td>33</td>
</tr>
<tr>
<td>5</td>
<td>RG30/RCC70</td>
<td>259</td>
<td>35</td>
</tr>
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<td>8</td>
<td>RG20/CR80</td>
<td>369</td>
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</tr>
<tr>
<td>7</td>
<td>CR100</td>
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<td>34</td>
</tr>
<tr>
<td>6</td>
<td>RG10/CR90</td>
<td>323</td>
<td>27</td>
</tr>
<tr>
<td>9</td>
<td>RG30/CR70</td>
<td>341</td>
<td>39</td>
</tr>
<tr>
<td>All</td>
<td></td>
<td>299</td>
<td>63</td>
</tr>
</tbody>
</table>

6.11.2 Pavement Stiffness

Deflection and deflection-difference plots are useful for showing relative differences in the condition of the layers, and enable delineation of the pavement trial sections with similar behaviour, and giving an indication of where structural weakness may be present.

Figure 6.63 compares the FWD maximum deflections measured on the finished pavement surfaces for all pavement trial sections. As can be seen in Figure 6.63, pavement deflections vary between 234 and 369 micron for all pavement sections. This indicates that the construction of the 200 mm base and 50 mm glassphalt has significantly increased the overall pavement stiffness. Generally, pavement sections
with recycled crushed concrete blends produced lower FWD maximum deflections and, hence, higher overall pavement stiffness, than pavement trial sections with crushed rock blends. This is possibly due to the partial re-cementing action that commonly occurs with recycled crushed concrete pavement layers resulting in higher pavement layer stiffness.

Similarly, Figure 6.64 compares FWD deflection curvatures measured on the finished pavement surfaces for all pavement trial sections. As can be seen in Figure 6.64, pavement deflection curvatures vary between 114 and 184 micron for all pavement sections. Generally, the pavement sections with recycled crushed concrete blends produced lower deflection curvatures and hence, higher base stiffness than the pavement sections with crushed rock blends. As mentioned earlier, this is might be due to the partial re-cementing action that commonly occurs with recycled crushed concrete pavement layers resulting in higher pavement layer stiffness.

Figure 6.63: Comparison of FWD maximum deflections measured on the finish pavement surfaces
Figure 6.64: Comparison of FWD maximum deflection curvatures measured on the finish pavement surfaces

6.11.3 Asphalt surface composite modulus using FWD maximum deflection ($D_0$)

The FWD maximum deflection ($D_0$) can be used to calculate the composite modulus ($E_0$) assuming constant loading on the Boussinesq’s theory (Fleming 2001; Livneh and Goldberg 2001; Deng 2006) using the following equation.

$$E_0 = \frac{2 \times (1 - \nu^2) \times \sigma_0 \times a}{D_0}$$

Equation 6.6

where,
- $E_0$ = composite modulus (MPa)
- $\sigma_0$ = contact pressure at load plate (kPa)
- $a$ = radius of the load plate (mm)
- $D_0$ = deflection at the center of the load plate (μm)
- $\nu$ = Poisson's ratio

In this research, the contact pressure at load plate was 700 kPa for asphalt surface. The radius of the load plate was 150 mm. Austroads assume the poison's ratio for subbase
quality material as 0.35 (Austroads 2008). Therefore, for simplification, the Poisson’s ratio was also assumed as 0.35 for asphalt surface.

Figure 6.65 to Figure 6.73 show the variation of composite modulus ($E_0$) with chainage along the longitudinal direction in different sections (Sections 1 to 9) on asphalt surfaces. Due to different boundary conditions and constant values used for calculating the composite modulus of FWD and CHM, they are not always comparable.

From the figures, it is observed that the composite modulus ($E_0$) shows a large variation with values ranging from 282 MPa to 818 MPa for the asphalt surface. This range of composite modulus along the longitudinal sections indicates that there were significant variations in subgrade, subbase as well as base modulus within each pavement section and between pavement trial sections.

![Figure 6.65: FWD Composite Modulus along section 1 (Base RCC100)](image-url)
Figure 6.66: FWD Composite Modulus along section 2 (Base RG20/RCC80)

Figure 6.67: FWD Composite Modulus along section 3 (Base RG10/RCC90)
Figure 6.68: FWD Composite Modulus along section 4 (Base RG15/RCC85)

Figure 6.69: FWD Composite Modulus along section 5 (Base RG20/CR80)
Figure 6.70: FWD Composite Modulus along section 6 (Base RG30/RCC70)

Figure 6.71: FWD Composite Modulus along section 7 (Base CR100)
Figure 6.72: FWD Composite Modulus along section 8 (Base RG10/CR90)

Figure 6.73: FWD Composite Modulus along section 9 (Base RG30/CR70)

Figure 6.74 shows the summary of the composite modulus of different base sections. As shown in Figure 6.74, the composite modulus varies from 375 to 658 MPa, 464 to 744 MPa, 391 to 614 MPa, 440 to 818 MPa, 409 to 675 MPa, 355 to 512 MPa, 351 to 553 MPa.
MPa, 282 to 690 MPa and 319 to 514 MPa for sections 1, 2, 3, 4, 5, 6, 7, 8 and 9 respectively.

In general, the pavement sections with RG/RCC blends produced higher composite modulus than the pavement sections with RG/CR blends. This might be due to the partial re-cementing action that commonly occurs with recycled crushed concrete pavement layers resulting in higher pavement layer stiffness.

**Figure 6.74:** Summary of the FWD Composite Modulus of Asphalt Surface with different blends of RG/RCC or RG/CR of base section

Figure 6.75 shows the variation of composite modulus with respect to RG content in RG/RCC blends. From Figure 6.75, it can be seen that composite modulus is increasing with increase in RG contents rather than decreasing as expected due to the brittle nature of the RG as compared to RCC. This might be due to the good quality in situ and imported materials used for the subgrade and subbase section for the sections composed of RG/RCC blends.

Figure 6.76 shows the variation of composite modulus with respect to RG content in RG/CR blends. From Figure 6.76, it can be seen that maximum composite modulus is decreasing with increase in RG contents as expected due to the brittle nature of the RG.
as compared to CR. On the other hand, the average and minimum composite modulus do not show any significant variation for the sections composed of RG/CR blends.

In general, the results were too scattered most probable due to the variations of the nature of the in situ and imported materials used for the subgrade. Consequently, it was difficult to isolate the effects of glass content on the composite modulus of the RG/RCC or RG/CR blends.

![Graph showing FWD Composite Modulus variation with RG content in RG/RCC blends]

Figure 6.75: Variation of FWD Composite Modulus with respect to RG content in RG/RCC blends
Figure 6.76: Variation of FWD Composite Modulus with respect to RG content in RG/CR blends

6.11.4 Placement of CR or RCC blends with RG additive

As the RG used as additive with CR or RCC blends was essentially non plastic, it was expected to detract from the workability and cohesion characteristics of the crushed rock and in particular the recycled crushed concrete blends. However, while some minor issues were experienced with the placement of the recycled glass blends, in reality, these issues were far less than originally anticipated.

In terms of the placement of the CR and RCC blends provided that the moisture content of the placed material was kept close to OMC, no perceived issues were observed. The pugmilling of products prior to placement obviously aided the uniformity of the placed material, with respect to both moisture content and grading, and so segregated or bony patches were rarely observed.

With regard to the final prepared surface, it was noticed that the RCC with RG was not as tight showing a tendency to ravel on the prepared surface in some areas as the crushed rock with RG additive. This is probably due to the additional PI in the crushed rock and the extra cohesion imparted by the clay component.
Although the levels of compaction achieved in both materials indicated that the materials could be potentially ‘density sensitive’ and may require close control of density (uniform and defined roller routine). Overall, the crushed rock/recycled crushed concrete blends with 15% glass additive appeared to be the easiest to place and yielded the best compaction results.

6.12 Findings based on field performance

The construction of the nine trial pavement sections was successfully completed and to date, no significant problems have occurred with the performance of each trial section. Some significant issues encountered during the construction of the pavement trial sections are summarised below:

- Significant variation was present in subgrade CBR within each pavement section and between pavement sections due to the nature of the in situ and imported materials used.
- Small variations were evident in base course thickness within each of nine pavement sections.

The results confirmed that some difficulties were experienced in the construction of the base layers using the RG/RCC blends and the RG/CR blends. While these problems are not desirable, they probably represent what happens in the real world with the construction of non-homogenous pavements on arterial and local roads. These types of roads would typically be the target use for crushed glass and other recycled products.

In comparing with the Local State Authority’s compaction criteria for base, it was found that:

- Most base pavement sections containing recycled glass content less than 20% achieved the target mean value of density ratio of 98% for subbase layer; whereas all base pavement sections struggled to meet the minimum required compaction criteria for base layer, i.e. less than the target mean value of density ratio of 100%.
This suggested that the recycled blends only meet VicRoads specifications on Class 3 subbase materials, but would not meet the VicRoads specifications on Class 2 base materials. The field trial also indicated that blends containing a recycled glass content greater than 20% would likely result in a lower field dry density being achieved.

In comparing with the VicRoads drying back specifications of base prior to bituminous surfacing, it appears that all pavement sections complied with the specifications concerned, possibly due to the timing construction to take place during the summer, which was expected to aid dry back. As a result of good drying back, all pavements developed quite high stiffness, with FWD mean maximum deflection ($D_0$) normalised to 700 kPa applied stress varying in the range of 365 to 585 $\mu$m. The crushed concrete blends also produced higher stiffness than the crushed rock blends. This was consistent with the results of resilient modulus determined with the laboratory RLT test methods. The Clegg Hammer Modulus (CHM) also shows the similar trend (Average CHM for RG/RCC blends is higher than the average CHM of RG/CR blends).

It is recommended that field performance monitoring (FWD test and roughness test) of the pavement sections will also be made to enable better interpretation of the long-term performance results. Although the nine test pavements are not uniform as desirable, there is sufficient detailed thickness, density and deflection data collected during the construction at exact locations along each 80 m pavement section for each pavement section to act as a realistic trial of crushed glass as an additive.

Finally, based on the field testing results, it can be concluded that limited blends of 20% RG with coarse sized RCC (RG20/RCC80) and CR, (RG20/CR80), appears to be the optimum limits of RG content with recycled aggregates for the trial pavements. Of the control sections, RCC performed well as a pavement base material while CR section was border line at some short stretches while still performing satisfactorily Both RCC and CR met the requirements as a subbase material. RG was found to be a viable additive when used in limited proportions with other recycled aggregates in pavement applications.
CHAPTER SEVEN

7 FINITE ELEMENT MODELLING OF RECYCLED GLASS BLENDS AS PAVEMENT MATERIALS

7.1 Introduction

Finite element modelling of the pavement was carried out using PLAXIS 2D (PLAXIS 2D Version 8.5, 2009) finite element software. Due to the circular contact area of the wheel load, the axisymmetric model was used to analyse the pavement in this research.

In the axisymmetric model, the deformations and stress state are assumed to be identical in any radial direction. The side vertical boundaries of the road pavement section were specified as free in vertical direction. The bottom boundary was specified as fixed in both directions. The geometry model of the typical road pavement section is presented in Figure 7.1.

![Figure 7.1: Typical geometry model of pavement structure](image)

Due to the much finer and much more flexible nature of the meshes composed of 15 node elements, finite element meshes were generated by using fifteen (15) nodes triangular elements instead of six (6) nodes elements. More fine mesh elements were generated in the clusters around the loading area as this is the critical area for analysis. The mesh coarseness is selected as “fine” for element distribution in the model as a finer mesh element gives more accurate results. Figure 7.2 and Figure
7.3 shows the finite element mesh and mesh with nodes for the geometry model of typical pavement structure, respectively. The stress points of mesh for geometry model of typical pavement structure are shown in Figure 7.4.

Figure 7.2: Finite element mesh for the geometry model of typical pavement structure

Figure 7.3: Finite element mesh with nodes for the geometry model of typical pavement structure

Figure 7.4: Finite element mesh with stress points for the geometry model of typical pavement structure
In this research, Mohr-Coulomb (linear elastic-perfectly plastic) material model was selected for granular materials and subgrade materials. Asphalt material was modelled with linear elastic material model.

7.2 Unbound granular pavement

Unbound granular pavement is a flexible pavement with thin bituminous seal surface and carries lightly traffic loads. Granular pavements with CR100, RCC100 and 15% RG in blends with 85% CR and 85% RCC as granular materials were modelled with axisymmetric concept in PLAXIS 2D (PLAXIS 2D Version 8.5, 2009).

As the sprayed seal surface does not act as a structural element and it prevents permeability of water through the top of the pavement surface, its thickness of the pavement had been neglected. The granular layer had been divided into five (5) layers with the same physical properties and shear strength parameters but with the different modulus values to consider the stress dependent behaviour of pavement structures.

Figure 7.5 shows the components of granular pavement and depth of locations selected for vertical deformation. The thickness of the granular materials was considered as 475 mm and was divided into equal 5 layers of 95 mm thickness.

![Diagram of granular pavement](image-url)
7.3 Recycled Materials as Unbound Granular Pavement Materials

The Mohr–Coulomb material model was used for all material in the analysis. Properties of the granular materials were derived from laboratory experiments and the properties subgrade material were assumed and the input parameters of the pavement materials for modelling are summarised in Table 7.1. Here, the dilatancy angle \( \psi \) has been calculated as follows:

\[
\text{Dilatancy angle, } \psi (\degree) = \phi' (\degree) - 30
\]  
Equation 7.1

<table>
<thead>
<tr>
<th>Parameters</th>
<th>CR100</th>
<th>RCC100</th>
<th>RG15/CR85</th>
<th>RG15/RCC85</th>
<th>Subgrade</th>
</tr>
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<tr>
<td>( \gamma_{\text{unsat}} ) (kN/m(^3))</td>
<td>22.56</td>
<td>19.33</td>
<td>21.97</td>
<td>19.33</td>
<td>16.00</td>
</tr>
<tr>
<td>( \gamma_{\text{sat}} ) (kN/m(^3))</td>
<td>24.52</td>
<td>22.00</td>
<td>23.85</td>
<td>21.69</td>
<td>18.03</td>
</tr>
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<td>( k_x ) (m/s)</td>
<td>6.1\times10^{-8}</td>
<td>7.0\times10^{-8}</td>
<td>6.8\times10^{-8}</td>
<td>3.8\times10^{-7}</td>
<td>1.0 \times 10^{-8}</td>
</tr>
<tr>
<td>( k_y ) (m/s)</td>
<td>6.1\times10^{-8}</td>
<td>7.0\times10^{-8}</td>
<td>6.8\times10^{-8}</td>
<td>3.8\times10^{-7}</td>
<td>1.0 \times 10^{-8}</td>
</tr>
<tr>
<td>( \nu )</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.45</td>
</tr>
<tr>
<td>( c_{\text{ref}} ) (kN/m(^2))</td>
<td>69.6</td>
<td>75.4</td>
<td>59.3</td>
<td>51.2</td>
<td>5.0</td>
</tr>
<tr>
<td>( \phi' ) (\degree)</td>
<td>48.3</td>
<td>51.0</td>
<td>47.2</td>
<td>48.1</td>
<td>35.0</td>
</tr>
<tr>
<td>( \psi ) (\degree)</td>
<td>18.3</td>
<td>21.0</td>
<td>17.2</td>
<td>18.1</td>
<td>0</td>
</tr>
<tr>
<td>Material model</td>
<td>MC</td>
<td>MC</td>
<td>MC</td>
<td>MC</td>
<td>MC</td>
</tr>
<tr>
<td>Material type</td>
<td>Drained</td>
<td>Drained</td>
<td>Drained</td>
<td>Drained</td>
<td>Drained</td>
</tr>
<tr>
<td>Strength</td>
<td>Rigid</td>
<td>Rigid</td>
<td>Rigid</td>
<td>Rigid</td>
<td>Rigid</td>
</tr>
</tbody>
</table>

The unbound pavement material cannot be properly compacted on a soft subgrade. Therefore, the modulus of any unbound pavement layer is not an intrinsic property of the component material, but is primarily dependent on the stiffness of the materials of underlying layers. Therefore, sub-layering is required for granular materials placed directly on the subgrade (Austroads 2008). Moffat and Jameson (1998a) used sub-layering in multilayer elastic models refining the original Austroads procedures for mechanistic design of new unbound granular pavements and proposed the following procedures:

(a) Granular materials should be divided into 5 layers of equal thickness
(b) The modular ratio of successive sub-layers should be determined as follows:

\[
R = \left( \frac{E_{\text{topgranularsublayer}}}{E_{\text{subgrade}}} \right)^{1/2}
\]

Equation 7.2

(c) The modulus of each layer beginning with that immediately overlying the subgrade should be calculated based on the known modulus of subgrade.

Here the modulus of subgrade was assumed as 50 MPa (Austroads 2008) and the modulus of other granular materials was taken from the secant modulus calculated from consolidated drained triaxial compression tests. The modular ratio, R of adjacent layers was calculated from equation 7.1. For example, the secant modulus of CR100 was calculated as 93 MPa based on the consolidated drained triaxial tests. Therefore the modular ratio, R for the CR100 granular material was calculated as 1.14. Hence, the modulus of the first granular sub layer for CR100 above the subgrade was calculated as follows:

\[
E_v \text{first sub layer} = R \times E_{v\text{subgrade}} = 1.14 \times 50 = 57 \text{ MPa}
\]

Equation 7.3

The modulus of the other granular sub layers were calculated according to the above calculation and summarised in Table 7.2.

<table>
<thead>
<tr>
<th>Table 7.2: Secant modulus of pavement materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>Materials</td>
</tr>
<tr>
<td>Sub Layers</td>
</tr>
<tr>
<td>1\text{st} sub layer</td>
</tr>
<tr>
<td>2\text{nd} sub layer</td>
</tr>
<tr>
<td>3\text{rd} sub layer</td>
</tr>
<tr>
<td>4\text{th} sub layer</td>
</tr>
<tr>
<td>5\text{th} sub layer</td>
</tr>
</tbody>
</table>

According to Austroads “Guide to Pavement Technology Part 2: Pavement Structural Design” (Austroads 2008) guidelines, a vertical load of 20 kN is applied to each circular area at a uniform vertical stress distribution of 750 kPa. The radius (r) of each loaded area is calculated as follows:
\[ r = 2523 \times p^{-0.5} \text{ (About 92.1 mm for highway traffic)} \]  

Equation 7.4

where,

\[
\begin{align*}
    r &= \text{radius (mm)} \\
    p &= \text{vertical stress (kPa)}
\end{align*}
\]

**Loading Stage**

An axisymmetric model, 750 kPa load had been applied over a loaded area of radius 92.1 mm. The deformed mesh after loading is shown in Figure 7.6. Figure 7.7 shows the vertical displacement profile as shaded regions.

![Deformed mesh of unbound granular pavement material (RCC100) while loading](image)

**Figure 7.6:** Deformed mesh of unbound granular pavement material (RCC100) while loading

It has been noted from Figure 7.7 that the extreme vertical displacement occurred on top of the granular layer (RCC100) as loading is applied over the granular sub layer and this sub layer is under high stress. The vertical displacement is gradually decreasing with depth of the granular material as well as with horizontal distance from the loading location.
Figure 7.7: Vertical displacement profile for unbound granular pavement material (RCC100) while loading

Figure 7.8 to Figure 7.11 shows the vertical displacement of various locations of pavement from outer edge of the load while loading for RCC100, RG15/RCC85, CR100 and RG15/CR85, respectively. The vertical displacement below the subgrade shows negligible displacement during loading phase and the displacements vary significantly with horizontal distance. Vertical deformation is higher at top of granular material while loading as the top of granular layer is directly beneath the loaded area and it carries the more stress while loading. Bottom of granular and top of the subgrade also show similar vertical deformation beneath the loading as the location of both of them is very close to each other. Vertical deformation is decreasing significantly with the distance from the load at top of granular layer, bottom of granular and top of subgrade. Out of the four types of granular materials modelled using PLAXIS software, the RCC100 material shows less vertical deformation and RG15/RCC85 shows maximum vertical deformation. This might be due to higher shear strength parameter of the RCC100 materials as compared to other granular materials.
Figure 7.8: Vertical displacement of the selected regions from the outer edge of the load under loading condition (RCC100)

Figure 7.9: Vertical displacement of the selected regions from the outer edge of the load under loading condition (RG15/RCC85)
Figure 7.10: Vertical displacement of the selected regions from the outer edge of the load under loading condition (CR100)

Figure 7.11: Vertical displacement of the selected regions from the outer edge of the load under loading condition (RG15/CR85)
Unloading Stage

An unloading phase was simulated by deactivating the applied load. The deformed mesh after unloading is shown in Figure 7.12. Figure 7.13 shows the vertical displacement profile as shaded regions for unloading condition.

Figure 7.12: Deformed mesh of unbound granular pavement material (RCC100) while unloading

Figure 7.13: Vertical displacement profile for unbound granular pavement material (RCC100) while unloading

Figure 7.14 to Figure 7.17 show the vertical deformation at various locations of pavement elements with RCC100, RG15/RCC85, CR100 and RG15/CR85, respectively as granular material after unloading. Similar to the loading phase, the vertical displacement below the subgrade shows negligible displacement during unloading phase and the displacements vary significantly with horizontal distance for top of
subgrade, bottom of granular and top of granular layers. The vertical displacements decrease significantly with horizontal distance. Similar vertical deformations have been identified at the bottom of granular and top of subgrade during the unloading phase due to the very close location of each other. On the other hand, a small heaving has been identified at top of the granular material, bottom of the granular and top of subgrade close to the inner edge of unloading location. It might be due to the relative movements in loaded and unloaded boundaries. Vertical deformation is decreasing significantly with the distance from the load at top of granular layer, bottom of granular and top of subgrade. Out of the four types of granular materials modelled using PLAXIS software, the CR100 material shows less vertical deformation and RG15/RCC85 shows maximum vertical deformation. This might be due to higher density and shear strength parameter of the CR100 materials as compared to other granular materials.

Figure 7.14: Vertical displacement unbound granular pavement material (RCC100) while unloading
Figure 7.15: Vertical displacement of the selected regions from the outer edge of the load under unloading (RG15/RCC85)

Figure 7.16: Vertical displacement unbound granular pavement material (CR100) while unloading
Figure 7.17: Vertical displacement of the selected regions from the outer edge of the load under unloading (RG15.CR85)

Figure 7.18 shows the loading and unloading cycles up to thirty (30) cycles of granular pavement with RCC100 and RG15/CR85 blends as granular materials. It has been noted that the extreme vertical deformation after each loading cycle for RG15/RCC85 is higher than that of the RCC100, but the pattern of extreme vertical deformation after each loading cycles is almost similar. However, the extreme vertical deformation after unloading cycle is increasing with number of cycles. It shows that more plastic deformation is occurring during loading and it would indicate that the pavement experienced higher permanent deformation during loading. Similar permanent deformation trend also had been identified in the repeated load triaxial testing.
Figure 7.18: Extreme vertical deformation of granular pavement (RCC100 and RG15/RCC85) after loading and unloading cycles.

Figure 7.19 shows the loading and unloading cycles of granular pavement with CR100 and RG15/CR85 blend as granular materials. Similar to the RG/RCC blends, it has also been noted that the extreme vertical deformation after each loading cycle for CR100 and RG15/CR85 are almost similar and the pattern of extreme vertical deformation after each loading cycle is also similar. However, the extreme vertical deformation after unloading cycle is increasing with number of cycles. It shows that more plastic deformation is occurring during loading and it would indicate that the pavement experienced higher permanent deformation during the loading phase.
Figure 7.19: Extreme vertical deformation of granular pavement (CR100 and RG15/CR85) after loading and unloading cycles

Comparison of vertical displacement in unbound granular pavements

Figure 7.20 shows the extreme vertical displacement of granular pavements with various granular materials (e.g., CR100, RCC100, RG15/CR85 and RG15/RCC85). It can be noted that RG15/CR85 and RG15/RCC85 granular pavements show higher extreme vertical displacement and RCC100 shows the lower extreme vertical displacement during loading. Figure 7.20 also shows the extreme vertical displacements stay almost constant with number of cycles. During the unloading phase, even though, all the granular pavements show similar extreme vertical displacement at initial cycles, the extreme vertical displacement of all pavement materials is slightly increasing with cycles. It might be due to the fact that some plastic deformation has occurred during the loading phase.

It could be noted that the shear strength properties of RCC100 are higher as compared to the other blends considered among these four granular materials. It could be concluded that RCC100 granular materials performed well compared to other three granular materials in terms of vertical displacement and loading and unloading cycles.
Table 7.3 shows the extreme vertical displacement of the four blends modelled using Plaxis. Among the four blends RG15/RCC85 shows the higher extreme vertical displacement during the first loading and unloading phases and RCC100 shows the lowest extreme vertical displacement during the same loading and unloading phases.

<table>
<thead>
<tr>
<th>Materials</th>
<th>During loading (µm)</th>
<th>During unloading (µm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CR100</td>
<td>1530</td>
<td>18.18</td>
</tr>
<tr>
<td>RG15/CR85</td>
<td>1540</td>
<td>21.37</td>
</tr>
<tr>
<td>RCC100</td>
<td>1350</td>
<td>21.37</td>
</tr>
<tr>
<td>RG15/RCC85</td>
<td>1540</td>
<td>25.56</td>
</tr>
</tbody>
</table>

**Findings and Discussions**

Granular pavements with thin bituminous or sprayed seal pavements carry the load to the subgrade through granular layers. Significant contribution of shear strength properties of granular materials has been found in granular pavements in terms of
vertical deformation. Higher the shear strength properties of granular material indicate lower the deformation. In addition, deformation of granular pavement while loading is almost consistent. On the other hand, deformation after unloading is slightly increasing as more permanent deformation could occur under repetitive loading.

The deformation of RG15/RCC85 granular pavement after unloading cycle is comparably higher and slightly increasing with cycles, compared to the conventional crushed rock and other granular materials. In contrast, RCC100 granular material showed comparably lower deformation than that of the other granular materials.

It would suggest that RCC100 would perform well as granular material in terms of deformation compared to the conventional recycled crushed rock and RG15/RCC85 granular materials.
CHAPTER 8

8 CONCLUSIONS

This research was undertaken to investigate the geotechnical properties of recycled glass in blends with recycled crushed concrete and crushed rock. Extensive laboratory tests had been undertaken on a suite of blended mixtures of 10%, 15%, 20%, 30%, 40% and 50% of recycled glass with recycled crushed concrete and crushed rock. Based on the geotechnical properties of blends, potential use of these blends as granular pavement material has been examined with referring to the local state road authority’s standards specifications. The following conclusions can be drawn from this research study.

8.1 Characterization of RG (Recycled Glass)

- RG possesses a specific gravity value of 2.49 which is approximately 10% lower than the specific gravity of natural aggregates ranging from 2.60 to 2.83.

- The maximum dry density obtained for RG sample is approximately 10% lower than the values of 1.93-2.04 Mg/m³ found for natural aggregate with the same soil classification (Craig 1997).

- RG is classified as well graded sand mixed with little amount of silt size particles.

- The CBR value of RG is far below that of natural aggregates.

- The Los Angeles abrasion value of RG is slightly higher than the natural aggregates but well below the VicRoads specifications. This might be due to the result of a higher debris level and brittle nature of RG particles.

- The internal friction angle of RG declined from 55° to 46° with normal stress increasing from 25 kPa to 400 kPa. The internal friction angle of recycled glass
is found to be similar to that of dense sand with angular grains (Das 2008). This suggests that the recycled glass source exhibits the satisfactory friction characteristics for usage in some geotechnical engineering applications.

- The properties of the RG could be enhanced by blending it with other recycled aggregates to improve its performance in pavement subbase applications.

### 8.2 Characterization of RG/RCC blends

The RG/RCC blends with up to 50% RG additive were tested using laboratory modified compaction test, grading test and CBR test, etc. The following test results were noted:

- The results of laboratory modified compaction tests indicated that RCC blends with a higher RG additive content would potentially produce a lower maximum dry density.

- The results of after compaction grading curves indicated that RCC blends with greater than 40% RG additive would potentially produce grading exceeding the upper limits specified by the road authorities.

- The results of CBR testing indicated RCC blends with a higher RG additive content would potentially reduce their soaked CBR values. For the RCC blend, an increase of 50% RG content would reduce the soaked CBR value by 50% (i.e. from CBR 200% to CBR 100%). However, all the blends had soaked CBR values greater than 80% which satisfies the requirements on CBR for Class 3 RCC subbase.

The RCC blends with up to 50% RG additive were tested using consolidated drained triaxial compression test method for the determination of shear strength parameters (effective cohesion, $c'$ and effective angle of internal friction, $\phi'$). The following results were noted:
The internal friction angle varies from 44.7˚ and 51˚ and the cohesion ranges between 33.4 kPa and 91.2 kPa. Generally, all blends attain high shear strength parameters.

The RCC blends with up to 30% RG additive were also tested using advanced laboratory repeated load triaxial test methods for mechanical properties including permanent strain and resilient modulus. The following results were noted:

- The laboratory performances (in terms of both permanent deformation and resilient modulus) of the RCC blends tested are superior compared to those of natural CR used for the construction of subbase.

- For the RCC blends, permanent strain was not sensitive to RG content. However, higher RG content would potentially produce lower resilient modulus.

Based on the above results obtained from laboratory compaction, after compaction grading and performance tests (including CBR test, RLT permanent strain and resilient modulus tests), it was decided to select CR and RCC blends with up to 30% RG additive for further field testing to assess the constructability of the RG blends and to determine their long-term field performance for comparison with the laboratory performance.

### 8.3 Characterization of RG/CR blends

The RG/CR blends with up to 50% RG additive were also tested using laboratory modified compaction test, grading test and CBR test, etc. The following test results were noted:

- The results of laboratory modified compaction tests indicated that RG/CR blends with a higher RG additive content would potentially produce a lower maximum dry density.
The results of after compaction grading curves indicated that RG/CR blends with greater than 50% RG additive would potentially produce grading exceeding the VicRoads upper limits.

The results of CBR testing indicated that RG/CR blends with a higher RG additive content would potentially reduce their soaked CBR values. For the RG/CR blends, an increase of 50% RG content in the CR would reduce the soaked CBR value by 30% (i.e. from CBR 180% to CBR 120 %). However, all the blends had soaked CBR values greater than 80%. This satisfies the Local State Authority’s requirements on CBR for Class 3 CR subbase.

The CR and RCC blends with up to 50% RG additive were tested using consolidated drained triaxial compression test method for the determination of shear strength parameters (effective cohesion, \( c' \) and effective angle of internal friction, \( \phi' \)). The following results were noted:

- The internal friction angle varies between 46.5° and 50.3° and the effective cohesion ranges between 29.8 kPa and 59.3 kPa. Generally, all blends attain high shear strength parameters.

The CR blends with up to 30% RG additive were also tested using advanced laboratory repeated load triaxial test methods for mechanical properties including permanent strain and resilient modulus. The following results were noted:

- The laboratory performances (in terms of both permanent deformation and resilient modulus) of the CR blends tested are comparable to those of natural CR used for subbase.

- For the CR blends, higher RG content would potentially produce higher permanent strain. However, this trend is not consistent throughout the results especially for stage 1 and this might be due to the variation of moisture content which makes it difficult to establish a trend for permanent strain. On the other hand, the resilient modulus was not sensitive to changes in either moisture content or RG content.
Plaxis finite element modelling suggests that RCC100 would perform well as granular material in terms of deformation compared to the conventional recycled crushed rock and RG15/RCC85 granular materials.

8.4 Field Performances

The construction of the nine trial pavement sections was successfully completed and to date, no significant problems have occurred with the performance of each trial section.

Some significant issues encountered during the construction of the pavement trial sections are summarised below:

- Significant variation was present in subgrade CBR within each pavement section and between pavement sections due to the nature of the in situ and imported materials used.

- Small variations were evident in base course thickness within each of nine pavement sections. This was perhaps largely due to the difficulties related to placing the subbase and the variation in subbase and subgrade thickness.

The results confirmed that some difficulties were observed in the construction of the base layers using the RG/RCC blends and the RG/CR blends. While these problems are not desirable, they probably represent what happens in the real world with the construction of non-homogenous pavements on arterial and local roads (equivalent to VicRoads Scale C roads). These types of roads would typically be the target use for crushed glass and other recycled products.

In comparing with the Local State Authority’s compaction criteria for subbase and base, it was found that:
Most pavement base sections containing recycled glass content less than 20% achieved the target mean value of density ratio of 98% for subbase layer; whereas all base pavement sections struggled to meet the minimum required compaction criteria for base layer, i.e. less than the target mean value of density ratio of 100%. This suggested that the recycled blends only meet VicRoads specifications on Class 3 subbase materials, but would not meet the VicRoads specifications on Class 2 base materials. The field trial also indicated that blends containing a recycled glass content greater than 20% would likely result in a lower field dry density being achieved.

In comparing with the drying back specifications of base prior to bituminous surfacing, it appears that all pavement sections complied with the specifications concerned. As a result of good drying back, all pavements developed quite high stiffness, with FWD mean maximum deflection ($D_0$) normalised to 700 kPa applied stress varying in the range of 365 to 585 µm. The crushed concrete blends also produced higher stiffness than the crushed rock blends. This was consistent with the results of resilient modulus determined with the laboratory RLT test method.

### 8.5 Recommendation for Future Study

Present research suggest that initially up to 30% “4.75 mm minus” RG could be safely added to Class 3 CR or Class 3 RCC. Depending on the results of future field trials with long term performance monitoring, it may be possible to increase the percentage of RG in the mixtures. Therefore, further research is required to monitor the long term performance of subbase/light duty base constructed using the recycled material.
REFERENCES


VicRoads (2006c). Standard Specifications for Road works and Bridge works, Section 812, Crushed rock for base and subbase pavement, July.


VicRoads (2009a). Standard Specifications for Road works and Bridge works, Section 812, Crushed rock for base and subbase pavement, July.


WSDOT Test Method T 611-11. “Method of Test for Determination of the Resistance (R-Value) of Untreated Bases, Subbases, and Basement Soils by the Stabilometer.”

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Appendix 1: Refereed Journal Articles


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Appendix 2: Refereed Conference Papers


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