Faculty of Science and Engineering

Department of Civil Engineering

Investigation of Effects of Rehydration of Cement in Recycled Crushed Concrete Road base

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DECLARATION

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

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

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ABSTRACT

The use of recycled crushed concrete (RCC) as a road base has been increasing slowly in Western Australia, despite efforts by recycling companies and the Waste Authority to divert this fully recallable material from landfill. Landfill space is becoming increasingly difficult to find, and environmental requirements placed on landfill with leachate control and high cost lining make the volume of landfill too valuable to be taken up by inert materials.

Despite the high structural strength of RCC roadbase, it has not been adopted by many road Engineers and authorities due to the tendency to gain very high strength and exhibit shrinkage cracking. This research is aimed at investigating ways to limit the excessive strength gain exhibited by RCC roadbase. This strength gain is thought to stem from the rehydration of the Portland cement content contained in the RCC. Two methods are examined; firstly by blending with inert non pozzolanic materials to limit the capability of cementitious bonds to reform, and secondly by the forced development of microcracks by recompaction during the early stages of curing.

Two materials were investigated for the blending of non pozzolanic materials, a blend of crushed brick and tile also sourced from recycling of demolition materials, and ferricrete, a material produced from the crushing of lateritic caprock generated by the excavation of cells for landfill at the Red Hill waste disposal site operated by the Eastern Metropolitan Regional Council. The research showed that the blending of either of the crushed brick and tile or ferricrete reduced linear shrinkage but the strength of the materials as determined by the Unconfined Compressive Strength Test (UCS) was little affected by the blending of these materials.

In order to determine the effects of post construction recompaction to induce microcracks, a nonstandard test was developed. Again this test failed to show the desired effects as strength remained high, but reasons for this are developed and guidance on future extension of this method is provided.

Keywords: Recycled crushed concrete (RCC), Brick and Tile, Ferricrete, Unconfined Compressive Strength Test (UCS).

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RESEARCH PUBLICATIONS

Conference Paper

- ✓ Effects of Rehydration of Cement in Recycled Crushed Concrete Road Base "2014 AASRI International Conference on Applied Engineering Sciences (ICAES 2014) July 23-24, 2014 Hollywood (Los Angeles)".

Report

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Acronyms and Abbreviations

C&D	Construction and demolition
CF	Curvature function
CRB	Crushed road base
CRC	Crushed recycled concrete
IPWEA	Institute of Public Works Engineering Australia
LA	Los Angeles abrasion test
LL	Liquid Limit
LS	Linear Shrinkage
MDCS	Maximum dry compressive strength
MDD	Maximum dry density
MRWA	Main Roads Western Australia
Ev	Elastic compressive modulus
OMC	Optimum moisture content
PL	Plastic limit
PSD	Particle size distribution
RBMA	Recycled brick masonry aggregate
RBMAC	Recycled brick masonry aggregate concrete
RLTT	Repeat load triaxial test
RCA	Recycled crushed aggregate
RCC	Recycled crushed concrete
UCS	Unconfined compressive strength
UTM	Universal testing machine
WALGA	Western Australian Local Government Association
C&I	Commercial & Industrial
RCP	Reinforced Concrete Pavement
CRCP	Continually Reinforced Concrete Pavement

USGBC	US Green Building Council
ABS	Australian Bureau Council
RAP	Reclaimed Asphalt Pavement
PCP	Plain Concrete Pavement
HCC	Humilton City Council
CDRB	Crushed Demolished Road Base
FDR	Full Depth Reclamation
CGRB	Crushed Granit Road Base
GCB	Ground Clay Brick
RLT	Repeated Load Traxial
FWD	Falling Weight Deflectometer
FRCA	Fine Recycled Concrete Aggregates
CTB	Cement Treated Base
HCTCRB	Hydrated Cement Treated Crushed Road Base
AAPA	Australian Asphalt Pavement Association
UU	Unconsolidated Undrained Static Trial
Er	Resilient Modulus
ANOVA	Analysis of Variance

Standard test methods cited in this research

Test number	Description
WA 100.1	Sampling Procedures for Soil and Manufactured Granular Materials-(MRWA, 2011p)
WA 105.1	Preparation of Disturbed Soil and Manufactured Granular Material Samples for Testing-(MRWA, 2011c).
WA 110.1	Soil and Granular Pavement Material Moisture Content: Convection Oven Method-(MRWA, 2011 L)
WA 115.1	Particle Size Distribution: Sieving and Decantation Method-(MRWA, 2011d).
WA 120.1	Liquid Limit-(MRWA, 2012e).
WA 120.2	Liquid Limit-Cone Penetrometer method-(MRWA, 2012f).
WA 121.1	Plastic Limit-(MRWA, 2012g).
WA 123.1	Linear Shrinkage-(MRWA, 2012h).
WA 132.1	Dry Density/Moisture Content Relationship: Standard Compaction Fine and Medium Grained Soils-(MRWA, 2012)
WA 133.1	Dry Density/Moisture Content Relationship: Modified Compaction- Fine and Medium Grained Soils-(MRWA, 2012i).
WA 143.1	Determination of the Unconfined Compressive Strength of Laboratory Compacted Specimens-(MRWA, 2012j).
WA 220.1	Los Angeles Abrasion Value-(MRWA, 2012k).
WA 122.1	Plasticity Index-(MRWA, 2012m)

CHAPTER 1

INTRODUCTION

1.1 **Introduction**

The construction of roads in Australia and elsewhere in the world consumes considerable quantities of natural resources, whilst at the same time, the demolition of structures due to age; redevelopment or failure to meet current standards generates large amounts of material that has the potential to be reused for road making materials. These materials are termed as Construction and Demolition waste (C&D waste), but are in fact 100% recyclable for use in many areas of new construction; however the focus of this research is in the reuse of this material in roads.

In 2008–09, Australia produced a total of 19 million tonnes of C&D waste, 8.5 million tonnes of which was disposed of in landfill while 55% was recycled (Hyder, 2011). During 2008–09, Western Australia recycled a total of 1,832,155 tonnes. Approximately 44% was from the C&D sector and 31% from the Commercial and Industrial waste(C&I waste) sector. In 2008-2009, 86% of recycled material was from municipal and 14% from non-metropolitan sources. Furthermore, the recycling rate per capital in WA rose steadily in the years 2004to 2009 (Hyder, 2010).

In 2011, the Department of Sustainability, Environment, Water, Population and Communities (DSEWPaC) and the Queensland Department of Environment and Resource Management (DERM), commissioned Hyder Consulting (Hyder), and its project partners Encycle Consulting and Sustainable Resource Solutions to prepare a status report on the management of construction and demolition waste in Australia (C&D Waste Status Report).

Analysing this report, and considering only the masonry component, this being the majority component of the C&D waste stream suitable for use in road pavement construction, showed that for the major mainland states, Western Australia compared poorly with regards to the recycling of these materials. (Hyder 2011) Table 1.1 shows the comparison between states of the relative recycling rates for masonry type materials. The figures expressed are based on 2008-2009 data, and show that of the mainland states, WA lags the rest in recycling of C&D materials by a significant amount.

Table 1.1: Masonry products recycled by state

State	Total disposed (tonnes)	Total recycled (tonnes)	Recycling rate (%)
ACT	21,311	155,816	88
SA	290,999	1,253,750	81
NSW	1,078,156	4,344,952	80
VIC	1,003,806	1,762,228	64
QLD	1,275,229	1,128,916	47
WA	1,935,621	738,949	28
TAS	33,738	9,216	21
NT	N/A	N/A	N/A
All states	5,638,860	9,393,827	62

Source: Hyder (2011)

In 2008, the Department of Environment and Conservation (DEC) contracted Cardno WA Pty Ltd (Cardno 2008) conduct a detailed investigation into the existing and potential markets for recycled construction and demolition (C&D) material in Western Australia.

Figure 1.1 is taken from that report and shows slightly different figures than those of Hyder, as the C&D material in the Cardno report may be different from that of Hyder. Nevertheless, it confirms the fact that WA has a poor record of recycling.

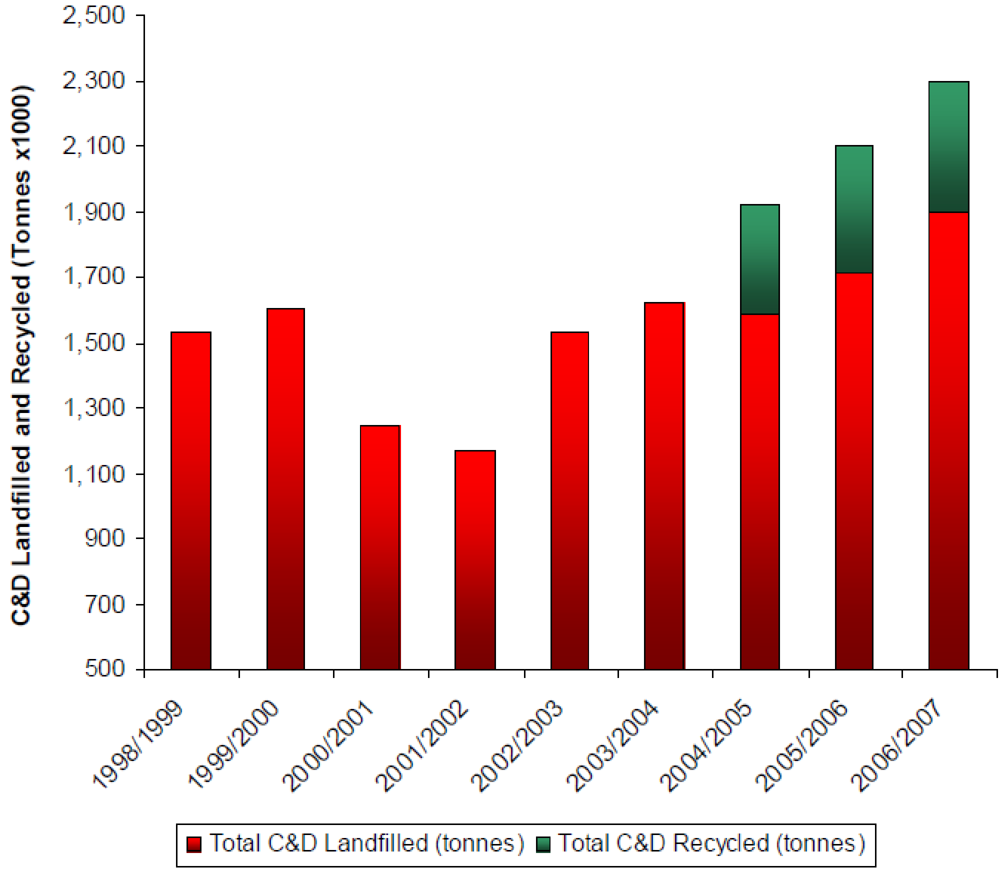


Figure 1.1: C&D waste generation and recycling in Western Australia

C&D waste represents 50% of the waste that is sent to landfill in Western Australia. The Department of Environment estimates that "*approximately 1,500,000 tonnes of construction and demolition waste is disposed each year in Perth*". The Australian State of the Environment Report notes that approximately 30-40% of solid waste disposed of in landfill came from buildings. In general, WA has the highest waste per capita generation in Australia (Cardno, 2006, p. 2).

A road pavement is a multi-layered structure made up of base, subbase, and subgrade. Natural materials such as particular gravel and manufactured materials such as quarried crushed rock, concrete and asphalt are consumed in considerable quantities during pavement construction for roads, airports, car parks, industrial pavements and floors. Many studies have investigated the potential of using materials such as recycled concrete in road base and subbase courses. The studies show that construction and demolition materials, such as concrete, brick and tile can be used in road base and concrete production, as recycling of these materials is simple, yielding products suitable for using in road and construction projects.

Recycled concrete is mostly used as aggregate in road base and subbase. It can also be used to manufacture concrete for kerbs, footpaths, island infill and such like. The properties of concrete, being sourced from often the same high strength rock used in pavement materials, make it suitable for use as aggregate in road construction and different construction projects when it is recycled and crushed into suitable size ranges. The quality of aggregate produced from recycled concrete depends on the quality of the original materials and on how these materials are processed. In some areas these aggregates are used to make new concrete.

Recycling concrete can not only decrease the use of virgin aggregates, but can also have a significant effect on transportation and utilization costs; recycling facilities are often in close proximity to major developed areas where the source of the original virgin materials may in cases be quite remote. Various blends of broken building products with recycled materials are used in pavement construction. Several recycled materials with additives such as cement, lime, fly ash, slag, etc. have been used for the construction of pavement layers. The use of additives has a significant effect on the properties and strength of recycled materials and thus on pavement life.

However in many applications of recycled concrete as a base, post construction shrinkage cracking has occurred and some road agencies have trialled recycled concrete base, and due to the evidence of block type cracking, have ceased using the material. City of Gosnells in Western Australia is a case in question. This shrinkage coupled with considerable strength gains is thought to occur due to the rehydration of the Portland cement content of the recycled concrete component; recycled concrete makes up the majority of the demolition material that is recycled.

Main Roads WA proposed that when recycled crushed concrete was used as a base, it must be surfaced with a geotextile reinforced seal. Main Roads WA concerns were two fold, one being the shrinkage cracking, and secondly was the strength gain with time, where concern was expressed that the material would stiffen, become effectively bound, and then be subject to fatigue type failure, breaking up into small discrete blocks. This strength gain is again attributed to rehydration of the Portland cement content.

This study investigates and reports on effect of cement rehydration in recycled crushed concrete in roadbase. In this research, the effects of curing time on recycled crushed concrete were determined without using additives. This study also tested various blends of recycled concrete with crushed brick, tile and ferricrete for suitability in road base production.

1.2 **Objectives**

1.2.1 ***General objective***

The main objective of this research is to investigate the effect of rehydration of RCC when used as a base material in pavement construction. Two methods are to be investigated, blending with non pozzolanic materials, and forced development of multiple microcracks by post construction compaction.

1.2.2 ***Specific objectives***

The specific objectives of this research can be separated into three separate components:

1. To assess the physical and mechanical properties of RCC, ferricrete, brick and tile using standard test methods.
2. To assess the physical and mechanical properties of different blends of RCC and varying proportions of brick & tile, and RCC and varying proportions of ferricrete.
3. To assess the effect of post construction compaction on UCS, bending strength and stiffness of pure RCC beams.

This research was carried out in two stages. Stage 1 addressed the first and second specific objectives by means of index testing, while stage 2 covered the third specific objective.

1.3 **Outline of the thesis**

The research is reported in five sections as follows:

1. General overview of the project consisting of the project's aims and scope.
2. Review of the background of using recycled crushed concrete (RCC), and different blends of recycled material with RCC as base course materials.
3. General introduction to the research methodology.
4. Presentation of non-standard and standard test data, followed by analysis and discussion of results.
5. General conclusion and recommendations for further research.

CHAPTER 2

BACKGROUND STUDY

2 Background to the research

Recently, the use of recycled crushed concrete as a road base in Western Australia and other states has seen an upward trend. Many studies have been observed into the use of recycled products and demolition materials in the construction of road base and paths.

Recycling aggregates and materials can help preserve land, reduce energy consumption, reduce waste and conserve natural resources, thereby creating many economic and environmental benefits. In fact, recycled aggregates and materials are more suitable for road construction than any other use. Moreover, the performance properties of recycled materials have illustrated that rehydration gives improvements in the strength and mechanical behaviour of these materials.

Crushed concrete provides an alternative source of aggregate for the construction industry. Construction and demolition waste, including concrete, brick, tiles, glass and asphalt, can be used in road base and replace virgin aggregates.

2.1 Outline of background study

Many substantial (mechanical and non-mechanical) aspects of various recycled materials have been considered with regard to extending pavement life. Much research has also been done into the economic and environmental benefits realised by using recycled materials in road constriction. The aim of this chapter is to review previous research with regard to the following:

1. The definition and function of pavement.
2. Background and main studies on the properties of recycled concrete.
3. Background and main studies on the properties of recycled brick and ferricrete. Recycled concrete aggregate as a road base course material.
4. Rehydration of recycled concrete.
5. Background and main studies into the use of microcracking to reduce shrinkage cracking in cement treated bases.
6. The control of cracking in cement stabilized pavement.
7. Background and main studies on different experimental pavement tests such as:
    - Unconfined compressive test (UCS)
    - Compaction test
    - Modified beam test
    - Repeated load triaxial test (RLTT).

This chapter is divided in different phases as follows:

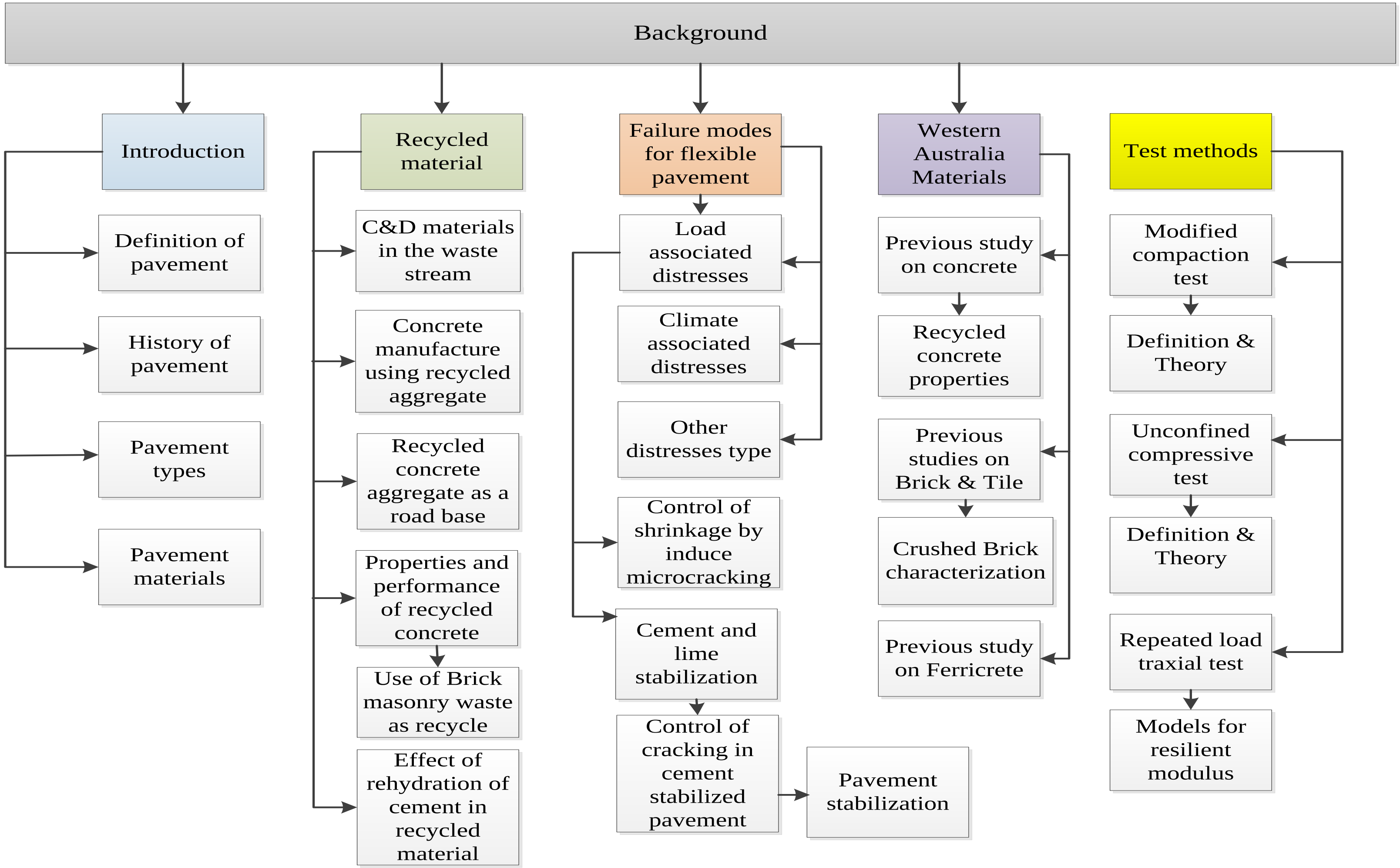


Figure 2.1: Diagram of this research

2.2 **Introduction**

Utilization of recycled materials as aggregates not only reduces waste disposal and saves natural resources, but also can have a role in reducing construction costs. Although most of the research has been on recycled crushed concrete (RCC), some studies have also been done on concrete blended with other recycled aggregates such as brick aggregates (Cavalline and Weggel, 2013). The utilization of recycled materials for constructing the base layer has been developed over the last twenty years(Edil, Tinjum and Benson, 2009).

Investigations into the use of RCC with different blends of demolished materials have been significant in the expanding usage of recycled concrete in Western Australian pavements. Many laboratory and experimental tests have been done to analyze the performance and durability of pavement materials, yielding a greater understanding of the properties of recycled road base constructed from C&D materials. The results of investigations show that materials perform similarly in Western Australia compared to materials tested in other parts of Australia(Leek, Siripun, Nikraz and Jitsangiam, 2011).

Different test methods have been applied to recycled materials and natural aggregates used in road construction, and the stiffness of recycled concrete products has been evaluated in field trials. The potential of recycled concrete has led to many studies being undertaken to develop a bound material with the addition of cement making use of the rehydration process. The rehydration process in recycled crushed concrete raobase has not been studied in any depth, but has been recognised that the process may result in high stiffness properties. (Leek and Siripun, 2010).

2.2.1 ***Definition of pavement***

Pavement is defined as the load carrying structure of a road placed on the sub grade to allow the load on the subgrade to be distributed sufficiently to within the stress limits of the subgrade. In the years before vehicular traffic, stone paths were primarily used by pedestrians and animal traffic. Nowadays, pavement structures are used by pedestrians, cycles, passenger vehicles, trucks, aircraft and heavy duty equipment in factories and freight terminals.

The properties required of pavement materials is very much dependent on the type, loads and volume of traffic using the pavement. The strength and thickness of the pavement layers have an important effect on the lifetime of the pavement, and the maintenance regime and performance of a pavement depends on factors such as subgrade strength material properties, vehicle loads and environmental effects including moisture, frost and temperature.

2.2.2 ***History of pavement***

Concrete asphalt and granular materials are used for roadway pavement (Skinner, 2008). The first modern concrete highway was constructed at the end of the 19th century. Concrete mixes consisting of Portland cement, sand, aggregate and water can be used for rigid pavement. Asphalts have been used in road construction as a waterproof layer since the 1800s. Asphalt concrete mixes consist of a mixture of aggregate (gravel or crushed stone), sand and cement (asphalt binder) and can be used for flexible asphalt pavement. The first asphalt road was constructed in Paris in 1854. The materials used included "*natural rock, i.e., asphalt, limestone rock saturated with asphalt*" (Johnson, 2012, p. 11). "*In the 1890s* *the first asphalt concrete specifications appeared in the US*". At the same time, use of aggregate blends was considered in Europe. Moreover, "*utilization of the first hot-mix asphalt was advanced in the late 1920s*" (Johnson, 2012, p. 12).

Europeans settling in Australia built the first road in Sydney in 1788. Road networks were significantly improved and increased in the period after the 1860s. Australian cities and towns grew and developed along with increases in the population and migration. Increases in traffic and transportation vehicles motivated the upgrading and development of the roads network (Watts, 2006).

More than 3000 miles of roads were constructed by the Roman Empire in Britain in 200AD. They constructed these roads with in-built canals to provide efficient drainage. The structure of roads of different thicknesses above the fragile and weak soils indicated the Romans' basic knowledge of soil mechanics (Johnson, 2012). Since then, pavement design has been gradually improved, slowly changing from an art to a science. In the past, the thickness of pavement was determined based on experience. Much research and various methods have been devoted to determining the optimal thickness of pavement (Huang, 1993).

The thickness of the pavement and the material used in the layers play an important role in pavement life, which is also affected by increased traffic volume due to population growth, increased use of heavy vehicles, and environmental effects. The performance and maintenance of a pavement structure is strongly dependent on the pavement responses and strength of the pavement layers. The thickness design procedure is based on controlling the critical pavement reactions in pavement layers. The main role of pavement material is to receive the dynamic and vertical traffic load and conduct it to the base layer, while at the same time the base acts as a preserver cover (Gibbons, 1999).

The extensive roads network in Australia is approximately "*(0.06km per user in Australia compared with 0.03km per user in both Canada and New Zealand) with a total of over 800,000km*" Australian Bureau of Statistics (ABS, 2012a, p. 1).

For a long time, industrial wastes and recyclable materials have been used in pavement projects. For instance, crumb rubber from old tires is often used as a stabilizer in hot mix asphalt pavement design. In California and Arizona, asphalt rubber hot mix is used in pavement construction to reduce highway noise. Experimental studies have shown that "*reclaimed asphalt pavement (RAP) is often used in different countries as an additional material for aggregate and a portion of the asphalt binder in hot mix asphalt, including Superpave mixes*"(Skinner, 2008, p. 10).

In Victoria, recycled crushed concrete is used for constructing unbound or cement stabilized pavement layers (VicRoads, 1997). The use of recycled aggregate as a granular base course in pavement projects has also been considered. Further studies show that when a road is built on wet subgrade areas, recycled aggregates will stabilize the base course and make a better working surface for pavement construction (Shing Chai NGO, 2004).

2.2.3 ***Pavement types***

Three major types of pavement are flexible (granular, spray seals and asphalt), rigid (concrete) and composite pavement (Huang, 1993). A flexible pavement structure is usually composed of several layers of material. The performance of natural granular and modified materials depends on technical, environmental or economic conditions. Each layer is impacted by the load from the overhead layer and distributes the load to the next lower layer until the load is distributed over a large enough area of subgrade so as to be within the capacity of the subgrade to withstand the load. The strength and thickness of each layer varies. In general, the quality of layers close to the surface should be higher in order to better resist traffic loads and environmental conditions. In a conventional flexible pavement, the materials in the top layer have better quality compared to the material at the bottom due to the high concentration of stress on top as shown in Figures 2.2 (Huang, 1993).

Rigid pavements are so named because the pavement structure bends very little under loading due to the high modulus of elasticity of the surface course. Rigid pavements generally consist of Portland cement concrete and can be understood by plate theory instead of layered theory. In plate theory the concrete slab is a medium thick plate with a plane which remains plane before and after loading. Rigid pavements are placed on the subgrade in very low traffic situations, but more usually on a single layer of granular or stabilized material (Huang, 1993). Even on relatively good subgardes, a working platform to support the weight of the paving equipment is needed. Because of the inflexibility, the pavement structure distributes loads over a wide area which is non-uniform as shown in Figure 2.3.

Rigid pavements may be plain concrete pavements (PCP) where most slabs are unreinforced, reinforced concrete pavements (RCP) where the essential joints are wider spaced that the case with PCP,s, or continually reinforced (CRCP) where reinforcement is continuous over multiple joints. The reinforcement in concrete pavements is to control cracking; it does not affect slab thickness. The thickness of the slab is however affected by the presence of dowelled joints and integral shoulders. Rigid pavements constructed with hydraulic cement concrete are also able to be reinforced, unreinforced, or post-tensioned (Samarin, 1999).

Rigid pavements are used extensively on the heavier trafficked freeways and highways in NSW, where a significantly developed and experienced workforce is able to maintain the quality required.

A great majority of Australian pavements are flexible, being unsealed granular or granular with a thin bituminous surface.

"*Flexible pavements usually consist of three major layers: bituminous surfacing, base and subbase. The base is the layer of material under the surface, and can be made of crushed stone, crushed slag, or other untreated or stabilized materials. The subbase course is the layer of material beneath the base course. For economic reasons, two different granular materials are used. Local and cheaper materials can also be utilized as a subbase course on top of the subgrade rather than having an expensive base course material*" (Huang, 1993, P.10).

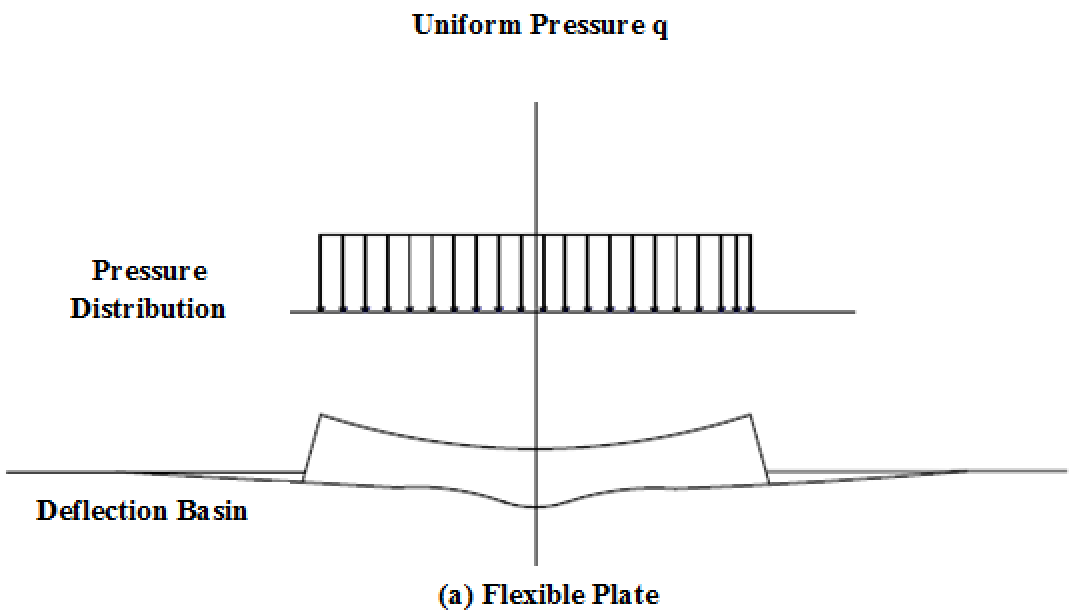


Figure 2.2: Flexible Pavement Load Distribution (Huang, 1993)

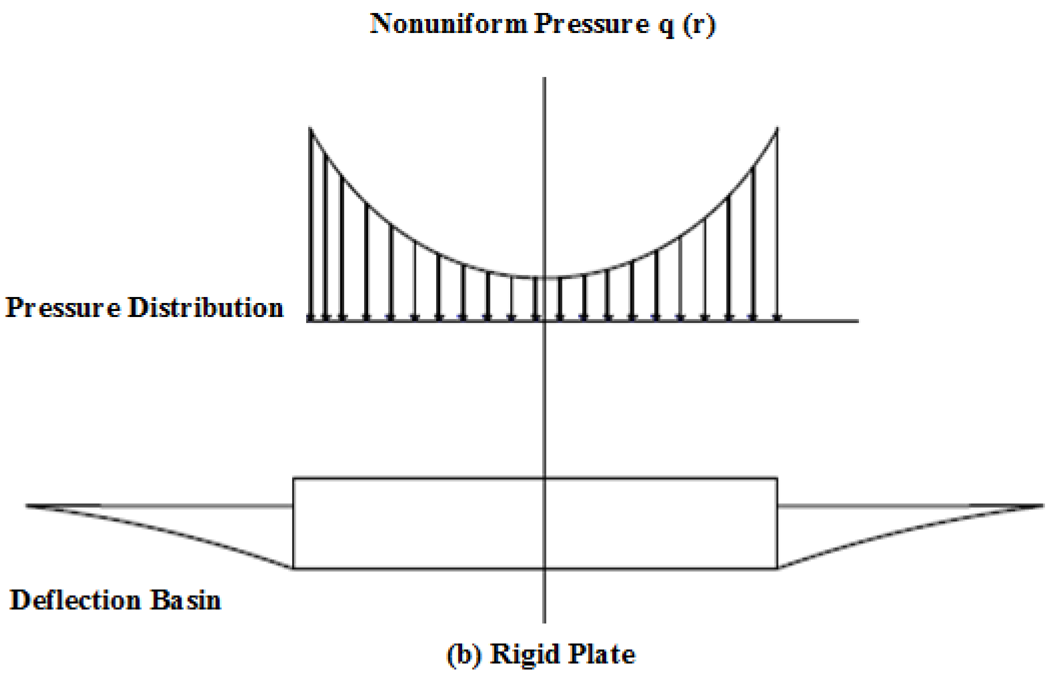


Figure 2.3: Rigid Pavement Load Distribution (Huang, 1993)

Flexible pavements also encompass bound materials, such as highly stabilised granular materials and deep layers of asphalt. Whilst these pavements may be very stiff, they are still classified as flexible, however the bound layers are designed to limit the strain such that they can withstand a given number of load repetitions, whereas in a granular pavement, the only design criteria is to limit subgrade strain.

2.2.4 *Pavement materials*

As mentioned earlier in this chapter, pavement layers consist of base, subbase and surface. Each pavement layer uses different materials and has different roles in improving the pavement's structure and strength to resist traffic loads. Pavement materials should also tolerate environmental influences such as high temperature, moisture, humidity, frigidity as well as being resistant to chemical disintegration by moisture. The surface layer provides stable, dust free and erosion resistant surface for easy vehicle and pedestrian travel. The base layer supporting the surface layer, provides sufficient support for the surface layer, but must also have suitable drainage properties according to the moisture regime in the locality.

Whilst the base could generally provide the structure to support the load transfer to the subgrade, it is often an expensive material, and a cheaper subbase is used to provide the necessary thickness over the subgrade. However in wet conditions, the subbase may have particular properties to provide for drainage of the pavement layers above, and to reduce the transmission of water by capillary action to the base.

Because the surface has to resist environmental conditions and heavy traffic loads, surface materials should be of a higher quality than the other layers. The materials mostly used for the surface layer include select granular materials in the case of unsealed roads, spray seals, asphalt mixtures, and Portland cement concrete.

Unbound granular materials, crushed rock, natural gravels, or soil aggregate are the commonly used base materials. Modified granular materials, materials which have been partially treated with bituminous, chemical, or pozzolanic materials to improve their performance to be suitable as an unbound granular material may also be used. However these materials are still considered unbound. Highly modified materials that have stiffness values significantly above those of granular materials such as bitumen stabilized materials, chemically modified materials, cement, lime, lime/fly ash or slag-modified materials are termed bound and can be used in pavement construction, generally in cases of high traffic loads or poor subgrade conditions. (AASHTO, 2007, p.5).

Studies have revealed that the properties of unbound and modified granular materials are similar in terms of the performance is generated by the shear strength developed by particle interlock, although in the case of some natural materials used in arid regions, may result from a degree of natural cohesion. In general, the shear strength of cemented granular materials is due to both interlocking of particles and chemical bonding. The tensile strength of these materials is significant.

Asphalt consists of blends of graded aggregates, sand, filler and bitumen. It is clear that some factors could have a significant effect on the strength of asphalt, such as particle interlock, viscosity of binder, cohesion and adhesion between binder and stone. Experimental studies have revealed that the modulus of this visco-elastic material increases with an increase in the rate of loading. Increasing the temperature caused a decrease in the strength against the shear; however it led to greater resistance against fatigue (Leek, personal communication, 2012).

Unbound graded granular materials generally consist of a specific quantity of fines and water and are used for base and subbase layers (Samaris, 2004). The properties of these materials depend on pavement stress conditions. The performance of granular materials depends on some important factors. Particle size distribution has a significant effect on the strength and finish of a pavement layer, and a maximum aggregate size of 25 mm for the base and 75 mm for the subbase is generally adopted as a compromise between strength and surface finish.

The best shape for these materials is cubic and angular. Although a high quantity of fines decreases internal friction and permeability, it can also lead to increased moisture sensitivity. The quantity and type of fines can also have a significant effect on plasticity. The action of modified granular materials is the same as granular materials, and can be improved by modification for undesirable properties such as low stiffness and high plasticity (Leek, personal communication, 2012).

There are a number of important factors to take into consideration when choosing pavement materials. These include gradation, type of crushing, moisture content and sensitivity, density, permeability and strength. In a well graded material, the gradation of aggregates should be such that increasingly smaller particles fill the voids left by larger particles, but some residual voids should remain, and be interconnected to allow water to dissipate crushed aggregates with proper angular shapes provide a better and greater surface area for contact between the aggregates, leading to greater density and strength of the pavement layers.

The optimum moisture content (OMC) and maximum dry density are important factors which can have a major influence on the strength and deformation of pavement. When the moisture content of soil is lower than the OMC, the soil will need more compaction effort in order to gain a reasonable density. Moisture content higher than the OMC also leads to instability and weakness in pavement layers. The permeability of the soil should be such that moisture content does not influence the compaction of layers (AASHTO, 2007).

Cemented granular materials used in the base layer can improve low cohesion of base materials and decrease moisture sensitivity. Cement, lime or chemicals can be used to stabilise the subgrade to improve the subgrade strength (AustSab 2012). Different types of aggregates with particular features are used for creating the base layer. Natural aggregates consist of workable particles of gravel and sand, crushed quarry stone being one type of base material that can be used in pavement construction. Studies in this field show that recycled aggregate materials can also be used for constructing a base layer with blends of natural aggregates, bituminous blends, crushed concrete and recycled glass.

In situ recycling of existing pavement has become common in recent years. Base layers may be treated with bituminous products or various cementitious materials consisting of Portland cement, lime and lime-fly ash to improve pavement properties. On occasions, specific aggregate sizes can be blended in a process called mechanical stabilisation to improve the properties of an existing unbound granular base.

Portland cement can be used for treatment of both fine and coarse aggregates; however it may not provide enough strength and durability for the base layer. Although this base layer can have the capacity to tolerate loads, the development of cracks in the base layer can be still a problem. Lime can be used as a stabilizer for modifying fine-grained soils or fine-grained granular soils. The combination of lime and soil has significant advantages such as increased durability and strength and reduction of the plasticity index. Lime and fly ash also can be used to treat fine-grained materials, particularly silty soils, in a pozzolanic reaction (AASHTO, 2007).

2.3 **Recycled material**

The recycling and reuse of various materials plays a significant role in preserving virgin materials and maintaining the environment. Use of recycled materials can also reduce the cost of construction. Recycled concrete, brick, glass and asphalt can be reused for road construction and buildings (VicRoads, 2011).

2.3.1 ***C&D materials in the waste stream***

The growth of population and economy increases waste production. Waste can be categorized by its different utilization sources, including industrial, commercial, municipal, construction and demolition. Due to environmental and social factors, Australia has one of the highest rates of waste production in the world (ABS, 2012b). Construction and demolition (C&D) waste materials are produced as a result of new building construction, renovation and demolition.

It is not easy to provide precise statistics for waste generated by construction projects; however it is estimated that it makes up more than 30% of the entire weight of construction materials sent to the construction site, (IWWG, 2013). In Australia, almost 44 million tonnes of solid waste was produced in 2007, an estimated 2080kg of waste per person. Comparison of waste generation in 2003 and 2007 reveals an increase of 35% in solid waste in both landfill disposal and recovery. In 2007, 48% of waste went to landfill and 52% was recycled, compared to 46% of waste recycled in 2003. In 2008–09, 19 million tonnes of C&D was produced in Australia.

During this year a total mass of 8.5 million tonnes of C&D waste was disposed of nationally and 10 468 186 tonnes recovered. Thus, 45% of this waste stream went to landfill and 55% was recycled and reused (Hyder 2011). During 2009–10, "*of 21.6 million tonnes of waste preserved at landfills, 34% was from domestic and municipal waste stream, 34% was from industrial waste stream and 26% was from construction and demolition waste stream*" (ABS, 2012c, p. 1).

Waste generation in Western Australia amounted to 5,247,000 tonnes in 2006–07, with 33% being recycled and 67% going to landfill. Western Australia's C&D waste totalled approximately 2,348,000 tonnes during 2006–07, of which 409, 000 tonnes were recycled. Production of waste was 2490kg per person in WA, (EPHC, 2010).

Australian waste and recycling figures show diverse recovery rates for each authority in 2008–09. The WA recovery rate was less than 30% compared to a rate of more than 70% for New South Wales, South Australia and the ACT. The Queensland and Victoria recovery rates were between 35% and 55%. The recovery rate for Tasmania was 15% and 1% for the Northern Territory (Hyder, 2011).

Also the municipal waste generation and recycling performance of Australia was compared with four countries in 2008. The figures indicated that the US had the highest municipal waste production at around 927kg per capita; however the US recycling rate was estimated to be lower than that of Germany. Australia generated 566kg municipal waste per capita and recycled 217kg per capita. Although Australia produced less municipal waste than England, it recycled more of its waste. After Germany by 61%, Australia had a higher recycling rate by 38% compared to other countries (EPHC, 2010).

More than 50% of waste in New Zealand comes from construction and demolition activities, more than 20% of which goes to landfill and 80% to clean-fill, Hamilton City Council (HCC, 2013). In Canada, approximately 27% of all solid waste is disposed of in landfill (Yeheyis, Hewage, Alam, Eskicioglu and Sadiq, 2012). The figures reveal that Canada recycles and reuses less of its waste than other industrialized countries, recovering and reusing only 22% of solid waste, compared to Denmark with up to 95% (Shamloo, 2010).

The United States produces more than 170 million tonnes of waste from construction and demolition projects every year, with nearly 50% of waste being recycled. Construction and demolition waste was the largest component of the total waste produced in Europe. In general, C&D waste is recognized as the largest waste stream in the European Union, making up approximately 49% of total waste production. Construction and demolition projects in the EU are estimated to generate more than 855 million tons of waste per year (IWWG, 2013).

The amount of waste produced varies from country to country, due to financial and national differences. Although the recycling rate is still only 25% in some countries, recovery rates of more than 80% have been attained in Germany, Denmark, Belgium and the Netherlands. At present, approximately 75% of C&D waste goes to landfill in Europe (IWWG, 2013).

2.3.2 *Concrete manufacture using recycled aggregate*

In most countries, the disposal of construction and demolition waste has become a severe concern. For environmental and economic reasons, these waste materials are recycled and reused in concrete products and road construction (Bazazi, Khayati, and Akrami, 2006).

Crushed concrete can be recycled as an aggregate for creating new concrete or pavement layers. Although recycled concrete can be combined with natural aggregate for making new concrete, mostly it used as an aggregate in subbase layers (ACPA, 2013). Studies have revealed that recycled concrete aggregates used instead of virgin aggregates in new concrete yield the same quality and performance, (FHWA, 2005).

Many countries have trialled the use of recycled aggregates in construction projects. An Indian study of concrete manufactures with recycled aggregate has compared some important properties such as compressive strength and workability. Replacing various percentages of natural aggregates with recycled aggregates 3, 7 and 28 days shows that concrete specimens with 40% replacement of recycled aggregate were the strongest at 28 days, and that the early compressive strength of concrete made of natural coarse aggregate and recycled coarse aggregate was the same (Patel, Vyas and Bhatt, 2013).

Various tests have been performed by Sagoe-Crentsil et.al. (2001) on concrete made of coarse recycled concrete aggregate and natural fine sand, both immediately and after a curing period. Compressive strength, drying shrinkage, expansion, splitting tensile strength and abrasion resistance of recycled concrete were determined. In this study, graded unwashed coarse recycled concrete aggregate and natural fines for making concrete mixtures were used. For mixes that had the same volumetric proportions and workability, there was no difference in 28-day compressive strength between concrete made with recycled aggregates. In addition, recycled concrete aggregates displayed higher drying shrinkage values in comparison to normal concrete mixtures. According to the test results, the drying shrinkage of specimens increased with time and stabilized at 91 days. The tensile strength of recycled concrete was determined by the splitting tensile method. Water absorption and carbonation rate of test samples indicated little difference between commercially produced recycled concrete and reference concrete which has high cement content (Sagoe-Crentsil, Brown and Taylor, 2001).

In order to use both fine and coarse recycled aggregates, Hansen (1992) determined that with equal water-cement ratios the compressive strength of recycled concrete aggregate reduces in comparison with concretes made with natural gravel and sand. In addition, compressive strength tests of recycled concrete aggregate with coarse recycled aggregates and a blend of 50% fine recycled aggregates and 50% natural sand revealed that the strength was 10–20% lower than the strength of recycled aggregates with coarse recycled aggregate and 100% natural sand.

Desmyter (1999) studied the use of recycled concrete and masonry aggregate in road construction. The efforts related to the production of ready mixed concrete and concrete products with recycled aggregates. Concrete specimens were fabricated with several kinds of C&D aggregates and tested for workability, strength, durability, creep and shrinkage. Different concrete prism expansion tests such as a modified NF P18-587 test and ATILH-LCPC Annex G were undertaken to show that recycled aggregate possessed a residual reactivity

2.3.3 ***Recycled concrete aggregate as a road base course material***

Approximately more than eight million cubic meters of concrete are made in the world every day. The use of large quantities of concrete in various projects is a major reason for the increase in waste concrete. Increasing quantities of concrete is discarded during various civil projects, such as renovation, demolition of old structures and construction of new buildings, bridges and dams. Concrete recycling in construction projects is an important aim in improving environmental and economic conditions (Schelmetic, 2012). Concrete is the most used construction material in different civil projects and activities (Tam, Wang and Tam 2008).

Recycled concrete aggregates (RCA) are produced from C&D waste, and are frequently used in road projects. RCA is produced from concrete pavement and different structures may consist of small amounts of various materials (Gabr and Cameron, 2012).

Many studies have been carried out comparing a combination of different recycled materials with a control section of conventional crushed (granite) road base (CRB). For upgrading Welshpool Road in Perth, Western Australia, the main aim was to replace the CRB base and limestone subbase with recycled materials such as pure crushed concrete, and/or commingled mixed concrete, brick, asphalt and tile. CRB was used as a base in one part only as a control section.

A number of tests, including repeat load triaxial testing, shear box testing and falling weight deflectometer testing was done on recycled crushed concrete and commingled recycled crushed road base under different moisture conditions and particle orientation, which showed that using recycled material would result in increased asphalt fatigue life. The high resilient modulus values demonstrated that the base manufactured from recycled concrete performed well under a range of moisture conditions. According to the constructors, recycled materials were as easily compacted as CRB, but had a reduced tendency for developing spongy patches during working (Leek, 2008).

The performance of recycled road base sourced from both concrete only and commingled concrete, brick and tile has been investigated and compared with the performance of CRB. The investigation illustrated that the recycled products had a significantly higher modulus and lower moisture sensitivity than CRB, and that the source of concrete made a significant difference to the strength of the recycled product. Recycled concrete road base sourced from structural grade concrete was significantly stiffer than material sourced from low grade concrete. Using recycled concrete can therefore reduce project costs (Leek and Siripun, 2010).

Similar studies have been carried out for determining the resilient modulus and permanent deformation characteristics of C&D waste as a road base, using CRB as a reference material. Test results for both materials were also compared. Research results stated more study should be carried out to find a compatible compaction method to reach the identical density condition of C&D waste also it indicated that because during the recompaction in preconditioning period, material become denser, the modified standard compacting test is not the best method for this reason.

The CIRCLY program was used by Jitsangiam et. al (2009) for pavement design and to determine the performance of the typical pavement model. Both CRB and C&D waste had the same permanent deformation value based on the Austroads (APRG 00/33) test method. The resilient modulus is a fundamental input for the CIRCLY program. This research used the K-Theta (K-θ) model for determining the appropriate depth of aggregate as a base layer (Jitsangiam, Nikraz and Siripun, 2009).

Similarly, Cheema (2004) carried out several laboratory tests on crushed recycled concrete manufactured from construction and demolition waste and used in the base construction of Gilmore Ave in Kwinana, WA. This trial included both CRB and recycled base materials in separate sections, with the CRB being the control section. Tests undertaken included consistency limit tests, particle size distribution (PSD), unconfined compressive strength (UCS), the Los Angeles abrasion test and maximum dry compressive strength (MDCS), in accordance with Main Roads Western Australia test methods.

Benkelman beam deflection testing was performed on the finalized pavement for several years after completion. Deflection and curvature function (CF) were monitored at the completion of the project until the strength of the pavement appeared to become constant. The deflection and curvature values for both CRB and crushed recycled concrete (RCC) sections of the constructed pavement were low. However the CF for the RCC section has consistently been significantly lower that the CRB sections, and this is reflected in the current condition of the pavement, where the CRB section has significant fatigue and rutting (Figure 2.4), and the RCC section has an occasional fine shrinkage crack.

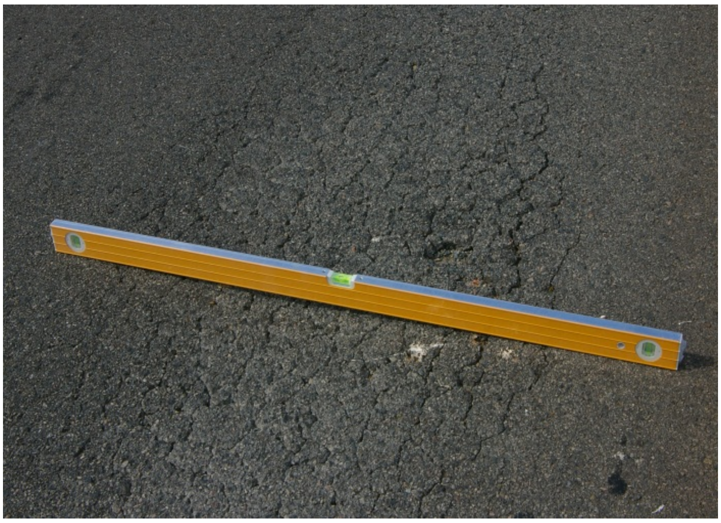


Figure 2.4: Gilmore Ave CRB section

Figure 2.5: Gilmore Ave RCC section

Moreover, tests to determine UCS parameters carried out for 0% to 15% brick content after 28 days curing showed an increase in strength. Some specifications were proposed for a base and subbase of CRC sourced from construction and demolition waste. It was concluded that a brick content of up to 15% was acceptable for a subbase material, but a base material required a brick component of no more than 5%. However no justification was provided to support these statements.

2.3.4 ***Properties and performance of recycled concrete with different blends of materials***

Recycled crushed concrete has been considered for different pavement projects in Western Australia. Different blends of recycled crushed concrete with brick and tile have been studied to upgrade pavement specifications and design (Leek, Siripun, Nikraz, and Jitsangiam, 2011). Laboratory tests determined and compared the properties and performance of recycled materials with those of natural materials. The studies revealed that recycled crushed concrete can be used as a road base layer. Some material properties have a significant effect on the development of pavement, such as gradation, shape, absorption, permeability, specific gravity, thermal properties, pH-level, solubility, particle strength, particle stiffness and freezing (Leek and Siripun, 2010).

Laboratory tests were performed on different blends of recycled crushed concrete and reclaimed asphalt pavement (RAP) to assess the appropriate blends for constructing the road base. The optimum moisture content decreased as the proportion of RAP increased and generally, high proportions of RAP could raise the permanent strain (Leek and Siripun, 2010). The comparison of recycled crushed demolished road base (CDRB) with crushed granite road base (CGRB) was investigated and results revealed that the optimum moisture content for CDRB is greater than CGRB. In addition results from repeated load triaxial testing (RLTT) stated that the resilient modulus for CDRB is significantly high compare to CGRB. The data showed that CDRB, with its higher base stiffness, could be a better pavement material compared with CGRB (Leek and Siripun, 2010).

Experimental studies done in WA show that a combination of different blends of materials with recycled crushed concrete not only can have a great influence on the rehydration process and stiffness, but also can control cracking. Thus, using brick and tile or sand as fine aggregates in recycled crushed concrete can control stiffness and limit the effects of rehydration (Leek, Siripun, Nikraz, and Jitsangiam, 2011).

2.3.4.1 ***Use of brick masonry waste as recycled aggregate in concrete***

Various mixures of brick and natural aggregates were tested to determine the effect of natural aggregate with brick on the properties of concrete. The results showed that the compressive strength of natural aggregates is always higher than crushed brick. The mixes of crushed brick as fine and coarse aggregates were also shown to be not as strong in comparison with all mixes. In general, the blend of 25% crushed brick, 25% natural coarse aggregates and 50% natural fine aggregates had highest compressive strengh. The blends of 25% crushed clay brick fine with 25% natural fine aggregates plus 50% crushed clay brick fine aggregate also the blend of 50% natural fine aggregate with 25% of natural coarse aggregate plus 25 % crushed clay brick coarse had highest tensile strength compare to the rest of blends (Ghazi, 2011).

Rashida, Hossain, and Islamb (2008). carried out tests on the characterization of high strength concrete made with crushed clay brick. The studies illustrated the effect of water cement ratio on the compressive strength of brick aggregate concrete at 28 days, demonstrating that water cement ratio has a contrary effect on compressive strength. Moreover, using brick as a coarse aggregate increases the strength of concrete.

There have been many tests, such as compressive strength, splitting tensile strength, suction, modulus of rupture and thermal conductivity, used to determine the mechanical properties of recycled brick masonry aggregate (RBMA) and recycled brick masonry aggregate concrete (RBMAC). Results revealed that reducing the water content of the admixture can have a desirable effect on pavement workability. Moreover, taking into consideration the acceptable compressive strength, using RBMA as a replacement for natural coarse aggregates can provide a strong concrete (Cavalline and Weggel, 2013).

Devenny (1999) has also produced a concrete containing crushed brick as the coarse aggregate, showing that crushed brick aggregate concrete had a relatively lower strength at early ages than normal aggregate concrete. The author attributed this characteristic to the higher water absorption of crushed brick aggregate compared to gravel which was used as the control aggregate. However, the investigation also found that crushed brick aggregate concrete had a relatively higher strength at later ages, which they attributed to the pozzolanic effect of the finely ground portion of the brick aggregate.

Moriconi and Corinaldesi (2004) determined that a good mortar-brick adhesion depends mainly on the quality of the interfacial zone. The recycled aggregate mortar, in spite of having the worst mechanical behaviour, showed the best mortar-brick bond strength. By means of rheological testing, it was shown that the presence of recycled material lowered the yield stress value for longer periods, enabling the mortar to better permeate the brick surface, thereby assuring a good physical interlock and as a consequence, an improved bond. Mortars containing fly ash or brick powder also showed better adhesion properties with brick than simply cementitious mortar, but bond strength was not as high as in the case of recycled aggregate mortar.

Bektas (2007) notes that recycled clay brick can be considered as a waste material obtained from demolished masonry or products used in unbound systems such as drainage layers and subbase in road construction. Based on his research, clay brick from demolished masonry can be recycled and used as a pozzolanic material in concrete. Investigations have indicated that utilization of 25% of ground clay brick (GCB) in concrete will not have any effect on the water demand and stability .In addition, a blend of pozzolanic materials such as fly ash, silica fume, metakaolin and natural pozzolan with portland cement can help increase the resistance of concrete to sulfate attack and the alkali-silica reaction.

2.3.5 ***Effect of rehydration of cement in recycled material***

Hydration is the result of the chemical and physical reactions between water and cement. Temperature has a significant effect on moisture. Heat transfer theory can be seen as an explanation for the impact of thermal and mechanical energy on the temperature of hydrating concrete (Mukhopadhyay, Dan Ye and Zollinger, 2006). The Heat of hydration can be controlled by selecting the necessary materials. Cement which contains more tricalcium silicate and tricalcium aluminate which is finer, as a higher fineness will have a higher rate of heating than other cements such as Type III cements(Skokie and Illinois, 1997).

Shui, Xuan, Wan and Cao (2008) used fine recycled concrete aggregates (FRCA) subjected to thermal treatment as the main component of building mortar. They used techniques of thermogravimetric-differential scanning calorimetry (TG-DSC) and X-ray diffraction for dehydrating and preheating FRCA. Rehydration of preheated FRCA was evaluated by examining the mechanical properties of mortar. The addition of fly ash (FA) and Portland cement to preheated FRCA was expected to raise the rehydration strength. The rehydration of preheated FRCA commenced with the addition of water, and the addition of FA to the preheated FRCA was found to increase adhesion, workability and mechanical properties.

In another study, Khater (2011) used waste concrete, grog (a product formed from fired and ground clay), hydrated lime and bypass cement dust instead of cement to manufacture building bricks. This investigation was intended to show that waste concrete and different materials could be used instead of cement in concrete brick-making. The results of testing showed that the compressive strength of mixes with variable amounts of burn dust increased with hydration age but decreased with the addition of burn dust. Furthermore, it could be seen that mixing of burnt cement dust with demolition waste was more suitable than reutilizing of dust with cement raw material.

A study has been carried out for determining the effect of hydration temperature on the solubility behaviour of Ca-, S-, Al- and Si-bearing solid phases in Portland cement pastes over 28 days at various tempratures from 5–50°C (Thomas, Rothstein, Jennings and Christensen, 2003). According to thermodynamic analysis, the saturation changes in the early period of hydration occurred faster than expected, with hydrated calcium sulfoaluminate phases showing supersaturation behaviour after the first hours of hydration. Temperature had a major effect on the curing of cement-based material, and a great impact on the hydration and properties of the hardened cement paste concrete.

Siripun, Jitsangiam and Nikraz (2009a) also performed laboratory tests to determine the mechanical behaviour of hydrated cement treated crushed rock base (HCTCRB). This material is specified for use as a base layer in Main Roads Western Australian pavements. The conventional triaxial test and repeated load triaxial test (RLTT) were performed to determine the resilient modulus and deformation of HCTCRB under simulated real traffic loading using the CIRCLY program. The properties and performance of HCTCRB was not similar to modified and stabilized materials.

Some important factors such as hydration period and the quantity of added water did not have a major effect on HCTCRB performance. In this study, an attempt was made to preserve the features of unbound material by breaking the cementitous bond produced during the hydration process in order to prevent shrinkage cracks. It is clear from this study that the properties and performance of HCTCRB are different with recycled materials. In fact, the hydration process, which depends on the amount of added water and cement content, were important aspects which have also been considered in various studies on the utilization of recycled materials in road base in WA (Siripun, Jitsangiam and Nikraz , 2009a).

The study on HTCRB showed that hydration period and the quantity of added water had no significant influence on strength, however these factors can have a significant effect on the strength and workability of recycled materials. Based on these important factors, some studies have revealed that the utilization of recycled concrete as a base course can lead to increased stiffness after the rehydration period.The test results indicated that increasing the hydration period not only creates a bound material but also may lead to base course fatigue failure. The UCS test results also illustrated the influence of curing time on increasing the strength of materials. Bearing this in mind, longer hydration periods can have a significant effect on the strength of materials. In general, the utilization of different blends of materials such as crushed brick and tiles may have potential to minimize the effects of rehydration (Leek and Siripun, 2010).

Thus, use of concrete with recycled materials can have a major effect on rehydration, stiffness and shrinkage cracking. Adding brick and tiles or sand to the recycled material as a fine material may be helpful in controlling excessive stiffness and limiting rehydration effects (Leek, Siripun, Nikraz, and Jitsangiam, 2011).

2.4 **Failure modes for flexible pavements**

In civil engineering, failure is described as "*fracture or break*". Failure occurs when loads surpass the determined acceptable rate. The loads applied to the pavement should not exceed the strength of the pavement materials. Increasing the applied load to unacceptable levels can lead to fracturing of the pavement (Erlingsson, 2013, p. 1). Studies have showed that pavement failure is not accidental, but is rather due to specific reasons whereby the pavement design could not tolerate the load. Pavement failure can be not only fundamental, such as "*deep structure rutting, alligator cracking, longitudinal or transverse cracks in slabs, etc*". but also functional, including "*surface rutting, roughness, loss of skid resistance, etc*" (Garg, Guo and McQueen, 2004, p. 18). Flexible pavement distresses can be caused by features such as "*environmental, material, structure, construction*"(Garg, Guo and McQueen, 2004, p. 19).

Various studies have shown that different factors can cause pavement failure, such as "*shrinkage cracking, thermal fatigue, top down cracking of bitumen pavement, etc*"(Vandhiyan, 2004, p. 55). Many factors can lead to crack formation, such as stresses from axle loads, temperature and moisture changes in pavement layers. Therefore, it is important to identify the type of cracks and the means required to control them . Various methods have also been considered for repairing pavement surface cracks, such as crack filling and crack sealing. Removal and replacement of damaged areas can be a useful way of controlling and repairing fatigue cracks (D0, 2010).

2.4.1 ***Load associated distresses***

"*The main structural distress is alligator cracking known as fatigue cracking. This type of distress can cause physical and mechanical pavement failure and consists of structural cracks produced by constant traffic loading in the wheel pathways*" (Garg, Guo and McQueen, 2004, p. 20). Horizontal tensile strain, which occurs at the base of the asphalt concrete, can lead to fatigue cracking(Caltrans Flexible Pavement Materials Program, 2003).

Rutting is caused by permanent strain accumulation which causes shear and this results in lateral displacement of pavement materials.With respect to traffic load, the permanent deformation of pavement layers or subgrade is caused by horizontal movement of materials or consolidation (Garg, Guo and McQueen, 2004; Erlingsson 2013). Permanent deformation can occur in different layers, and major rutting can cause structural failure of the pavement. In general, rutting happens only on flexible pavement, and can be seen by rut depth in the wheel load pathway (Caltrans Flexible Pavement Materials Program, 2003, p. 12).

Edge cracking can be observed where there is erosion, too-heavy loads and shear failure in the pavement edge(Caltrans Flexible Pavement Materials Program, 2003).

2.4.2 *Climate associated distresses*

Pavement surface which is divided into rectangular pieces is called block cracking. Environmental and material issues are the main factors causing block cracking and can lead to "*functional failure of the pavement*"(Garg, Guo and McQueen, 2004, p. 20). This type of distress is caused by shrinkage of asphalt due to the temperature variation during a day (Shahin, 2005).

Thermal stresses can lead to longitudinal and transverse cracking in the wheel path in flexible pavement.Major causes of these distresses are asphalt shrinkage because of low temperatures or hardening of asphalt or daily temperature variation (Shahin, 2005). Surface reflection cracking happens in response to the movement of cracks in underlying pavement(Caltrans Flexible Pavement Materials Program, 2003).

2.4.3 *Other distress types*

It is clear that the combined effects of traffic loads and an unsteady surface or base layer can be another reason for fundamental pavement failure. Sometimes pavement failure can be observed during road construction. When some road surfaces are lower than surrounding areas, this is called depression. "*Settlement of the foundation soil*" can cause this. Another type of distress that can make dark areas on the surface is named "*jet-blast erosion*". Some environmental factors, construction and types of materials can contribute to the creation of these cracks. Removing and replacing original areas of pavement with the same materials can also lead to pavement failure (Garg, Guo and McQueen, 2004, p. 21).

Slippage cracking can be seen when the bond between the surface and lower layers is weak (Garg, Guo and McQueen, 2004).

2. 5 **Control of shrinkage cracking by induced microcracking**

In cement treated base (CTB), shrinkage cracking happens due to desiccation and cement hydration. Microcracking is an important means of reducing shrinkage cracking, and can be done by making several passes with a vibratory roller after a short curing time. Microcracking can prevent the formation of severe wide cracks and reduce unpredictable cracking through the pavement surfacing. In order to investigate this, Sebesta (2005) used a falling weight deflectometer (FWD) to calculate the base modulus, and investigate and control the microcracking process. Microcracking had a significant impact on reducing shrinkage cracking in the base. Two factors were determined to be important: time and the amount of rolling.

Three passes of the vibratory roller after three days curing led to a significant reduction in shrinkage cracking associated with the cement treated aggregate base. Microcracking reduced the severity of shrinkage cracks in the base. The extent of cracking is related to cement content, and the inducing of microcracking can reduce total crack length. The design and construction of pavement in cold regions is of special concern, due to frost and heavy traffic. Many investigations have been done using various methods and standards in order to find appropriate aggregates and materials and determine the optimal thickness of pavement layers. In order to minimize the shrinkage cracking of cement, a specify curing time with suitable with suitable moisture levels have been considered for pavement layers. In addition, a minimal amount of Portland cement, geogrids, and lower UCS value, microcracking and stabilized subbase layers have all been used to reduce cracking (Litzka and Haslehner, 1995).

In exploring the rection of shrinkage cracking, Scullion (2001) has illustrated that there are many factors that can have an enormous effect on the severity of cracks, such as weather conditions, moisture content, curing processor, amount of cement used and aggregate characterization. According to experimental investigations, compacting the base layer at or below optimum moisture content has been suggested as a useful strategy to minimize cracking. Base modulus (stiffness) was determined with both the Humboldt stiffness gauge and falling weight deflectometer. The results indicated that five roller passes can cause microcracks to form and this can prevent the formation of wide cracks.

Halsted (2010) has also performed many studies into reducing pavement cracks, and has observed that microcracking in pavement is an important method for minimizing wide shrinkage cracks. Use of several passes of a vibrating roller one or two days after the final compaction of treated base material makes a crack pattern which helps to minimize the development of wide shrinkage cracks. An additional important factor in using the roller is to improve full-depth reclamation (FDR) bases by reducing the length and width of the cracks. FDR is a procedure with cement that allows the recycling of asphalt pavement and stabilization with cement, making a new base with a strong foundation for long pavement life. Microcracking reduces the risk of reflective cracking in the surface layer (Halsted, 2009) .

2.6 ***Cement and lime stabilization***

The use of cement stabilization in Australia has a long history. The main functions of cement stabilization are to improve the quality of pavement base and subbase materials, and reduce the base thickness needed to achieve design strength (AustStab, 2012).

Different types of cement have been used for stabilization in order to increase the strength and durability of soil (Army, 1994). Chai, Oh and Balasubramaniam (2005) found that the utilization of cement stabilized base material had a significant role in the strength of pavement.

The Australian Asphalt Pavement Association (AAPA, 2012, P. 34) has investigated some practical applications for the stabilization of pavement as part of its construction program. Lime stabilisation of the subgrade can be an effective method of increasing subgrade strength, and on occasions, premixing prior to adding lime can be an advantage. Stabilisation can be undertaken by the use of hydrated lime or quicklime, but where quicklime is used, it must be slaked with sufficient water to completely hydrate the lime prior to mixing into the soil. The lime must be well blended and thoroughly mixed by a purpose made stabilizer. After mixing compaction is essential, and depending on the layer, and a pad foot roller should be used. Whilst lime stabilisation is most often applied to a clay subgrade, it can also be used to modify plastic pavement materials.

Cement can be used for some subgrade materials. As stated in NCHRP 20-07 report (2009), cement stabilization is better applicable to coarse aggregates with sand size or larger (larger than 75µm) and this stabilization should be done by bonding coarse materials with a mixture of cement paste and fine soil aggregates. Moreover, it has been recommended in this report, cement stabilization is not suitable for the soils with high organic content, clays with high plasticity and sandy soils with poor reacting.

2.6.1 ***Control of cracking in cement stabilized pavement***

Various types of cracks can occur in pavement, such as thermal cracking, fatigue cracking or cracks which may lead to base failure so that the pavement needs to be maintained and repaired. However, some other types of crack, such as reflective cracks, can stay in place for a long time without maintenance, without leading to a decrease in pavement resistance (Halsted, 2010). The main issue is cracks of more than 6mm (wide cracks), which are not able to transfer loads. These wide cracks increase stresses and lead to performance problems (Adaska, Luhr et al. 2004).

In general, utilization of cement stabilized bases not only reduces fatigue cracking and decreases base failure, but also can improve base material performance in freezing and saturated conditions. Decreasing the vertical deflection and tensile strain can be an effective way of preventing pavement surface failure (Halsted, 2010). However they are very likley to exhibit shrinkage cracking, and the spacing and width of these cracks is a function of cemenet content, layer thickness and friction generated between the stabilised layer and the supprting layer. Generally, the wider the crack spacing, the wider the crack will be.

Adaska, Luhr et al. (2004) researched ways in which to reduce cracks in cement stabilized pavements. They found several factors had an influence, such as type of soil, cement content, degree of compaction and curing. Temperature and moisture have a great influence on the degree of shrinkage. Means of crack control included proper construction and curing of the stabilized base, and using pre-cracking for reducing crack size. The result of compaction showed that compacting cement stabilized soil using the modified Proctor method reduced shrinkage more than standard Proctor. Moreover, having moisture content at modified Proctor compaction is less than standard Proctor compaction. Another method for reducing cracking was pre-cracking to form microcracks instead of single transversal cracks.

2.6.1.1 ***Pavement stabilization***

Stabilization is a method that has an effect on the stability and strength of pavement material. Stabilization techniques have been used in road construction around the world, and particularly in Australia. Base and subgrade stabilization are used to provide a modified pavement course with a maximum compressive strength of 1 MPA. When the UCS exceeds 1MPa, the pavement is considerd bound, although the actual value does vary between road agencies as to where this boundary actually sits. In addition, lime, bituminous or cementitious binding agents such as cement, fly ash or slag, are used alone or in combination for the stabilization of the base and subbase. An important consideration in stabilization is the use of recycled materials, which can save natural resources and reduce the cost of construction (Wilmot, 2006).

In addition, subgrade stabilized materials, granular stabilized materials, modified stabilized materials and bound stabilized materials are the most common means of stabilizing pavement used in Australia (Adamson, 2012).

Guyer (2011) has indicated that mixing additives such as lime, cement, bitumen and fly ash into soil can improve some soil properties, such as strength, texture, workability and plasticity. It is obvious that using stabilization increases the strength of layers, thus helping to prevent deflection and permanent deformation of the layers. In this regard, there are some important factors to consider, such as cost, environmental conditions and type of soil (Army, 1994).

2.7 **Western Australian materials**

Many types of materials are used in highway construction in Western Australia. Recycled products obtained from construction and demolition (C&D) have been utilized in some WA road networks, and the performance of recycled crushed concrete in pavement construction has been considered. There are some practices in WA of recycling building products from C&D, and it is increasing with the increasing importance of preserving virgin materials and the natural aggregates.

Brick, tile and other hard rock sourced materials are the most widely available materials in WA. Waste materials such as concrete, brick and tile are sent either to landfill, or to a recycling facility where they are processed to make recycled aggragtes or roadbase.. Ferricrete as a natural material is produced by Quarry Park Pty Ltd at Red Hill, and is excavated at Red Hill Waste Disposal Site. It is surplus material generated by the excavation of cells for landfill use.

Experiments have shown that the use of recycled concrete can increase the stiffness and performance of pavement. The rehydration of cement content was the main reason for increased stiffness, however extreme stiffness can lead to fatigue over time. In general, blends of small amounts of waste products and natural materials can reduce excessive rehydration of recycled concrete (Leek and Siripun, 2010). In order to explore this issue, many experimental studies have done on different blends of concrete with brick and tile. In general, ferricrete, brick and tile can be mixed with recycled concrete as high density foreign materials. The investigations indicated that the combination of RCC with these foreign materials can limit the effect of rehydration and control cracking. In Western Australia, many investigations are required to determine the optimum percentage of foreign materials to add to physically sourced concrete in order to reduce the risk of developing bound highway construction layers (Leek, Siripun, Nikraz and Jitsangiam, 2011).

2.7.1 ***Previous studies on concrete characterization***

Concrete consists of Portland cement, water and aggregates. The chemical reaction between water and cement (hydration) can easily change the properties of concrete from plastic to solid. Curing is also a major factor in increasing the strength of concrete and depends on time, temperature and moisture. It is clear that the ratio of water to cement plays a significant role in the strength of concrete. Nemati (2013) has indicated that the degree of compaction also has a significant effect on the strength and durability of concrete of a given mix proportion.

Although workability and consistency are the most important concrete properties, some other factors such as maximum size, surface, shape and grade of aggregates should also be considered. With regard to the use of concrete in pavements, Lofsjogard (2003) has indicated that the differences in quality, shape, size and colour of aggregates can have an enormous effect on the functional properties, uniform strength and brightness of concrete. In fact, a uniform grading of aggregate particles can improve workability. In addition, it can be seen that time can change the strength of concrete. Water content also has an impact on the workability of concrete mix; increasing the water content will increase the flow and compatability of the mix, and can also be a reason for decreased concrete strength.

2.7.1.1 ***Recycled concrete properties***

Many studies have been carried out to investigate the density, strength and permeability of recycled concrete aggregate (RCA) in pervious concrete. The investigations showed that RCA has a higher water absorption than virgin aggregates due to the cement paste. Recycled fine aggregates also have a great effect on the performance of concrete. Results have shown that raising RCA content can lead to a decrease in hydraulic conductivity and compressive strength (Berry, Suozzo, Anderson and Dewoolkar, 2012).

As a consequence, there have been many investigations into the use of different percentages of recycled aggregates, which have shown that tensile strength and modulus of elasticity improves when the amount of water used in recycled aggregates mixes is decreased. The compressive strength, tensile strength and modulus of elasticity decreased with an increase in the precentage of recycled aggregate used in the specimens (Shing Chai NGO, 2004).

Cervantes, Roesler and Bordelon (2007) discovered that a paving concrete using recycled concrete as a coarse aggregate had the same properties as that made with virgin coarse aggregate. Their research also revealed that a blend of 50% virgin and 50% recycled coarse aggregate had the same properties as 100% virgin coarse aggregates. Other results indicated that the free drying shrinkage of the virgin and blended coarse aggregates were very similar over 28 days.

2.7.2 ***Previous studies on brick and tile properties***

Bricks are made from clay to a range of different work dimensions. Salt attack is a major durability problem for brick (Boral, 2008). In compare with the other building materials brick and concrete have better thermal insulation property.In addition, brick is very durable and because of strong ceramic bonds of this building material, it has a very high wear residance (ClayBricks, 2007).

2.7.2.1 ***Crushed brick characterization***

Recycled clay brick is obtained from demolished masonry or non-standard products.. This waste material can be used in unbound systems such as drainage blankets and road constructions. It can also be utilized as a pozzolan in concrete. Studies show that the usage of ground clay brick (GCB) in concrete can increase mortar durability (Bektas, 2007).

Khalaf (2001) investigated the use of recycled brick in asphalt. This illustrated that the main properties of recycled brick as an aggregate, such as strength, grading, density, shape and surface texture, will have an enormous effect on asphalt concrete. Abdul Kadir and Mohajerani (2011) also found that different recycled materials have a great effect on the physical and mechanical properties of fired clay brick.

2.7.3 ***Previous studies on ferricrete properties***

Crushed ferricrete is utilized as a road base material and is produced by Quarry Park Ltd at Red Hill in Western Australia. The ferricrete is generated by crushing a mixture of massive ferricrete and sandy gravel. "*According to the AS1726-987, ferricrete can be defined as a* *material generated by the deposition of iron oxides dissolved by groundwater, transported in solution and deposited by physical or chemical means* ". A huge amount of ferricrete in Western Australia is identified as "*laterite, caprock, massive laterite, duricrust and ironstone*". This material can be defined as "*hardpan laterite*". In Western Australia, Pisolitic ferricrete is considered to be "*lateritic gravel*" or sometimes just "*gravel*" (sic). It is more than 30 years since the combination of small amounts of crushed ferricrete and virgin lateritic gravels began to be commonly used for constructing pavement in Western Australia (Coffey Partners International Pty Ltd, 1995, p. 4).

There are various types of ferricrete—"*Y-type, Q-type and B-type*"—which are different in their physical properties. Y-type is smaller than the other types and is parti-colored dark red to purple to gray due to the presence of manganese oxides such as pyrolucite and ferrihydrite.The Q-type is "*clast rich and cemented by bright orange oxides of perhaps limonite and goethite, and clay*".The B-type "*consists of shelves of orange, clast-rich ferricrete*". Many studies carried out for presenting the iron minerals in the ferricrete. Laboratory tests, XRD and SEM results illustrated the different types of minerals present in each type (Pierce, 1995, p. 2).

Ferricrete is composed of different "*sub-horizons, namely mottled, mixed nodular and pseudo-pisolitic*". It consists of Fe_2O_3 (30–50%), Al_2O_3 (more than 25%) and SiO_2 (more than 30%). Ferricrete develops on ferruginous sandstone with iron oxide/hydroxides forming a coating around rock fragmentsduring successive rise and fall in groundwater levels. (Konka, Gebreselassie and Hussen 2013, p. 16).

Various tests have been performed to determine the quality performance of lateritic gravel, with the aim of extending the utilization of natural/or non-standard pavement materials. The laboratory tests included repeated load triaxial (RLT) to determine the resilient modulus and permanent strain properties, after which the pavement performance in the field and in the laboratory were compared. According to the RLT test, the influence of density on the performance was not significant compared to the effect of variation in moisture content. The FWD maximum deflections were similar on the surface of the cement-treated subbase (CTSB) at different sites (Sharp, Vuong, Rollings, Baran and Metcalf, 1999).

As ferricrete is a manufactured material by crushing, screening and blending, particle size distribution can be controlled. The presence of sesquioxides Fe_2O_3 and Al_2O_3 can aid resistance to wear and erosion because due to self-cementing properties. In some sealed roads, meandering cracks developed due to the self-cementing properties of lateritic gravel. These cracks had no adverse influence on pavement performance. There has been no observed increase in these cracks in roads constructed with crushed ferricrete. According to the resilient modulus test results, ferricrete was stiffer than bitumen stabilized limestone at 70% OMC, however this material showed the same stiffness with 80% OMC indicating moisture sensitivity. (Coffey Partners International Pty Ltd, 1994, p. 9).

2.8 **Test methods**

In this research project, a number of standard and non-standard tests were applied to investigate material behaviour. These were :

- Compaction test;

- Unconfined compressive test;
- Modified beam test;
- Repeated load traxial test.

2.8.1 ***Modified compaction test***

"*Compaction is the densification of soil materials by the use of mechanical energy*" (Reynolds, 2012, p. 2). Soil compaction is an important part of the construction process. In practice, most building and road projects use mechanical compaction techniques. Soil is used as fill material in many projects. Compaction is essential for achieving an acceptable soil density. In order to deliver suitable results at a reasonable cost, the precise degree and type of compaction necessary should be considered. In general, compaction is a method for improving the engineering properties of soil by increasing the density (Head, 2006)

The modified compaction test was performed to determine the maximum dry density, optimum moisture content relationship for a fine, medium and coarse grained soil by using a modified compaction process, test method 133.1 WA (MRWA, 2012i). The main objectives of the compaction process include increasing the load-bearing capacity, stability, avoiding soil settlement and frost damage, and reducing permeability, swelling and contraction. Some factors have a considerable effect on compaction such as the type of soil, water content and the compaction energy applied (Budinger and Associates, 2011).

2.8.1.1 Definition and theory

The modified compaction test is the process of combining dry soil with different percentages of water, through which the dry and solid soil particles become closely packed together, this process increasing the soil density (Head, 2006). In modified compaction, the soil is compacted in five equal layers by applying 25 uniformly distributed blows of a steel rammer weighing 4.9 kg, dropped from a height of 450mm as per test method 133.1 WA (MRWA, 2012i).

The mechanical energy applied by modified hammer is derived as follows in equation 2.1:

$$E = (N_b \times \mathrm{N_L} \times W_h \times H)/V \qquad (2.1)$$

Where:

E = Energy of compaction

N_b= Number of blows per layer

$\mathrm{N_L}$= Number of layers

W_h= Drop height of hammer

V= Volume of mould

The degree of compaction can be determined by dry density, which in turn can be determined from soil density and water content. Soils are stiffer and compaction is more difficult when water content is low. Increasing the water content will therefore improve compaction and make soils more effective, and the dry density of soil will decrease with a higher water content (McMahon, 2010).

It is also obvious that specific gravity of the soil and the water density are constant, based on the equation below, and the zero-air-void density is inversely related towater content. The reasonable compaction curve will always be below the air-void curve. Compaction is easier with added water, because the water lubricates the particles. When the water content increases to beyond the optimum value, the void spaces fill with water and further compaction is impossible.

$$\gamma_{zav} = \frac{G_s \gamma_w}{1 + wG_s} \tag{2.2}$$

where:

$\gamma_{z.a.v}$ = Density at zero air voids

G_s = Specific gravity of soil solids

γ_w= Water density

w= Moisture content

In relation to the above, reducing the air voids in the soil by compaction should not be confused with consolidation. The air voids can be reduced to a minimum with a suitable control by compaction, but not completely eliminated (Head, 2006).

Figure 2.6 shows the influence of different compaction energy. As mentioned in the test method 133.1 WA (MRWA, 2012i) , the modified Proctor compaction test uses a compaction energy of 2703 kJ/m^3, much higher than the compaction energy used in the standard Proctor test (596 kJ/m^3) outlined in the test method 132.1 WA (MRWA, 2012). In addition, it can be observed that optimum moisture is reduced by increasing the energy although the maximum density is increased. The compaction curve shows the relationship between dry density (γd) and water content (w) for a particular soil compacted at continuous energy.

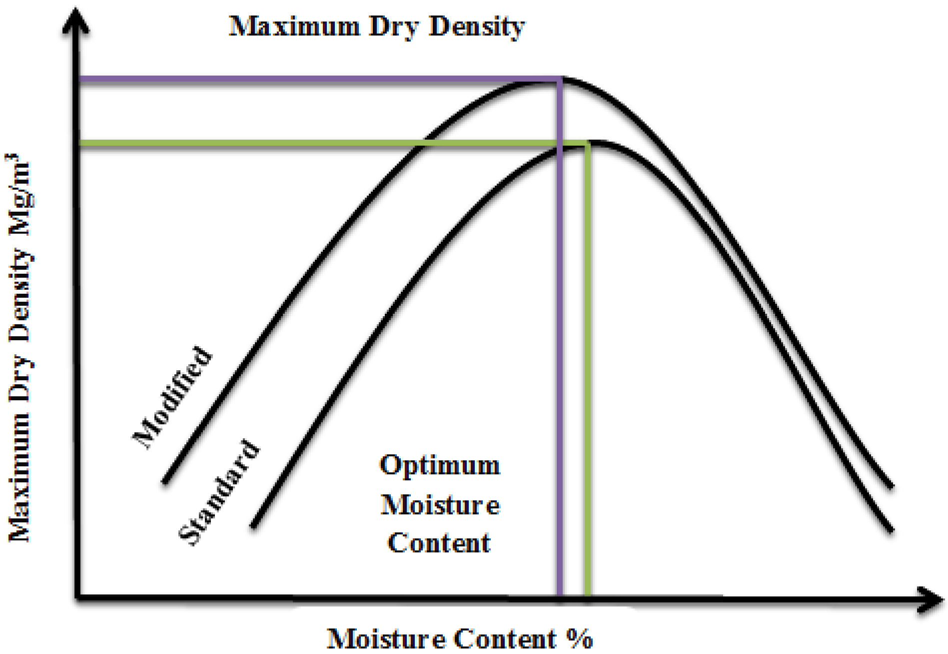


Figure 2.6: Effect of compaction energy on compaction curves

2.8.2 *Unconfined compressive test*

The unconfined compressive strength (UCS) test is important for geotechnical engineering purposes and is used for estimating the soil strength. The UCS test WA 143.1 (MRWA, 2012j) is the simplest, fastest and cheapest laboratory testing method to determine the mechanical properties of compacted specimens of unbound, bound, self-cementing materials, rocks, and fine-grained soils. It provides measurements of the undrained strength and the stress-strain properties of the rock or soil. The UCS test is frequently used in laboratory-testing programs for geotechnical investigations, especially when dealing with rocks. It is not suitable for dry sands or fragile clays because the materials would collapse under the application of the first load. "*This test is also used to evaluate the unconsolidated, undrained shear strength of clay under unconfined conditions. According to the ASTM standard, the unconfined compressive strength (qu) is defined as the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test*" (Reddy, 2002, p. 145).

Some studies indicate that the UCS test is not suitable for noncohesive and coarse grained soils. It can be observed that "*the UCS test is strain controlled and when the soil sample is loaded rapidly, the pore pressures (water within the soil) do not have enough time to dissipate*"(Sargent, 20012, p. 2).

2.8.2.1 Definition and theory

The unconfined compression test is one of the most common methods of soil shear testing. Specimens are prepared by extruding from the compaction mould into very thin membranes(Bowles, 2009).

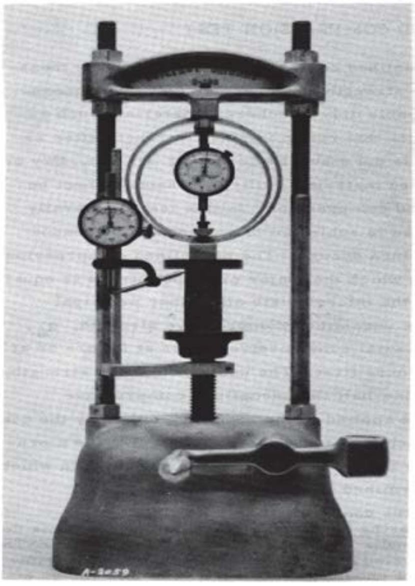


Figure 2. 7: Antique loading device **Figure 2. 8: Modern loading device**

The old and modern unconfined compressive strength devices are shown in Figure 2.7 and Figure 2.8. Undisturbed and disturbed specimens can be prepared in the same way as for triaxial tests. For performing the test on disturbed specimens, the sensitivity of the soil should be taken into consideration. For this reason, undisturbed specimens are larger than test specimens. In fact, preparation of the specimens is an important part of the test. For example, the specimen may be covered with a thin layer of grease or petrolateum while drying it before and during the test, in order to preserve the moisture content (Vulcanhammer, 2001).

After preparing the sample for the unconfined compressive strength test, determining the average diameter and length of the samples is important. In this test, there should be zero deformation when the test specimen is placed on the lower bearing block of the compression testing device, and when the top of the specimen comes into contact with the platen. The load should be applied at a continuous rate until the loads drop off in two continuous readings or two readings further than the 15% strain value.The average of these values is considered to be the compressive strength (Bowles, 2009; MRWA, 2012j).

Based on this experiment, some factors can be calculated as follows:

- Determine strain ε

$$\varepsilon = \frac{\Delta L}{L} \tag{2.3}$$

Where:

ΔL = change in length of the specimen as read from the dial gauge

L = length of the test specimen

- Determine the average cross-sectional area, A' for each reading determined

$$A' = \frac{A_0}{1-\varepsilon} \quad (2.4)$$

Where:

A_0 = the original average area of the specimen in mm^2

- the load per unit area (unconfined compressive stress), q_u for each reading intended

$$q_u = \frac{P}{A'} \times 1000 \ kpa \quad (2.5)$$

Where:

P = Applied Axial load N

- The shear strength, τ_f equals to half of the unconfined compressive strength calculated

$$\tau_f = \frac{q_u}{2} \quad (2.6)$$

In addition, the shear strength from the Torvane and the Pocket Penetrometer tests are compared. The results indicated that the the maximum possible Torvane reading should be $1kg/cm^2$ (Bowles, 2009).

2.8.3 ***Repeated load triaxial test***

The repeated load triaxial test (RLT) was developed to simulate and determine the effect of heavy traffic on base coarse aggregates. According to Austroads (2000), this test can determine the permanent deformation and resilient modulus of unbound pavement materials with a maximum particle size of 19mm.

In general, RLT is used to evaluate the various stresses affecting the base and subbase. For pavement design, there are some important factors such as type of materials, environmental conditions, stiffness and the strength of pavement layers. Whilst pavement design can be based on finite element methods, and much work is currently in progress to use this method to refine models, currently, studies have illustrated that a multilayer linear system can provide the simplest simulation of flexible pavement and are compatible with computer power that has been available. In this system, the linear elastic behaviour of all layers is considered. In addition, the stresses and strains can be Investigated by using a multilayer elastic computer program.

Ozel and Mohajerani (2011, p. 10) showed that the resilient modulus of different fine-grained subgrade soils can also be an important factor in pavement design. In order to explore this issue, stress levels were considered as a percentage of the confined and/or unconfined soil static strengths The resilient modulus was determined based on a semi-logarithmic model, unconfined compressive strength and deviator stress.

The result of Unconsolidated undrained static triaxial (UU) and unconfined compressive strength (UCS) test results illustrated that the same stress levels. Both "*octahedral stress model and a semi-logarithmic model*"presented and developed for determining the resilient modulus (Er) of fine-grained soil in flexible pavement design.

Huang (1993) has also stated that the resilient modulus of base and subgrade material are important parameters in the design of new pavements. The resilient modulus is the elastic modulus to be used with elastic theory. Most paving materials are not elastic, but experience some permanent deformation after each load application. However, if the load is small compared to the strength of the material and is repeated a large number of times, the deformation under each load repetition is nearly recoverable, such that the material can be considered elastic.

The researches showed that "*field resilient modulus was typically associated with a repeated deviator stress of 42Kpa and that unconfined compression tests would yield an accurate estimate of the resilient modulus*" (Nazarian, Pezo and Picornell, 1996, p. 4).

2.8.3.1 Models for resilient modulus

Nazarian, Pezo and Picornell (1996) revealed that in a repeated load traxial test, the resilient modulus of subgrade and base materials can be determined by placing a specimen in a cell and applying repeated axial loads.The resilient modulus is calculated from:

$$\mathrm{M_r} = \frac{\sigma_d}{\varepsilon_a} \tag{2.7}$$

Where:

$\boldsymbol{\sigma}d$= axial deviatoric stress

The resilient axial strain can be determined from:

$$\varepsilon_a = \frac{\Delta \mathrm{L}}{L} \tag{2.8}$$

Where:

ΔL = recoverable axial deformation along a gauge length

The Poisson's ratio υ determined from:

$$\upsilon = \frac{\varepsilon_l}{\varepsilon_a} \tag{2.9}$$

Where:

ε_1 = lateral strain

Siripun, Jitsangiam and Nikraz (2009a) also mentioned that the resilient modulus characteristics can be determined by K-Theta (K-θ) model using equation 2.10 as follows:

$$M_r = k_1\theta^{k_2} = 7.684\theta^{0.591} \tag{2.10}$$

Where:

M_r = resilient modulus

θ = bulk stress ($\sigma_1 + \sigma_2 + \sigma_3$)

$\sigma_1 = \sigma_2$ = major principal stresses

σ_3 = confining stress

K_1 & K_2 = regression coefficients

CHAPTER 3

RESEARCH METHODOLOGY AND EXPERIMENTAL PROGRAM

3.1 **Overview**

This chapter describes the research methodology and experiments conducted in this research. The major aim of this study was to develop and determine a method to increase the stiffness of recycled materials, and investigate effect of rehydration of cement in road base. Based on standard and non-standard techniques, the strength and durability of Western Australian recycled road base materials were assessed and evaluated to control and reduce the effect of microcracking and shrinkage. The study focuses on recycled crushed concrete (RCC) which is widely used as an unbound granular road base material in Western Australia.

Sophisticated laboratory tests were carried out as shown in Figure 3.1. Firstly, the basic properties and characteristics of materials such as RCC, ferricrete, crushed brick and tile were investigated. Subsequently, the main tests were performed to investigate the mechanical behaviour of the materials under various conditions were conducted. Once laboratory testing was complete, the test results were analysed and their application to and importance for pavement design was determined. Lastly, conclusions were drawn and recommendations were made. The initial and main tests on recycled materials were carried out in the Geomechanics laboratory at Curtin University. The experimental techniques including methods and testing procedures are presented as shown in Figure 3.1

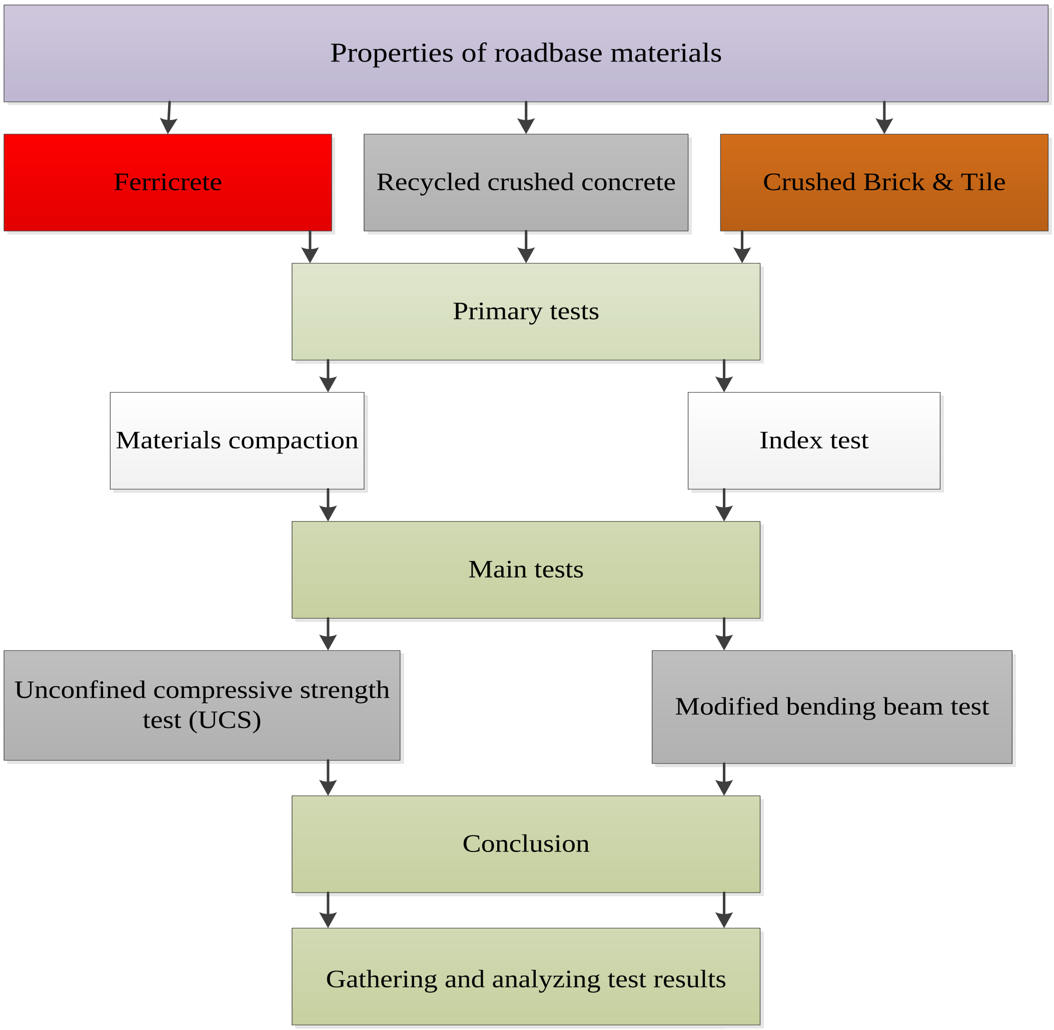


Figure 3.1: Diagram of the research process

3.2 Materials and Methods

The experimental design for this research is shown in Figure 3.2.

3.2.1 Stage 1: *Characterisation of individual recycled aggregates and blends of recycled aggregates*

As shown in Figure 3.1. Index testing was carried out in stage 1 covering the following objectives:

- To assess the physical and mechanical properties of RCC, brick&tile and ferricrete using standard test methods.
- To assess the physical and mechanical properties of different blends of RCC and varying proportions of brick&tile and ferricrete as shown in Table 3.1.

Table 3.2 shows the standard tests used to assess the physical and mechanical properties, while Table 3.3 presents the number of replicates for each blend of aggregates tested for UCS.

Table 3.1: Proportion of aggregates (% w/w) present in each blend subjected to physical and mechanical tests

Name of Blend	Aggregates		
	RCC (%)	Ferricrete (%)	Brick and Tile (%)
A	100		
B	90	10	
C	70	30	
D	50	50	
E	90		10
F	70		30
G	50		50

Table 3.2: Physical and Mechanical Tests (Index tests) and Standards Used

Physical properties	
Name of test and acronym	Name of Standard and reference (author/org and year)
Particle size distribution (PSD)	Test method WA 115.1– (MRWA, 2011d)
Los Angeles abrasion (LA)	Test method WA 220.1– (MRWA, 2012k)
Plastic limit LPL)	Test method WA 121.1– (MRWA, 2012g)
Linear Shrinkage (LS)	Test method WA 123.1- (MRWA, 2012h)
Liquid Limit (LL)	Test method WA 120.1-(MRWA, 2012e) & WA 120.2– (MRWA, 2012f)
Maximum dry density (MDD)	Test method WA 133.1– (MRWA, 2012i)
Optimum moisture content (OMC)	Test method WA 133.1– (MRWA, 2012i)
Mechanical properties	
Unconfined compressive strength	Test method WA143.1– (MRWA, 2012j)

Table 3.3: Number of samples for each blend subjected to UCS

Name of blends	Number of replicates
A	5
B	7
C	7
D	6
E	8
F	7
G	7

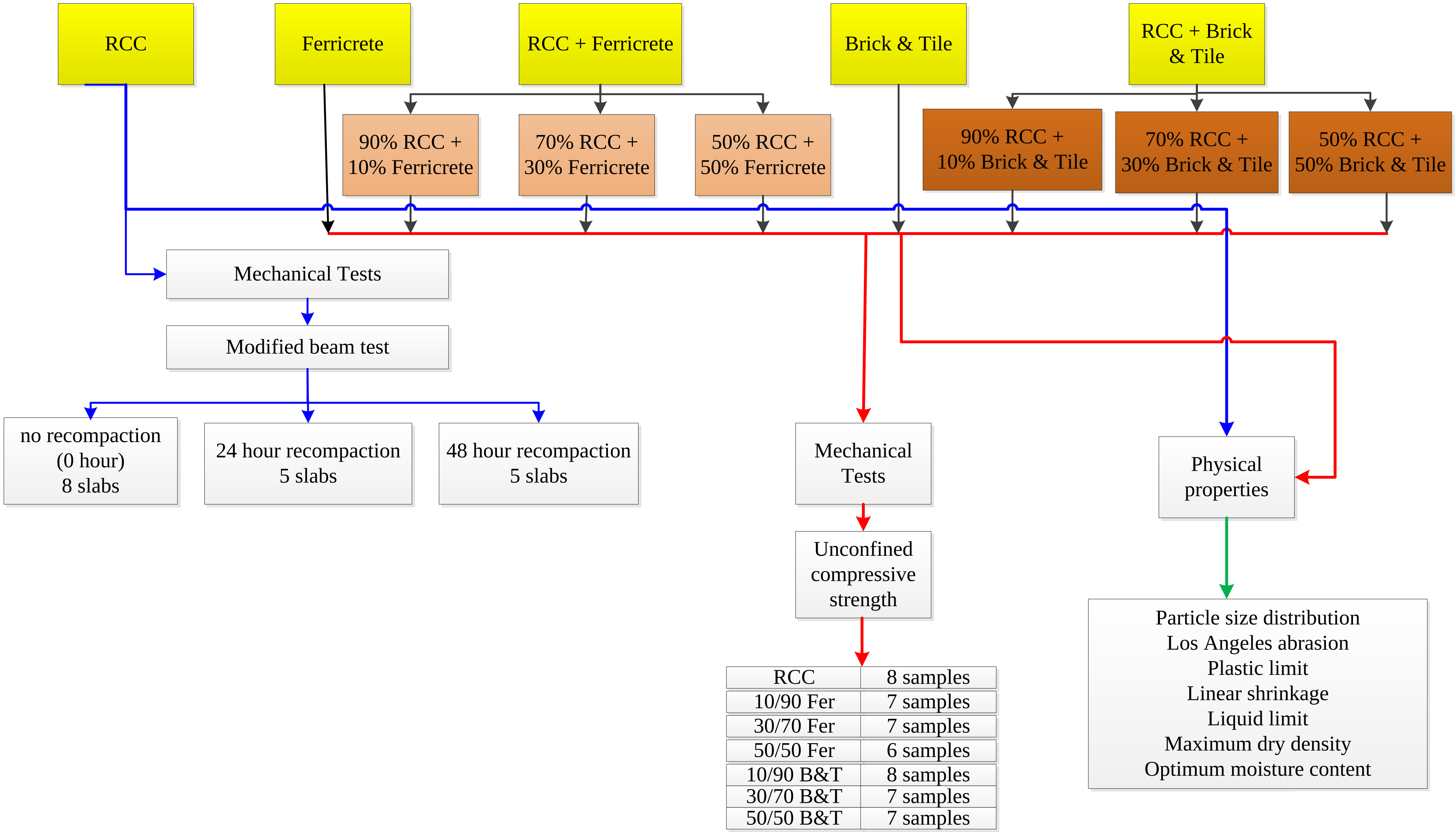


Figure 3.2: Experimental design

3.2.2 **Stage 2:** ***Investigation of the effect of recompaction on pure RCC***

This stage addressed the effects of post construction compaction on UCS, bending strength and stiffness of pure RCC beams. Unlike the previous stage, only RCC beams were tested in this stage as the intention was to examine if straight RCC could be utilised with confidence by the application of post construction trafficking. Bending strength and stiffness were tested using a modified test method described as follows.

Modified bending beam test

Some experimental tests have been undertaken to assess the crack position and bending load capacity in concrete slabs manufactured from recycled crushed concrete only. Raghu Prasad, Kumar Saha and Gopalakrishnan (2010) determined the crack position and peak load of plain concrete beams with different thicknesses. The beams were placed on steel roller supports, and three- point loading was applied. The aim was to determine the life of concrete pavements and partially cracked slabs and overloaded slabs on highways. In addition, this method can also evaluate the feasibility of using cementitious-based materials such as recycled aggregate concrete, geopolymers, and fiber reinforced concrete in pavement applications.

As the aim of the current research was to determine the effects of post construction compaction on the ability to control strength gain due to rehydration, a non-standard test method was required. There is currently no standard test available to undertake this type of testing, and a non-standard test was developed. This test will be used in future research as identified in this report.

It is thought that inducing microcracks can prevent the development of shrinkage cracking (Sebesta, 2005). This experimental and non-standard test was performed and analysed by using a Cooper slab compactor to manufacture slabs of recycled pure crushed concrete and then recompacting to induce microcracking.

The applications of several vibratory roller passes was to replicate the situation in the field where a completed pavement would be recompacted some period after initial compaction was completed to cause the creation of microcracks, and reduce the incidence of larger shrinkage cracks. The slabs were prepared at OMC, and then cured for 24 and 48 hours respectively prior to recompaction assess the effect on bending strength. The slabs were then left for 56 days to determine if rehydration continued after recompaction.

The Table 3.4 presents three different methods of manufacturing the slabs. The test set up is shown schematically in Figure 3.3.

Table 3.4: Different compaction conditions for manufacture of slabs

Curing (hrs)	Manufacturing method
0	Slabs made, cured 48 hours, moulds stripped and tested at 56 days
24	Slabs made, cured 24 hours, recompacted, moulds stripped at 48 hours and tested at 56 days
48	Slabs made, cured 48 hours, recompacted, moulds stripped at 48 hours and tested at 56 days

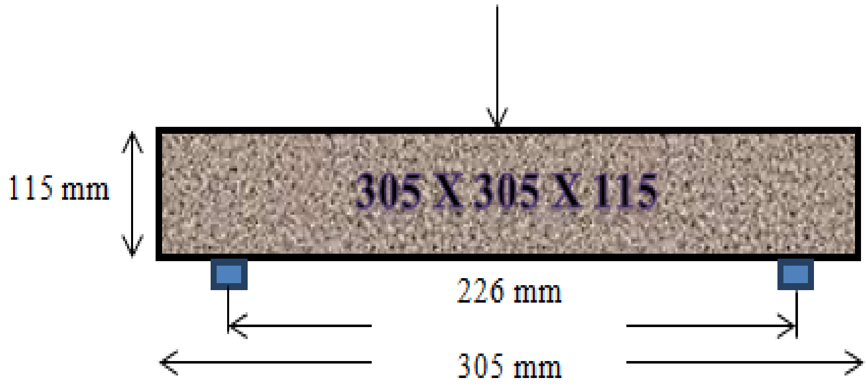


Figure 3.3: Schematic layout for modified beam test

The Cooper slab compactor is designed to allow asphalt slabs to be compacted to a defined density. It was considered that this device could be adopted to manufacture slabs to a density, and then recompact the slabs after a curing period to ascertain the effects of creating microcracks in the slab post construction. This was intended to simulate in the laboratory the proses in field, where a roller is applied to a pavement after a curing period to induce microcracking. The theory was that if microcracks are induced into the pavement after the initial hydration period, the development of larger single cracks could be eliminated.

The success of the development of microcracking was intended to be determined by the vertical load required to break the slab when subjected to a line load applied centrally and parallel to supports on either edge of the slab as shown in Figure 3.4.

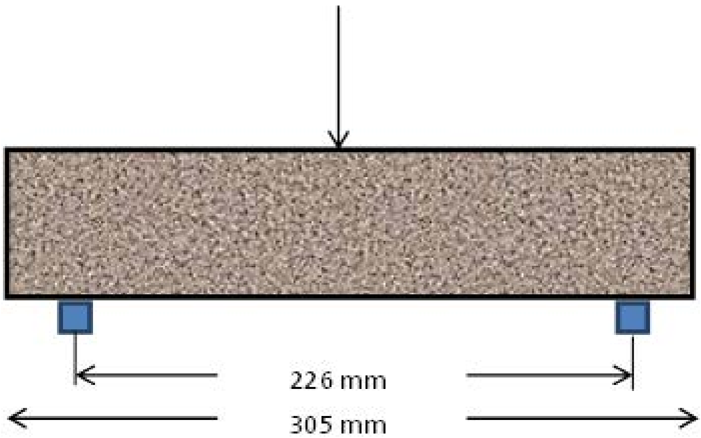


Figure 3.4: Test set up for modified beam strength

The load was applied by gradually adding water to a 50mm wide, 305 long, 300mm high container and determining the mass of water required to crack the slab, as it was anticipated that the breaking force would be low. As will be discussed in chapter 4, the test required some modification due to issues encountered during the initial tests.

The objective was to determine the effect of delayed recompaction (or post construction compaction) on the strength and stiffness of slabs (305x305x115mm) made of various blend of recycled materials using a non-standard beam bending test described above.

Table 3.5 shows the number of replicates tested for samples that were (i) not subjected to recompaction or Condition 0, (ii) subjected to recompaction after 24 hours, and (iii) subjected to recompaction after 48 hours.

Table 3.5: Number and labels of RCC beams subjected to modified beam bending.

Recompaction delay	Number of replicates (slabs)	Beam Labels
no recompaction (Condition 0)	8	5,6,9.10, 17, 18, A, B
24 hour recompaction	5	2, 4, 8, 15, 18
48 hour recompaction	5	7, 11, 12, 13, 14

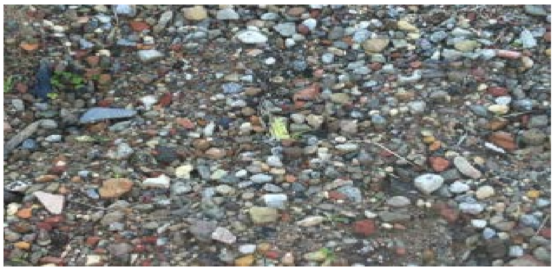
3.3 **Materials in this research**

The materials used in this study were chosen for two reasons: the economic aspect, and reduction in the consumption of virgin materials in Western Australia. All of the selected materials were available in Perth, and were already widely used in various structures and road construction projects.

3.3.1 ***Recycled crushed concrete (RCC)***

Recycled crushed concrete materials are manufactured from construction and demolition waste and are increasingly commonly used in road applications. Once the recycled crushed concrete has been sieved or decontaminated, it can be used as base, subbase material in road construction, aggregate for new concrete and for other purposes. The recycled crushed concrete (RCC) used in this research was collected from a local stockpile at the Capital Recycling Station in Perth, Western Australia.

(a)



(b)

Figure 3.5: (a) Recycled crushed concrete (RCC) (b) Commingled brick and tile with RCC used in this research

3.3.2 ***Crushed brick and tile***

Crushed brick and tile was also utilized in this research, and was collected from a local stockpile at the Capital Recycling Station in Perth, Western Australia. In this study, different percentages of crushed brick and tile with recycled crushed concrete were used and many tests were performed to determine the properties and strength of these materials in accordance with the Australian standards for use in future pavement projects.

Figure 3.6: Crushed brick and tile used in this study

3.3.3 *Crushed ferricrete*

Ferricrete is a hard, erosion-resistant layer of material that consists of sediments cemented by iron oxide near the surface of the soil. Ferricrete contains sediments and other non-indigenous materials, which have been transported from outside the immediate area in which it occurs. The iron oxide cements are derived from the oxidation of percolating solutions of iron salts. The word ferricrete is derived from the combination of ferruginous and concrete. Ferricrete is used widely in South Africa for road constructions, and is also found in the western and remote eastern areas of Australia. The ferricrete used in this research was produced and collected from the Red Hill Waste Management Facility in Western Australia.

Figure 3.7: Ferricrete used in this research

3.4 **Non-mechanical material properties**

The non-mechanical behaviour of soil has a significant effect in road construction and other applications. Knowledge of the characteristics of pavement materials is essential for classifying the categories, physical form and index properties of materials. A number of laboratory tests were therefore done to determine the non-mechanical properties of the materials in this study.

Primary tests investigated the properties of recycled crushed concrete, crushed brick and tile and crushed ferricrete as follows:

- Particle size distribution
- Los Angeles abrasion
- Liquid limit
- Plasticity index

- Plastic limit
- Linear shrinkage
- Compaction

The laboratory trials carried out to identify these characteristics were performed in accordance with Main Roads Western Australia Standards.

3.4.1 ***Particle size distribution test***

Particle size distributions of materials were determined by using a sieve analysis test procedure in accordance with test method WA 115.1 (MRWA, 2011d). In this research, the maximum aggregate size of the material was determined as < or > 19 mm, and the required 5 kg of material was used to determine the PSD in accordance with test method WA 115.1 (MRWA, 2011d) and test method WA 105.1(MRWA, 2011c). Figure 3.8 and Figure 3.9 show the equipment were used for hygroscopic moisture content and Decantation tests in this research.

Figure 3.8: 30g of sieved material used for determining the hygroscopic moisture content

Figure 3.9: Decantation test and equipment used in this study

Figure 3.10: Particle sieves and sieve shaker machines

Particle size distribution of ferricrete, crushed brick and tile was also performed in accordance with test method WA 115.1 (MRWA, 2011d). The sample preparation and test method WA 110.1 (MRWA, 2011L) was the same as for the recycled crushed concrete (RCC). In accordance with test method WA 105.1. A minimum test portion mass of 5.0 kg was adopted as the nominal maximum size was less than19 mm. The fine and coarse sieving, hygroscopic moisture content and decantation were performed similarly to the tests carried out on RCC.

3.4.2 ***Los Angeles abrasion test***

The Los Angeles abrasion test is appropriate for coarse aggregate of different sizes and is not used for fine aggregate. This test is used for determining the Los Angeles abrasion value of aggregate particles derived from crushed rock or gravel. In this study, the Los Angeles abrasion value of 5 kg of recycled crushed concrete (RCC) was determined in accordance with test method WA 220.1 (MRWA, 2012k).

Figure 3.11: The Los Angeles abrasion device used in this research

3.4.3 *Liquid limit*

The liquid limit (LL) is the water content at which a soil changes from plastic to liquid behaviour. The liquid limit is one of the most generally performed of the Atterberg Limits along with the plastic limit. These two tests are used to classify soil. The original liquid limit test method is Atterberg's method, later refined by Casagrande. In this study, two different liquid limit test methods (the cone penetrometer method and Casagrande) were used to determine the liquid limit of recycled crushed concrete with different blends of brick and tile and ferricrete.

The Casagrande method was undertaken in accordance with MRWA test method WA 120.1 (MRWA, 2012e) and the cone penetrometer method in accordance with MRWA test methods WA 120.2 (MRWA, 2012f)

Figure 3.12: Casagrande apparatus and equipment used in this study

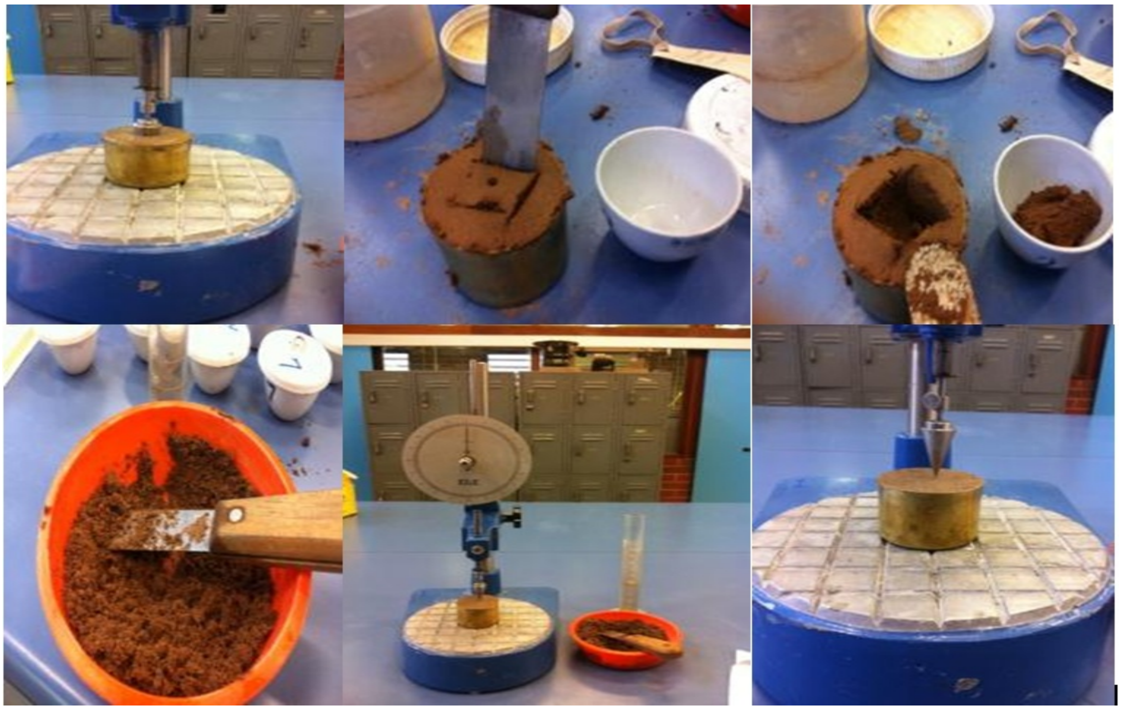


Figure 3.13: Cone penetrometer apparatus and equipment used in this study

3.4.4 *Plastic limit*

The plastic limit is defined as the moisture content at which the mechanical properties of soil change. In this research, test method WA 121.1 (MRWA, 2012g) was adopted for determining the plastic limit of soils and granular pavement materials.

Figure 3.14: Rolling the recycled crushed concrete to form a 3 mm diameter thread with crumbling

3.4.5 *Plasticity index*

The numerical difference between the liquid limit (LL) and the plastic limit (PL) of a soil or granular pavement is defined as the plasticity index, based on test method WA 122.1 (MRWA, 2012m). As a result of the plastic limit and liquid limit tests on recycled materials in this research, the determination of the plasticity index was investigated.

3.4.6 *Linear shrinkage*

Linear shrinkage can be defined as a measure of the swelling properties of a soil. It is the decrease in a single dimension of a soil sample after oven drying removes the moisture content. This test gives the linear shrinkage of a soil and can be used for soil of low plasticity, including silts, as well as for clays. In this research, the test method WA 123.1 (MRWA, 2012h) used for determining the linear shrinkage of soils and granular materials.

Figure 3.15: Preparation of linear shrinkage recycled materials in this study

Figure 3.16: Linear shrinkage of pure RCC with different blends of ferricrete and brick and tile after drying in the oven

3.4.7 *Compaction*

Figure 3.17: Modified compaction equipment

Standard and modified compaction tests are two common methods used for engineering purposes. The test processes are different in the energy input to the soil layers. The modified compaction test method, which was mentioned in Chapter Two, was performed to determine the maximum dry density (MDD) and the optimum moisture content (OMC) of the materials (Figure 3.17).

In order to ensure greater repeatability, the automatic soil compactor as shown in Figure 3.18 was used, as this applies a repeatable distribution of blows over the sample which cannot be achieved with the manual hammer, this device was also used for making cores for UCS testing and determining the OMC and MDD in this research.

Figure 3.18: Modified Compaction process by the automatic soil compactor machine

Figure 3.19: Samples prepared and cured for the modified compaction test in this research

3.5 Mechanical characterization and properties

The mechanical characteristics of pavement materials are important factors affecting pavement structure. In fact, pavement longevity depends upon the quality and long-term performance of the materials used. The mechanical reactions of pavement materials are determined according to their uses in different pavement constructions. Several experimental laboratory tests were performed to assess the significant mechanical properties of pure recycled crushed concrete (RCC), as well as blends of RCC with crushed brick and tile and ferricrete.

In this study, the unconfined compressive strength test (UCS) was performed to determine the relative strength of pure crushed concrete and the different concrete blends containing brick and tile and ferricrete. A modified beam test was also performed as a non-standard test on the pure RCC. Laboratory processes were carried out to investigate the mechanical properties and strength of recycled materials in accordance with Australian Standards and the Main Roads Western Australia methods for testing aggregates for engineering purposes.

3.5.1 ***Unconfined compressive strength (UCS) test***

The standard method WA 143.1 (MRWA, 2012j) was used for the unconfined compressive strength (UCS) test in this research. The UTM-25 universal testing machine in the Geomechanics Laboratory, Department of Civil Engineering at Curtin University was used, as shown in Figure 3.20. This apparatus was used for two different tests in this research: the bending test for slabs and the UCS test for various cores consisting of both pure crushed concrete and different blends of brick and tile and ferricrete with crushed concrete.

Figure 3.20: UTM-25 device

3.5.1.1 Preparation of sample cores

In this study, many cores were made with different blends of recycled crushed concrete, brick and tile and ferricrete. All sample cores were prepared in accordance with the automatic sample compactor in accordance with test method WA 143.1 (MRWA, 2012j) and the unconfined compressive strength of the blends was determined. The blend ratio included 100% recycled crushed concrete, 10%, 30% and 50% crushed brick and tile, and 10%, 30% and 50% ferricrete with recycled crushed concrete. In general a total of six cores were made and compacted for each blend.

Figure 3.21: Cured samples

Figure 3.22: Making cores by Automatic soil compactor device

Figure 3.23: Compacted RCC core

Figure 3.24: Extrusion apparatus and equipment

- After extruding the core, the experimental wet mass of each core was determined with a balance as shown in Figure 3.25.

Figure 3.25 : Wet compacted cores

- The moisture and weight of the cores were determined and calculated at 60% OMC.
- The cores were placed in the oven at a temperature of 110ºC, for drying back to 60% OMC.

Figure 3. 26:: Cores dried back to 60% OMC

- On achieving the required mass, the cores were taken out of the oven and wrapped to maintain 60% of OMC

Figure 3. 27: Wrapping the cores

- Finally the cores were cured and stored for 56 days at room temperature.

Figure 3.28: Curing and keeping the specimens for 56 days

3.5.2 ***Modified bending beam test***

The modified bending beam test used in this study is a non-standard test method. In this research, many slabs of recycled pure crushed concrete were manufactured by using a Cooper compactor device, after which the slabs were re-compacted under different curing conditions to induce microcracking. For this investigation, many attempts were made to extrude the beams and cores in order to test these samples for strength and effect of microcracking.

3.5.2.1 Preparation of sample slabs for bending beam test

Sample slabs were prepared as follows:

- The recycled crushed concrete was prepared and dried in the oven at a temperature of 110°C for 24 hours based on the sample preparation test method WA 105.1 (MRWA, 2011c). The samples were dried and allowed to cool.

Figure 3.29: Cooling the RCC after drying in the oven

- The maximum dry density (MDD) and optimum moisture content (OMC) of pure RCC, paramount factors in preparing the samples, were determined by the modified compaction test.

- The mass of pure RCC required for compacting in the slabs was determined, based on the dimensions of the slab moulds (305 mm x 305 mm x 115 mm) and an MDD of 98%. It was determined that 19.82 kg of RCC would be required for each slab. The dried RCC was placed in a plastic bag and the quantity of water required for optimum moisture content was added to the samples and mixed completely. The samples were cured for two hours.

Figure 3.30: Curing the samples

- The height of the sample was set to be 115 mm and as the manufactured height of the slab moulds was 120 mm, plywood with a thickness of 5 mm was placed in the bottom of each mould. The plywood was then covered with a thin layer of plastic to allow for easy removal of the slabs after stripping the moulds.

Figure 3.31: Plywood placed in the bottom of a slab

The maximum height of the roller compactor device used for compacting the samples was 120 mm; the compaction process was performed in two separate layers of 75mm followed by 40mm. The height of the sample to produce the required density was set on the Cooper slab compactor and the machine continues compaction until that height (and hence density) is achieved.

(a): Compaction of RCC slabs to a height of 7.5 cm **(b): Compaction of RCC slabs to a height of 115 cm**

Figure 3.32: Compaction of pure RCC slabs by (Cooper) roller compactor device

Figure 3.32 illustrates the two steps for slab compaction by a Cooper compactor device. Figure (a) shows the first part of the compaction to a height of 7.5 cm and Figure (b) shows the second part of the compaction to a height of 115 cm.

- Slabs were compacted to reach a MDD of 98%. Slabs were compacted by vibration at four levels to reach a proper height.

In this research, 20 slabs were made under three different curing conditions. Under the first condition, six slabs were made and cured for 48 hours, recompacted and the moulds stripped. Under the second condition, six slabs were compacted and cured for 24 hours, recompacted and the moulds stripped at 48 hours. Under the last condition, 6 slabs were made and cured for 48 hours, and the moulds stripped. In addition, two extra slabs were made for materials which were cured more than 24 hours for the first condition.

Figure 3.33: Cooper compactor device used for compacting the slabs

All of the slabs were kept at room temperature and cured for 56 days.

Figure 3.34: Curing and storing the slabs for 56 days

As a non-standard bending beam test was to be used, some equipment had to be manufactured, and was used for the modified beam test as follows:

- A container was made of perspex with dimensions of 305 mm x 305 mm x 50 mm and a weight of 796.4 g, as shown in Figure 3.35.

Figure 3.35: Perspex container used in this research

- Two steel beams were used with dimensions of 50 mm x 35 mm x 300 mm. Beam 1 weighed 4193 g and beam 2 weighed 4117 g. The beams are shown in Figure 3.36.

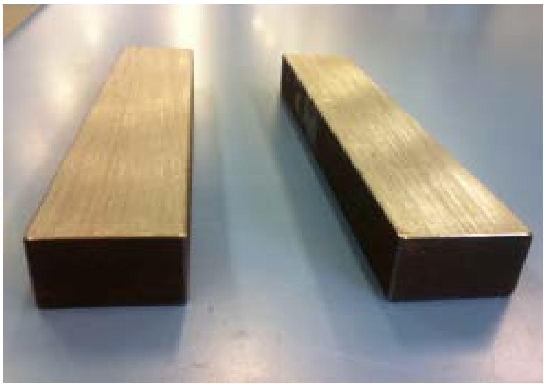


Figure 3.36: Steel beams

- Aluminium support beam and baseplate with specific dimensions as shown in Figure 3.37 were manufactured and used in this research.

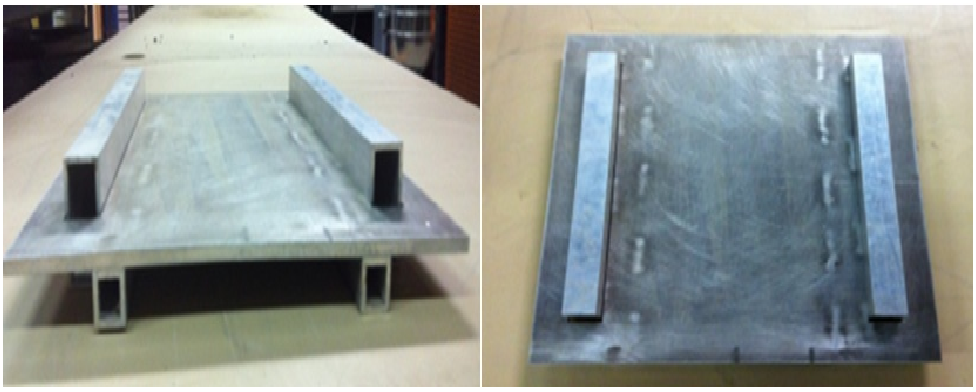


Figure 3.37: Aluminium support beam and baseplate

- Two steel bars as rollers and support beams were manufactured for use in this study, as shown in Figure 3.38.

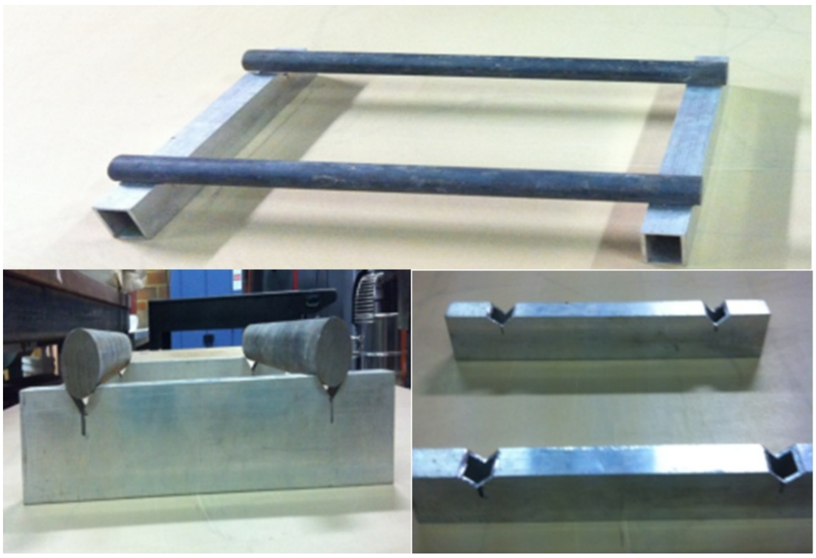


Figure 3.38

This part of the research concerns the various processes performed for the modified bending beam test. In the first step, the maximum strength of the slabs was to be determined by adding water gradually to the perspex container. However the weight was insufficient, and a steel beam was placed under the container and the container refilled. This was still insufficient to cause failure and a second steel beam added at the process repeated The total load of over 12kg was still was not sufficient to fail the sample, and it was then decided to use the UTM25 for the application of higher loads.

The samples were set up in the test rig of the UTM 25 using the same base plate as previously used for the water container method. The load was applied using a steel bar to spread the load linearly, parallel and centrally between the supports. However after the first test, it was decided that the rectangular supports under the block were responsible for frictional forces resisting bending, and the rectangular sections were replaced by steel rollers.

For all subsequent samples, the sample slabs were placed on the roller and the UTM is capable of applying maximum loading at a rate of 25 KN (equivalent to 2.5 tonnes). The load was continuously recorded and the load at break was recorded as the fracture load.

3.5.5 ***A summary of the test objectives***

A summary of the objectives and methodology of the selected tests is shown in Table 3.6.

Table 3.6: Summary of the thesis plans

Test objectives	Methodology	Section
Non-mechanical and Primary tests		
To assess a suitable gradation of recycled materials.	Particle size distribution test	3.4.1
To determine the durability and abrasion characteristics of recycled materials.	Los Angeles abrasion test	3.4.2
To assess the properties and behaviour of different recycled materials.	Liquid limit test	3.4.3
To assess the properties of different recycled materials.	Plastic limit test	3.4.4
To measure the plasticity of different recycled materials.	Plasticity index test	3.4.5
To determine the linear shrinkage of different recycled materials.	Linear shrinkage test	3.4.6
To assess the MDD and OMC of recycled materials and making cores.	Compaction test	3.4.7
Mechanical and main tests		
To determine the strength of various blends of recycled materials with RCC and the effect of rehydration to limit shrinkage.	Unconfined compressive strength test (UCS)	3.5.1, 3.5.1.1
To determine the strength, effect of rehydration and effect of microcracking on pure RCC to control and reduce cracks.	Modified bending beam test	3.5.2, 3.5.2.1

CHAPTER 4

RESULTS AND DISCUSSION

4.1 **An overview of the chapter's content**

This chapter presents and discusses the results of the experimental laboratory tests performed on recycled materials such as pure recycled crushed concrete (RCC) and various blends of RCC with crushed brick and tile and ferricrete. The analysis of the rehydration of cement in recycled crushed concrete was carried out by a series of standard and non-standard test methods.

This research was a thorough investigation aimed at providing a method for determining the effect of rehydration on recycled concrete with different blends of materials.

The experimental investigations were separated into two main parts:

- Non-mechanical characteristics
- Mechanical characteristics

Non-mechanical test results were used to determine the basic characteristics of RCC and other recycled materials. Subsequently, the more complicated reactions of RCC and different blends of materials were also investigated under various conditions.

The investigations into the properties of the recycled materials were carried out in two main steps:

- Determining the physical and mechanical properties of the recycled materials (i.e., concrete, brick and tile, ferricrete), which were analyzed separately.
- Examining the laboratory specimens after the primary tests, to determine the effect of rehydration of cement on recycled crushed concrete in road base.

The relationship between the material characterization data and experimental results was carefully studied to determine how to extend pavement life.

4.2 **Non-mechanical behaviour**

4.2.1 ***Particle size distribution***

In general, PSD is a part of the pavement material specifications and is used as an index test to ensure consistency of materials to be used for pavement construction. In addition, the limits of the grading envelope are designed to ensure maximum density is achieved. MRWA has established PSD specifications for base course, subbase course and earthwork materials. In this research, sieve analysis was used to identify the particle size distribution properties of RCC, crushed brick and tile and ferricrete.

PSD test results are presented in gradation graphs showing the percentage of soil masses passing through the sieves plotted against the sieve sizes. All PSD results are attached in Appendix B.

4.2.1.1 Particle size distribution of recycled crushed concrete

The first study on the properties and gradation of recycled materials was carried out using the particle size distribution test.

In general, particle size distribution test results are presented in gradation graphs illustrating the percentages of soil passing through the different sieve sizes.

As mentioned in the previous chapter, the particle size distribution test was performed on dry and wet RCC in order to determine a suitable gradation for use as a base course material. The first particle size distribution (PSD) test showed that the sample was not within the IPWEA/WALGA specified particle size distribution (IPWEA/WALGA, 2012), as shown in Figure 4.1.

All of the plastic bags containing RCC materials were distributed and mixed completely with a sample divider. Fines and coarse aggregates were mixed thoroughly and the PSD was again determined as being out of specification. These samples were discarded and a new sample of material obtained.

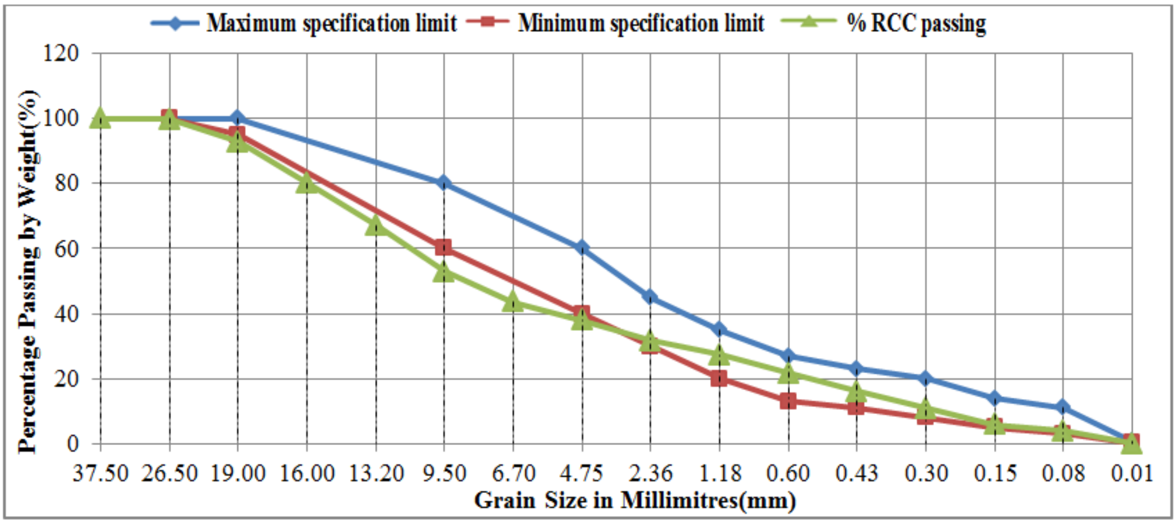


Figure 4.1: Unsuitable gradation of prior dry RCC samples

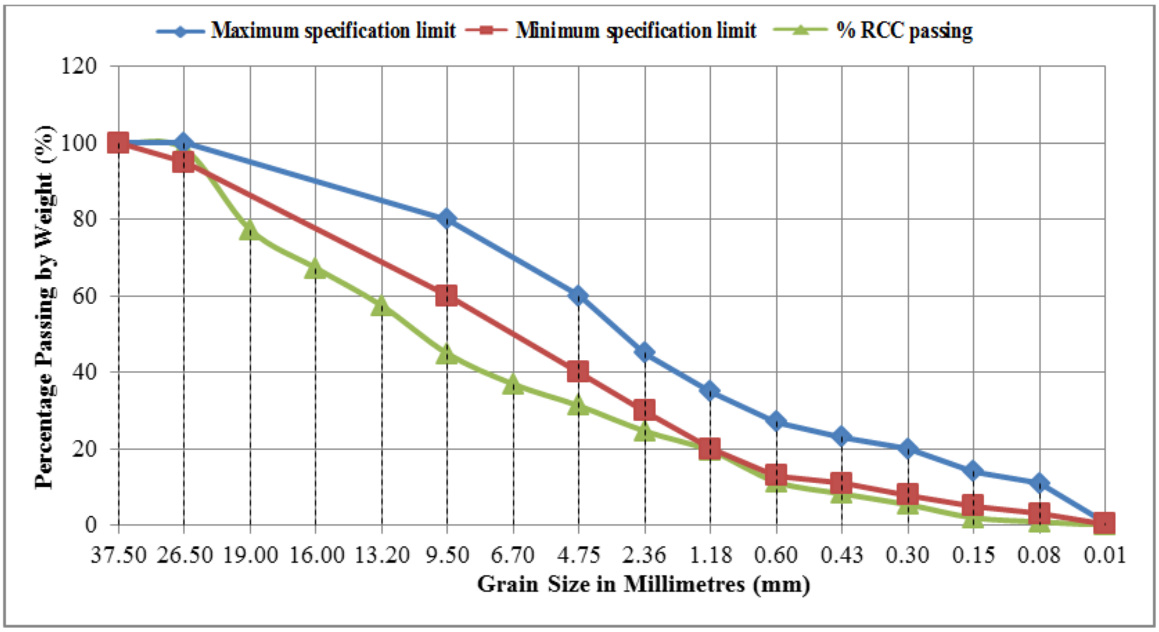


Figure 4.2: Gradation of wet RCC

Figure 4.3 illustrates the gradation of the new batch of RCC supplied as conforming to the specification for a base course material. This sieve analysis test result is represented in terms of a gradation chart which shows the relative sieve sizes and percentages of new RCC masses passing through each sieve. A further three repetitions of the PSD tests as described above, all showed very similar gradation. As discussed in Chapter Three, PSD tests were undertaken in accordance with MRWA test method WA 105.1 (MRWA, 2011c). The results are shown in Figure 4.3 and show that the new sample of RCC complied with the specification for a base course material.

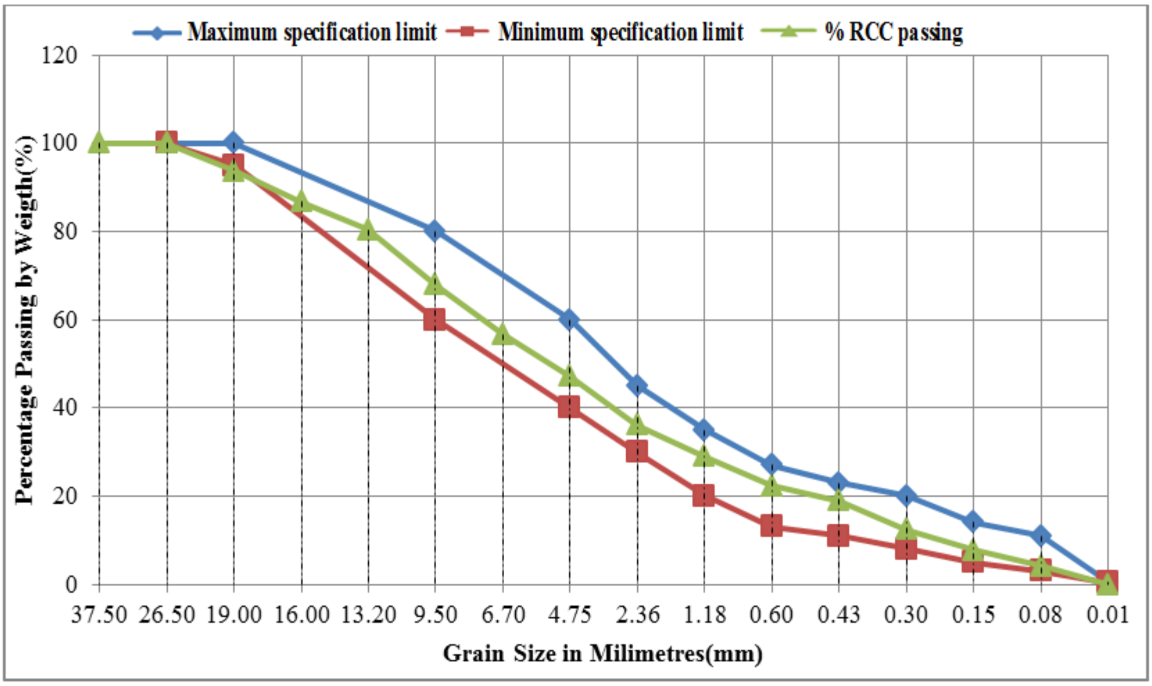


Figure 4.3: Gradation of new RCC

4.2.1.2 Particle size distribution of crushed brick and tile

The PSD for crushed brick and tile was also determined as shown in Figure 4.4. This indicates that the gradation of crushed brick used in this study also conforms to the Specification.

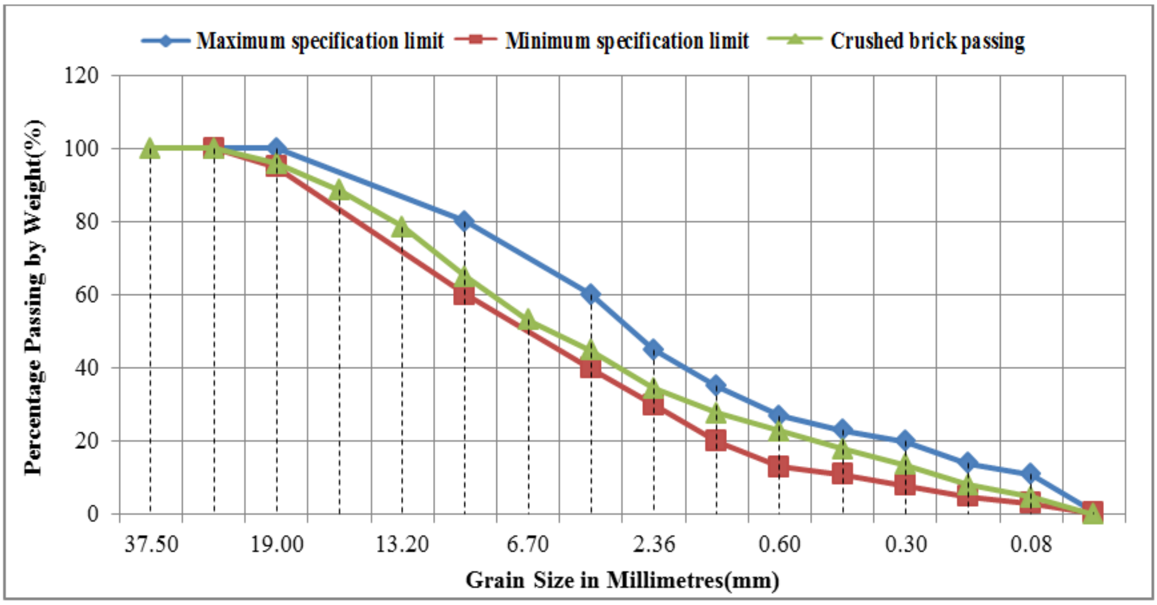


Figure 4.4: Gradation of pure crushed brick and tile

The research required preparation of blends of RCC and brick and tile with brick and tile making up 10%, 20%, 30% and 50% by weight of the RCC and brick and tile blends. After the proportions were blended, the PSD of each blend was determined as shown in Figures 4.5, 4.6, 4.7 and 4.8

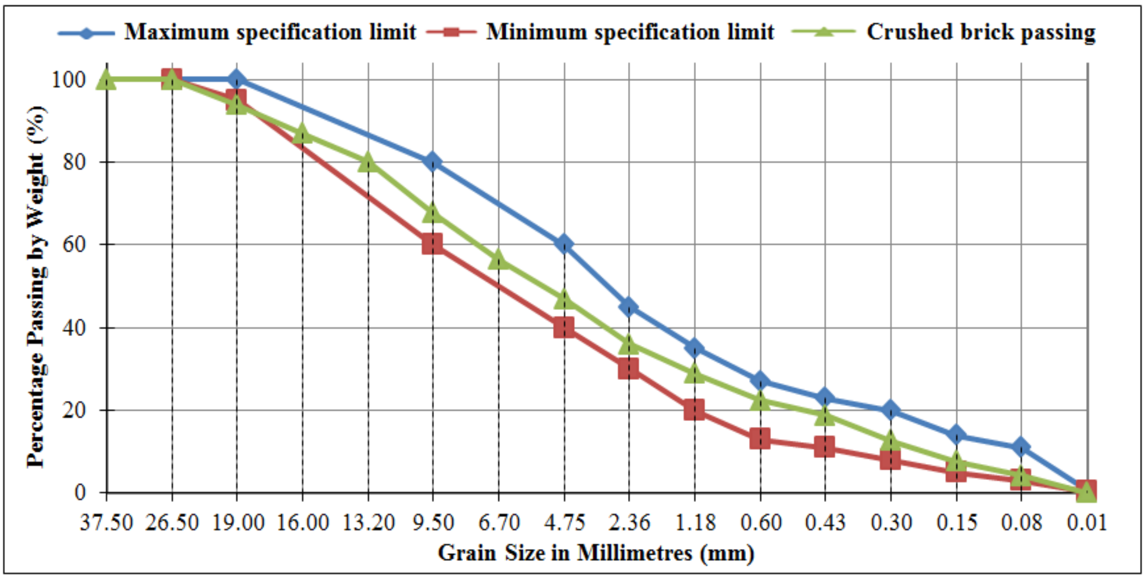


Figure 4.5 : Gradation of 10% crushed brick and tile mixed with RCC

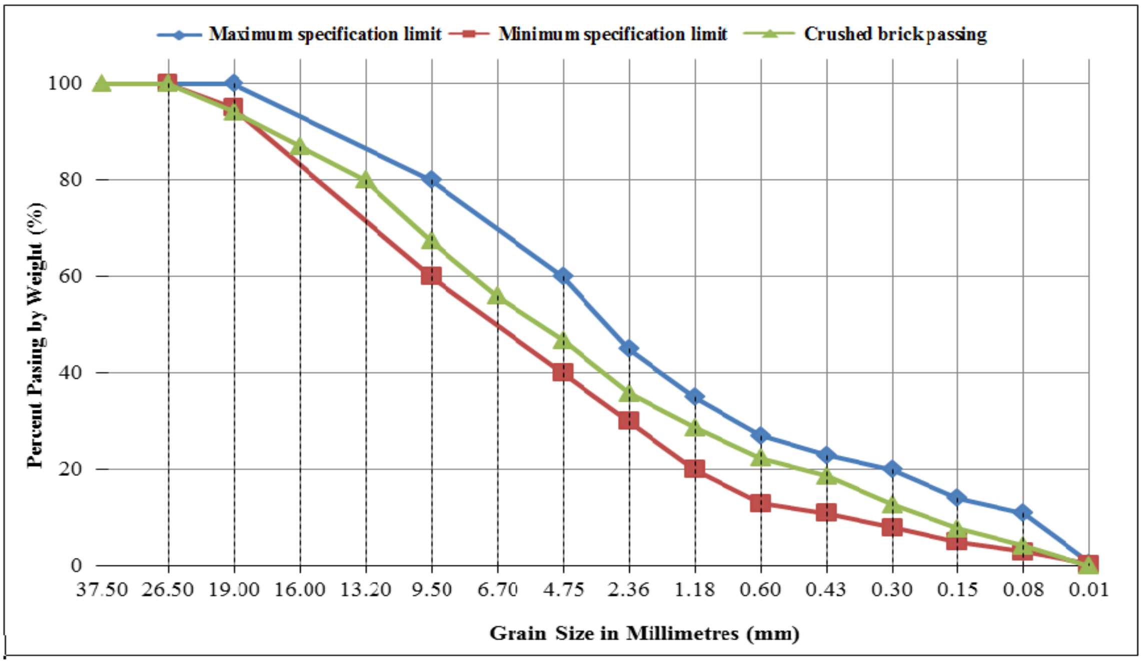


Figure 4.6: Gradation of 20% crushed brick and tile mixed with RCC

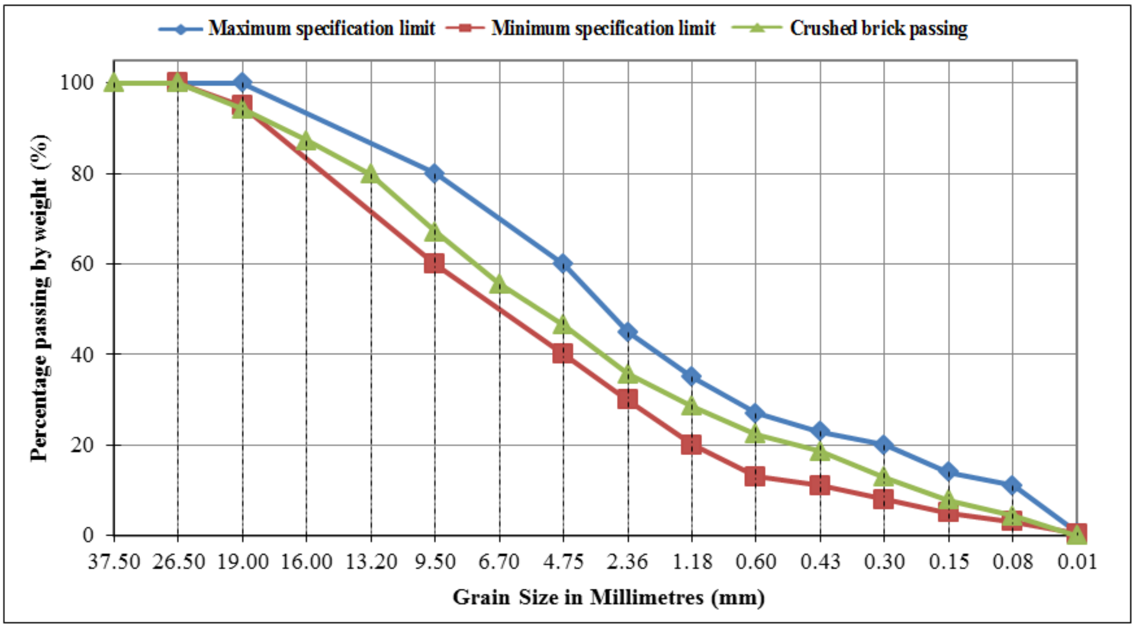


Figure 4.7: Gradation of 30% crushed brick and tile mixed with RCC

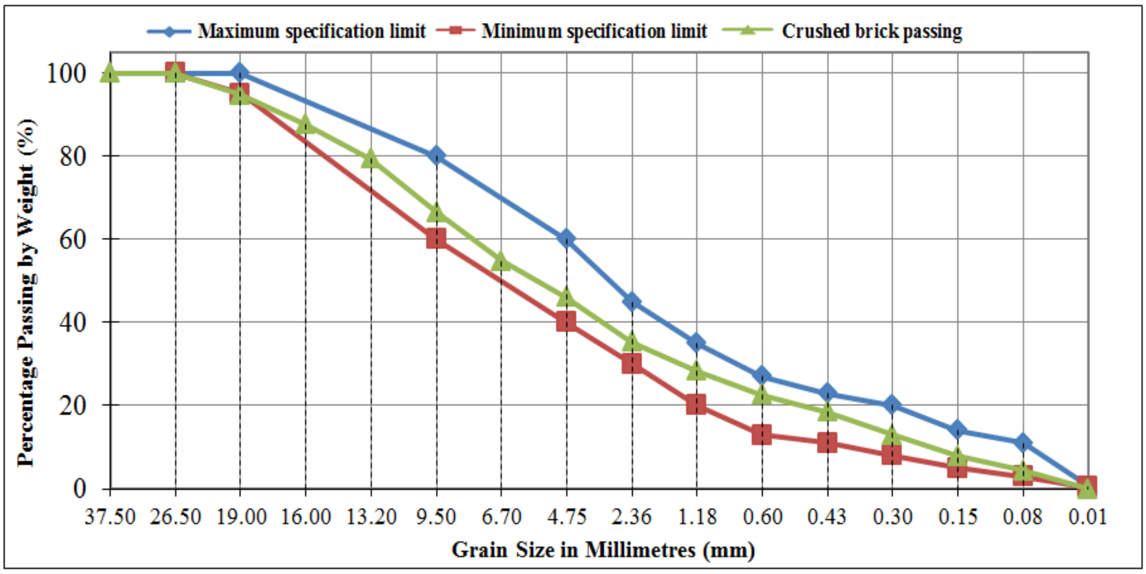


Figure 4.8: Gradation of 50% crushed brick and tile mixed with RCC

4.2.1.3 Particle size distribution of ferricrete

The PSD ferricrete and blends of 10%, 30% and 50% of ferricrete with RCC was also determined. Figure 4.9 shows the PSD of ferricrete, and Figure 4.10, Figure 4.11 and Figure 4.12 show the PSD of ferricrete/RCC blends. All of the prepared mixes fell withing the limits of the specification for a base material.

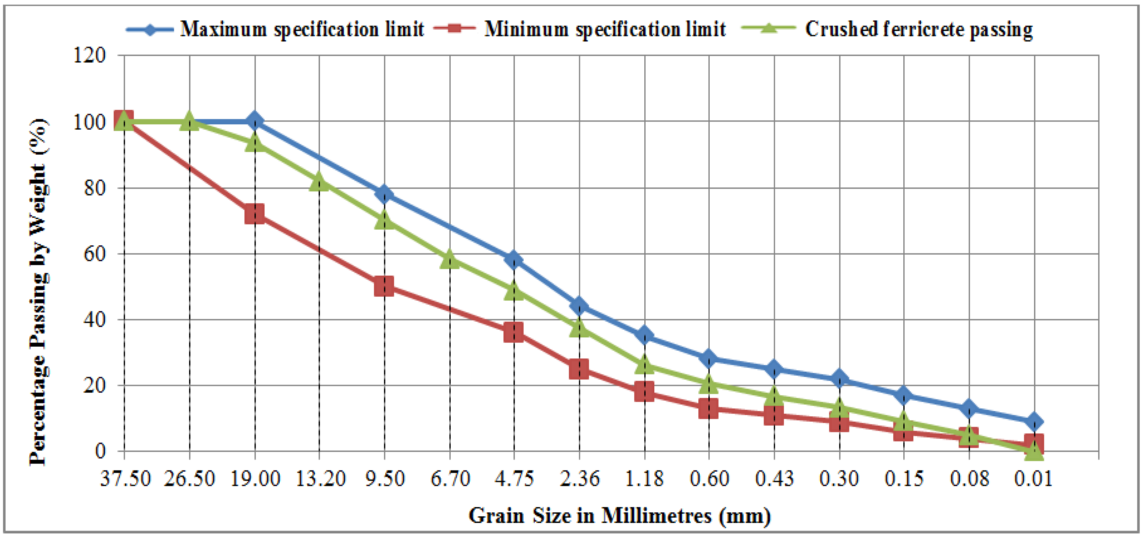


Figure 4.9: Gradation of pure ferricrete

As all of the component materials met the requirements of the specification, blends of the materials

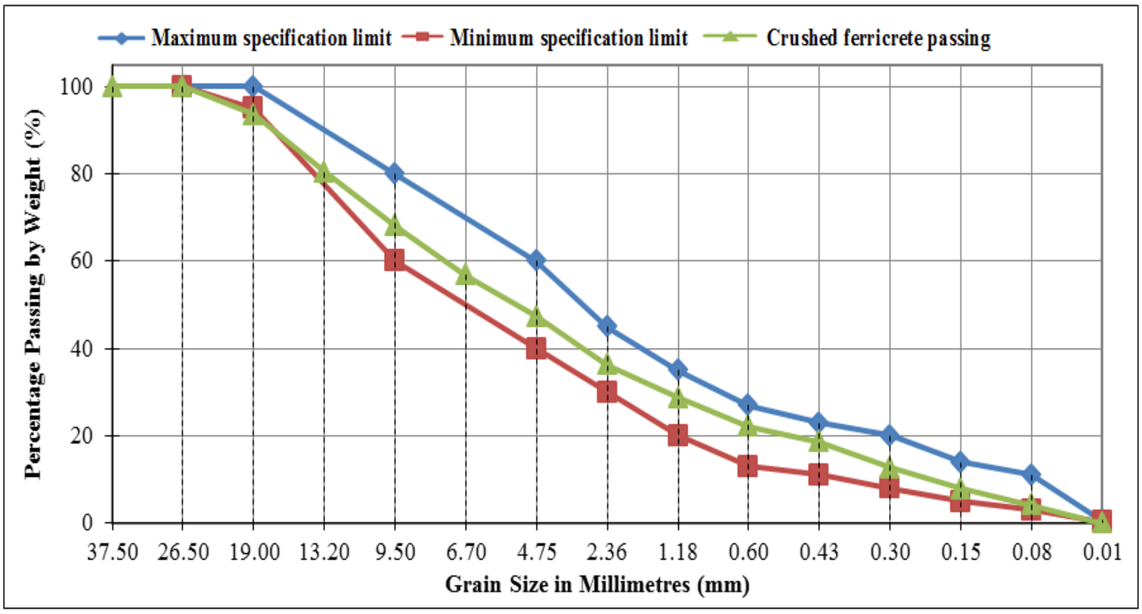


Figure 4.10: Gradation of 10% ferricrete mixed with RCC

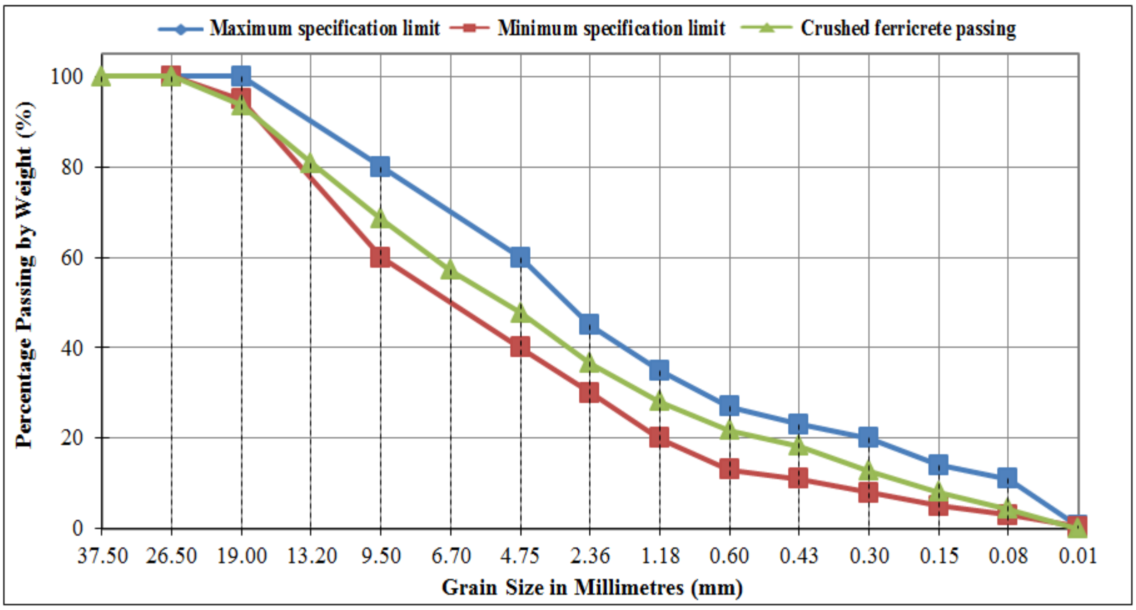


Figure 4.11: Gradation of 30% ferricrete mixed with RCC

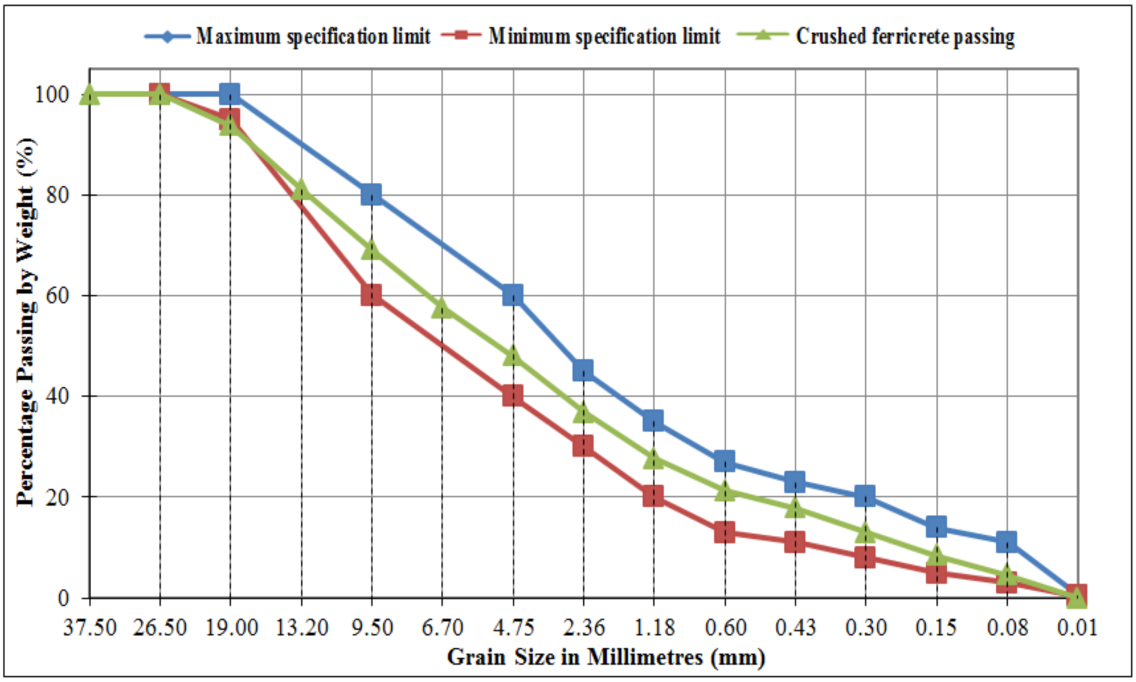


Figure 4.12: Gradation of 50% ferricrete mixed with RCC

4.2.1.4 Hygroscopic moisture content

After the particle size distribution test was carried out, the hygroscopic moisture content (w %) of the materials was determined in accordance with test method WA 110.1 (MRWA, 2011L). Table 4.1 presents the hygroscopic moisture content of the materials, and shows that the hygroscopic moisture content of RCC, crushed brick and ferricrete were not significant after drying in the oven.

Table 4.1: Hygroscopic moisture content (%)

Type of material	Hygroscopic moisture content (%)
Dry RCC	0.34
New dry RCC	0.67
Wet RCC	9.65
Crushed brick and tile	0.33
Ferricrete	0.33

4.2.2 *Los Angeles abrasion test*

The Los Angeles abrasion value of recycled crushed concrete was determined in accordance with Test method WA 220.1 (MRWA, 2012k).

The LA test indicates the resistance of the coarse aggregate to mechanical degradation. The result of the LA tests gave an average value of 33.6% which falls within the specification limits. Table 4.2 shows the Los Angeles abrasion value of three repeated tests on the RCC samples.

Table 4.2: Los Angeles abrasion value of RCC

Los Angeles abrasion value (%) Test 1	33.16
Los Angeles abrasion value (%) Test 2	34.10
Average Los Angeles abrasion value (%)	33.63

The IPWEA/WALGA (2012), specification for recycled materials to be used in road pavements requires a Los Angeles abrasion value of less than 40%. The result of the Los Angeles abrasion test for RCC in this research was less than 40%, and the material complies with the specification.

4.2.3 *Liquid limit test*

The liquid limit value of materials in this research was determined using the Casagrande WA 120.1(MRWA, 2012e) and cone penetrometer method WA 120.2 (MRWA, 2012f).

4.2.3.1 Casagrande method

Table 4.3 lists the liquid limit test results for pure RCC and different blends of crushed brick and tile with RCC using the Casagrande test method.

Table 4.3: Liquid limit values (Casagrande method)

Type of material	Liquid limit (%)
100% recycled crushed concrete (RCC)	30.3
10% crushed brick and tile with RCC	30.3
20% crushed brick and tile with RCC	30.3
30% crushed brick and tile with RCC	30.4

The MRWA Specification 501 prior to review in December 2011 required that the liquid limit value of recycled concrete should be less than 35%. The liquid limit value of all samples in this research fell within the specification limit, being less than 35%. The liquid limit value of pure RCC, determined by the Casagrande test method, was 30.3%. Increasing the crushed brick and tile content of the RCC from 10% to 20% and 30% yielded no significant difference in liquid limit value as shown in Figure 4.13.

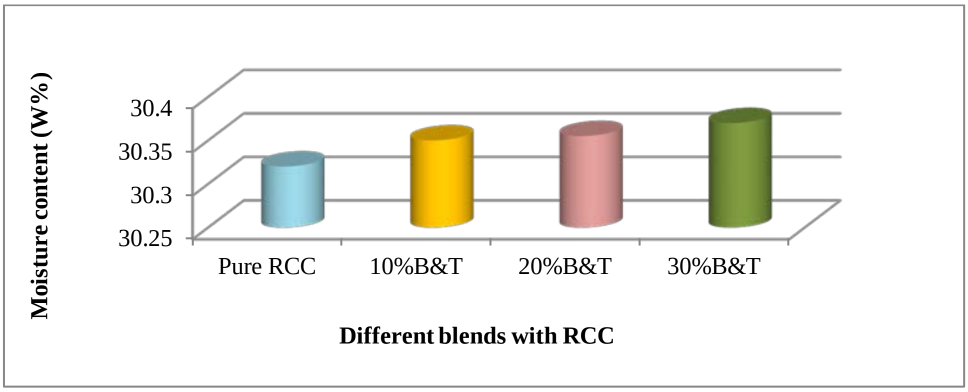


Figure 4.13: Liquid limit value of RCC blends with Cassagrande test method

4.2.3.2 Cone penetrometer method

The liquid limit value of the materials used in this research was also determined using the cone penetrometer test method. The liquid limit value of brick and tile mixtures with RCC obtained by the cone penetrometer test method was similar to the liquid limit value of samples obtained by the Casagrande test method. The scatter of data makes it difficult to ascertain a specific LL by this method, but all values for LL fall within the specification limits that existed prior to the IPWEA/WALGA specification.

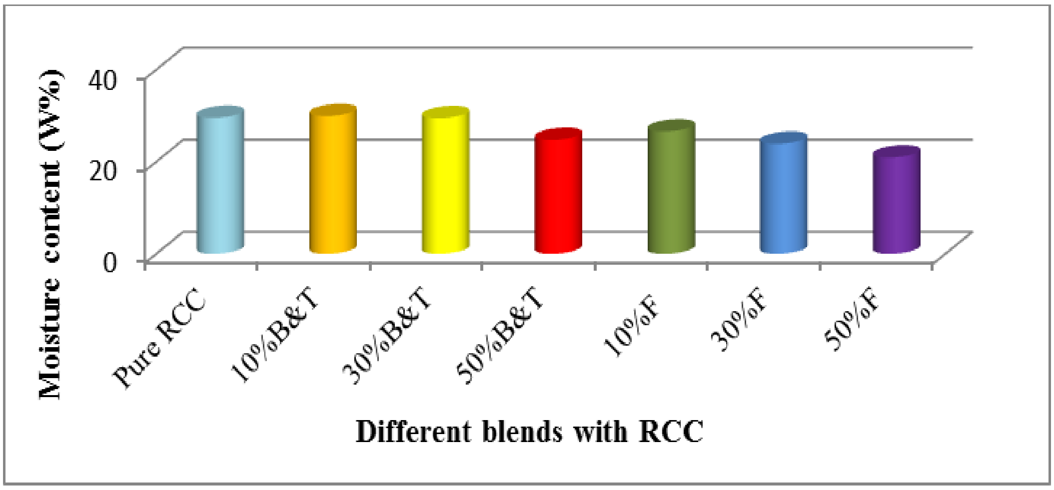


Figure 4.14: Liquid limit value of RCC blends with Penetrometer test method

However IPWEA/WALGA argued that the liquid limit was not reliable assessment criteria for RCC, as the liquid limit was affected by the cement content and at times in past testing had shown a value higher than the specified limit, but this did not affect performance. Liquid limit is not now a requirement of the specification. In general, it can be seen that overall, all liquid limit values fell within the original MRWA specification limits. All liquid limit results are attached in Appendix C.

Tables 4.4 and 4.5 present the liquid limit values for pure RCC and the different mixtures of crushed brick & tile and ferricrete with RCC determined by the cone penetrometers method. Increasing the ferricrete content had a possible effect on decreasing the liquid limit as indicated in Table 4.5.

Table 4.4: Liquid limit value (cone penetrometer method)

Type of material	Liquid limit (%)
100% recycled crushed concrete (RCC)	29.5
10% crushed brick and tile with RCC	29.9
30% crushed brick and tile with RCC	29.4
50% crushed brick and tile with RCC	24.7

Table 4.5: Liquid limit value (cone penetrometer method)

Type of material	Liquid limit (%)
100% recycled crushed concrete (RCC)	29.5
10% crushed ferricrete with RCC	26.5
30% crushed ferricrete with RCC	23.9
50% crushed ferricrete with RCC	20.9

4.2.4 *Plastic limit test*

As mentioned in Chapter Three, the plastic limit was also determined for different blends of RCC with ferricrete and crushed brick and tile. The results presented in Table 4.6 show that all of the recycled materials used in this study are non-plastic. Laboratory plastic limit tests demonstrated that these materials, could not be rolled into a thread of approximately 3 mm in diameter.

Table 4.6: Plastic limit test results for recycled materials used in this study

Type of material	Degree of plasticity	Plasticity index
Pure recycled crushed concrete	Non plastic	NP
RCC+10% brick&tile	Non plastic	NP
RCC+30% brick and tile	Non plastic	NP
RCC+50% brick and tile	Non plastic	NP
RCC+10% ferricrete	Non plastic	NP
RCC+30% ferricrete	Non plastic	NP
RCC+50% ferricrete	Non plastic	NP

4.2.5 ***Plasticity index***

Plasticity index (PI) is the numerical difference between the liquid and plastic limit, and indicates the range of water content within which the soil remains plastic. The equation is: PI = LL – PL. However where the PL is determined as not plastic, the PI is also defined as non plastic. WA 122.1 (MRWA, 2012m).

4.2.6 ***Linear shrinkage***

The specification for recycled materials in road pavements requires a linear shrinkage value for base course materials to be in the range of 0.2–1.5%. Figure 4.15 shows the linear shrinkage values of RCC with different percentages of crushed brick and tile and ferricrete that were determined using test method 123.1 WA (MRWA, 2012h). The actual values are shown in Table 4.7 and Table 4.8.

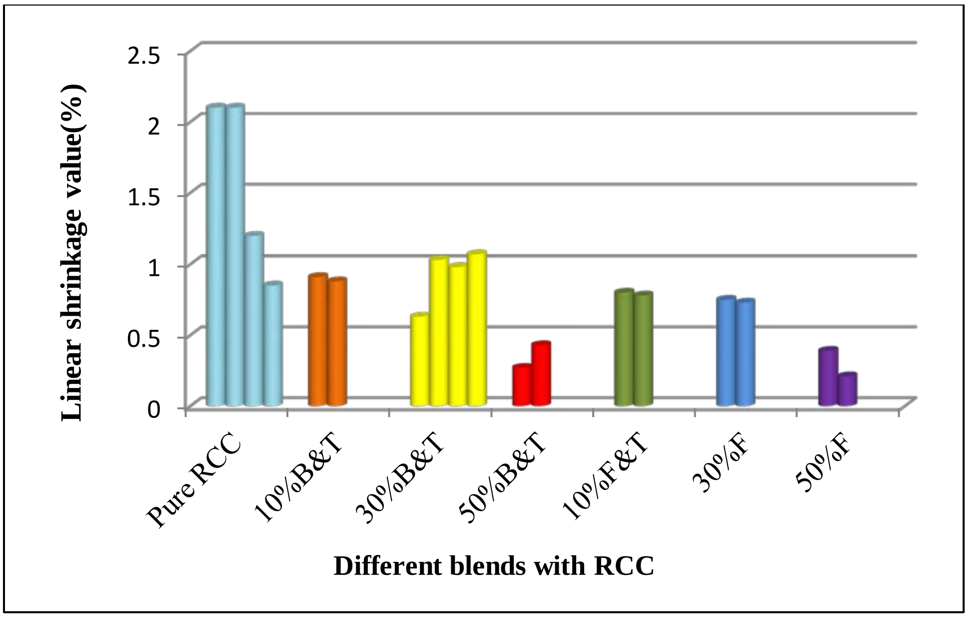


Figure 4.15: Linear shrinkage value of different RCC blends

Two RCC samples fell slightly above the specification limit of 2% (MRWA) and 1.5% (IPWEA/WALGA) being 2.1% each. The other two test results fell within the specification limits, as did all of the blends with both brick and tile and ferricrete.

Table 4.7: Linear shrinkage value for RCC

Type of material	Specification	Test results					
		15/11/2012		27/11/2012		27/12/2012	
Recycled crushed concrete (RCC)	0.2%–1.5%	M1	M2	M3	M4	M3	M4
		2.1	2.1	1.2	0.85	1.92	1.38

Table 4.8: Linear shrinkage values for different blends of crushed brick and tile with RCC

Test results								
Type of material	**RCC+10% crushed brick and tile**		**RCC+30% crushed brick and tile**				**RCC+50% crushed brick and tile**	
Samples	M5	M6	M7	M8	M9	M10	M11	M12
Linear shrinkage value	0.91	0.88	0.63	1.03	0.98	1.07	0.27	0.43

The linear shrinkage results for RCC with ferricrete illustrate that increasing the ferricrete led to a decrease in the linear shrinkage values (Table 4.9). The average linear shrinkage value for 10% ferricrete with RCC was lower than 0.8%, compared 0.91% for 10% crushed brick and tile. It is clear that the linear shrinkage values of different blends of ferricrete are lower than those of the mixtures of brick and tile with RCC. Table 4.9 shows that the linear shrinkage value for 50% ferricrete with RCC is around 0.3%, while the value for 30% ferricrete with RCC is 0.74%.

Table 4.9: Linear shrinkage values for different blends of ferricrete with RCC

Test results						
Type of material	RCC+10% ferricrete		RCC+30% ferricrete		RCC+50% ferricrete	
Samples	M13	M14	M15	M16	M17	M18
Linear shrinkage value	0.8	0.78	0.75	0.73	0.39	0.21

The linear shrinkage tests would indicate that the addition of either ferricrete or brick and tile does result in a decrease in the linear shrinkage. Linear shrinkage is generally indicative of a material with a degree of plasticity, but in this case the RCC is non plastic, and the shrinkage is more likely associated with the cohesive forces resulting from hydration of cement. Thus it is a possible indication that the combining of brick & tile or ferricrete with RCC is having an effect limiting the effects of rehydration.

4.2.7 *Modified compaction test*

All materials consisting of pure RCC and different blends of ferricrete, brick and tile with RCC were analysed for maximum dry density (MDD) and optimum moisture content (OMC) by test method 133.1 WA (MRWA, 2012i).

*4.2.7.1 **Modified compaction test of RCC***

The results of the determination of the Maximum Dry density (MDD) and Optimum Moisture Content (OMC) are shown in Table 4.10 and Table 4.11.

Table 4.10: Laboratory compaction results for RCC with crushed brick and tile

Type of material	Optimum moisture content (%)	Maximum dry density (t/m³)
First pure RCC	11.70	2.01
New pure RCC	12.16	1.89
RCC with 10% crushed brick and tile	12.27	1.93
RCC with 20% crushed brick and tile	12.34	1.92
RCC with 30% crushed brick and tile	12.38	1.91
RCC with 50% crushed brick and tile	12.89	1.85

Table 4.11: MDD and OMC test results

Blend of materials			MDD (t/m³)	OMC (%)
RCC (%)	Brick and Tile (%)	Ferricrete (%)		
100			1.89	12.16
90	10		1.93	12.3
80	20		1.92	12.3
70	30		1.91	12.4
50	50		1.85	12.8
90		10	2.00	10.8
70		30	2.10	10.6
50		50	2.16	10.2

Analysing the results shown in Table 4.11 shows that the addition of brick and tile has the effect of a small increase in the OMC of the RCC, but that the addition of ferricrete reduces the OMC. Being a denser material, the ferricrete also has the effect of a slight increase in density as the percentage increases, but the effect on density of the brick and tile is negligible. The MDD and OMC of all blends are shown in Appendix D of this research.

4.3 **Mechanical behaviour**

This section explains the mechanical behaviour of pure RCC, as well the different blends of recycled materials such as crushed brick and tile and ferricrete with RCC, in order to evaluate and define the materials' responses for use as road base material in Western

Australia. Initially, the modified bending beam test was carried out as a non-standard test on pure RCC. This experimental and non-standard test was performed by using a Cooper slab compactor to manufacture slabs of recycled pure crushed concrete and then re-compacting to induce microcracking. The slabs were then subjected to an ultimate bending strength test as detailed in Section 4.3.1. The results were used to determine the strength of pure RCC before and after recompaction. The effects of rehydration and microcracking on material strength also were determined.

Investigations were carried out into the mechanical characterization of different blends of recycled materials such as crushed brick and tile and ferricrete with RCC. The strength and stress-strain relationships of cylindrical specimens of these materials were investigated using the UCS test on the various blends of materials at a range of curing periods.

4.3.1 ***Modified bending beam test***

4.3.1.1 ***Determination of maximum strength of slabs with distribution load of container***

As described in Chapter Three, different non-standard tests were performed on slabs of pure RCC which were manufactured using a Cooper slab compactor device. The slabs were compacted under three different sets of conditions and then were cured for 56 days. Table 4.12 presents the three different conditions under which the slabs were manufactured.

Table 4.12: Different compaction conditions for making slabs in this study

Compaction conditions	Method of manufacturing
0	Slabs made, cured 48 hours, the moulds stripped
24	Slabs made, cured 24 hours, re-compacted, the moulds stripped at 48 hours
48	Slabs made, cured 48 hours, re-compacted, the moulds stripped

As outlined in Chapter 3, the testing on the slabs involved some continual trials until sufficient force could be applied to cause failure. By utilising the UTM25 device, the breaking stress for each of the slabs was determined.

The UTM25 can apply a load of 25kN (equivalent to 2.5 tonnes). The strength of compacted slabs was determined and analysed under different compaction conditions after 56 curing days with the UTM25 device. In the first stage, the effects of support beams on the slabs were analysed. A 50mm wide steel beam was used for applying the actuator loads to the slab, and distributed the load uniformly across the slab parallel to the support beams.

4.3.1.2 ***Modified bending beam with support beam***

In the test, rectangular support beams were used to support the slab. Figure 4.16 shows the test process and crack patterns under the applied load.

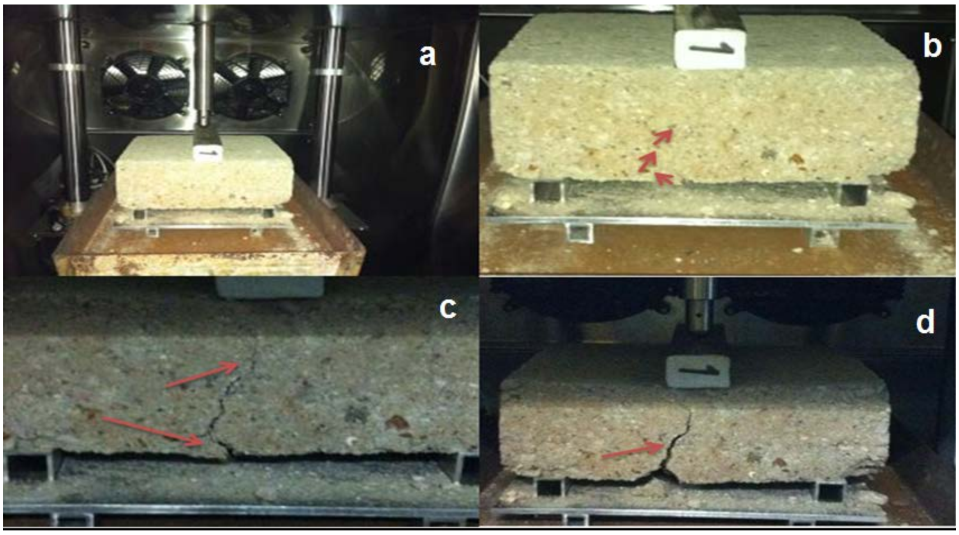


Figure 4.16: Modified bending beam test of beam 2 with support beam. (a) The first step of applying loads (b) Increase of microcracks with rise of applying loads (c) Increased cracks in the middle of beam (d) Failure of RCC beam

Figure 4.16a shows the minor cracks which occurred in the slabs after the loads were increased. It is understood that concrete is strong in compression, but weak in tension, and as expected cracks developed from the tension face of the slab. However it was noted that in the first trial, where fixed rectangular supports were used, the condition resulted in an indeterminate condition, as the frictional forces generated by the fixed supports could not be resolved. The Figure 4.16d shows the failure mode of the slab number 2 which had been one of those recompacted at 24 hours. Cracks occurred in a direction parallel to the applied load, in the areas where the pure RCC slab was weak in tension as shown in Figure 4.16b. In this case, the flexural, compression and tension failure can be seen. In Figure 4.16c, the red arrows present the initiation of microcracks while applying the load. However bearing failure occurred around the supports due to the support beam having sharp edges as shown in Figure 4.16b and Figure 4.16d. Figure 4.17 shows the strain generated in by the applied vertical load on the RCC slab at yield point.

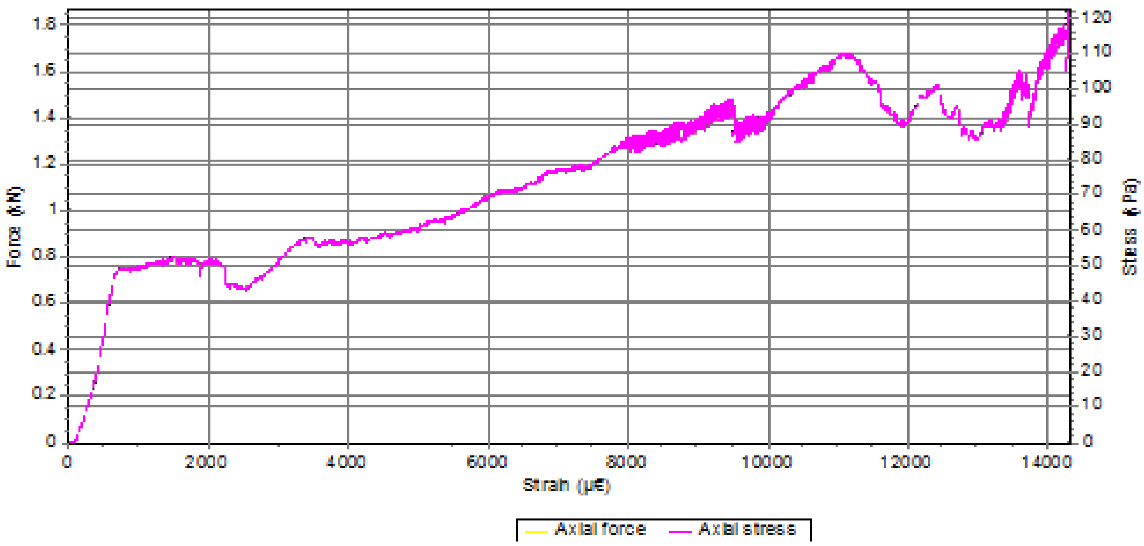


Figure 4.17: The stress-strain curve for beam 2 with support beam

The brittle nature of a non reinforced concrete slab should show immediate drop in load after brittle failure. However the confining forces generated by friction between the slab and support beams tended to restrain the movement of the slab, resulting in a resulting increase in load as the slab deformed. Failure is taken to be at approximately 0.75kN.

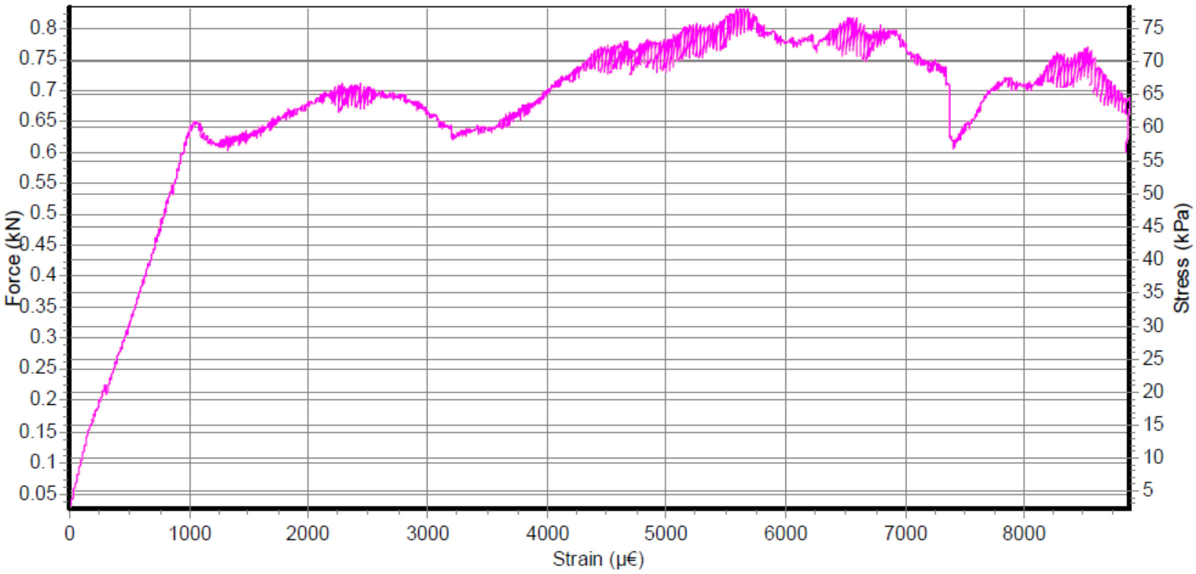


Figure 4.18: Stress-strain curve for beam 4 with support beam

RCC beam 4 was tested using the same setup, with similar results as shown in Figure 4.18 from the results, it can be observed that initial failure occurred at 0.64 KN. The failure of slab 4 is shown in Figure 4.19.

Figure 4.19: Modified bending beam test for beam 4 with support beam

Table 4.13 presented the bending beam results of these two RCC beams which were subjected to recompaction after 24 hours.

Table 4.13: Bending beam results of RCC 24hr recompaction with square supports beams

Sample (condition 24)	Stress (Kpa)	Maximum deflection (mm)	Moments (Nmm)	Maximum axial force (KN)	Modulus of elasticity (Ev) (MPa)
Beam 2	53.41	0.077	35904	0.74	72.45
Beam 4	46.86	0.081	31504	0.64	54.40

It was then decided for subsequent tests to use roller supports such that friction forces would be minimised. Figure 4.20 presents the load-deflection behaviour of RCC beam16 (recompaction at 24hrs) with roller supports. Figure 4.21 shows the actual failure of the slab. This shows that the effects of the confining forces from support friction were significant. All subsequent beams were tested in a similar manner and the results are shown in Table 4.14 (Slabs with no recompaction) Table 4.15 and Table 4.16 (slabs with recompaction at 24 hours) and Table 4.17 (slabs recompacted after 48 hours.

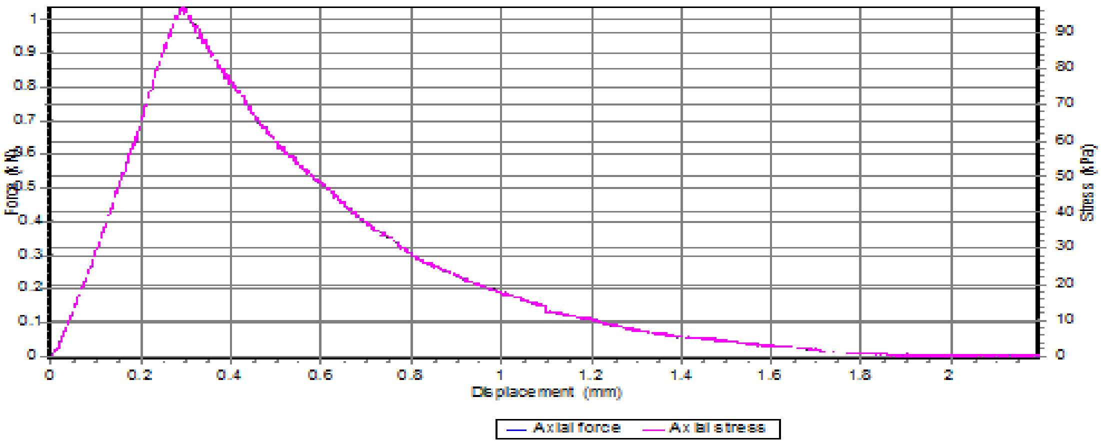


Figure 4.20: The load-deflection curve of RCC beam 16 (condition 24) with roller supports

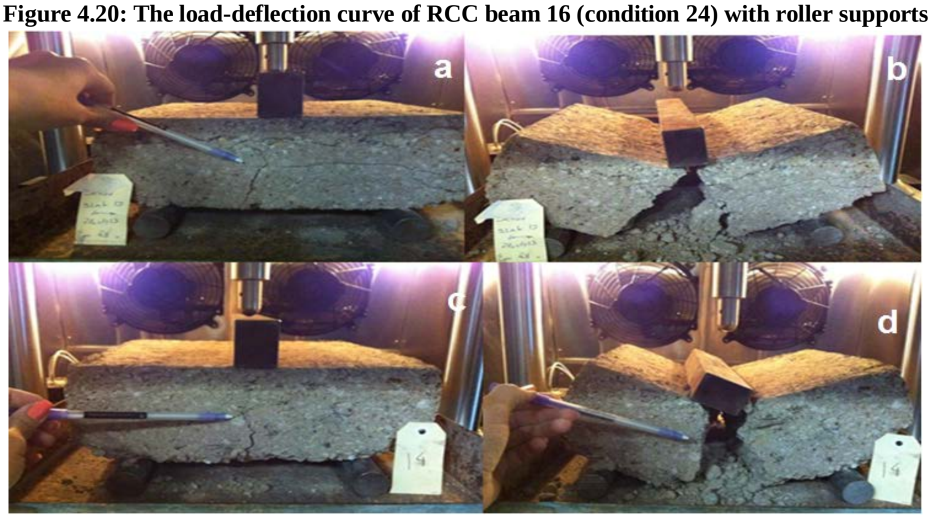


Figure 4.21: : Modified bending beam test with roller .(a) Increase of cracks with rise of applying loads.(b) Failure of RCC beam (c) Increase of cracks with rise of applying loads (d)Failure of RCC beam

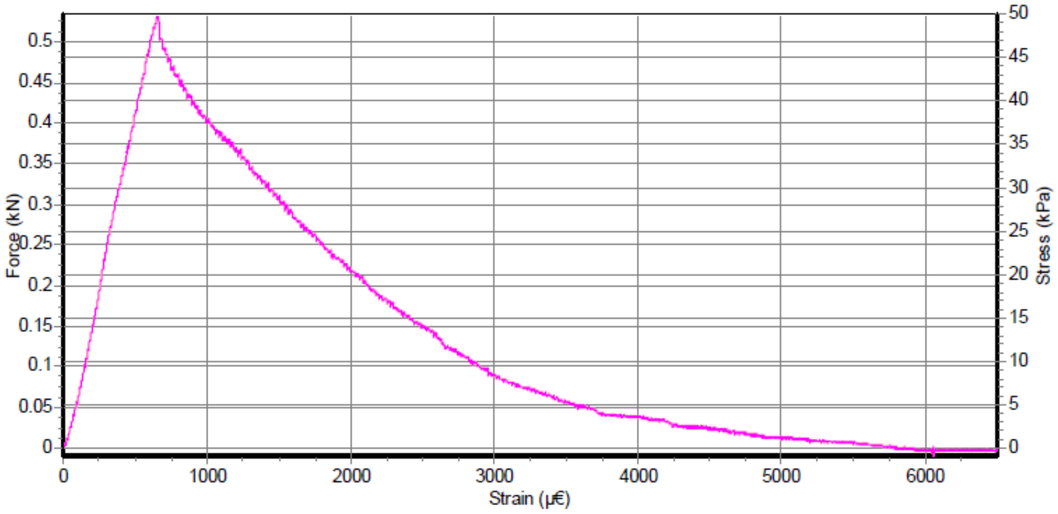


Figure 4.22: Stress-Strain curve of RCC beam 6

Table 4.14: Bending beam test results of RCC no recompaction with roller supports

Sample (condition 0)	Stress (KPa)	Maximum deflection (mm)	Moments (Nmm)	Maximum axial force (KN)	Compressive modulus EV (MPa)
Beam 5	40.19	0.328	27016	0.538	19.52
Beam 6	39.92	0.102	26840	0.534	80.28
Beam 9	27.36	0.143	18392	0.342	40.20
Beam 10	34.10	0.094	22924	0.445	81.70
Beam 17	43.52	0.121	29260	0.589	63.70
Beam 18	52.75	0.175	35464	0.730	60.65
Beam A	33.97	0.126	22836	0.443	62.80
Beam B	31.35	0.100	21076	0.403	74.28

Table 4. 15: Bending beam results of RCC 24hr recompaction with roller supports

Sample (condition 24)	Stress (Kpa)	Maximum deflection (mm)	Moments (Nmm)	Maximum axial force (KN)	Compressive modulus EV (MPa)
Beam 8	46.67	0.116	31372	0.637	75.66
Beam 15	53.80	0.120	36168	0.746	104.23
Beam 16	72.85	0.152	48972	1.037	107.08

Table 4.16: Bending beam results of RCC 24hr recompaction with square supports beams

Sample (condition 24)	Stress (Kpa)	Maximum deflection (mm)	Moments (Nmm)	Maximum axial force (KN)	Compressive modulus EV (MPa)
Beam 2	53.41	0.077	35904	0.74	72.45
Beam 4	46.86	0.081	31504	0.64	54.40

Table 4.17: Bending beam results of RCC 48hr recompaction with roller supports

Sample (condition 48)	Stress (KPa)	Maximum deflection (mm)	Moments (Nmm)	Maximum axial force (KN)	Compressive modulus EV (MPa)
Beam 7	46.21	0.102	31064	0.63	89.50
Beam 11	36.13	0.083	24288	0.476	57.88
Beam 12	32.33	0.040	21736	0.418	48.41
Beam 13	41.63	0.112	27984	0.560	71.83
Beam 14	39.86	0.085	26796	0.533	49.20

Recompaction of RCC beams after 24 hours may have made the beam stronger by further densification, and allowed the rehydration process to continue, particularly as the dryback period was insufficient. The compaction process reduces the pore space between aggregates, which means that recompaction or heavy compaction with higher moisture content can reduce the void ratio. In fact, recompaction increased the shear strength of the RCC beams. Thus the increased strength may result from increased density.

The maximum axial force of RCC beams under different conditions shows that the RCC beams under the condition of no recompaction had the lowest breaking load compared with the other RCC beams. The following Table 4.14, Table 4.15, Table 4.16 and Table 4.17 present the bending test results consist of maximum axial force, modulus elasticity and stress-strain behaviour of RCC slabs under different conditions.

The modulus of elasticity of RCC beams was determined graphically as a slope of the linear portion of the stress-strain as shown in Figure 4.23. The modulus of elasticity of RCC beam 10 is approximately 81.70 MPa which shows highest strength of RCC beam comparing the other RCC beams in condition 0.

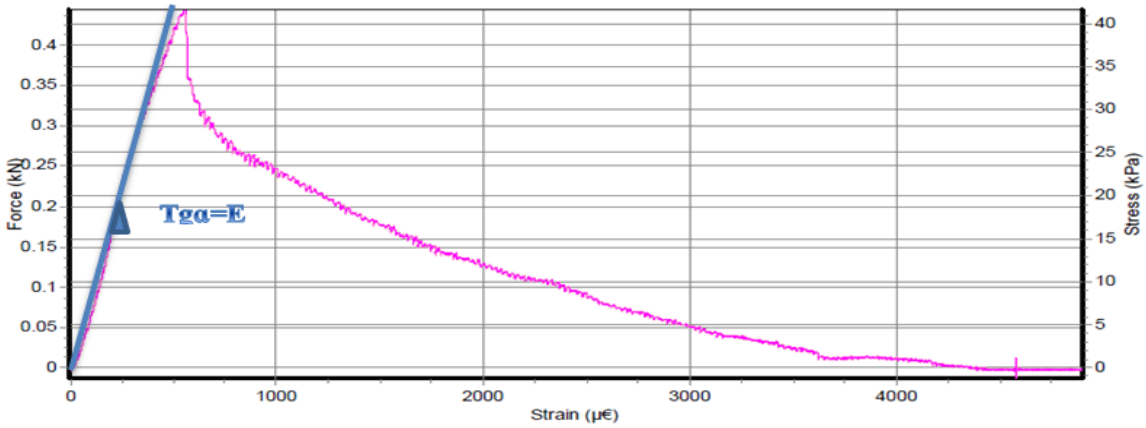


Figure 4.23: Stress-Strain curve of RCC beam 10

In the stress-strain curve, the stresses were plotted on the y axis and the strains, on the x axis. The OMC, aggregate properties, compaction process had significant effect on modulus of elasticity of recycled concrete in this research. The modulus of elasticity of RCC beams was determined and compared in different compaction conditions as shown in Table 4.14, Table 4.15, Table 4.16 and Table 4.17.

In this research there are three groups of recompaction conditions (0 hour, 24 hours and 48 hours). To make sure that the results are comparable there is a need to prove that the results are following the normal distribution. Normally this examination should be done by Analysis of Variance (ANOVA) at the first time for all the groups which is called F-Test method. If the F-Test indicated that the results are not following the normal distribution the T-Test should be considered for comparing the groups two by two.

Table 4.14, Table 4.15, Table 4.16 and Table 4.17 presents the maximum axial force of all RCC beams in three different recompaction conditions. Based on the ANOVA results for mean maximum axial force values, the calculated F- value was more than Fcrit and P-value was less than ($p=0.05$); indicating that mean maximum axial force values are not following normal distribution so there is a need to run the T-Test as mentioned before. All mean maximum axial force values for RCC beams were analysed and compared two by two by T-Test in different recompaction conditions. The T_{sta} of all RCC beams were less than T_{cri} and the P values were more than $p=0.05$.Therefore, T-Test analysis shows the normal distribution for all RCC beams which means increasing the recompaction time from 0hr to 24hr can increase the maximum axail force and strength of RCC beams. It can be seen that by extending the recompaction time from 0hr to 48hr the strength of RCC beams will increase, however this change is not significant. This statistical analysis presents that increasing the time from 24hr to 48hr cannot cause a significant increase in the maximum axial force as shown in Figure 4.24 and Figure 4.25.

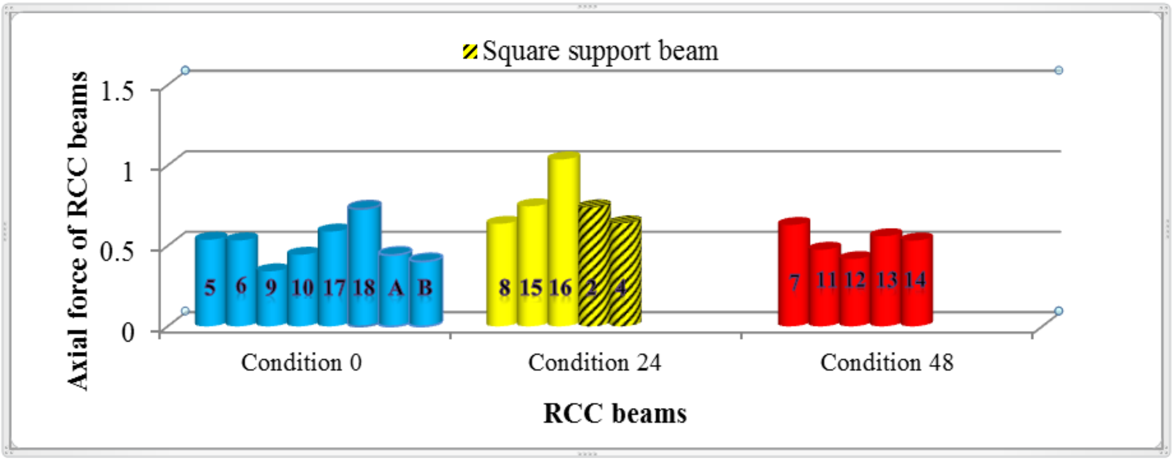


Figure 4.24: Comparison of axial force of RCC beams under three different conditions

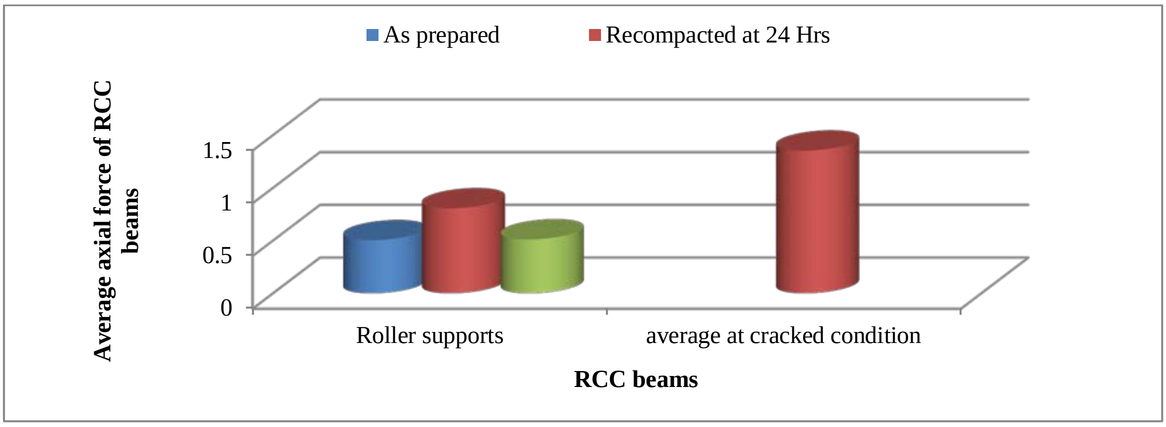


Figure 4.25: Comparison of average axial force of RCC beams under three different conditions

4.3.2 *Unconfined compressive strength test (UCS)*

The unconfined compressive strength test (UCS) was performed to determine the unconfined compressive strength and shear strength of pure crushed concrete and different blends containing brick, tile and ferricrete as shown in Figure 4.26. In this test, different blends of RCC with Brick& Tile and Ferricrete were compacted in cylinders. All cores were kept in the oven and dried back to 60% of OMC and then wrapped and cured for 56 days.

As MRWA specification 501 (MRWA, 2012b) requires the minimum dry back of 60% prior to surfacing, cores were manufactured and dried back to 60% OMC. The tests for UCS were undertaken on samples after 56 days' curing in order to investigate the effects of non-reactive materials blended with RCC. Results showed that there was some degree of rehydration action for these recycled materials.

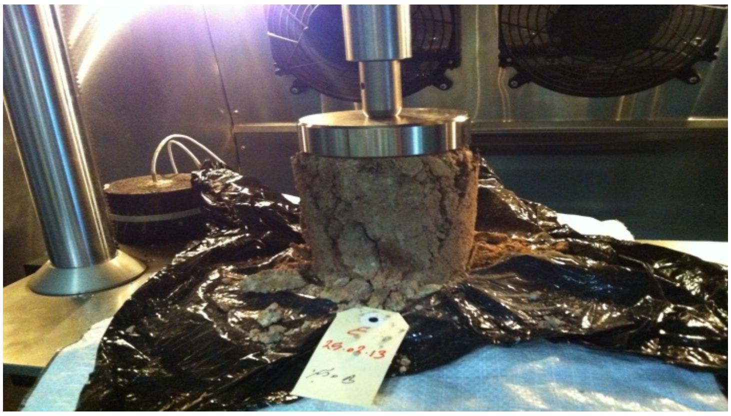


Figure 4.26: UCS test of RCC with 30% brick and tile core

The aim of this testing was to determine if the rehydration process can be controlled by the addition of non-pozzolanic materials such as brick and tile and ferricrete. This would be demonstrated by a reduction in the UCS values recorded for blends of RCC and non pozzolanic materials relative to pure RCC.

Road note 9 (MRWA, 2012a) provides useful guidance on what has been identified by past practice as practical limits for the UCS value of a material. Materials can be classed as natural, modified or stabilised. MRWA requires that for a modified granular material the 7-day unconfined compressive strength (UCS) should not exceed 1.0 MPa, and additionally if stabilised, should be within the range of 0.6MPa to 1.0MPa. It is important to note that these values are 7-day UCS values, and do not perhaps reflect the strength growth with time that occurs with pozzalanic materials. However Guide to Austroads Pavement Technology Part 4a: Granular Base and Subbase Materials (Austroads, 2008) suggest that materials that fall within the range of a 28 day UCS of 0.7MPa to 1.5MPa are modified materials. Materials with a UCS of greater than 1.5MPa are considered stabilised or bound granular.

*4.3.2.1 **Unconfined compressive strength test on pure RCC***

The UCS values of pure RCC cores were determined within the range of 0.62MPa to 0.93MPa in accordance with test method WA 143.1 (MRWA, 2012j) as shown in Table 4.19. The IPWEA/WALGA, (2012) specification for performance of recycled materials in road pavement requires the UCS value of to fall between 0.2 to 1MPa. The Table 4.18 illustrates the UCS result of pure RCC cores. Figure 4.27 shows that the typical failure mode for the UCS tests.

Table 4.18: UCS result of 100% RCC cores at 56 day cure

Code	UCS(MPa)	Maximum axial force(KN)	Axial strain,EA(%)	Deviator stress,SD(KPa)	Compressive modulus E_V(MPa)
M	0.87	7.475	0.035	878.25	49.20
N	0.7	6.068	0.042	709.23	28.25
O	0.86	7.379	0.037	864.01	40.06
K	0.66	5.68	0.036	665.75	37.86
J	0.78	6.65	0.136	779.84	41.38

Table 4.19: Valid range of UCS value for pure RCC cores

Average UCS	20% average UCS value	Minimum range of UCS value	Maximum range of UCS value
0.77	0.15	0.62	0.93

Figure 4.27 shows the stress-strain curve for sample M. It can be seen that this sample has the highest maximum axial force of all of the samples.

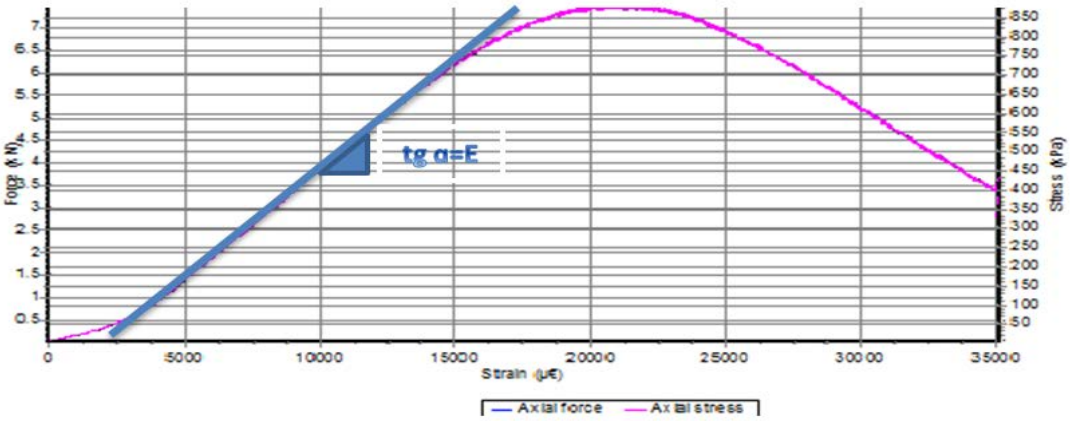


Figure 4.27: Stress-strain curve for RCC core M

Figure 4.28: Crack patterns in pure RCC cylinders

The elastic compressive modulus (E_V) was also determined as shown in Figure 4.27 which represents a constant ratio of the stress and strain of the cores. The modulus of elasticity for the cores is basically the slope of the stress-strain curve. The initial straight-line portion of the curve is the elastic range for the pure RCC cores. It is important to consider that a material's modulus of elasticity is not a measure of its strength. Strength is the stress needed to break or rupture a material (as illustrated in Figure 4.28). Table 4.20 presents the UCS results for all pure RCC cores.

Table 4.20: 56 day cured UCS results for pure RCC cores

Sample Code	Deviator stress, SD (kpa)	Axial strain, EA (%)	Maximum axial force (KN)	Moisture content at test	OMC (%)	Dryback (%)	UCS (Mpa)	Comp. Modulus E_V (Mpa)
10% brick & tile - 90% RCC								
M	878.25	0.035	7.475	7.30	12.2	59.8	0.87	49.20
N	709.23	0.042	6.068	7.30	12.2	59.8	0.7	28.25
O	864.01	0.037	7.379	7.32	12.2	60	0.86	40.06
k	665.75	0.036	5.68	7.30	12.2	59.8	0.66	37.86
J	779.84	0.136	6.65	7.30	12.2	59.8	0.78	41.38
					Mean	59.9	0.774	39.35
					Upper 90th %ile		0.9	46.10
					Lower 90th %ile		0.7	32.10

*4.3.2.2 **Unconfined compressive strength test on different blends of brick and tile and RCC***

The unconfined compressive strength test was also carried out on RCC with different blends of brick and tile in the same manner as previously described for RCC. Table 4.21 presents the UCS results for all combinations of brick and tile with RCC.

Figure 4.29: Crack patterns in blend of 10% brick and tile with RCC cylinders

Figure 4.29 shows the cracking pattern in and failure of cylinders consisting of 10% crushed brick and tile with RCC. The shape and size of the cracks were the same as for the pure RCC cylinders. Under pure unconfined compressive loading, the failure cracks formed at various angles to the applied loads. Table 4.21 presents the UCS results for all combinations of brick and tile with RCC.

Table 4.21: 56 day cured UCS results for mixtures of crushed brick and tile with RCC

Sample Code	Deviator stress, SD (kpa)	Axial strain, EA (%)	Maximum axial force (KN)	Moisture content at test	OMC (%)	Dryback (%)	UCS (Mpa)	Comp. Modulus Ev (Mpa)
10% brick & tile - 90% RCC								
S	763.26	0.04	6.505	7.4	12.3	60.2	0.76	33.40
T	666.74	0.039	5.677	7.4	12.3	60.2	0.66	31.75
U	675.04	0.046	5.758	7.4	12.3	60.2	0.67	25.40
P	702.76	0.024	5.98	7.4	12.3	60.2	0.7	36.94
Y	687.15	0.037	5.848	7.4	12.3	60.2	0.68	31.30
R	763.39	0.036	6.527	7.4	12.3	60.2	0.76	40.99
Q	714.21	0.04	6.101	7.4	12.3	60.2	0.71	30.05
Test1	529.45	0.042	4.489	7.4	12.3	60.2	0.53	23.30
Mean						60.2	0.7	31.64
Upper 90th %ile							0.8	38.15
Lower 90th %ile							0.6	24.80
30% brick & tile - 70% RCC								
F	1022.28	0.035	8.726	7.7	12.4	62.1	1.02	52.45
G	763.61	0.04	6.509	7.4	12.4	59.7	0.76	32.80
H	912.71	0.032	7.772	7.4	12.4	59.7	0.91	53.01
I	825	0.035	7.031	7.4	12.4	59.7	0.82	41.78
L	944.01	0.032	8.009	7.4	12.4	59.7	0.94	52.306
X	1076.52	0.032	9.189	7.4	12.4	59.7	1.075	57.70
6	660.44	0.051	5.668	7.4	12.4	59.7	0.66	24.82
Mean						60.0	0.9	44.98
Upper 90th %ile							1.0	54.41
Lower 90th %ile							0.7	29.60
50% brick & tile - 50% RCC								
A	702.61	0.046	5.97	5.9	12.8	46.1	0.7	23.32
B	736.86	0.034	6.295	6.0	12.8	46.9	0.73	36.95
C	827.34	0.041	6.99	5.9	12.8	46.1	0.82	33.12
D	594.24	0.11	5.05	6.0	12.8	46.9	0.59	19.175
E	868.6	0.041	7.442	6.4	12.8	50	0.86	37.48
W	1087.1	0.034	9.28	6.0	12.8	46.9	1.08	51.57
7	568.7	0.049	4.82	5.9	12.8	46.1	0.56	18.33
Mean						47	0.8	31.42
Upper 90th %ile							0.9	43.12
Lower 90th %ile							0.6	18.85

*4.3.2.3 **Unconfined compressive strength test on different blends of ferricrete and RCC***

The unconfined compressive strength test was also carried out on RCC with different blends of Ferricrete in the same manner as previously described for Brick and Tile. Table 4.22 presents the UCS results for all combinations of Ferricrete with RCC.

Table 4.22: 56 day cured UCS results for mixtures of Ferricrete with RCC

Sample Code	Deviator stress, SD (kpa)	Axial strain, EA (%)	Maximum axial force (KN)	Moisture content at test	OMC (%)	Dryback (%)	UCS (Mpa)	Comp. Modulus E_V (Mpa)
10% Ferricrete - 90% RCC								
Z1	725.47	0.046	6.198	6.5	10.8	60.18	0.72	25.43
Z2	1339.23	0.038	11.412	6.5	10.8	60.18	1.33	64.01
Z3	1034.93	0.037	8.759	6.5	10.8	60.18	1.03	50.31
Z4	399.58	0.054	3.394	6.71	10.8	62.1	0.39	14.50
Z5	765.48	0.035	6.516	6.5	10.8	60.2	0.76	36.14
Z17	979.99	0.11	8.314	6.52	10.8	60.4	0.98	47.96
OO TEST	897.26	0.039	7.660	6.37	10.8	59.0	0.89	37.27
					Mean	60.3	0.9	39.37
					Upper 90th %ile		1.2	55.80
					Lower 90th %ile		0.6	21.05
30% Ferricrete - 70% RCC								
Z6	839.18	0.038	7.15	6.37	10.6	60.1	0.83	41.77
Z7	1005.95	0.034	8.595	6.37	10.6	60.1	1.005	51.82
Z8	1088.13	0.034	9.31	6.37	10.6	60.1	1.087	55.25
Z9	1060.77	0.031	8.967	6.36	10.6	60.0	1.061	60.74
Z10	1002.88	0.041	8.574	6.37	10.6	60.1	1.003	43.94
Z18	826.63	0.039	7.044	6.37	10.6	60.1	0.82	36.45
02 TEST	508.99	0.042	4.283	6.37	10.6	60.1	0.5	19.79
					Mean	60.1	0.9	**44.25**
					Upper 90th %ile		1.1	**57.446**
					Lower 90th %ile		0.7	**29.786**
50% Ferricrete - 50% RCC								
Z11	786.67	0.044	6.702	6.15	10.2	60.30	0.78	35.79
Z12	814.47	0.046	6.934	6.15	10.2	60.30	0.81	35.57
Z13	847.21	0.044	7.23	6.13	10.2	60.10	0.84	35.31
Z14	707.16	0.047	6.048	6.13	10.2	60.10	0.7	32.69
Z15	654.29	0.049	5.630	6.13	10.2	60.10	0.65	26.02
Z16	837.75	0.042	7.139	6.13	10.2	60.10	0.83	35.35
			Mean			60.16	0.8	**33.455**
					Upper 90th %ile		0.8	**35.68**
					Lower 90th %ile		0.7	**29.355**

Figure 4.30 illustrates the failure and crack patterns in cylinders consisting of 10% ferricrete with RCC. Due to the increase in shear force, the microcracks extended parallel to the applied loads and the cracks were the same as for the other cylinders.

Figure 4.30: Crack patterns in cylinders

The results of the UCS test for 30% ferricrete with RCC are shown in Table 4.22. In this case the UCS values were higher compared to those for the other blends of ferricret. Increasing the axial forces led to an increase in the stresses.

Figure 4. 31: Crack patterns in cylinders

Cylindrical samples of the mixture of 30% ferricrete and RCC under unconfined compressive loads sustained many cracks which caused failure of the samples. In figure 4.31, many parallel cracks are visible. These shear and cone cracks were dense on one side of the cylinders.

In this research the strength and stiffness of these recycled blends were determined and compared after increasing the ferricrete content to 50%. The table 4.22 presents the UCS values of different samples. It can be seen that applying the load led to an increase in the stress, which also led to an increase in the UCS values of the samples.

Figure 4.32 shows the shear cracks in the cylindrical samples with the mixture of 50% ferricrete and RCC. Parallel cracks led to failure on two sides of the samples. The extension of microcracks led to the failure of samples with an increase in the axial force.

Figure 4.32: Crack patterns in cylinders

*4.3.2.4 **The comparison of UCS value of different blends of brick and tile, ferricrete and RCC***

The effect on unconfined compressive strength of different blends of ferricrete and crushed brick and tile with RCC was investigated and analysed. The UCS results showed that the strength of base course materials can be increased by adding 30% of these blends, but increasing the percentages of these materials to 50% did not have any effect on strength. Figure 4.33 illustrates the comparison of mean UCS values of different blends of recycled materials with RCC.

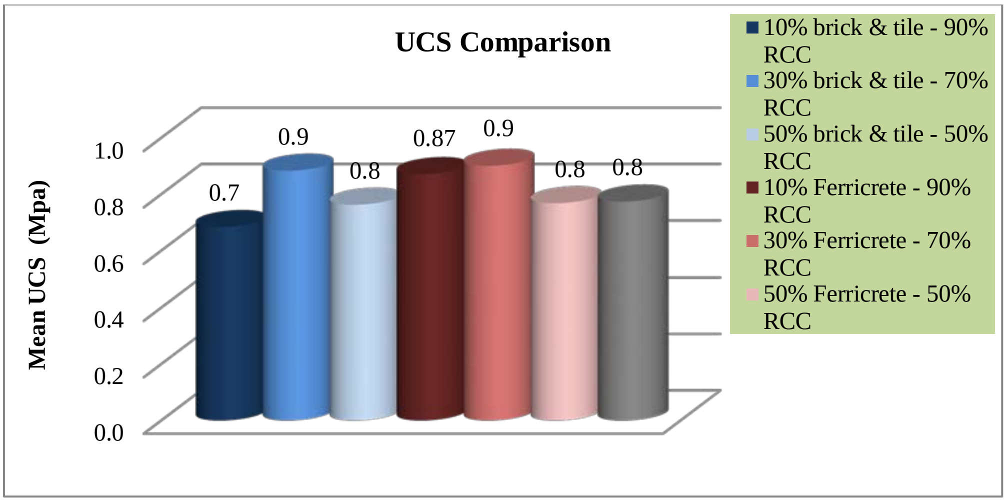


Figure 4.33: Comparison of mean UCS values of recycled materials cured for 56 days

Figure 4.34presents load-deflection curve of RCC core with 30% brick& tile. The obtained UCS values illustrated the greater strength of ferricrete compared to crushed brick and tile. In general, the highest MDD of ferricrete and the rehydration process were paramount factors for increasing the strength of these materials compared to crushed brick and tile. A combination of 10% brick and tile with RCC, compared to ferricrete, demonstrated the lowest UCS, but given the variability within the tests, this should not be viewed as significant. The UCS values for all the materials fall within the Austroads modified range (Austroads 2008).

Table 4.23 and Figure 4.35 shows and compares the UCS values of different blends of recycled materials. Based on the ANOVA of mean UCS values, the calculated F-value is less than the critical F value (p=0.05); indicating that the within sample variation and between sample variation of UCS values indicate normal distribution. Hence comparison of mean values is valid. The UCS results of all blends are shown in Appendix F of this report.

Table 4.23: Test results of unconfined compressive strength tests on RCC cores and blends

Material	100% RCC	10% Brick and tile	30% Brick and tile	50% Brick and tile	10% Ferricrete	30% Ferricrete	50% Ferricrete
Max (MPa)	0.87	0.76	1.02	0.86	1.03	1.061	0.84
Min (MPa)	0.66	0.66	0.76	0.7	0.72	0.82	0.65
Inter Quartile Range (MPa)	0.16	0.06	0.12	0.1075	0.22	0.175	0.105
SD (MPa)	0.094	0.041	0.102	0.075	0.135	0.111	0.077
Mean (MPa)	0.77	0.70	0.89	0.77	0.87	0.94	0.76
MRWA 7 day max (MPa)	1.0	1.0	1.0	1.0	1.0	1.0	1.0
MRWA 28 day max (MPa)	1.3	1.3	1.3	1.3	1.3	1.3	1.3
IPWEA 7 day (MPa)	0.2– 1.0	0.2 – 1.0	0.2 – 1.0	0.2 – 1.0	0.2 – 1.0	0.2 – 1.0	0.2– 1.0

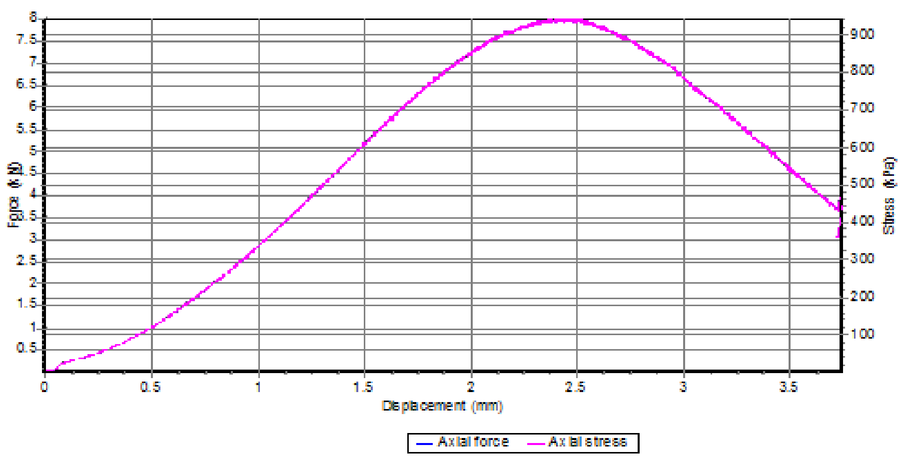


Figure 4.34: load-deflection curve of 30% crushed brick and tile with RCC-core L

Based on the UCS test results and the ANOVA (see Appendix G), it can be concluded that blending 70% RCC with either 30% brick and tile or 30% ferricrete results in higher UCS compared to pure RCC. Infact, increase of 10% and 30% of Ferricrete increased the UCS value. The comparison of UCS results of brick& tile and ferricrete present the highest value for 30% and lowest UCS value for 50%.

Blending 10% brick and tile to 90% RCC and 50% ferricrete to 50% RCC resulted in the lowest UCS values.

No significant trend could be determined with the addition of either ferricrete or brick and tile to RCC on the unconfined compressive strength of the materials as shown in Figure 4.35. However by Austroads specification, the materials fall within the modified range, but by MRWA standards, the upper values of some blends fell marginally into the bound range.

In accordance with (Austroads, 2008) Guide to Pavement Technology Part 4a: It should be noted that materials are considered to have been modified if sufficient amount of stabilising binders have been added so as to improve the performance of the materials without causing significant increase in tensile capacity (i.e. producing a bound material). There are no firmly established criteria to differentiate between modified and bound materials. However, Part 2 of the Guide considers modified materials to have a 28 day Unconfined Compressive Strength greater than 0.7 MPa and less than 1.5 MPa.

Based on (MRWA, 2012b) Specification 501 Cement stabilisation can be applied to any pavement layer, but typically only to the basecourse layer. The specimens are to be compacted at the specified density and 100% of OMC. The 7-day UCS must be in the range of 0.6 – 1.0MPa.

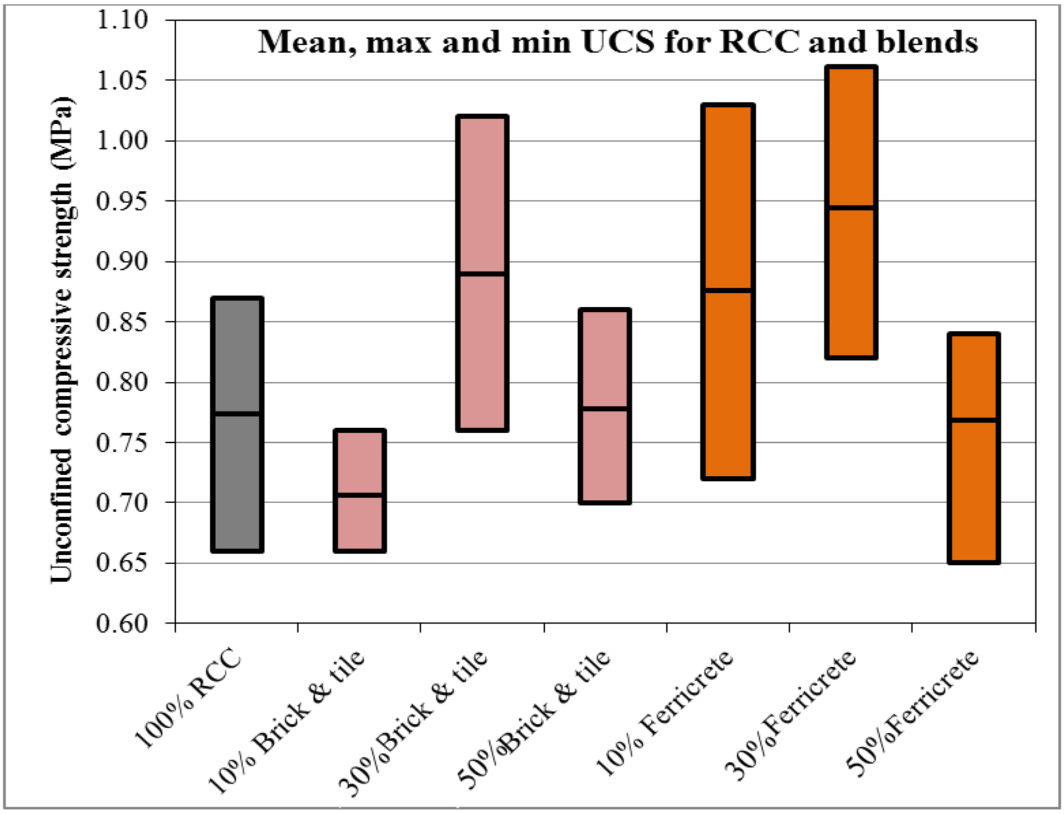


Figure 4.35: Comparison of UCS values of recycled materials cured for 56 days

CHAPTER 5

CONCLUSION AND RECOMMENDATION

5.1 **Summary and conclusion**

This study aimed to assess and evaluate the effects of rehydration of cement in recycled crushed concrete as road base material. The investigation was carried out on different blends of RCC with crushed brick and tile as one project, and Ferricrete as a second project. The physical and mechanical properties of pure RCC and different blends of recycled materials and virgin Ferricrete with RCC were evaluated and analysed in this research. A number of RCC, crushed brick and tile and Ferricrete samples were collected from stockpiles in Western Australia. Using Main Roads Western Australia (MRWA) test methods, the study first established that the recycled materials used in this study, which were obtained from C&D waste sources within the state, were within acceptable limits for particle size distribution and Atterberg limits.

The investigations were undertaken on the RCC which contains previously hydrated Portland cement and blended with non-cementitious materials in an attempt to limit rehydration, which was determined by applying the Unconfined Compressive Strength (UCS) test. A nonstandard laboratory test to investigate the control of shrinkage cracking by induced microcracking was developed. The UCS testing examined samples of various blends of recycled concrete with crushed brick and tile and with Ferricrete to determine and the optimum blends to control shrinkage cracking.

This section consists of a summary of the previous chapters, presented in six separate subsections as follows:

- non-mechanical behaviour and index properties of materials in this research
- modified compaction test results
- modified bending beam test results
- modified bending beam test results using an UTM25 device**.**
- effects of rehydration on material strength with extract cores and/or beams
- unconfined compressive strength (UCS) test results

5.1.1 ***Non-mechanical behaviour and index properties of the materials used in this research***

Particle size distribution:

The particle size distribution was determined for all constituent materials, and the following points are noted:

- The particle size distribution of the RCC samples conformed to the requirements of the IPWEA/WALGA (2012) specification for recycled base materials.
- The particle size distribution test on crushed brick and tile conformed to the requirements of the IPWEA/WALGA specification for recycled base materials.
- The particle size distribution test showed that the gradation of Ferricrete conformed to the requirements of the IPWEA/WALGA (2012) specification for recycled base materials
- As all of the constituent materials conformed to the specified PSD, all the blends tested in this research conformed to the specification.

The average Los Angeles abrasion value for RCC was 33.63% and complied with the limit of 40% as required by the IPWEA/WALGA (2012) specification

Atterberg Limits:

The Atterberg limits, liquid limit, plastic limit and linear shrinkage were determed and the following points are noted:

- The Casagrande test was selected to determine that the liquid limit value of the straight RCC was 30.3%. IPWEA/WALGA do not include a liquid limit for recycled pavement materials, but MRWA Specification 501 recomends a maximum liquid limit of 35%.
- The plastic limit tests on RCC showed that RCC is non plastic.
- When tested with the Casagrande test method, the liquid limit values of blends of RCC with crushed brick and tile at 10%, 30% and 50% by weight of crushed brick and tile there was no significant change to the resulting liquid limit.
- The plastic limit for all blends of RCC with both brick and tile, as well as Ferricrete showed the materials to be non plastic.
- The liquid limit was also determined using the cone penetrometer test method. This method gave a liquid limit value for pure RCC of 29.5% which was very close to the value obtained by the Casagrande method. To the liquid limit determined on the 10% and 30% brick and tile showed no change, but at 50% brick and tile, the liquid limit reduced slightly to 25%. The addition of Ferricrete had a similar effect of reducing the liquid limit values. Increasing the Ferricrete to 50% resulted in a liquid limit value of 21%.

Linear shrinkage:

For some base course materials, MRWA allows a maximum of 3% shrinkage, where IPWEA/WALGA limits linear shrinkage to the range of 0.2% to 1.5%. All blended samples of RCC containing either Ferricrete or brick and tile fell within the range of the IPWEA/WALGA specification. Two of the five samples of RCC tested gave shrinkage values of 2.1%, and one of 1.9%, while three others conformed to the IPWEA Specification. The blended materials showed a slightly lower shrinkage tendency than the straight RCC.

In future, it would be recommended that large size test beds are established using blends of materials to examine cracking potential due to rehydration. The test beds would be recommended to be 20m × 1m compacted at OMC to 98% of MDD and constructed in an exposed position. The elongated shape is recommended as the cracking observed in the field is always initially transverse. The development of cracking should be monitored by careful mapping of cracks for total length and width over at least a 12 month period to determine the relative shrinkage crack potential for each mix

5.1.2 ***Modified compaction test results***

Maximum dry density (MDD) and Optimum Moisture Content (OMC)

The straight RCC samples yielded a MDD of 1.89 t/m³ and a relatively high OMC of 12.16%. The addition a brick and tile had minimal effect on the MDD and OMC of the

RCC, whereas the Ferricrete blends showed an increase in MDD and a decrease in OMC as the proportion of Ferricrete increased.

5.1.3 *Modified bending beam test results*

A series of tests for simulating microcracking and determining the maximum strength of materials were performed on slabs of pure RCC slabs which were made under various conditions as described in Chapter Three. This testing procedure involved manufacture of slabs using the Cooper slab compactor, and then subsequently recompacting one set of slabs after 24 hours curing, a second set at 48 hours curing, whilst the third set (the control) was left to cure. Slabs were subjected to an ultimate bending strength test after a period of 56 days.

This testing showed that by recompacting at 24 hours, an increase in strength occurred, and at 48 hours, the strength again showed a possible slight increase over the control. This testing did not achieve the intended outcome of inducing microcracking and limiting strength gain, and it is considered that the reason for this was that the testing was undertaken too soon. It is recommended that the modified beam test be undertaken on more slabs be manufactured and tested at an extended period post initial compaction, 48 hours, 56 hours, 72 hours and 96 hours to determine if the inducement of microcracking can reduce the strength gain over time. This should be undertaken while duplicate sets of samples are cured in a humidity cabinet and in a low heat oven at around 30oC to determine if the period of hydration in the field needs to be adjusted according to the prevailing conditions at the time of construction. At least six slabs should be tested at each condition to ensure data is sufficient to analyse statistically, but nine replicates would be preferred.

5.1.4 *Unconfined compressive strength (UCS) tests on blended RCC*

The aim of this part of the research was to determine if the excessive strength gain of RCC with blends of either brick and tile or Ferricrete could limit the strength gain with time compared to the contol sampes of pure RCC. The unconfined compressive strength test was adopted to test the theory that by the addition of non pozzalanic materials, the UCS should reduce compared to that of the RCC when tested at 56 days.

However this was not the case, all blends showed a mean UCS similar to the control of straight RCC. None of the mean UCS values exceeded the limits described in the Austroads Guide to Pavement Technology Part 4 D : Stabilised materials (2006) as falling in the modified range specified as between 0.6MPa and 1.5MPa, and could best be described as slightly modified in all cases.

The compressive modulus values were also determined as part of this test. The RCC showed the lowest modulus. However if the results of the brick and tile blends are considered, increasing the brick and tile content resulted in a lower modulus, but the 10% blend showed a much higher modulus than the control, indicating that the control samples may have been compromised.

5.2 *Recommendations*

1. It is recommended that the modified beam test be undertaken on more slabs be manufactured and tested at an extended period post initial compaction, 48 hours, 56 hours, 72 hours and 96 hours to determine if the inducement of microcracking can reduce the strength gain over time. This should be undertaken while duplicate sets of samples are cured in a humidity cabinet and in a low heat oven at around 30^{o}C to determine if the period of hydration in the field needs to be adjusted according to the prevailing conditions at the time of construction. At least six slabs should be tested at each condition to ensure data is sufficient to analyse statistically, but nine replicates would be preferred.
2. The potential to use repeated load triaxial testing be investigated as a means of inducing microcracking in the samples.
3. Large size test beds are established using blends of materials to examine cracking potential due to rehydration. The test beds would be recommended to be 20m × 1m compacted at OMC to 98% of MDD and constructed in an exposed position. The elongated shape is recommended as the cracking observed in the field is always initially transverse. The development of cracking should be monitored by careful mapping of cracks for total length and width over at least a 12 month period to determine the relative shrinkage crack potential for each mix.

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Appendix A

IPWEA/WALGA SPECIFICATION

Specification for the supply of road making aggregates sourced from by-products of the construction and demolition industry

The following specification is that developed jointly by IPWEA and WALGA for the supply of road base materials manufactured predominantly from recycled crushed concrete sourced from the demolition of structures.

IPWEA/WALGA specification

Specification for the supply of road making aggregates sourced from by-products of the construction and demolition industry

1 General

The material shall consist of a uniformly blended mixture of coarse and fine aggregate resulting from the crushing of recycled concrete from construction and demolition material. It may contain other materials such as clay brick and tile, sand and glass according to the limits specified in Table 2.

2 Material classes

The material classes shall be determined according to the end use of the product which will be determined by the pavement design, traffic conditions and level in the pavement. The recommended material class required for a specific application is specified in Table

Table 1: Material class for given application

Level in pavement	Traffic (ESA/day)			
	> 500	< 500	50-100	< 50
Base < 50 mm asphalt or spray seal	Class 1	Class 1	Class 1	Class 1
Base ≥ 50 mm asphalt	Class 1	Class 1	Class 1	Class 2
Subbase	Class 2	Class 2	Class 2	Class 2

3 Limits on source material composition

Limits on the material composition are shown in Table 2.

Table 2: Limits on constituent materials based on material class

Material	Class 1	Class 2
	Maximum %	
Crushed Recycled Concrete (RCC)	95	95

Recycled Asphalt Pavement (RAP)	10	15
High density clay brick and tile	10	15
Low density materials (plastic, plaster, etc.)	1	1.5
Organic Matter (Wood, etc)	0.5	0.5 as base
Unacceptable high density materials (metals, glass, ceramics > 4 mm)	2	3
Asbestos and other hazardous materials	0	0

4 Particle size distribution (PSD)

PSD shall conform to the limits of Table 4. The PSD curve shall be classified by the descriptive classification as shown in Table 3. The PSD shall be determined in accordance with MRWA test method WA 115.1

Coarse aggregate (retained 4.75 mm sieve) shall consist of clean, hard, durable, angular fragments of recycled concrete or asphalt produced by crushing sound recycled materials originally made from sound unweathered rock and shall not include materials which break up when alternately wetted and dried.

Fine aggregate (passing 4.75 mm sieve) shall consist of crushed rock fragments or a mixture of crushed recycled concrete, asphalt or brick fragments produced by crushing sound recycled materials originally made from sound unweathered rock, clays or natural sand.

Table 3: Shape variability class for PSD

Shape variability descriptor	Shape attributes of PSD curve
Low	Where the grading curve fits smoothly within the envelope, and may gradually move from the high limits to the low limits or from the lower limits to the higher limits but does not wander between extremes
Medium	Where the grading curve changes from the higher limits to the lower limits or the lower limits to the higher limits in one sieve size that is above the 2.36 sieve
High	Where the grading curve fits outside the envelope for one or two sieve sizes above the 2.36 mm sieve, or where the grading envelope changes from the lower limits to the higher limits for any sieve size 2.36 mm or less, or where the grading curve changes from the higher limits to the lower limits or the lower limits to the higher limits on more than one instance
Unacceptable	Where the grading curve falls outside the envelope for any sieve size 2.36 mm or less or for more than two sieve sizes above 2.36 mm

Table 4: Limits for particle size distribution

Material Class	Class 1	Class 2
AS sieve size (mm)	% passing by mass minimum and maximum limits	
75		
50		
37.5		
26.5	100 - 100	100 - 100
19.0	95 - 100	95 - 100

9.50	60 - 80	59 - 82
4.75	40 - 60	41 - 65
2.36	30 - 45	29 - 52
1.18	20 - 35	20 - 41
0.600	13 - 27	13 - 29
0.425	11 - 23	10 - 23
0.300	8 - 20	8 - 20
0.150	5 - 14	5 - 14
0.075	3 - 11	3 - 11
Ratio of 0.475:0.075	0.35 – 0.60	0.35 – 0.60
Shape variability	Low	Base: Low or medium Subbase: Low, medium or high

5 Linear shrinkage (LS)

Linear shrinkage shall be determined on the portion of material passing the 425μm sieve in accordance with MRWA test method WA 123.1 Limits for LS are given in Table 5.

Table 5: Limits for linear shrinkage

Linear shrinkage (7 day)	Class 1 & 2
Base (%)	0.2 – 1.5
Subbase (%)	0.2 – 4.0

6 Unconfined Compressive Strength (UCS)

The UCS of the material when tested in accordance with MRWA test method WA 143.1 (7 days cured and 4 hours immersed) shall conform with the requirements of Table 6.

Table 6: Limits for unconfined compressive strength

Unconfined compressive strength	Class 1 & 2
Base (kPa)	200 – 1000
Sub-base (kPa)	200 – 2000

7 Micro Deval loss or Los Angeles abrasion coarse aggregate

The Micro Deval test on coarse aggregate is determined on material retained on the 9.5 mm sieve. The test method shall be determined in accordance with ASTM D6928 - 08e1 *Standard Test Method for Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro Deval Apparatus.* Limits for the Micro Deval Loss are given in Table A.8a. Limits for Los Angeles abrasion are given in Table A.8b

Table 7: Limits for Micro Deval test

Micro Deval test	Class 1	Class 2
Micro Deval loss (%)	<15	<20

Table 8: Limits for Los Angeles abrasion test

Los Angeles abrasion	Class 1	Class 2
Los Angeles abrasion loss (%)	<40	<42

8 California Bearing Ratio (CBR)

The CBR shall be determined in accordance with MRWA test method WA 141.1. The sample shall be soaked for four days. The minimum requirements for CBR are detailed in Table 9

Table 9: Limits for CBR

California Bearing Ratio	Class 1 (98% MDD, 100% OMC)	Class 2 (98% MDD, 100% OMC)
Californian Bearing Ratio (CBR) (%)	>100	>100

9 Minimum performance requirements

Performance tests including the repeat load triaxial test (RLTT) and consolidated undrained triaxial test shall be used to determine the performance parameters of:

resilient modulus at 400 kPa normal stress and 150 kPa confining stress both at 60% OMC

resilient modulus at 400 kPa normal stress and 150 kPa confining stress both at 80% OMC

maximum permanent strain at end of stress stage modulus cycle (%)

maximum cohesion (c) @ 28 days and 60% OMC (kPa)

minimum angle internal friction (ϕ) @ 28 days and 60% OMC

The consolidated undrained triaxial test shall be undertaken in accordance with MRWA test method WA 151.1 *Main Roads Western Australia Triaxial Test: Consolidated Undrained* and shall be used to report maximum cohesion (c) and minimum angle internal friction (ϕ).

Table 10: Limits for modulus, permanent strain, friction angle and cohesion

Performance test limits	Class 1	Class 2
Resilient modulus @ 400 kPa normal stress and 150 kPa confining stress		
at 60% OMC	700 - 1000	650 - 1000

at 80% OMC	>600	>550
Maximum permanent strain at end of stress stage modulus cycle (%)	3%	3.5%
Maximum cohesion (c) @ 28 days and 60% OMC (kPa)	100	120
Minimum angle internal friction (f) @ 28 days and 60% OMC	55^0	53^0

The RLTT shall be undertaken in accordance with Austroads Repeated Load Triaxial Test Method – *Determination of permanent deformation and resilient modulus characteristics of unbound granular materials under drained conditions* (Vuong and Brimble 2000) and shall be used to report modulus and permanent strain values. The target values are given in Table A.10.

10 Maximum dry density (MDD) and optimum moisture content (OMC)

The MDD and OMC of the material shall be determined in addition to all other tests at Frequency A as outlined in A.2. MDD and OMC shall be determined in accordance with MRWA test method WA 133.2 *Dry density/moisture content relationship: modified compaction coarse grained soils.*

11 Test frequency and sampling methods

11.1 Sampling methods

Sampling shall be undertaken using one of three options:

one sample per time period

one sample per number of tonnes produced

certified stockpile.

Where a certified stockpile is used, the stockpile shall be manufactured, tested and certified, and no more material shall be deposited to the stockpile after certification.

Where a certified stockpile is used, the number of samples shall be related to the size of the stockpile. Samples shall be collected in accordance with MRWA test method WA 200.1 part 5 except that the limits on stockpile size shall not apply.

Each sample shall be collected and tested individually. The number of samples required shall be determined as follows:

stockpile <1,000 m^3, 3 samples

stockpile 1,000 – 2,000 m^3, 6 samples

stockpile >2,000 m^3, 6 samples + 1 sample per 1,000 m^3.

Where sampling is undertaken during continuous batching operations and forms part of a process control, sampling shall be undertaken in accordance with MRWA test method WA 200.1 part 2.

11.2 Sampling frequency

Where sampling is undertaken during continuous batching operations and forms part of a process control, sampling shall be undertaken on either a unit of time or unit of mass basis whichever is the most frequent. Testing shall be dependent on the importance of the test and shall be at Frequency A or Frequency B or Frequency C as follows:

Frequency A

particle size distribution (PSD)

linear shrinkage (modified)

percentage foreign materials (modified).

These shall be at a frequency of one sample per 1000 tonne or one sample per week or change in source material.

Frequency B

Micro Deval (preferred)

OR

Los Angeles Abrasion

Californian Bearing Ratio (CBR)

unconfined compressive strength.

These shall be at a frequency of one sample per 5000 tonne or one sample per month or change in source material.

Frequency C

repeat load triaxial test

consolidated undrained triaxial test

These shall be at a frequency of one sample per 15000 tonne or one sample per 3 months or change in source material.

11.3 Testing authority

All testing shall be undertaken by a NATA accredited laboratory.

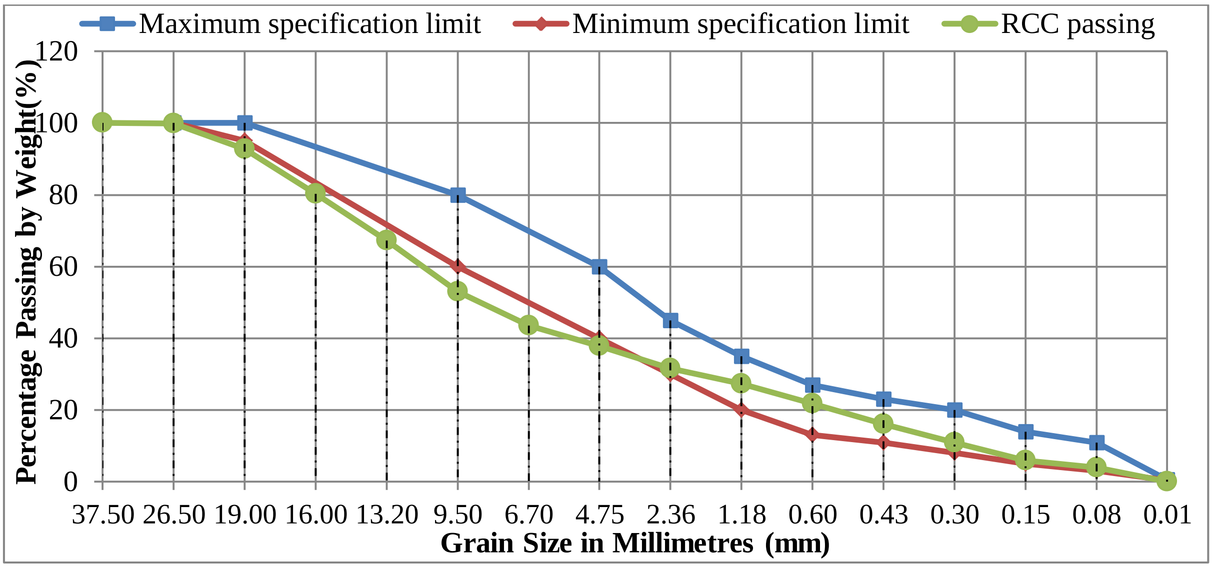
12 Reporting

All test results shall be kept on file and shall be distributed to the client organisation within 4 weeks of the sample date.

Appendix B: Particle size distribution curves

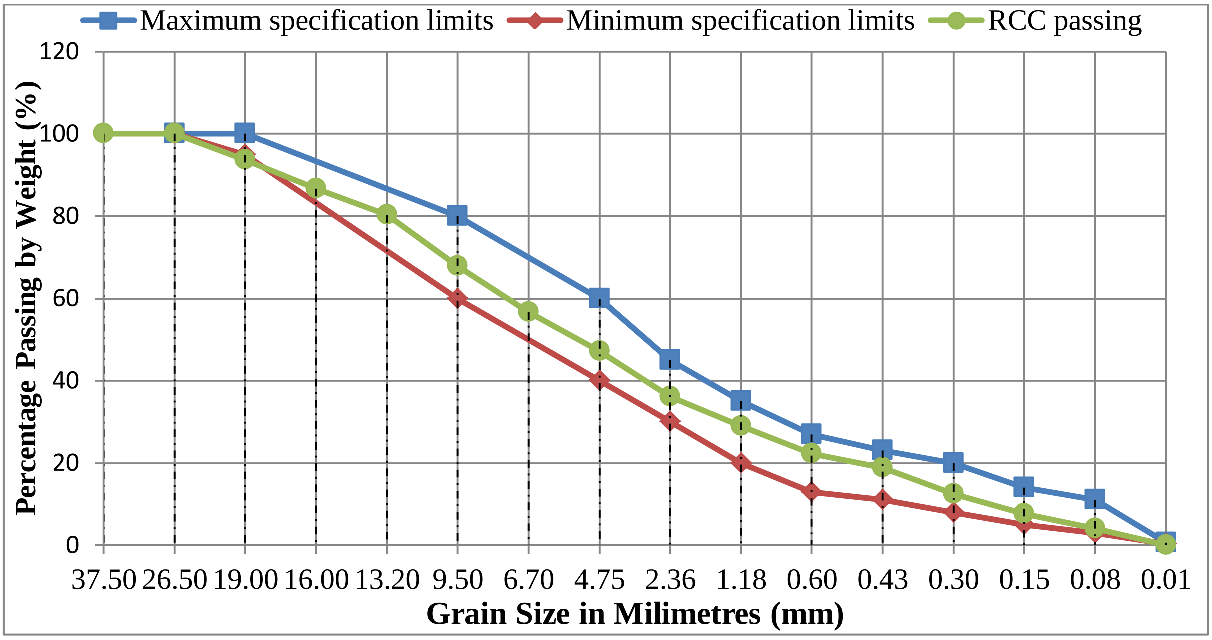
Appendix B1: RCC grading charts

Sample 1 - Rejected



Sieve Size	Mass retained	Mass passed	% passing
37.5	0.00	15855.08	100.00
26.5	22.00	15833.08	99.86
19	1130.00	14703.08	92.73
16	1966.25	12736.83	80.33
13.2	2067.00	10669.83	67.30
9.5	2274.00	8395.83	52.95
6.7	1487.00	6908.83	43.57
4.75	904.00	6004.83	37.87
2.36	987.17	5017.66	31.65
1.18	688.76	4328.90	27.30
0.6	873.28	3455.62	21.80
0.425	894.22	2561.40	16.16
0.3	843.02	1718.38	10.84
0.15	779.19	939.18	5.92
0.075	315.60	623.58	3.93
0.0135	623.58	0.00	0.00

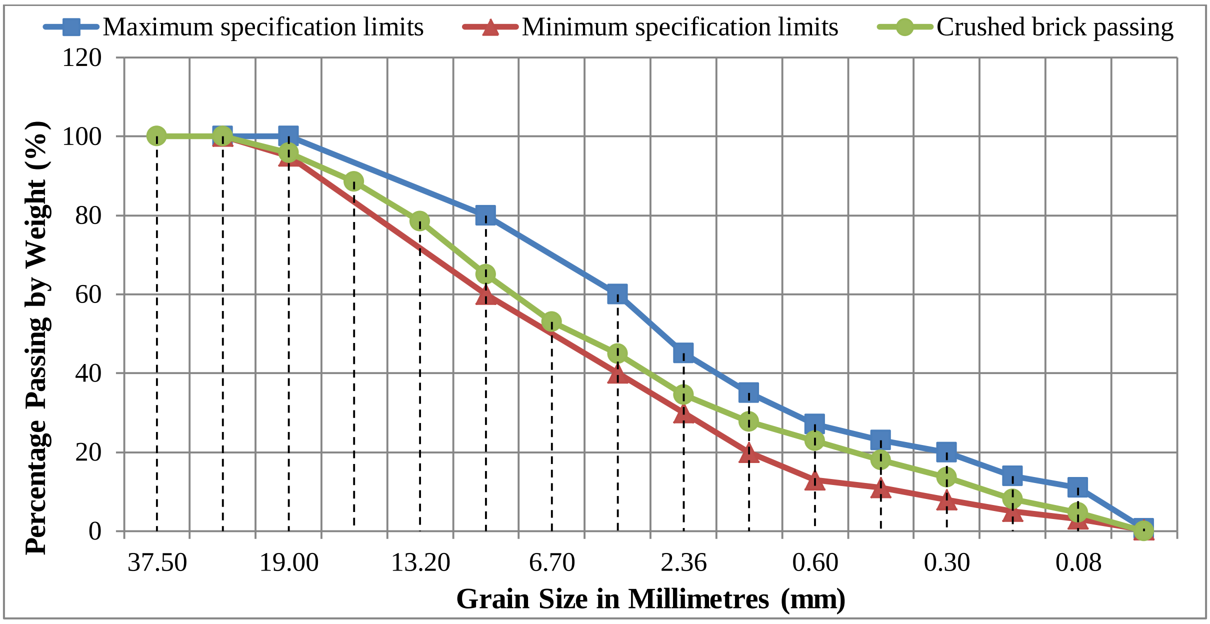
Sample 2: Accepted



Sieve	Mass retained	Mass passed	% passing
37.5	0.00	5980.41	100.00
26.5	0.00	5980.41	100.00
19	376.71	5603.70	93.70
16	419.29	5184.41	86.69
13.2	380.00	4804.41	80.34
9.5	743.06	4061.35	67.91
6.7	671.00	3390.35	56.69
4.75	566.83	2823.52	47.21
2.36	660.35	2163.17	36.17
1.18	432.00	1731.17	28.95
0.6	398.17	1333.00	22.29
0.425	206.00	1127.00	18.84
0.3	380.00	747.00	12.49
0.15	289.00	458.00	7.66
0.075	213.35	244.65	4.09
0.01	244.65	0.00	0.00

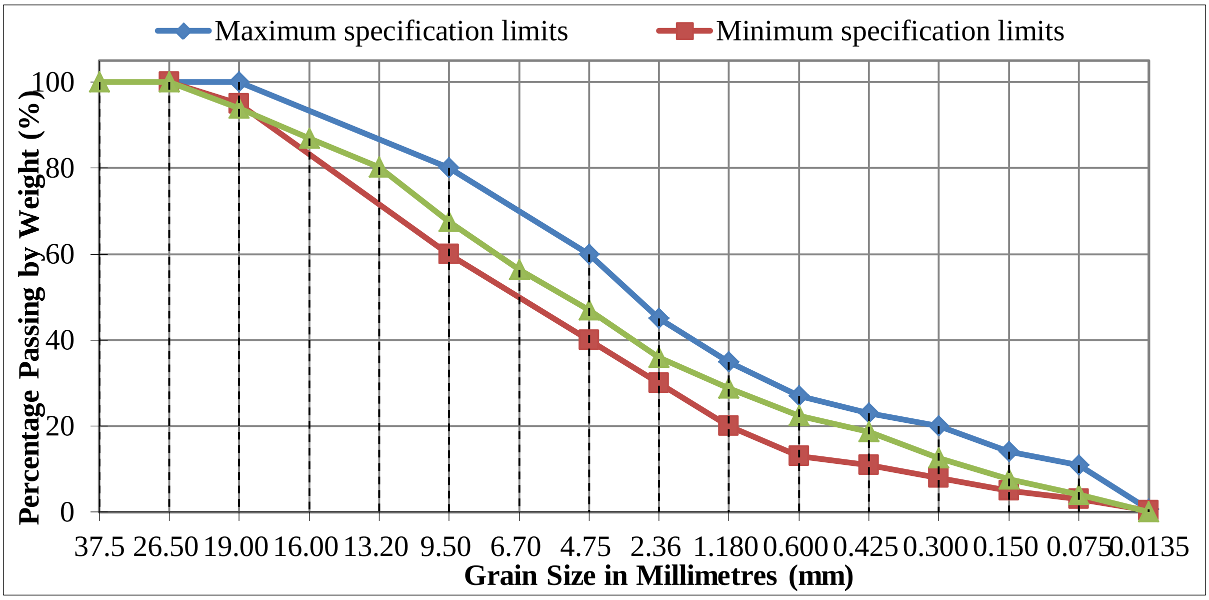
Appendix B2: Particle size distribution of crushed brick and tile with RCC

Pure crushed brick and tile sample



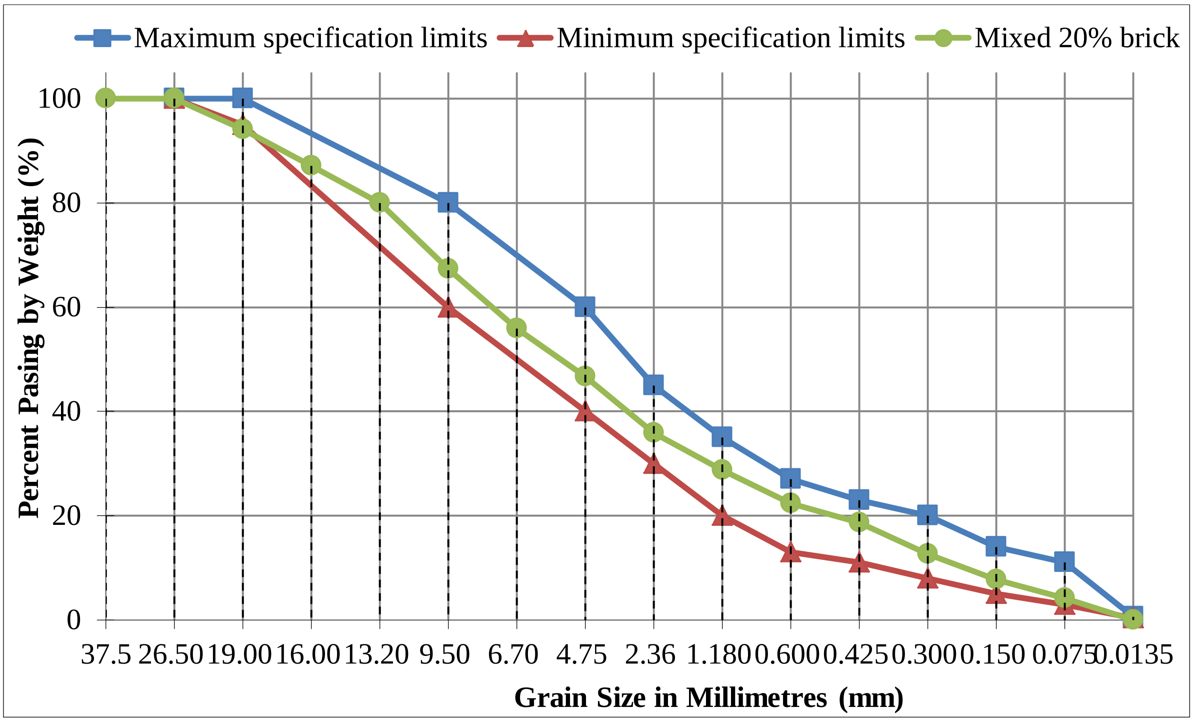
Sieve Size	Mass retained	Mass passed	% passing
37.5	0.00	5984.49	100.00
26.5	0.00	5984.49	100.00
19	253.00	5731.49	95.77
16	437.00	5294.49	88.47
13.2	596.00	4698.49	78.51
9.5	804.04	3894.45	65.08
6.7	720.00	3174.46	53.04
4.75	488.00	2686.46	44.89
2.36	621.00	2065.46	34.51
1.18	402.45	1663.01	27.79
0.6	296.78	1366.23	22.83
0.425	289.81	1076.42	17.99
0.3	265.78	810.64	13.55
0.15	322.48	488.16	8.16
0.075	209.68	278.48	4.65
0.0135	278.48	0.00	0.00

10% crushed brick and tile with RCC



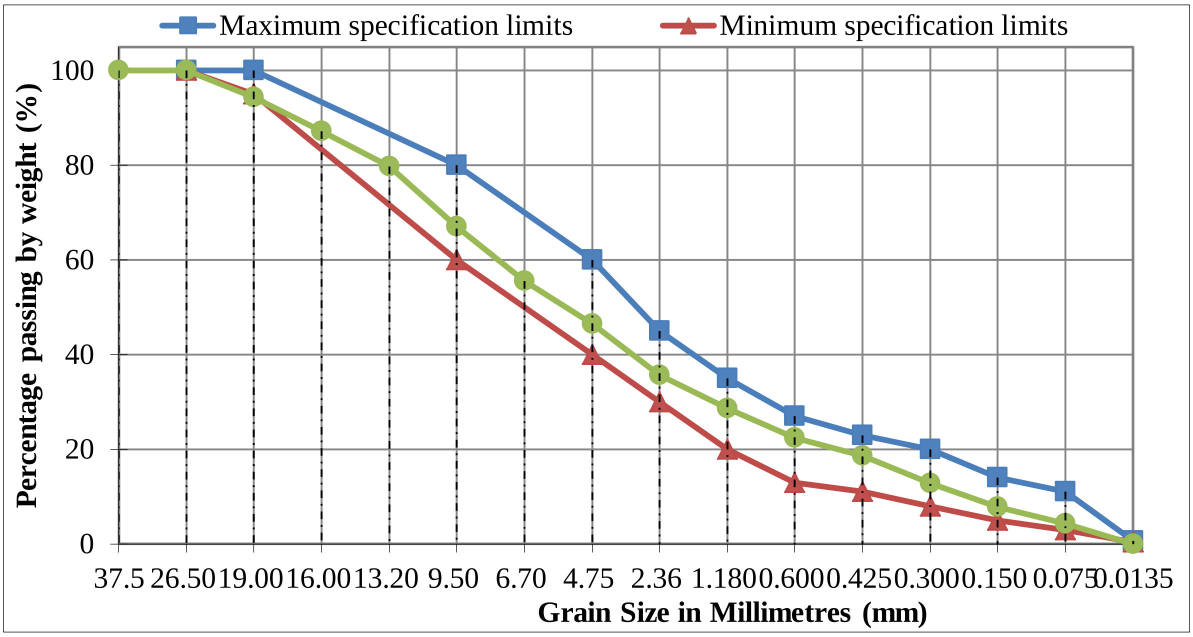
Sieve Size	Brick	%	concrete	%	Comb	Mid	Spec	Spec
(mm)	PSD	10.0	PSD	90.0	100.0	Point	Min	Max
37.5	100.0	10.0	100.0	90.0	100.0			
26.50	100.0	10.0	100.0	90.0	100.0	100.0	100	100
19.00	95.8	9.6	93.7	84.3	93.9	97.5	95	100
16.00	88.5	8.8	86.7	78.0	86.9	0.0		
13.20	78.5	7.9	80.3	72.3	80.2	0.0		
9.50	65.1	6.5	67.9	61.1	67.6	70.0	60	80
6.70	53.0	5.3	56.7	51.0	56.3	0.0		
4.75	44.9	4.5	47.2	42.5	47.0	50.0	40	60
2.36	34.5	3.5	36.2	32.6	36.0	37.5	30	45
1.180	27.8	2.8	29.0	26.1	28.8	27.5	20	35
0.600	22.8	2.3	22.3	20.1	22.3	20.0	13	27
0.425	18.0	1.8	18.8	17.0	18.8	17.0	11	23
0.300	13.6	1.4	12.5	11.2	12.6	14.0	8	20
0.150	8.16	0.8	7.66	6.9	7.7	9.50	5	14
0.075	4.65	0.5	4.09	3.7	4.1	7.00	3	11
0.0135	0.0	0.0	0.0	0.0	0.0	0.48	0	1

20% crushed brick and tile with RCC



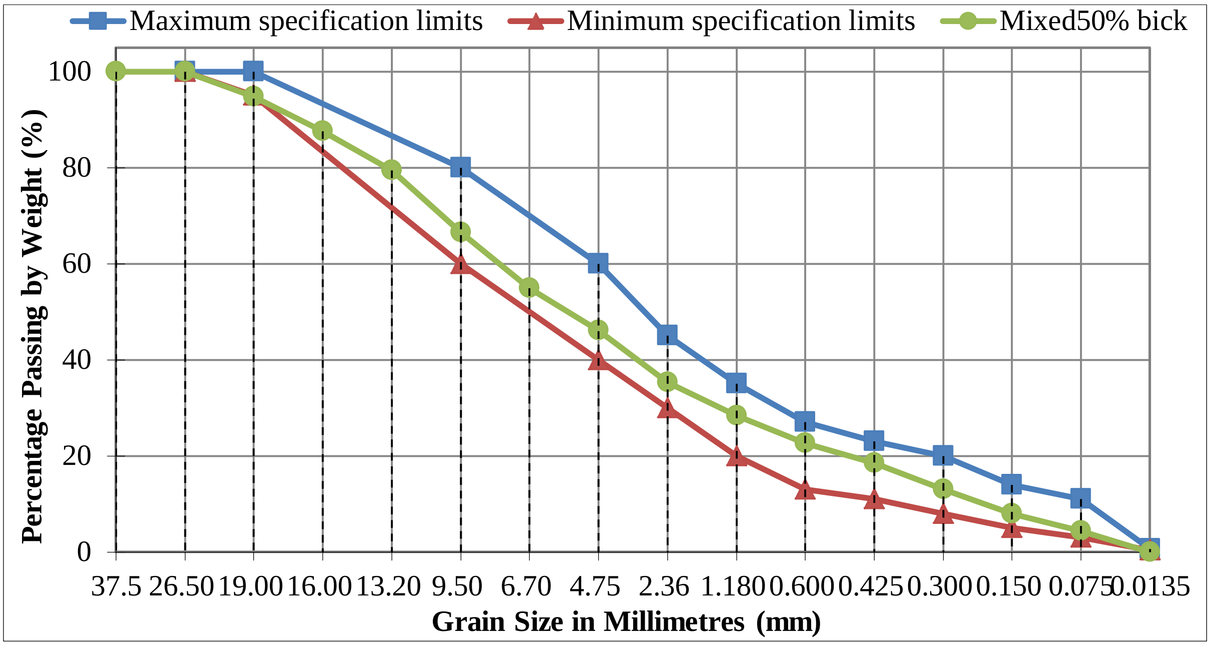
Sieve Size	Brick	%	concrete	%	Comb	Mid	Spec	Spec
(mm)	PSD	20.0	PSD	80.0	100.0	Point	Min	Max
37.5	100.0	20.0	100.0	80.0	100.0			
26.50	100.0	20.0	100.0	80.0	100.0	100.0	100	100
19.00	95.8	19.2	93.7	75.0	94.1	97.5	95	100
16.00	88.5	17.7	86.7	69.4	87.0	0.0		
13.20	78.5	15.7	80.3	64.3	80.0	0.0		
9.50	65.1	13.0	67.9	54.3	67.3	70.0	60	80
6.70	53.0	10.6	56.7	45.4	56.0	0.0		
4.75	44.9	9.0	47.2	37.8	46.7	50.0	40	60
2.36	34.5	6.9	36.2	28.9	35.8	37.5	30	45
1.180	27.8	5.6	29.0	23.2	28.7	27.5	20	35
0.600	22.8	4.6	22.3	17.8	22.4	20.0	13	27
0.425	18.0	3.6	18.8	15.1	18.7	17.0	11	23
0.300	13.6	2.7	12.5	10.0	12.7	14.0	8	20
0.150	8.16	1.6	7.66	6.1	7.8	9.50	5	14
0.075	4.65	0.9	4.09	3.3	4.2	7.00	3	11
0.0135	0.0	0.0	0.0	0.0	0.0	0.00	0	1

30% Crushed brick and tile with RCC



Sieve Size	Brick	%	concrete	%	Comb	Mid	Spec	Spec
(mm)	PSD	30.0	PSD	70.0	100.0	Point	Min	Max
37.5	100.0	30.0	100.0	70.0	100.0			
26.50	100.0	30.0	100.0	70.0	100.0	100.0	100	100
19.00	95.8	28.7	93.7	65.6	94.3	97.5	95	100
16.00	88.5	26.5	86.7	60.7	87.2	0.0		
13.20	78.5	23.6	80.3	56.2	79.8	0.0		
9.50	65.1	19.5	67.9	47.5	67.1	70.0	60	80
6.70	53.0	15.9	56.7	39.7	55.6	0.0		
4.75	44.9	13.5	47.2	33.0	46.5	50.0	40	60
2.36	34.5	10.4	36.2	25.3	35.7	37.5	30	45
1.180	27.8	8.3	29.0	20.3	28.6	27.5	20	35
0.600	22.8	6.8	22.3	15.6	22.5	20.0	13	27
0.425	18.0	5.4	18.8	13.2	18.6	17.0	11	23
0.300	13.6	4.1	12.5	8.7	12.8	14.0	8	20
0.150	8.16	2.4	7.66	5.4	7.8	9.50	5	14
0.075	4.65	1.4	4.09	2.9	4.3	7.00	3	11
0.0135	0.0	0.0	0.0	0.0	0.0	0.00	0	1

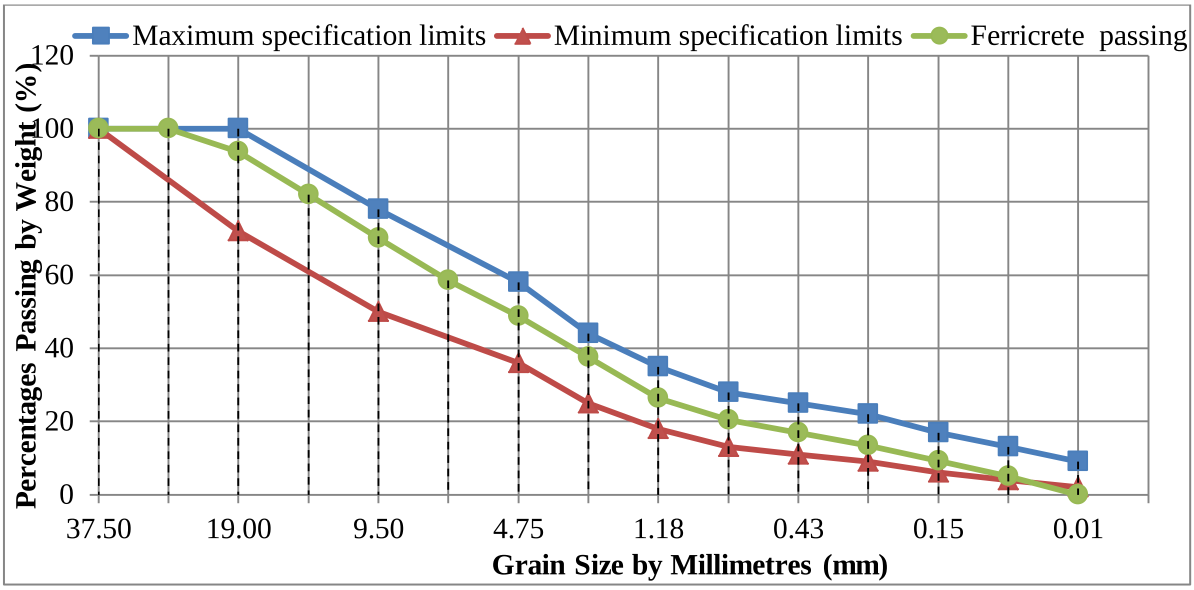
50% Crushed brick and tile with RCC



Sieve Size	Brick	%	concrete	%	Comb	Mid	Spec	Spec
(mm)	PSD	50.0	PSD	50.0	100.0	Point	Min	Max
37.5	100.0	50.0	100.0	50.0	100.0			
26.50	100.0	50.0	100.0	50.0	100.0	100.0	100	100
19.00	95.8	47.9	93.7	46.9	94.7	97.5	95	100
16.00	88.5	44.2	86.7	43.3	87.6	0.0		
13.20	78.5	39.3	80.3	40.2	79.4	0.0		
9.50	65.1	32.5	67.9	34.0	66.5	70.0	60	80
6.70	53.0	26.5	56.7	28.3	54.9	0.0		
4.75	44.9	22.4	47.2	23.6	46.1	50.0	40	60
2.36	34.5	17.3	36.2	18.1	35.3	37.5	30	45
1.180	27.8	13.9	29.0	14.5	28.4	27.5	20	35
0.600	22.8	11.4	22.3	11.1	22.6	20.0	13	27
0.425	18.0	9.0	18.8	9.4	18.4	17.0	11	23
0.300	13.6	6.8	12.5	6.2	13.0	14.0	8	20
0.150	8.16	4.1	7.66	3.8	7.9	9.50	5	14
0.075	4.65	2.3	4.09	2.0	4.4	7.00	3	11
0.0135	0.0	0.0	0.0	0.0	0.0	0.00	0	1

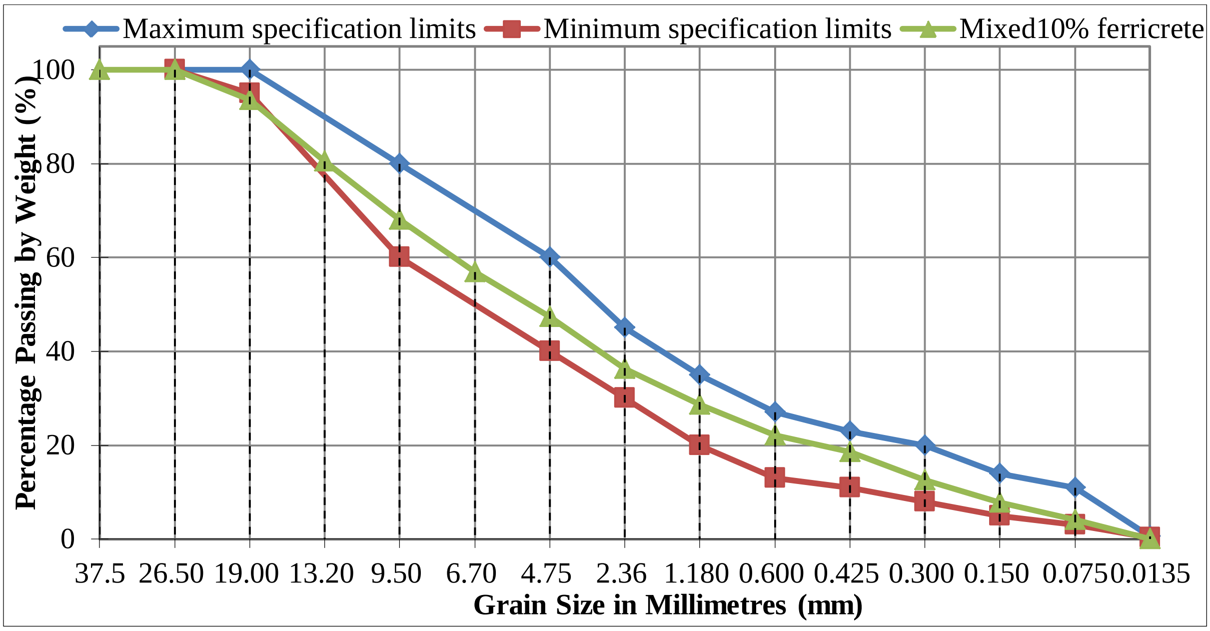
Appendix B3: Particle size distribution of ferricrete and RCC

Pure ferricrete



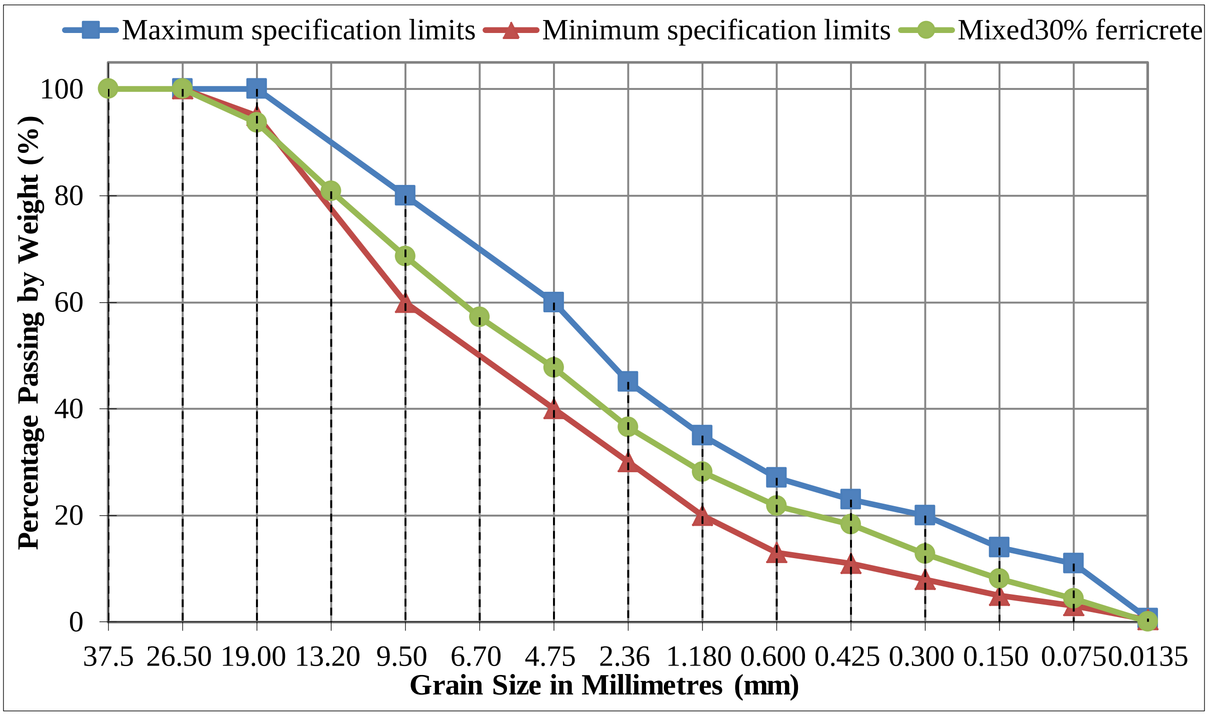
Sieve Size	Mass retained	Mass passed	% passing
37.5	0.00	5962.87	100.00
26.5	0.00	5962.87	100.00
19	379.94	5582.93	93.63
13.2	694.00	4888.93	81.99
9.5	704.65	4184.28	70.17
6.7	697.00	3487.28	58.48
4.75	576.10	2911.18	48.82
2.36	676.20	2234.98	37.48
1.18	663.50	1571.48	26.35
0.6	353.48	1218.00	20.43
0.425	214.20	1003.80	16.83
0.3	205.80	798.00	13.38
0.15	250.95	547.05	9.17
0.075	246.75	300.30	5.04
0.0135	300.30	0.00	0.00

10% ferricrete with RCC



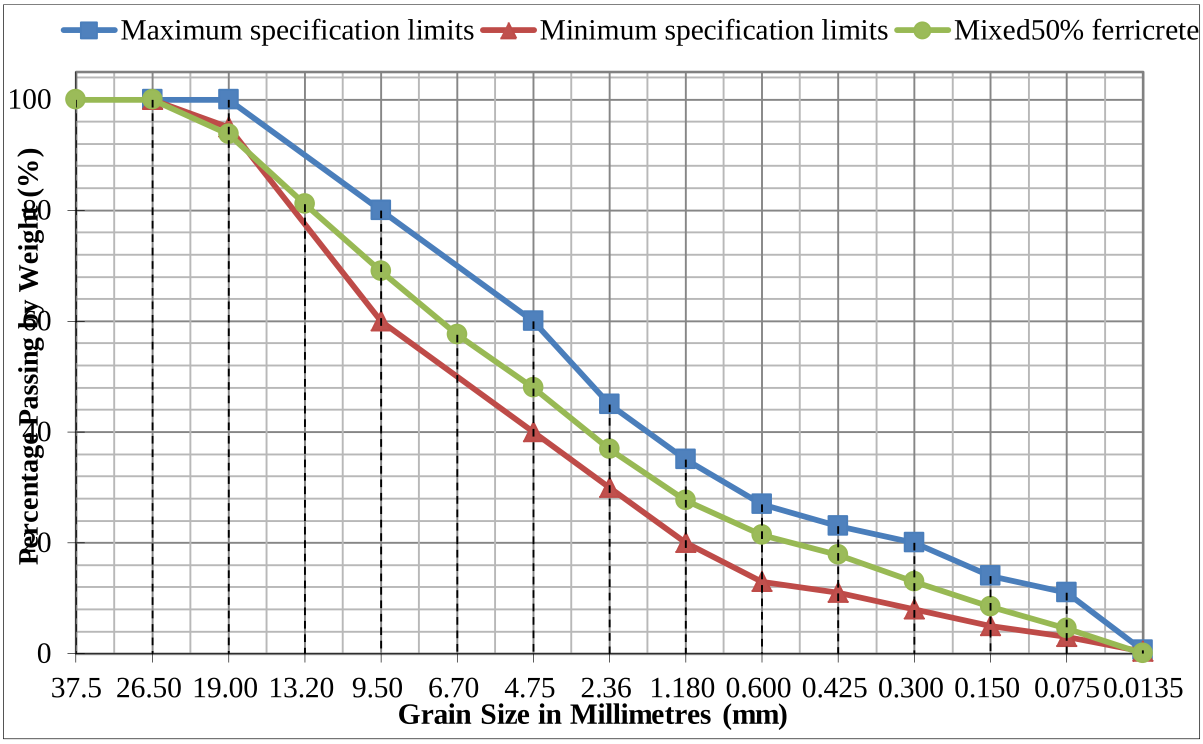
Sieve Size	Ferricrete	%	Concrete	%	Comb	Mid	Spec	Spec
(mm)	PSD	10.0	PSD	90.0	100.0	Point	Min	Max
37.5	100.0	10.0	100.0	90.0	100.0			
26.50	100.0	10.0	100.0	90.0	100.0	100.0	100	100
19.00	93.6	9.4	93.7	84.3	93.7	97.5	95	100
13.20	82.0	8.2	80.3	72.3	80.5	0.0		
9.50	70.2	7.0	67.9	61.1	68.1	70.0	60	80
6.70	58.5	5.8	56.7	51.0	56.9	0.0		
4.75	48.8	4.9	47.2	42.5	47.4	50.0	40	60
2.36	37.5	3.7	36.2	32.6	36.3	37.5	30	45
1.180	26.4	2.6	29.0	26.1	28.7	27.5	20	35
0.600	20.4	2.0	22.3	20.1	22.1	20.0	13	27
0.425	16.8	1.7	18.8	17.0	18.6	17.0	11	23
0.300	13.4	1.3	12.5	11.2	12.6	14.0	8	20
0.150	9.17	0.9	7.66	6.9	7.8	9.50	5	14
0.075	5.03	0.5	4.09	3.7	4.2	7.00	3	11
0.0135	0.0	0.0	0.0	0.0	0.0	0.00	0	1

30% ferricrete with RCC



Sieve Size	Ferricrete	%	concrete	%	Comb	Mid	Spec	Spec
(mm)	PSD	30.0	PSD	70.0	100.0	Point	Min	Max
37.5	100.0	30.0	100.0	70.0	100.0			
26.50	100.0	30.0	100.0	70.0	100.0	100.0	100	100
19.00	93.6	28.1	93.7	65.6	93.7	97.5	95	100
13.20	82.0	24.6	80.3	56.2	80.8	0.0		
9.50	70.2	21.1	67.9	47.5	68.6	70.0	60	80
6.70	58.5	17.5	56.7	39.7	57.2	0.0		
4.75	48.8	14.6	47.2	33.0	47.7	50.0	40	60
2.36	37.5	11.2	36.2	25.3	36.6	37.5	30	45
1.180	26.4	7.9	29.0	20.3	28.2	27.5	20	35
0.600	20.4	6.1	22.3	15.6	21.7	20.0	13	27
0.425	16.8	5.0	18.8	13.2	18.2	17.0	11	23
0.300	13.4	4.0	12.5	8.7	12.8	14.0	8	20
0.150	9.17	2.8	7.66	5.4	8.1	9.50	5	14
0.075	5.03	1.5	4.09	2.9	4.4	7.00	3	11
0.0135	0.0	0.0	0.0	0.0	0.0	0.00	0	1

50% ferricrete with RCC



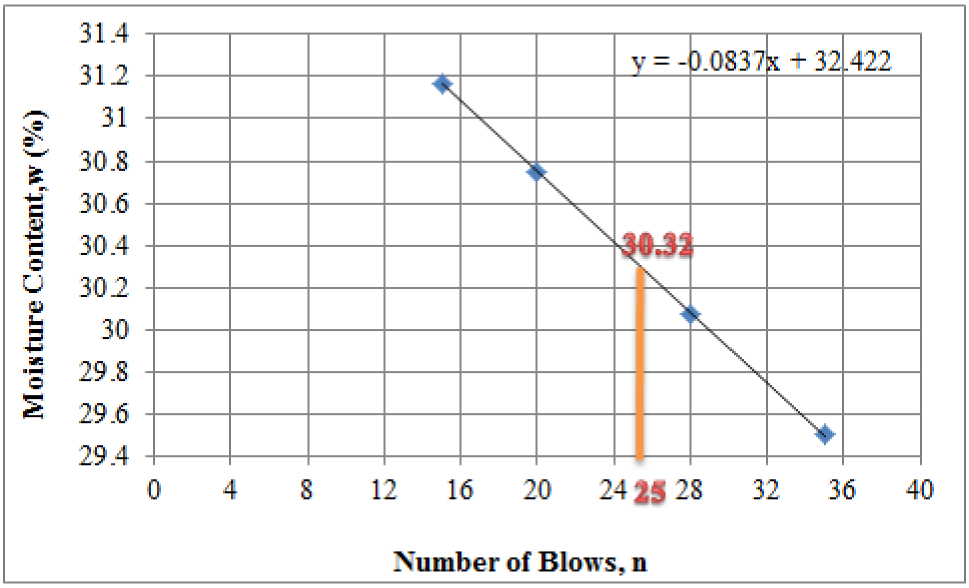
Sieve	Ferricrete	%	concrete	%	Comb	Mid	Spec	Spec
(mm)	PSD	50.0	PSD	50.0	100.0	Point	Min	Max
37.5	100.0	50.0	100.0	50.0	100.0			
26.50	100.0	50.0	100.0	50.0	100.0	100.0	100	100
19.00	93.6	46.8	93.7	46.9	93.7	97.5	95	100
13.20	82.0	41.0	80.3	40.2	81.2	0.0		
9.50	70.2	35.1	67.9	34.0	69.0	70.0	60	80
6.70	58.5	29.2	56.7	28.3	57.6	0.0		
4.75	48.8	24.4	47.2	23.6	48.0	50.0	40	60
2.36	37.5	18.7	36.2	18.1	36.8	37.5	30	45
1.180	26.4	13.2	29.0	14.5	27.7	27.5	20	35
0.600	20.4	10.2	22.3	11.1	21.4	20.0	13	27
0.425	16.8	8.4	18.8	9.4	17.8	17.0	11	23
0.300	13.4	6.7	12.5	6.2	12.9	14.0	8	20
0.150	9.17	4.6	7.66	3.8	8.4	9.50	5	14
0.075	5.03	2.5	4.09	2.0	4.6	7.00	3	11
0.0135	0.0	0.0	0.0	0.0	0.0	0.00	0	1

Appendix C: Liquid Limit test results

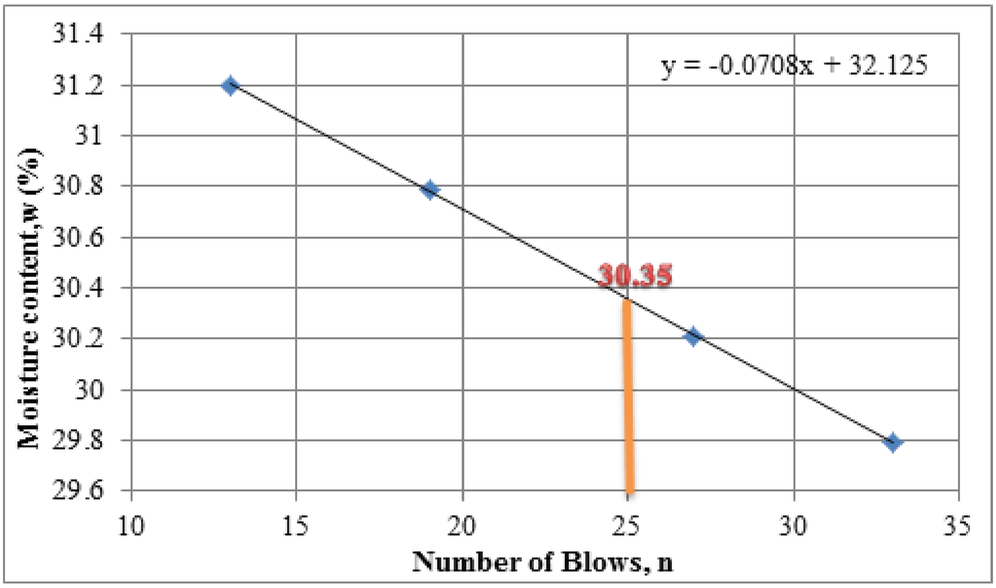
Appendix C1: Casagrande test method

100% Crushed brick and tile

Sample number: 1	7	4	1	14
Mc=Mass of empty, clean can+ lid (grams)	56.62	58.22	55.08	59.63
MCMS=Mass of can, lid , and moist soil (grams)	97	106.36	96.66	108.06
MCDS=Mass of can, lid, and dry soil (grams)	87.8	95.23	86.88	96.55
Ms=Mass of soil solids (grams)	31.18	37.01	31.8	36.92
Mw=Mass of pore water (grams)	9.20	11.13	9.78	11.51
W=Water content, w%	29.5	30.07	30.75	31.17
Number of drops (N)	35	28	20	15

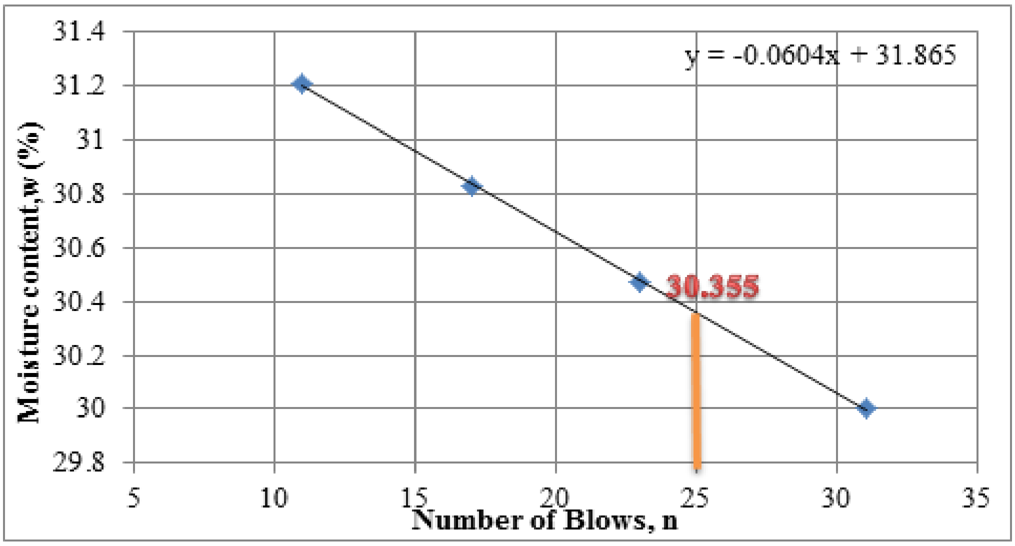


10% Crushed brick and tile with RCC



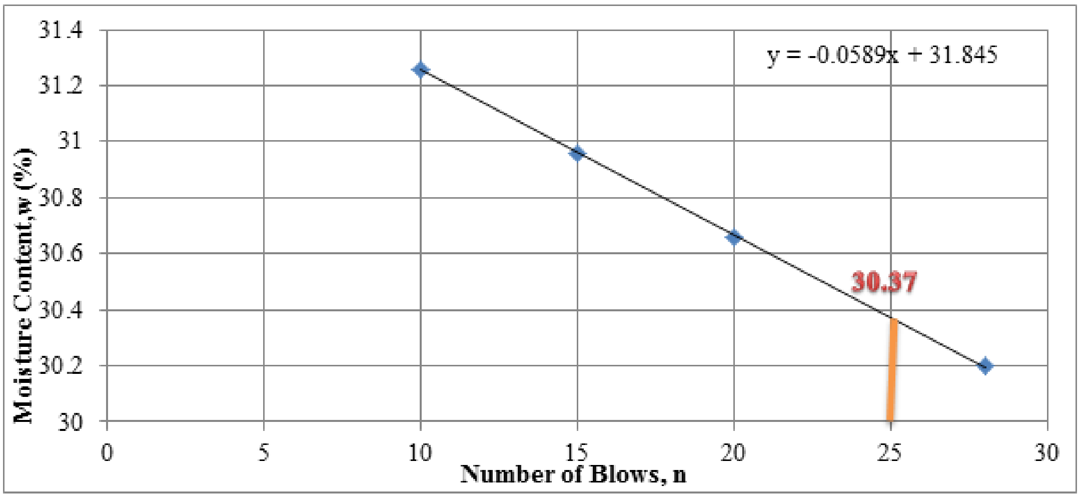
Sample number: 2	7	4	1	14
Mc=Mass of empty, clean can+ lid(grams)	56.62	58.23	54.67	59.64
MCMS=Mass of can, lid , and moist soil (grams)	85.68	80.432	78.5	86.76
MCDS=Mass of can, lid, and dry soil (grams)	79.01	75.28	72.89	80.31
Ms=Mass of soil solids (grams)	22.39	17.05	18.22	20.67
Mw=Mass of pore water (grams)	6.67	5.152	5.61	6.45
W=Water content, w%	29.79	30.21	30.79	31.20
Number of drops (N)	33	27	19	13

20% Crushed brick and tile with RCC



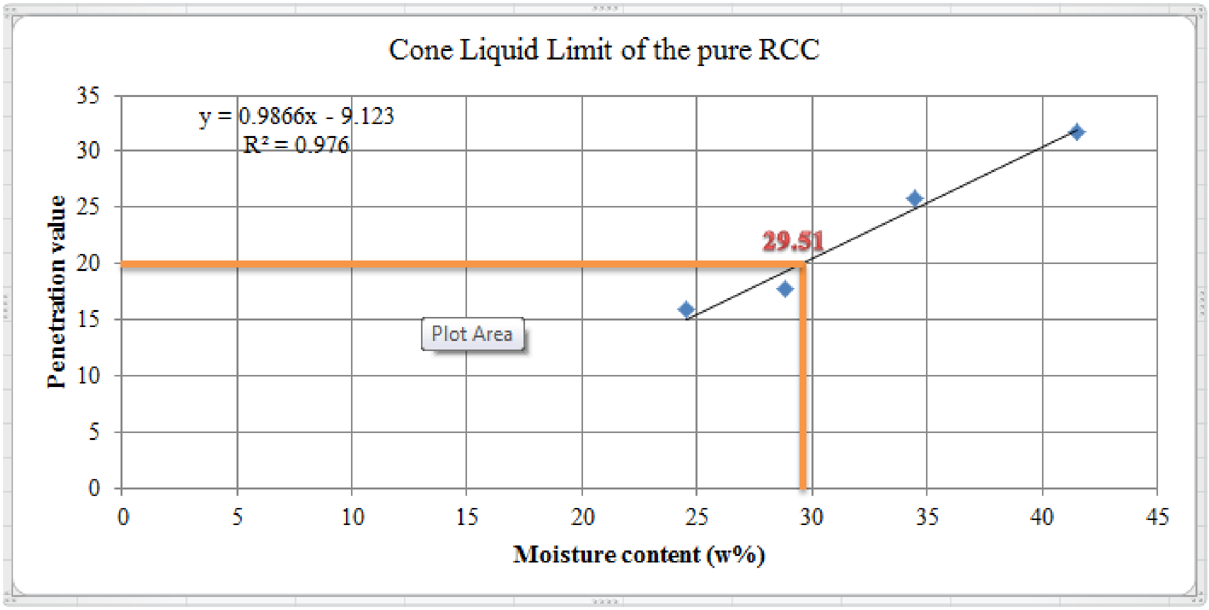
Sample number: 3	3	6	13	15
Mc=Mass of empty, clean can+ lid (grams)	54.18	61.99	60.38	62.50
MCMS=Mass of can, lid , and moist soil (grams)	80.388	87.903	90.957	93.61
MCDS=Mass of can, lid, and dry soil (grams)	74.34	81.85	83.75	86.21
Ms=Mass of soil solids (grams)	20.16	19.86	23.37	23.71
Mw=Mass of pore water (grams)	6.048	6.053	7.207	7.4
W=Water content, w%	30	30.47	30.83	31.21
Number of drops (N)	31	23	16	11

30% Crushed brick and tile with RCC



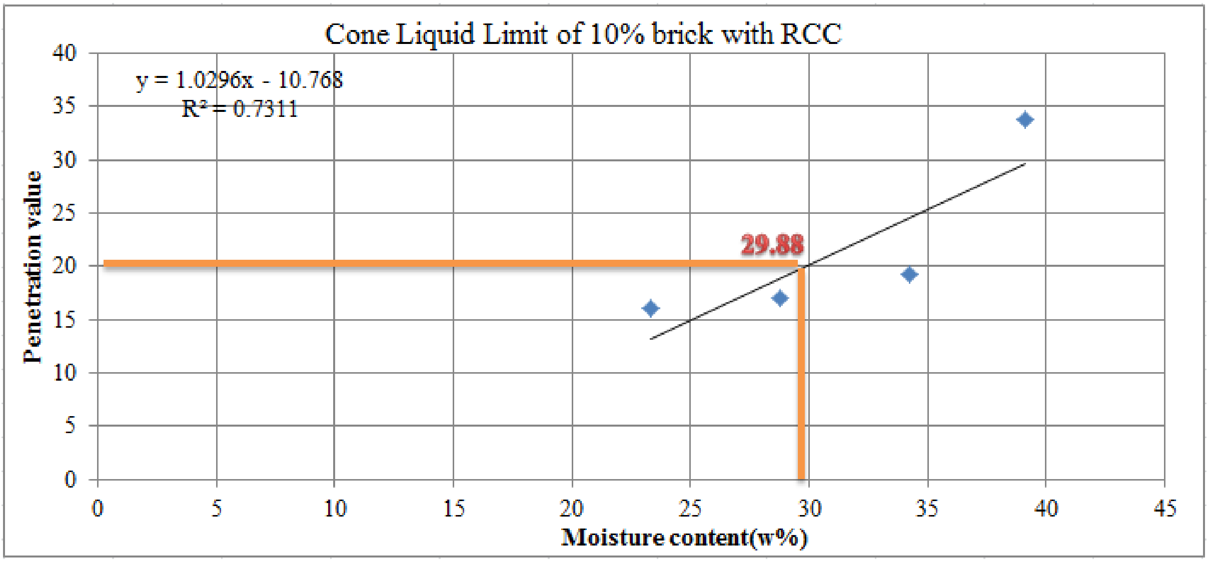
Sample number: 4	2	5	10	21
Mc=Mass of empty, clean can+ lid (grams)	57.76	51.36	58.46	59.56
MCMS=Mass of can, lid , and moist soil (grams)	85.115	82.27	96.36	89.12
MCDS=Mass of can, lid, and dry soil (grams)	78.77	75.02	87.40	82.08
Ms=Mass of soil solids (grams)	21.01	23.66	28.94	22.52
Mw=Mass of pore water (grams)	6.345	7.256	8.96	7.04
W=Water content, w%	30.20	30.66	30.96	31.26
Number of drops (N)	28	20	15	9

Appendix C2: Cone penetrometer test method



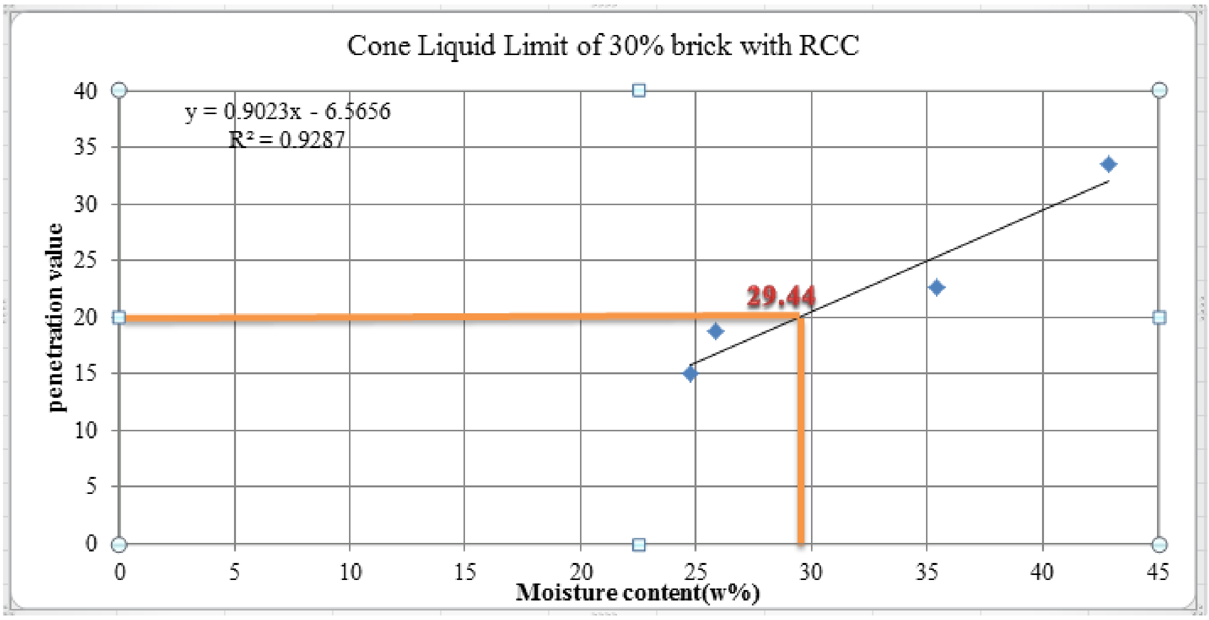
Liquid Limit	1	2	3	4
First Gauge: (mm)	15.4	18	25	31.2
Second	16.4	17.5	26.5	32.2
Average	15.9	17.75	25.75	31.7
Con number	4	22	14	3
Mass of con (g)	61.24	59.3	59.71	54.24
wet soil+ con (g)	76.1	72.62	88.5	90.01
dry soil +con (g)	73.17	69.64	81.12	79.52
Mass of water (g)	62.5	75	87.5	105
water content (W%)	24.55	28.82	34.46	41.49

10% Crushed brick and tile with RCC



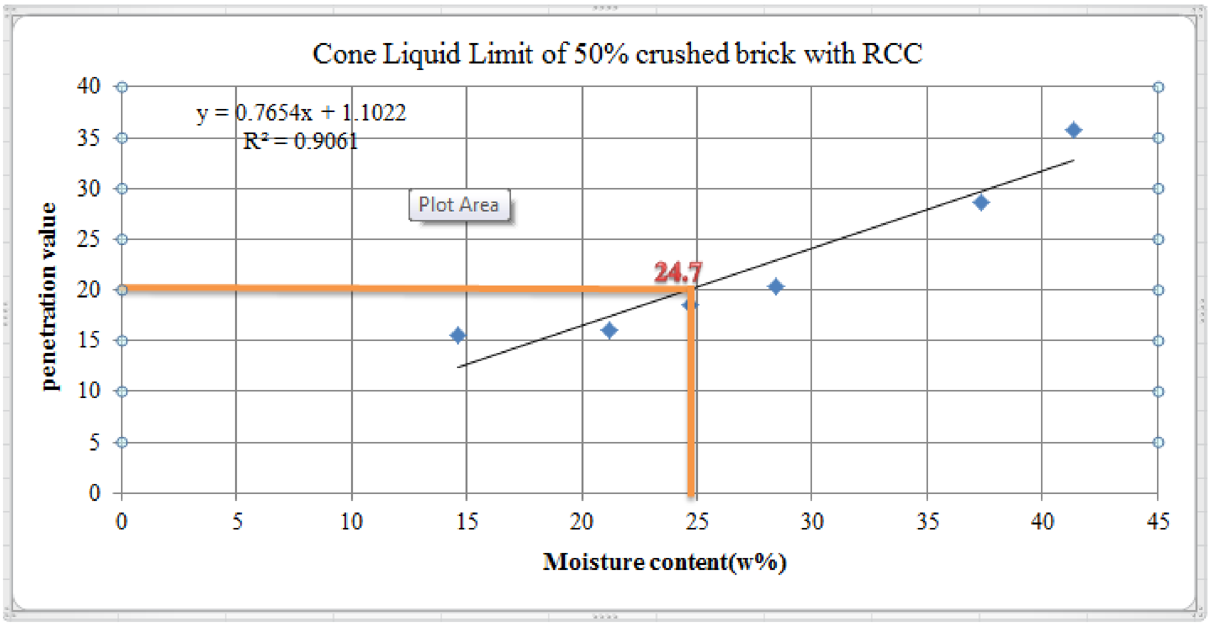
Liquid Limit test	1	2	3	4
First Gauge: (mm)	16.5	17.2	19.8	33.2
Second	15.5	17	18.8	34.2
Average	16	17.1	19.3	33.7
Con number	20	7	21	5
Mass of con (g)	58.33	56.69	59.59	51.44
wet soil+ con (g)	75.25	74.37	79.08	81.9
dry soil +con (g)	72.05	70.42	74.11	73.33
Mass of water (g)	62.5	75	87.5	100
Water content (W %)	23.32	28.76	34.22	39.15

30% Crushed brick and tile with RCC



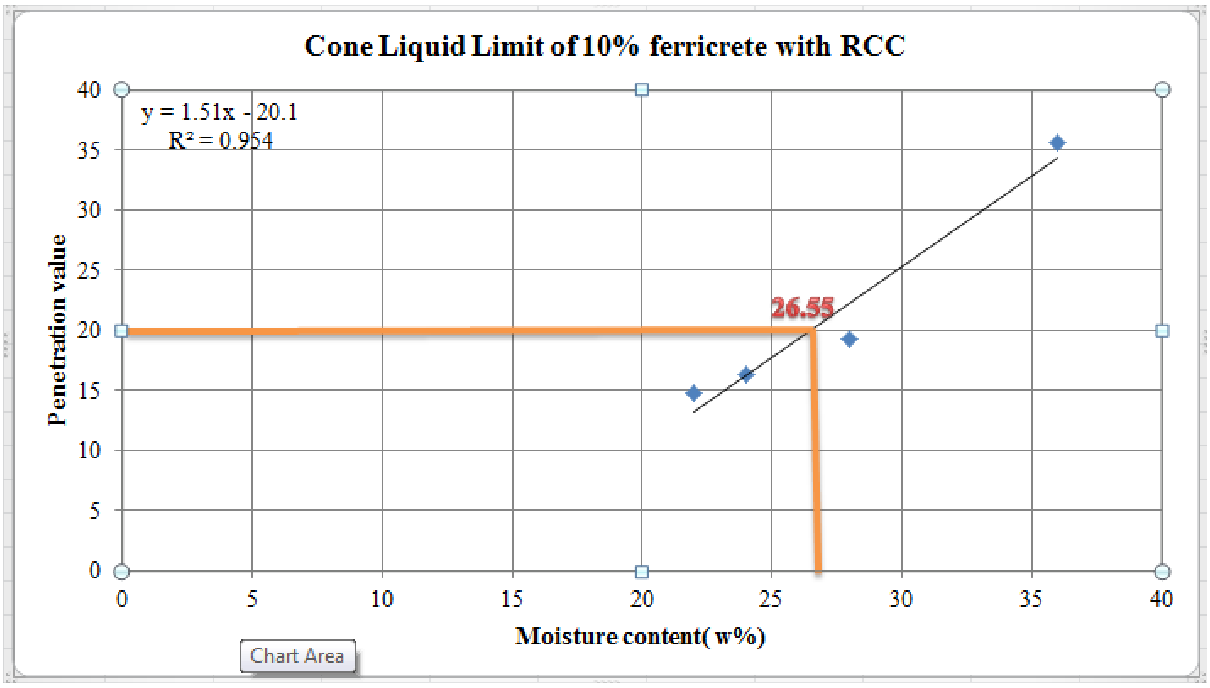
Liquid Limit	1	2	3	4
First Gauge: (mm)	14.5	18	22.4	33.5
Second	15.5	19.5	23	33.6
Average	15	18.75	22.7	33.55
Con number	16	5	11	5
Mass of con (g)	60.87	54.33	54.6	33.88
wet soil+ con (g)	78.17	74.03	76.71	69.45
dry soil +con (g)	74.74	69.98	70.93	58.78
Mass of water (g)	62.5	75	87.5	100
Water content	24.72	25.87	35.39	42.85

50% Crushed brick and tile with RCC



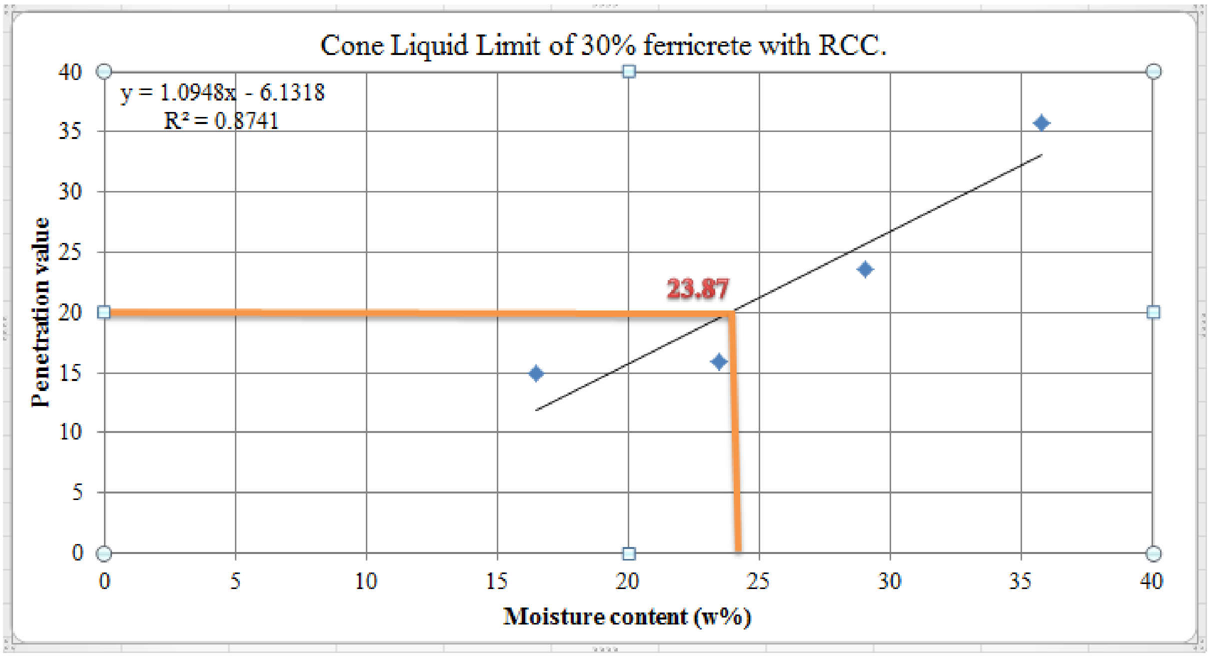
Liquid Limit	1	2	3
First Gauge: (mm)	20.9	28.7	36
Second	19.9	28.6	35.5
Average	20.4	28.65	35.75
Con number	D	E	F
Mass of con (g)	121.65	13.19	19.67
wet soil+ con (g)	145.21	46.83	57.73
dry soil +con (g)	140	37.68	46.6
Mass of water (g)	75	87.5	100
water content(W%)	28.39	37.36	41.32

10% Ferricrete with RCC



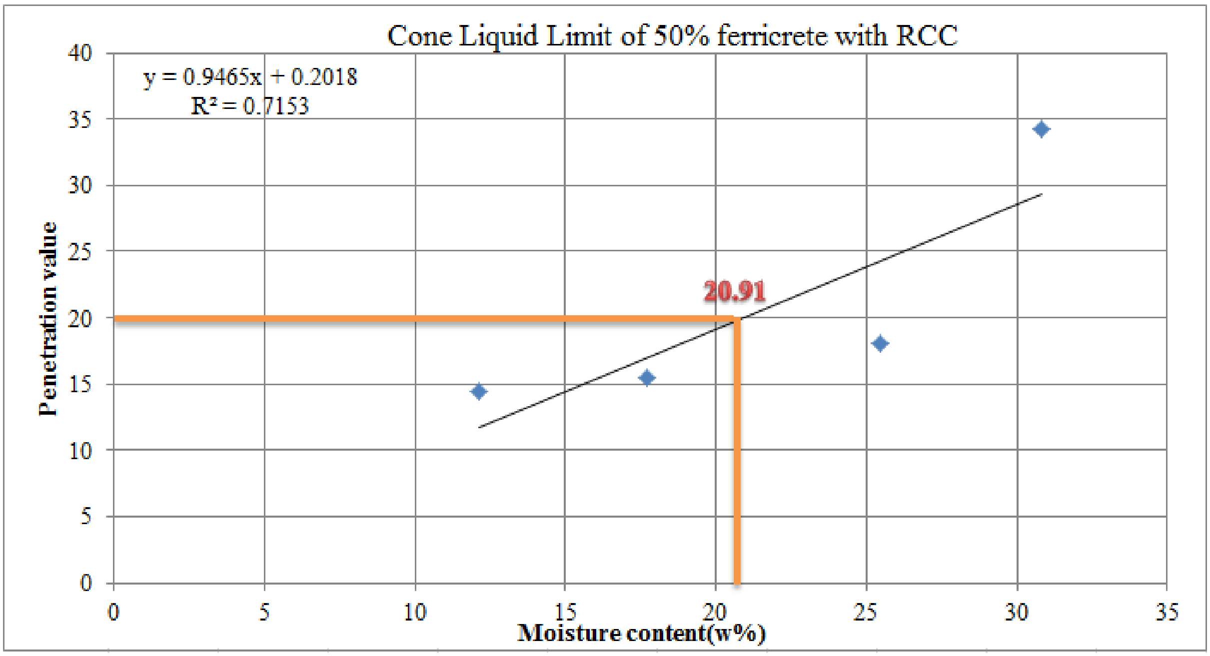
Liquid Limit	1	2	3	4
First Gauge: (mm)	14.2	16	18.7	35.4
Second	15.2	16.5	19.8	35.6
Average	14.7	16.25	19.25	35.5
Con number	H	I	J	K
Mass of con (g)	13.02	14.41	19.81	49.49
wet soil+ con (g)	28.55	28.19	37.66	93.49
dry soil +con (g)	25.77	25.51	33.75	81.8
Mass of water (g)	50	62.5	75	87.5
water content(W%)	21.8	24.14	28.04	36.18

30% Ferricrete with RCC



Liquid Limit	1	2	3	4
First Gauge: (mm)	14.5	15.7	23.1	36
Second	15.5	16	24	35.5
Average	15	15.85	23.55	35.75
Con number	M	N	O	P
Mass of con (g)	15.99	50.75	49.02	48.34
wet soil+ con (g)	29.84	73.66	77.97	78.77
dry soil +con (g)	27.88	69.31	71.45	70.75
Mass of water (g)	50	62.5	75	87.5
water content (W%)	16.48	23.43	29.06	35.78

50% Ferricrete with RCC

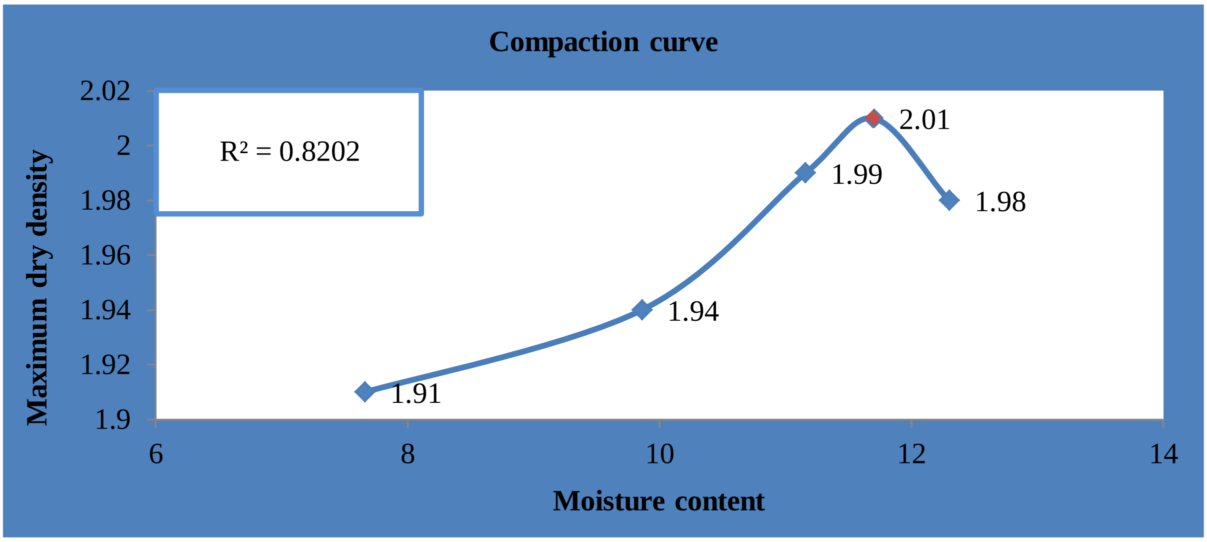


Liquid Limit	1	2	3	4
First Gauge: (mm)	14	15	18	34
Second	15	16	18.1	34.5
Average	14.5	15.5	18.05	34.25
Con number	Q	R	S	T
Mass of con (g)	58.62	15.88	53.12	51.96
wet soil+ con (g)	74.79	33.36	83.63	102.35
dry soil +con (g)	73.04	30.73	77.44	90.48
Mass of water (g)	37.5	50	62.5	0.3
water content(W%)	12.13	17.71	25.45	30.81

Appendix D: Modified compaction tests

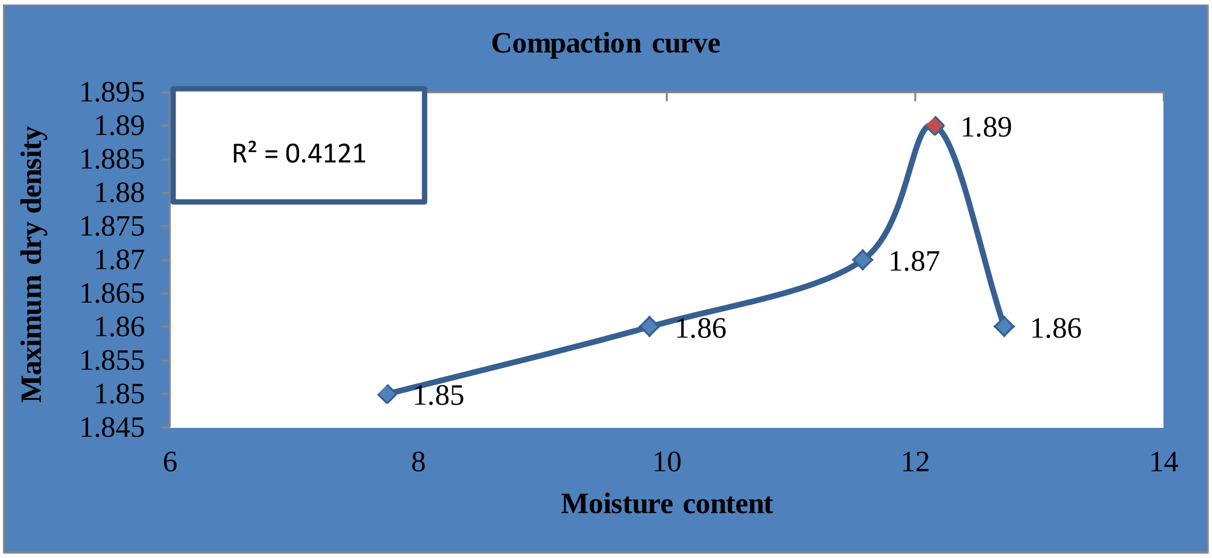
Pure prior RCC (rejected batch)

Assumed water content % / W1=4539 gr	8%	10%	12%	14%	16%
W2	6602	6675	6754	6795	6772
W3	246	241	247	831	829
W4	2297	2368	2448	3075	3019
W5	2151	2177	2227	2840	2779
w (%)	7.66	9.86	11.16	11.70	12.30
γ	2.06	2.13	2.21	2.25	2.23
γ_d	1.91	1.94	1.99	2.01	1.98



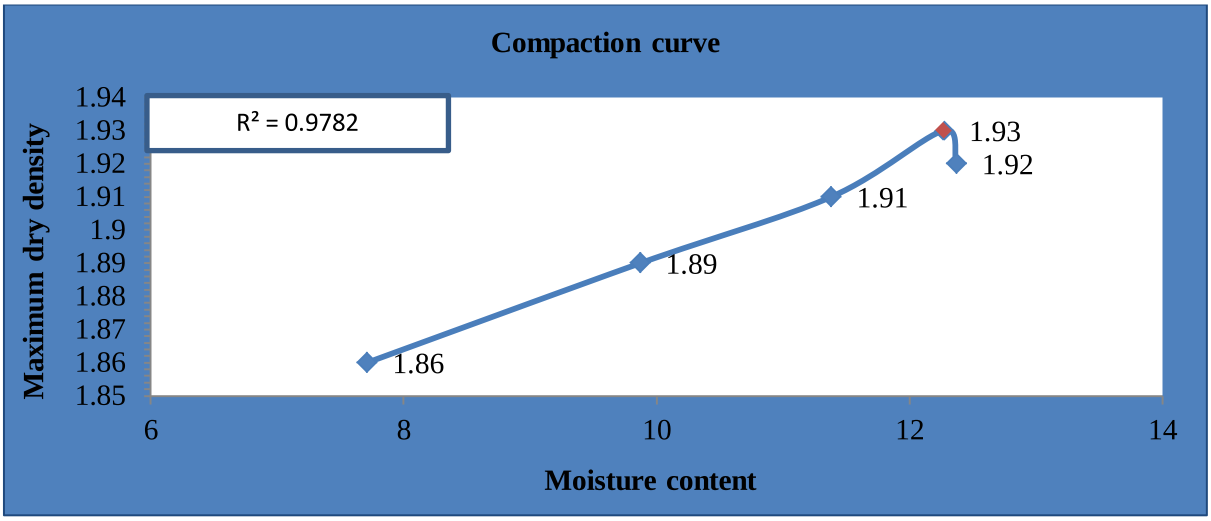
100% RCC (Trial batch)

Assumed water content % / W1=4539 gr	8%	10%	12%	14%	16%
W2	6529	6578	6632	6657	6638
W3	244	246	269	243	247
W4	2234	2285	2359	2354	2338
W5	2091	2102	2142	2125	2102
W (%)	7.75	9.86	11.58	12.16	12.72
γ	1.99	2.04	2.09	2.12	2.10
γd	1.85	1.86	1.87	1.89	1.86



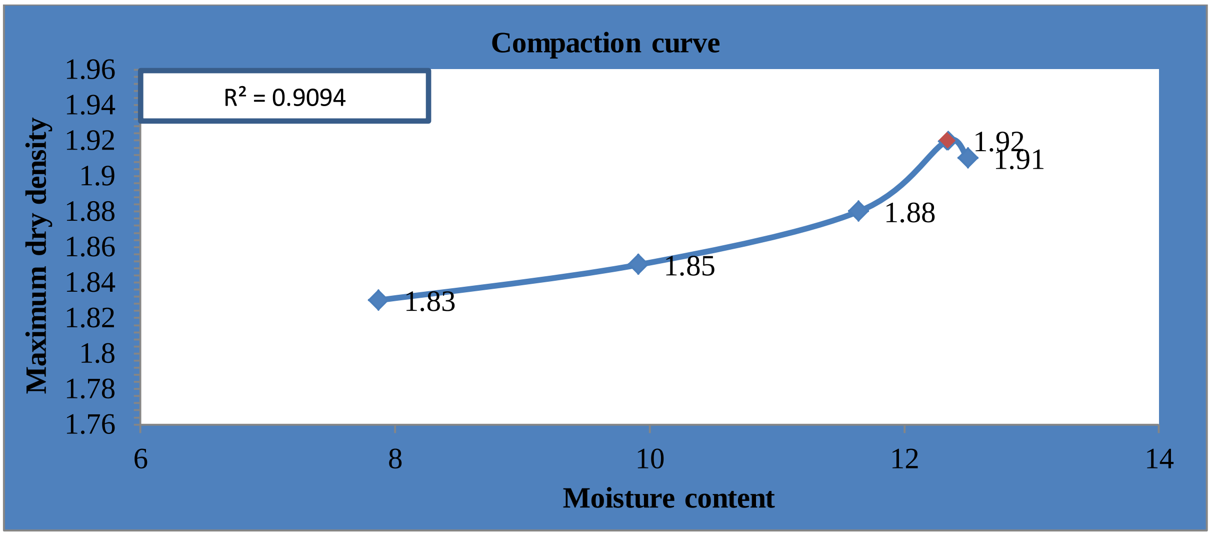
10% Crushed brick with RCC

Assumed water content %.	8%	10%	12%	14%	16%
W1=4537 gr					
W2	6554	6614	6667	6715	6700
W3	243	245	317	269	314
W4	2268	2292	2410	2418	2467
W5	2123	2108	2196	2183	2230
w (%)	7.71	9.87	11.38	12.27	12.37
γ	2.01	2.07	2.13	2.17	2.16
γ_d	1.86	1.89	1.91	1.93	1.92



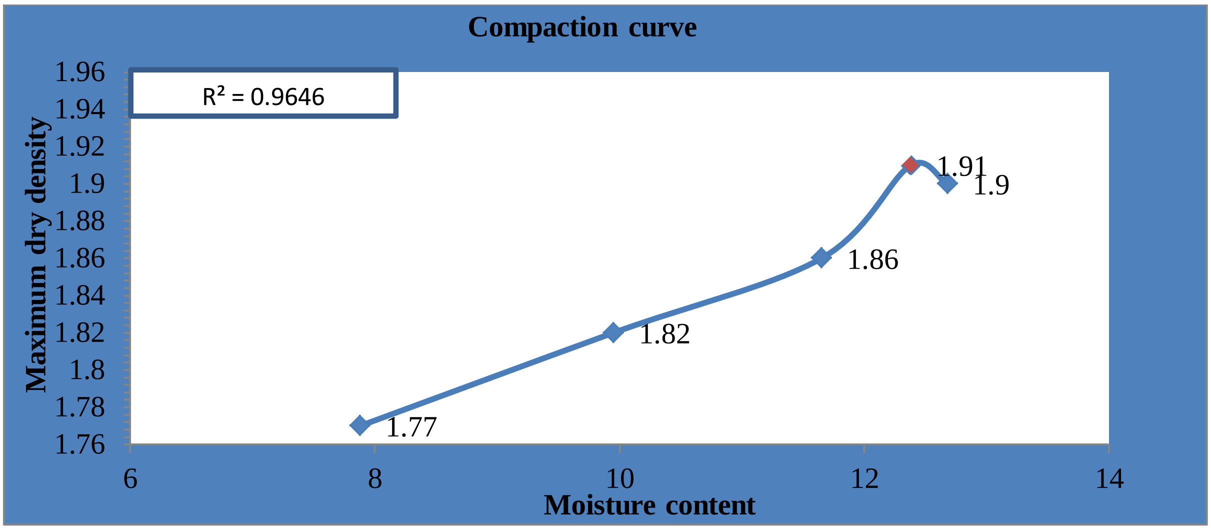
20% Crushed brick with RCC

Assumed water content %. W1=4537 gr	8%	10%	12%	14%	16%
W2	6517	6577	6643	6705	6694
W3	746	748	748	750	750
W4	2718	2699	2848	2897	2900
W5	2574	2523	2629	2661	2661
w (%)	7.87	9.91	11.64	12.34	12.50
γ	1.98	2.04	2.1	2.16	2.15
γ_d	1.83	1.85	1.88	1.92	1.91



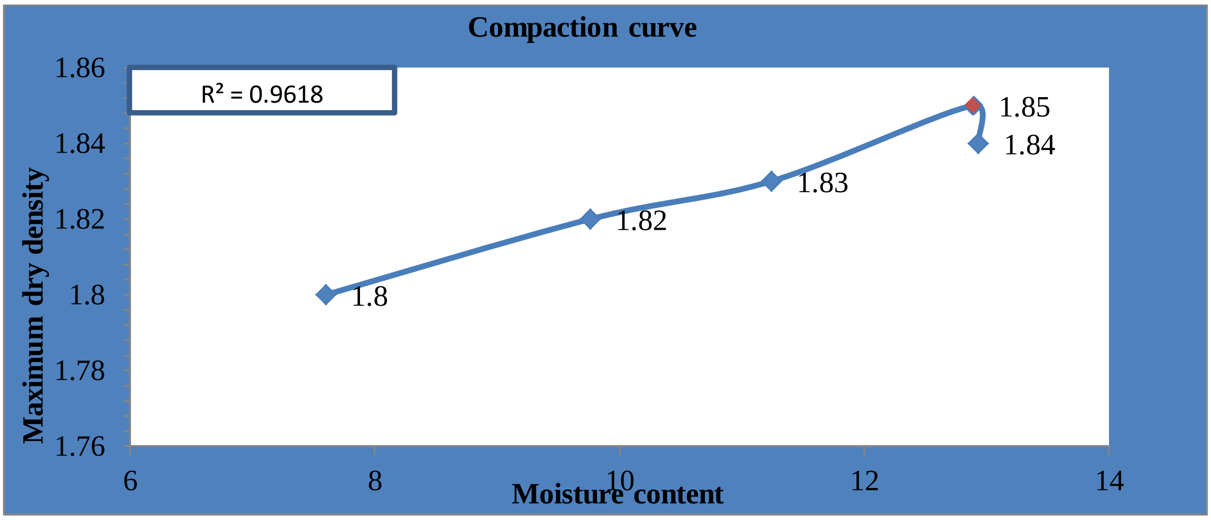
30% Crushed brick with RCC

Assumed water content %.	8%	10%	12%	14%	16%
W1=4537 gr					
W2	6453	6537	6621	6690	6680
W3	831	835	831	831	246
W4	2745	2834	2862	2972	2370
W5	2606	2653	2650	2736	2131
w (%)	7.83	9.95	11.65	12.38	12.68
γ	1.91	2.0	2.08	2.15	2.14
$γ_d$	1.77	1.82	1.86	1.91	1.9



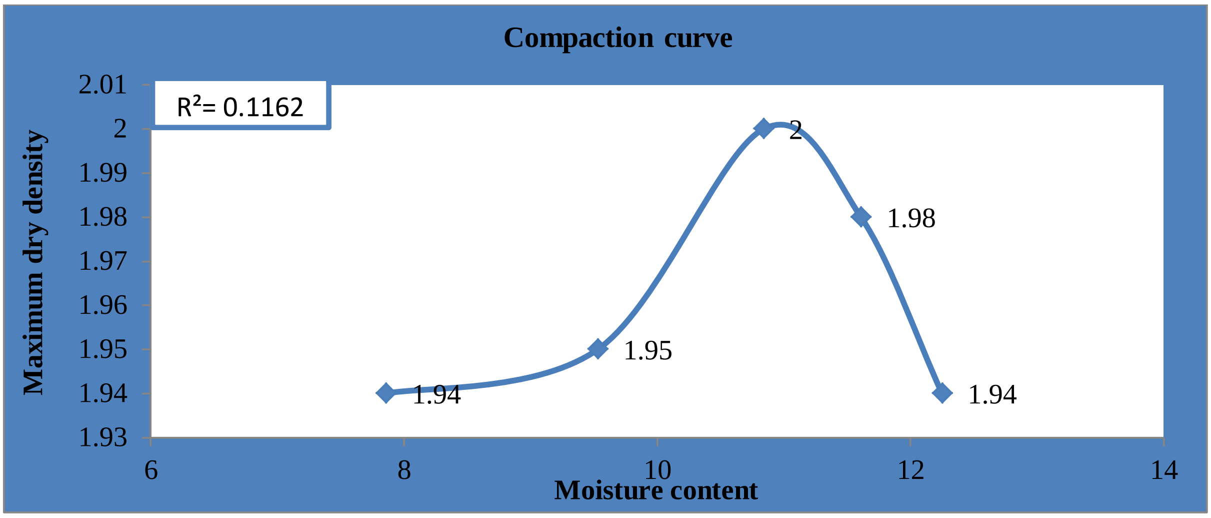
50% Crushed brick with RCC

Assumed water content %.	8%	10%	12%	14%	16%
W1=4539 gr					
W2	6480	6539	6585	6631	6620
W3	246	270	240	266	244
W4	2186	2215	2239	2350	2322
W5	2049	2042	2037	2112	2084
w (%)	7.60	9.76	11.24	12.89	12.93
γ	1.94	2	2.04	2.09	2.08
$γ_d$	1.80	1.82	1.83	1.85	1.84



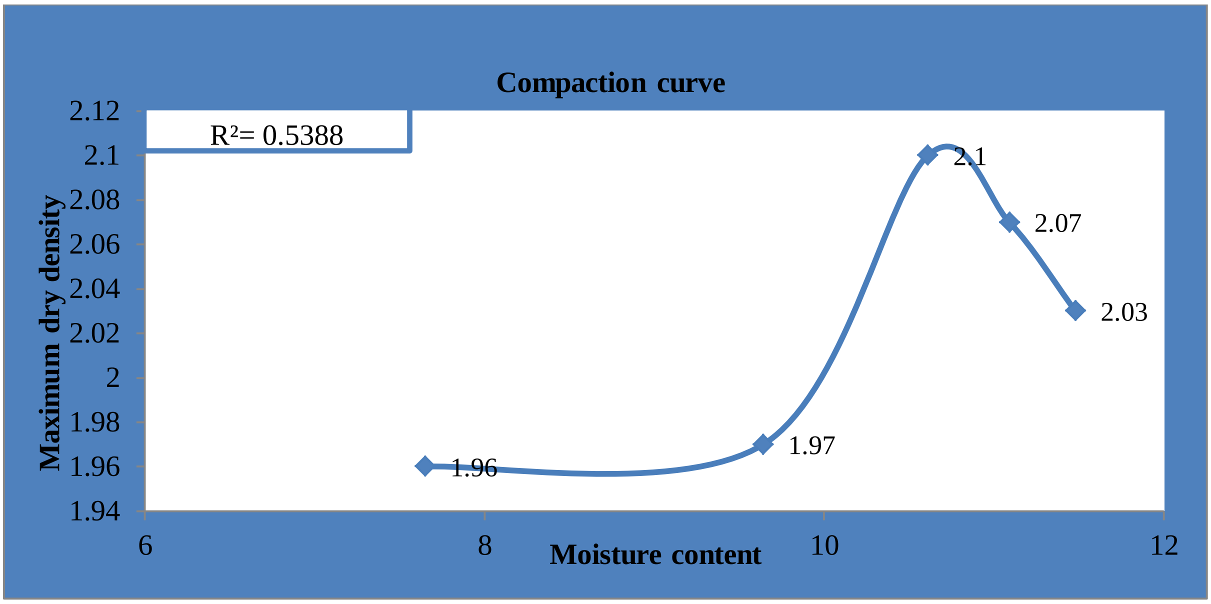
10% Ferricrete with RCC

Assumed water content %.	8%	10%	12%	14%	16%
W1=4527 gr					
W2	6633	6667	6754	6743	6715
W3	248	250	245	238	833
W4	2347	2387	2463	2420	3004
W5	2194	2201	2246	2193	2767
W (%)	7.86	9.53	10.84	11.61	12.25
γ	2.10	2.14	2.22	2.21	2.18
$γ_d$	1.94	1.95	2.00	1.98	1.94



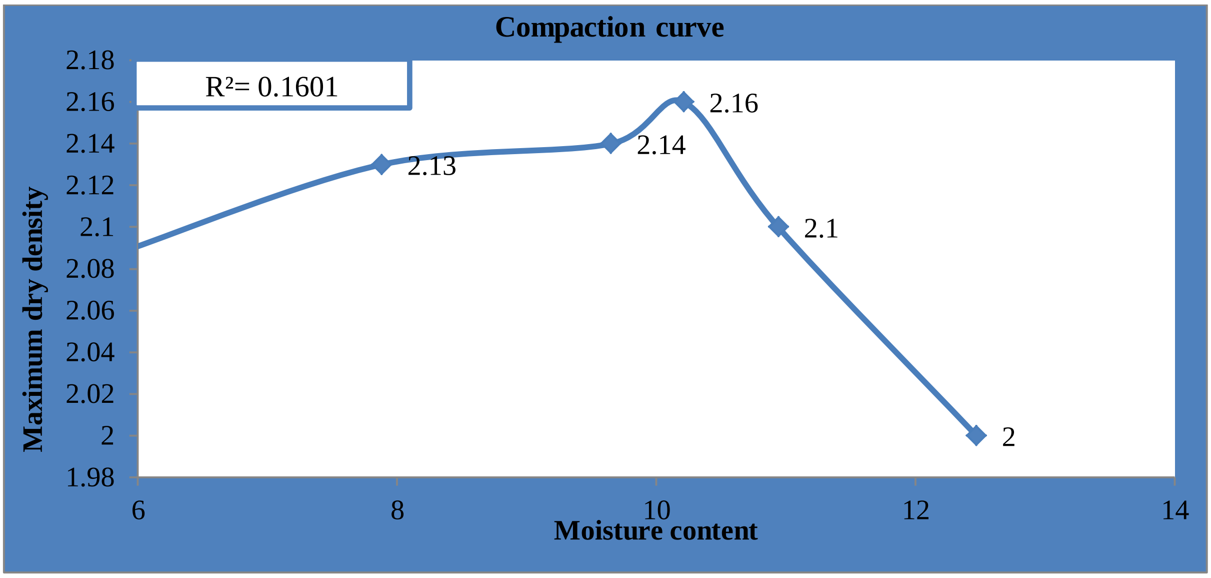
30% Ferricrete with RCC

Assumed water content %.	8%	10%	12%	14%	16%
W1=4527 gr					
W2	6649	6690	6852	6831	6800
W3	831	830	245	250	833
W4	2940	2990	2558	2534	3056
W5	2790	2800	2336	2306	2827
W (%)	7.65	9.64	10.61	11.09	11.48
γ	2.12	2.16	2.325	2.3	2.27
γ_d	1.96	1.97	2.10	2.07	2.03



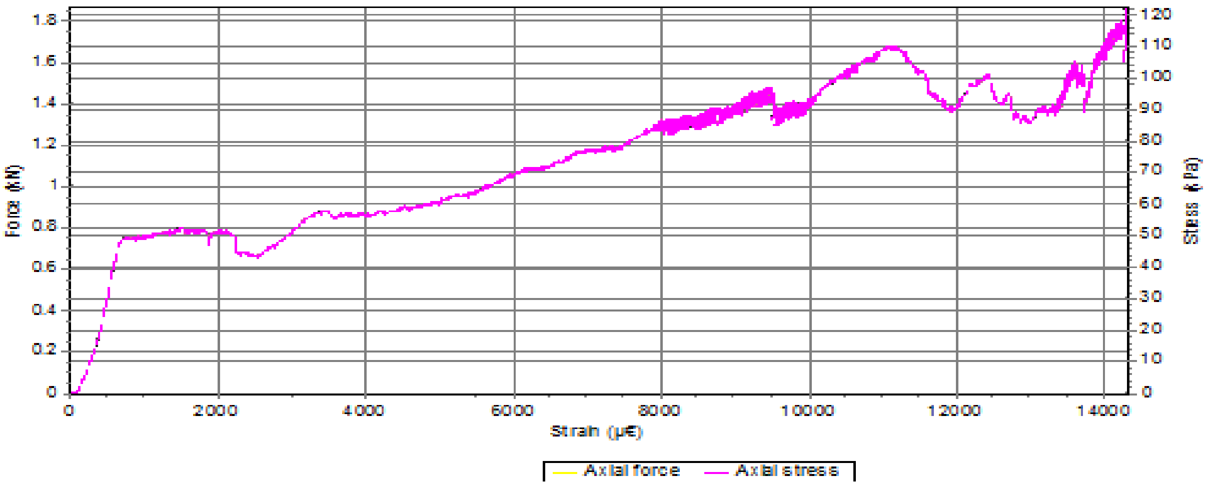
50% Ferricrete with RCC

Assumed W1=5085 gr	6%	8%	10%	12%	14%	16%
W2	7300	7393	7434	7471	7424	7336
W3	268	830	245	831	250	252
W4	2480.5	3155	2585	3204	2572	2470
W5	2356	2985	2379	2984	2343	2224
W (%)	5.96	7.88	9.65	10.21	10.94	12.47
γ	2.215	2.30	2.35	2.386	2.34	2.251
γ_d	2.09	2.13	2.14	2.16	2.10	2

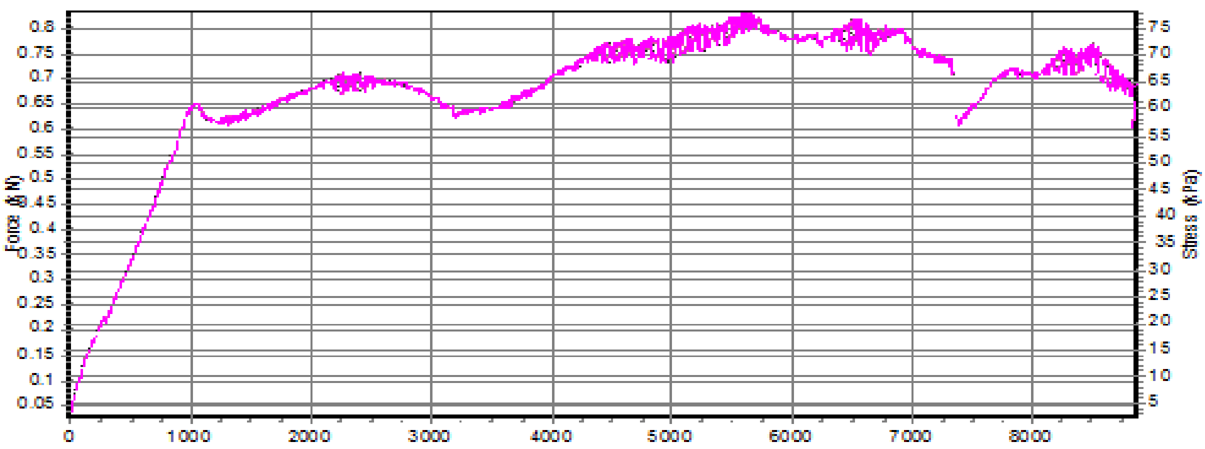


Appendix E: Modified bending beam test results

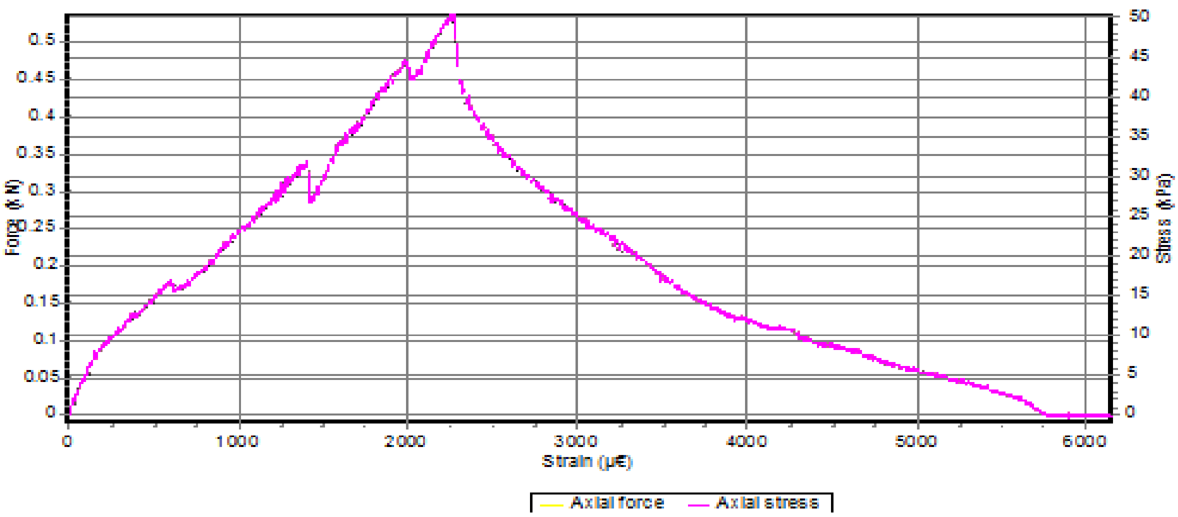
Modified bending beam 2 with support beam (compaction condition 24)



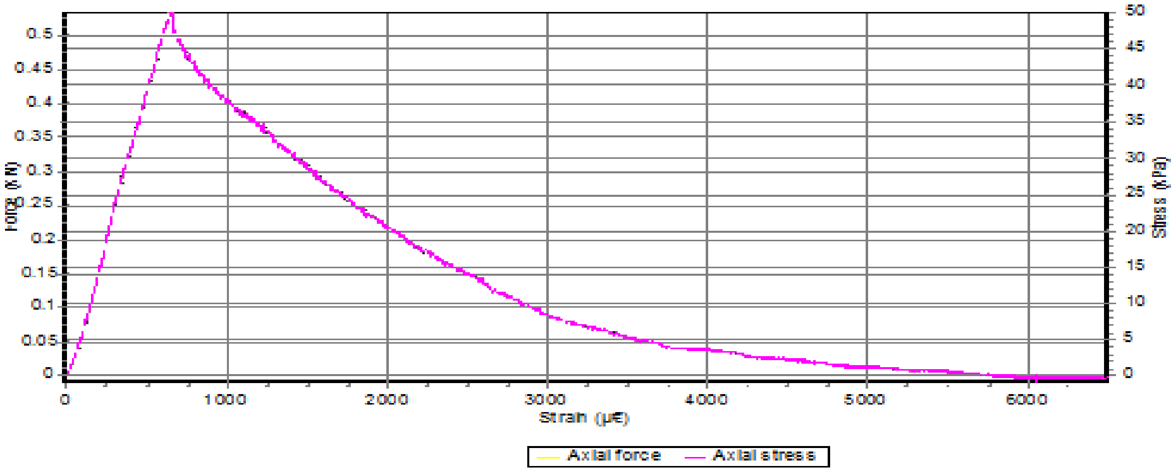
Modified bending beam 4 with support beam (compaction condition 24)



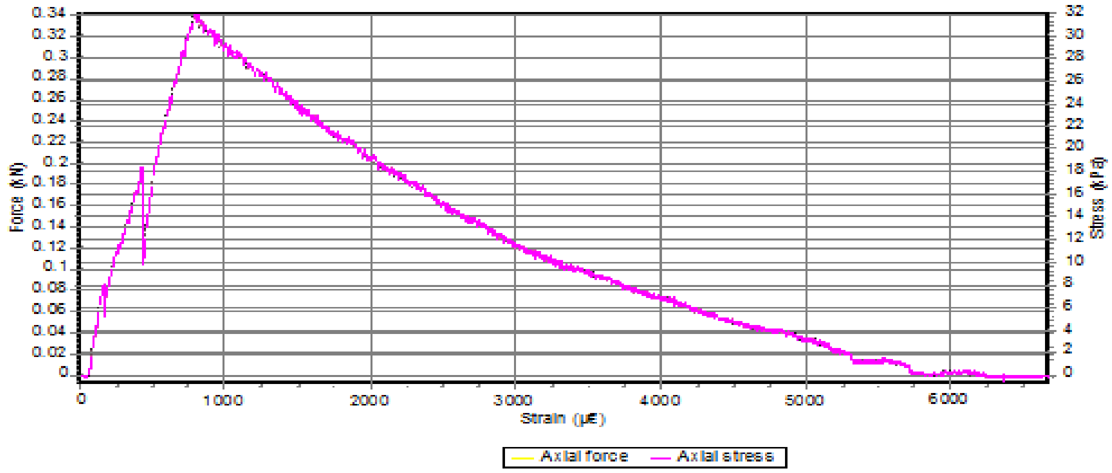
Modified bending beam 5 with roller (compaction condition 0)



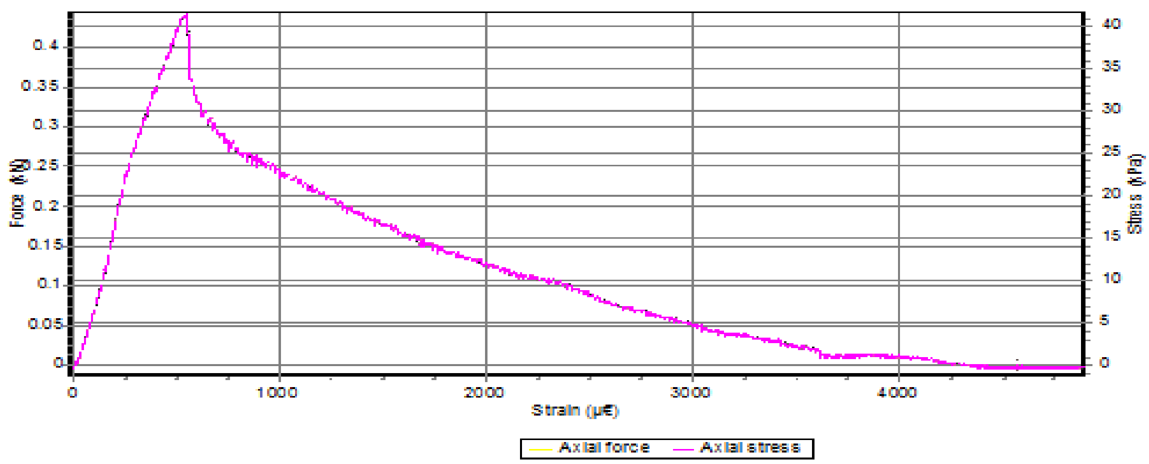
Modified bending beam 6 with roller (compaction condition 0)



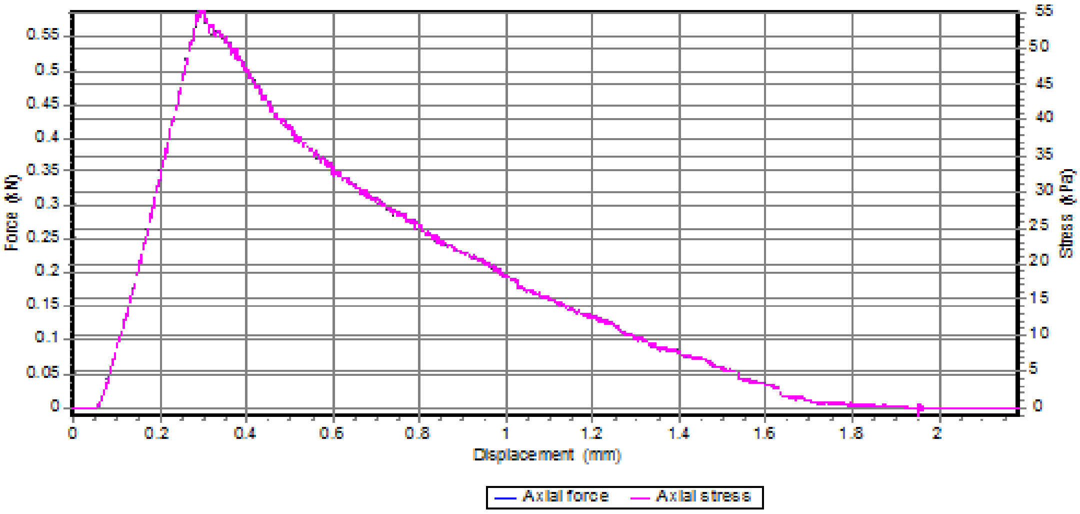
Modified bending beam 9 with roller (compaction condition 0)



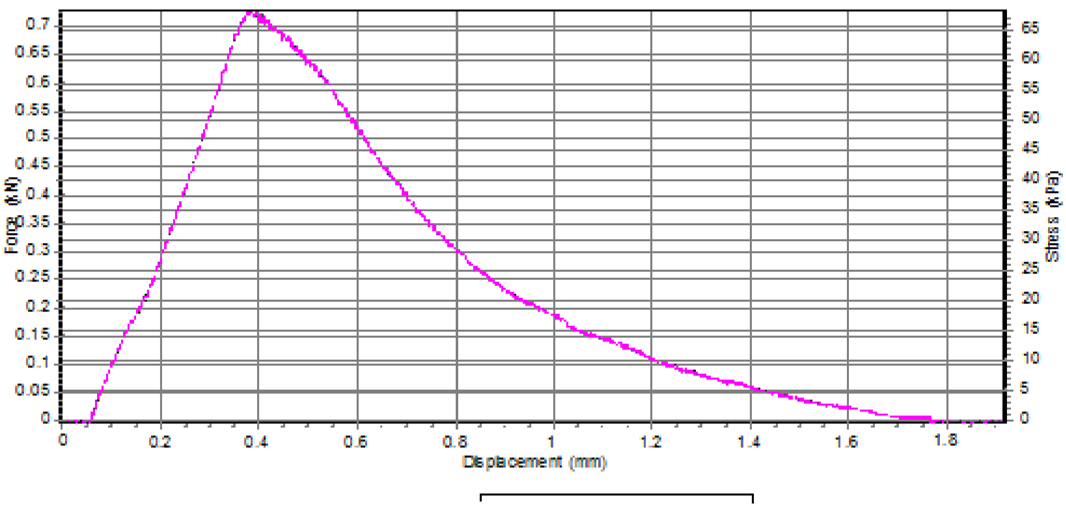
Modified bending beam 10 with roller (compaction condition 0)



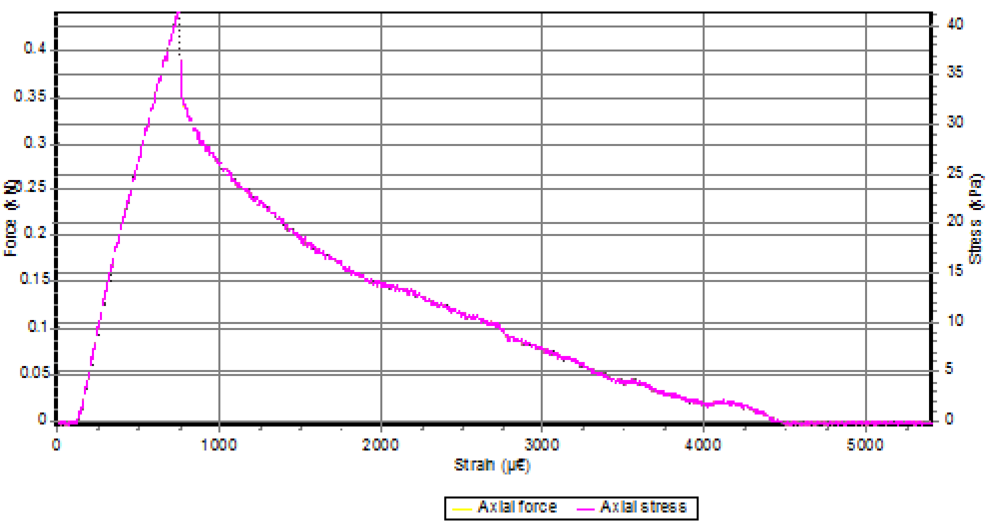
Modified bending beam 17 with roller (compaction condition 0)



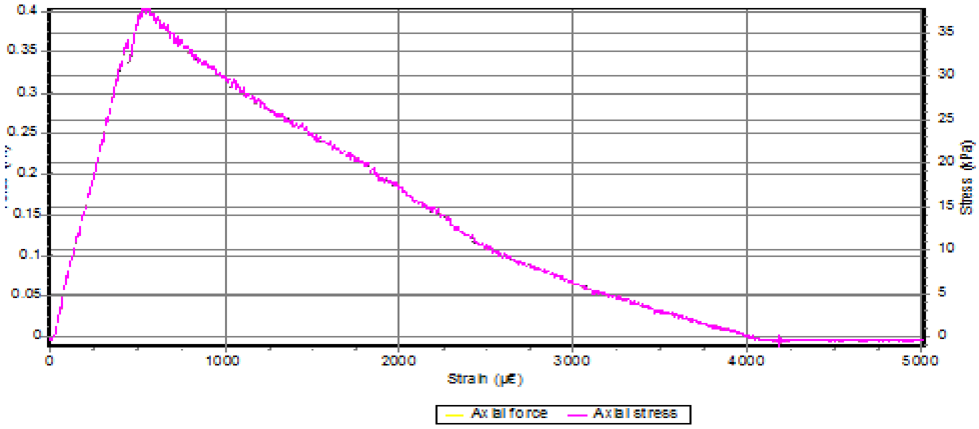
Modified bending beam 18 with roller (compaction condition 0)



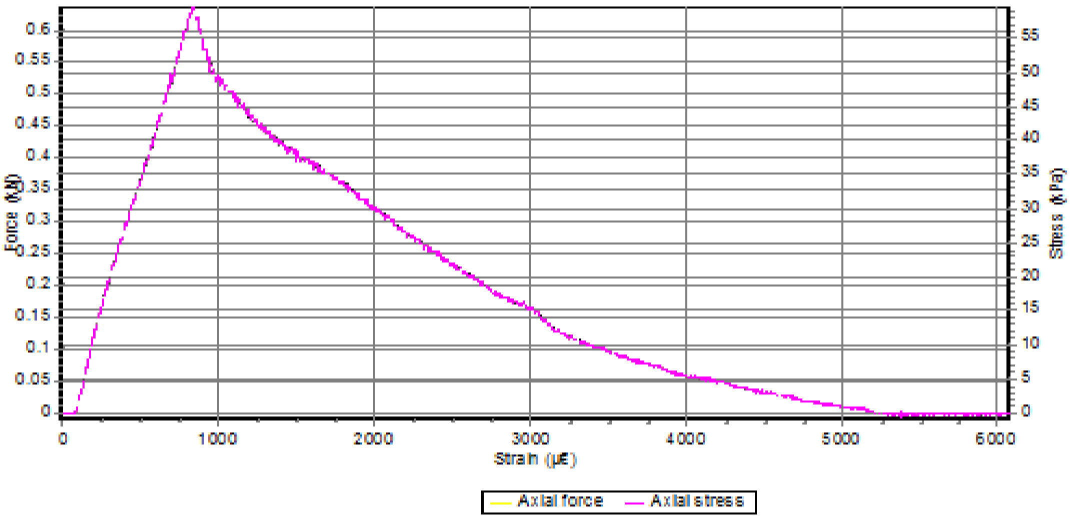
Modified bending beam A with roller (compaction condition 0)



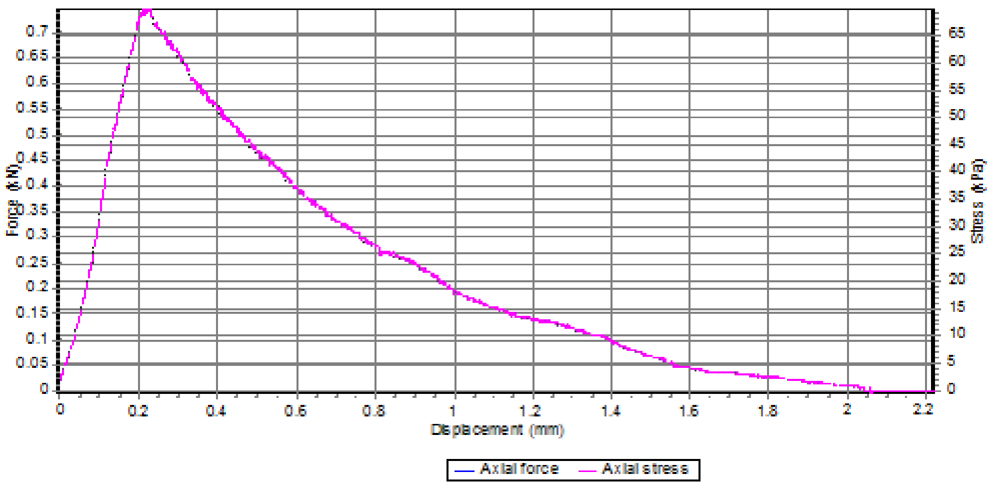
Modified bending beam B with roller (compaction condition 0)



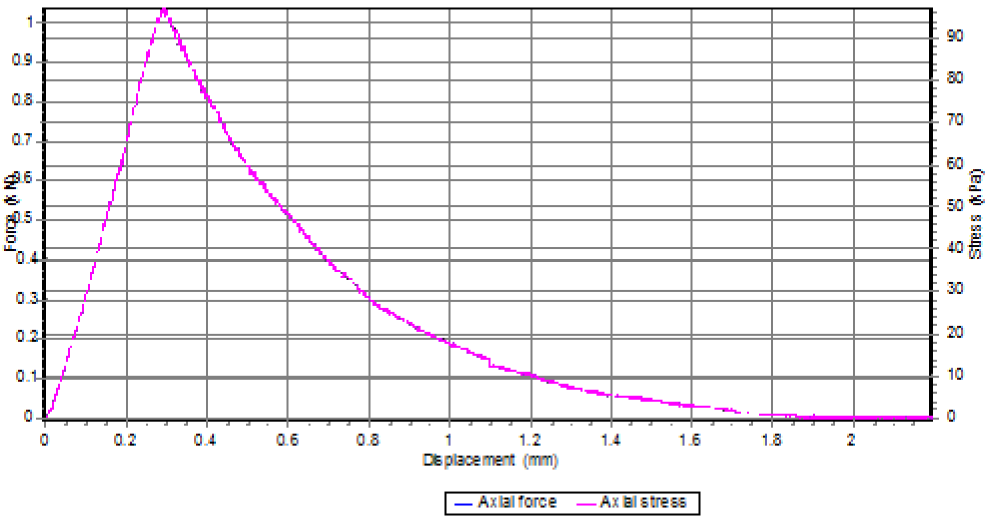
Modified bending beam 8 with roller (compaction condition 24)



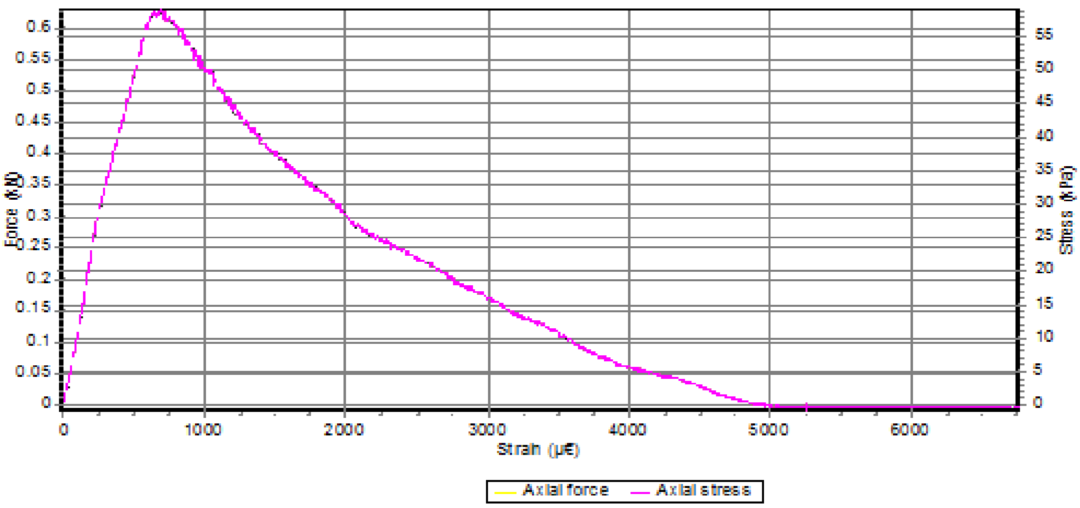
Modified bending beam 15 with roller (compaction condition 24)



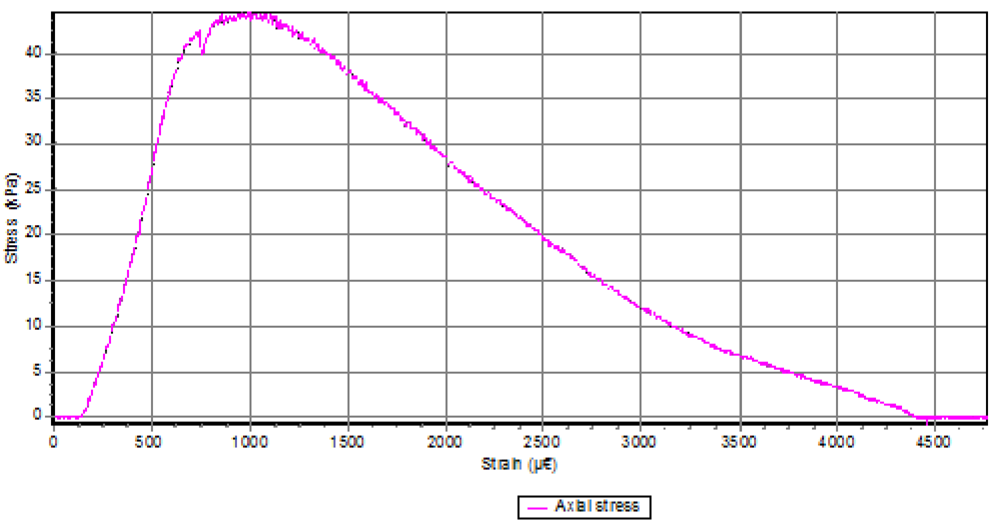
Modified bending beam 16 with roller (compaction condition 24)



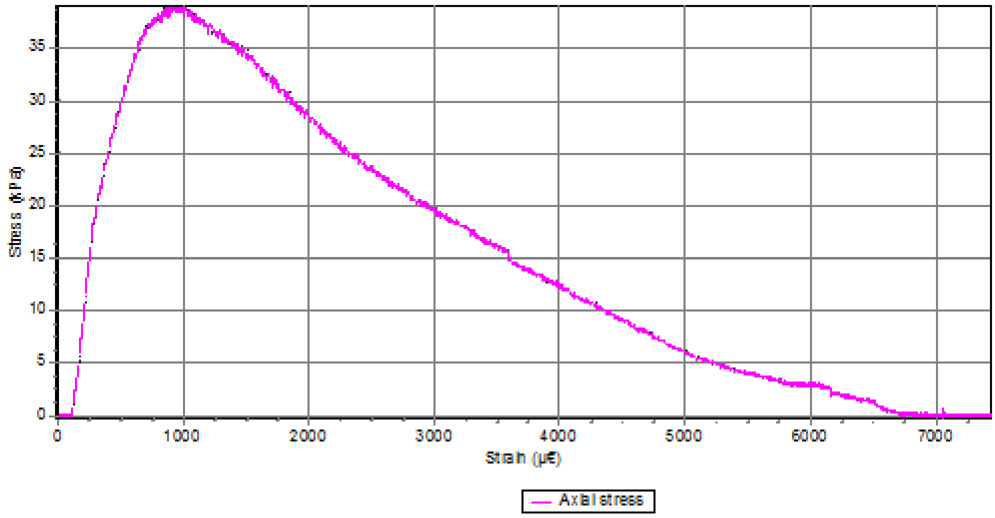
Appendix 45. Modified bending beam 7 with roller (compaction condition 48)



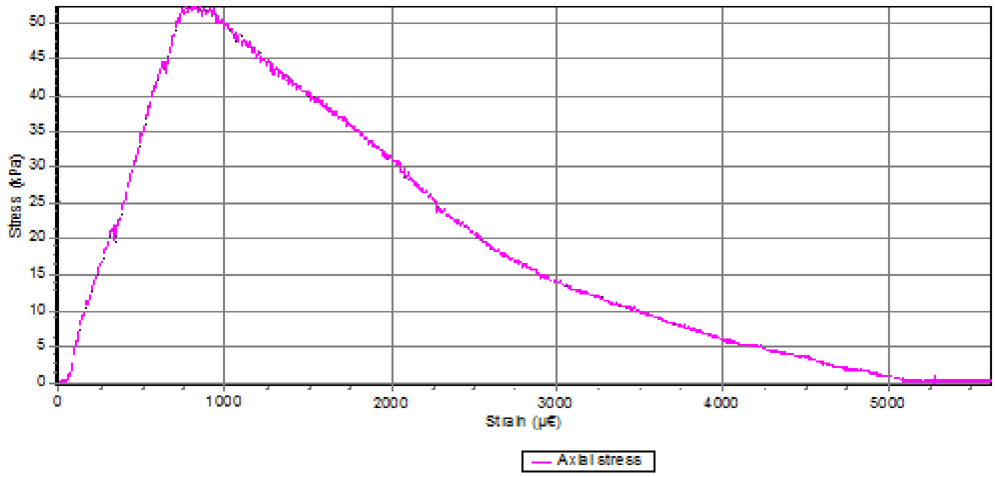
Appendix 46. Modified bending beam 11 with roller (compaction condition 48)



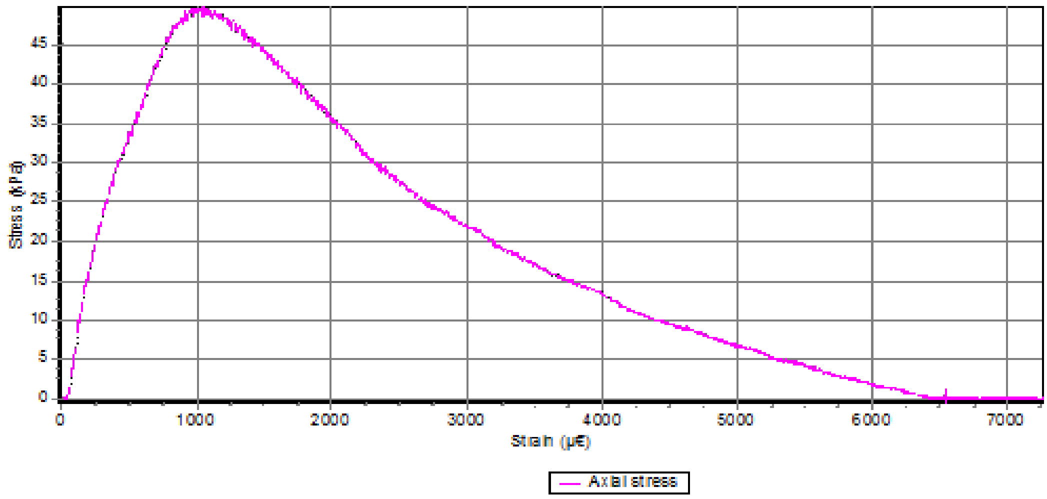
Appendix 47. Modified bending beam 12 with roller (compaction condition 48)



Modified bending beam 13 with roller (compaction condition 48)



Modified bending beam 14 with roller (compaction condition 48)



Appendix F. UCS and modulus elasticity results

Appendix F1: Data summary

Blends of brick and tile with RCC-Cores cured 56 days

Material	E(Mpa)		UCS(Mpa)	
	Average	SD	Average	SD
100% RCC	39.35	7.53	0.774	0.093
10% Brick and tile	31.64	5.73	0.70	0.072
30% Brick and tile	44.90	12.25	0.90	0.146
50%Brick and tile	31.40	11.99	0.778	0.177

Blends of ferricrete with RCC-Cores cured 56 days

Material	E(Mpa)		UCS(Mpa)	
	Average	SD	Average	SD
100% RCC	39.35	7.53	0.774	0.093
10% Ferricrete	39.38	16.47	0.871	0.292
30% Ferricrete	44.25	13.63	0.90	0.205
50%Ferricrete	33.45	3.81	0.768	0.076

UCS result of pure RCC cores

RCC Samples		100%RCC		56days curing	
Code	UCS(MPa)	Maximum axial force(KN)	Axial strain,EA(%)	Deviator stress,SD(KPa)	E_v (MPa)
M	0.87	7.475	0.035	878.25	49.20
N	0.7	6.068	0.042	709.23	28.25
O	0.86	7.379	0.037	864.01	40.06
k	0.66	5.68	0.036	665.75	37.86
J	0.78	6.65	0.136	779.84	41.38

UCS result of mixture 10%crushed brick& tile with RCC

Sample		curing 56 days	10% Brick& Tile		
Code	Ev(Mpa)	Deviator stress, SD (kpa)	Axial strain, EA (%)	Maximum axial force(KN)	UCS(Mpa)
S	33.40	763.26	0.04	6.505	0.76
T	31.75	666.74	0.039	5.677	0.66
U	25.40	675.04	0.046	5.758	0.67
P	36.94	702.76	0.024	5.98	0.70
Y	31.30	687.15	0.037	5.848	0.68
R	40.99	763.39	0.036	6.527	0.76
Q	30.05	714.21	0.04	6.101	0.71
Test1	23.30	529.45	0.042	4.489	0.53

UCS result of mixture 30%crushed brick& tile with RCC

Sample		curing 56 days	30% Brick& Tile		
Code	Ev(MPA)	Deviator stress, SD (kpa)	Axial strain, EA (%)	Maximum axial force(KN)	UCS(MPa)
F	52.45	1022.28	0.035	8.726	1.02
G	32.80	763.61	0.040	6.509	0.76
H	53.01	912.71	0.032	7.772	0.91
I	41.78	825.00	0.035	7.031	0.82
L	52.306	944.01	0.032	8.009	0.94
X	57.70	1076.52	0.032	9.189	1.075
6	24.82	660.44	0.051	5.668	0.66

UCS result of mixture 50%crushed brick& tile with RCC

Sample		curing 56 days	50% Brick& Tile		
Code	Ev(MPA)	Deviator stress ,SD (kpa)	Axial strain, EA (%)	Maximum axial force(KN)	UCS(MPa)
A	23.32	702.61	0.046	5.97	0.70
B	36.95	736.86	0.034	6.295	0.73
C	33.12	827.34	0.041	6.99	0.82
D	19.175	594.24	0.110	5.05	0.59
E	37.48	868.6	0.041	7.442	0.86
W	51.57	1087.1	0.034	9.28	1.08
7	18.33	568.7	0.049	4.82	0.56

UCS result of mixture 10% Ferricrete with RCC

Sample		curing 56 days	10% Ferricrete		
Code	Ev(MPA)	Deviator stress, SD (kpa)	Axial strain, EA (%)	Maximum axial force(KN)	UCS(MPA)
Z1	25.43	725.47	0.046	6.198	0.72
Z2	64.01	1339.23	0.038	11.412	1.33
Z3	50.31	1034.93	0.037	8.759	1.03
Z4	14.50	399.58	0.054	3.394	0.39
Z5	36.14	765.48	0.035	6.516	0.76
Z17	47.96	979.99	0.11	8.314	0.98
OO TEST	37.27	897.26	0.039	7.660	0.89

UCS result of mixture 30% Ferricrete with RCC

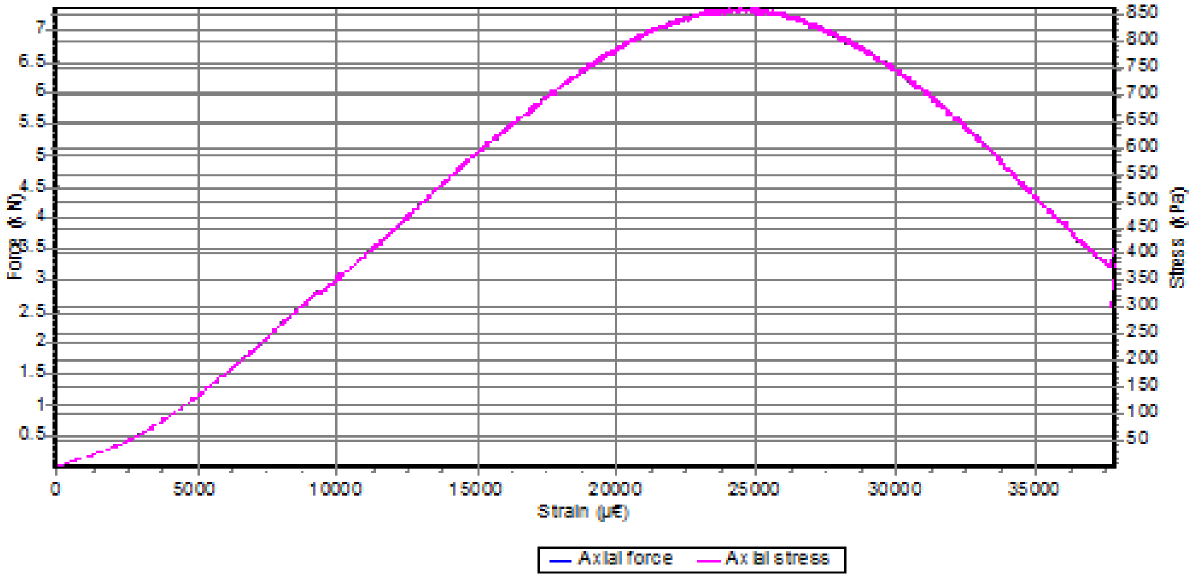
Sample		curing 56 days	30% Ferricrete		
Code	Ev(Mpa)	Deviator stress, SD (kpa)	Axial strain, EA (%)	Maximum axial force(KN)	UCS(MPA)
Z6	41.77	839.18	0.038	7.15	0.83
Z7	51.82	1005.95	0.034	8.595	1.005
Z8	55.25	1088.13	0.034	9.31	1.087
Z9	60.74	1060.77	0.031	8.967	1.061
Z10	43.94	1002.88	0.041	8.574	1.003
Z18	36.45	826.63	0.039	7.044	0.82
02 TEST	19.79	508.99	0.042	4.283	0.5

UCS result of mixture 50% Ferricrete with RCC

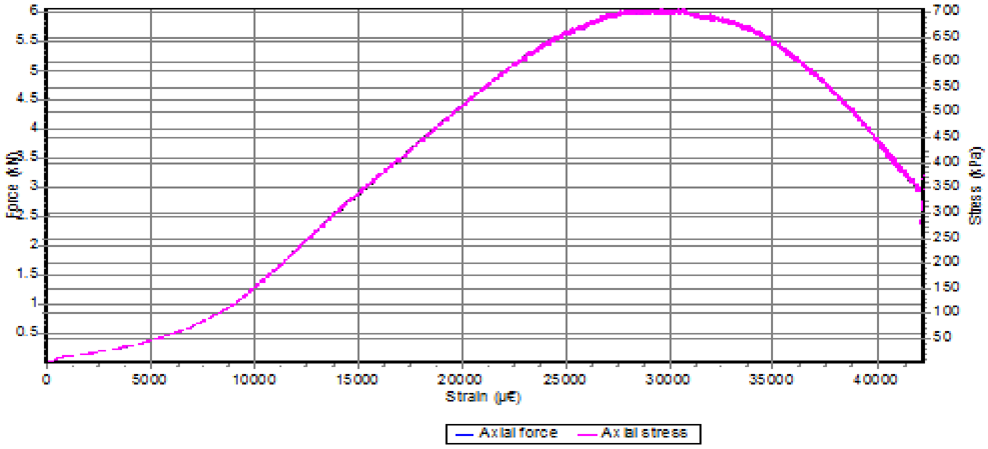
Sample		curing 56 days	50% Ferricrete		
Code	Ev(MPA)	Deviator stress, SD (kpa)	Axial strain, EA (%)	Maximum axial force(KN)	UCS(MPA)
Z11	35.79	786.67	0.044	6.702	0.78
Z12	35.57	814.47	0.046	6.934	0.81
Z13	35.31	847.21	0.044	7.23	0.84
Z14	32.69	707.16	0.047	6.048	0.7
Z15	26.02	654.29	0.049	5.630	0.65
Z16	35.35	837.75	0.042	7.139	0.83

Appendex F2: UCS test charts

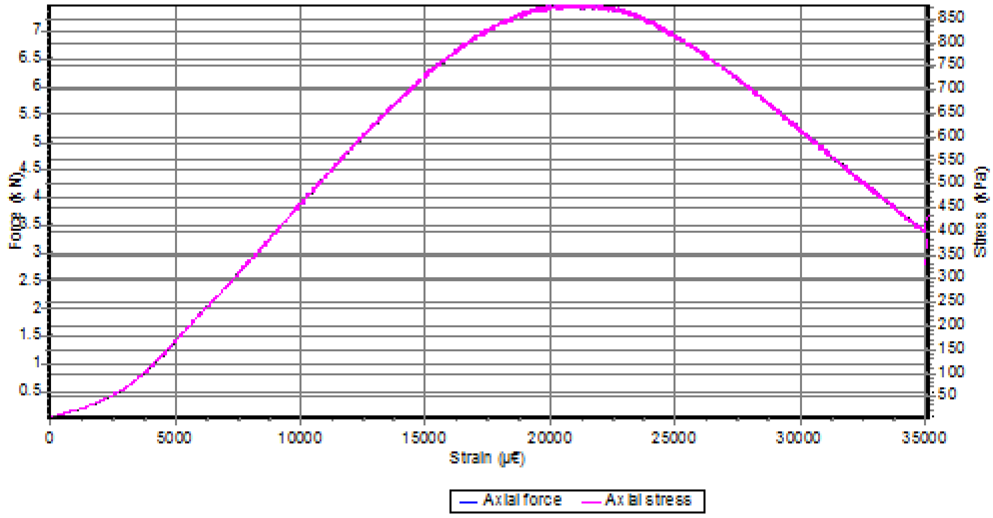
Unconfined compressive test (UCS) 100% recycled crushed concrete-core O



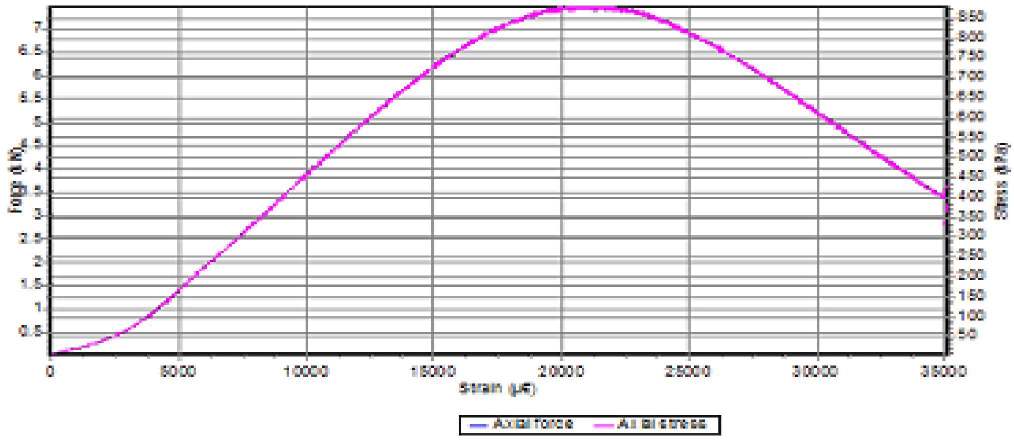
Unconfined compressive test (UCS) 100% recycled crushed concrete-core N



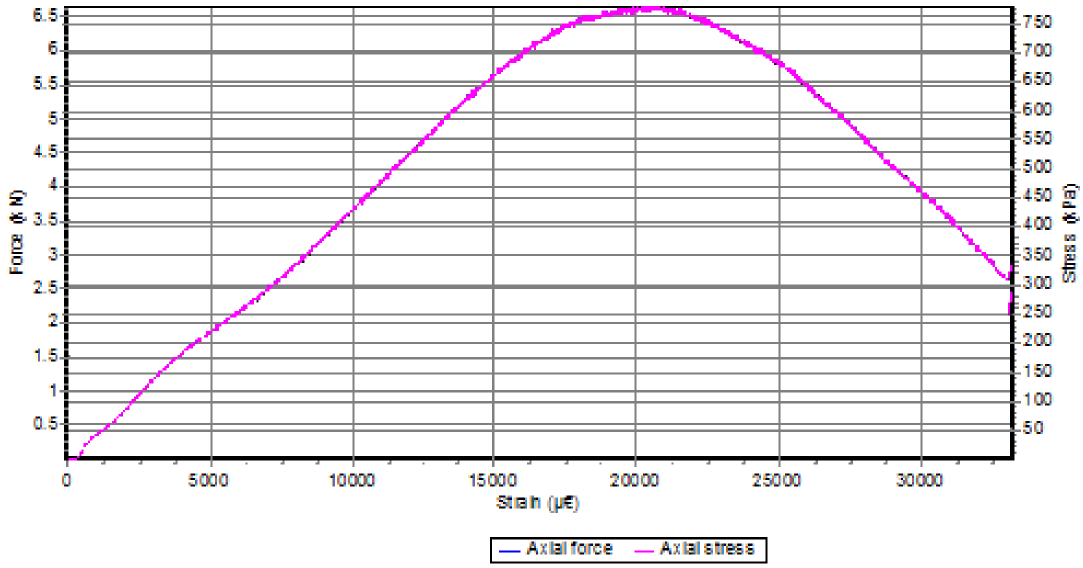
Unconfined compressive test (UCS) 100% recycled crushed concrete-core M



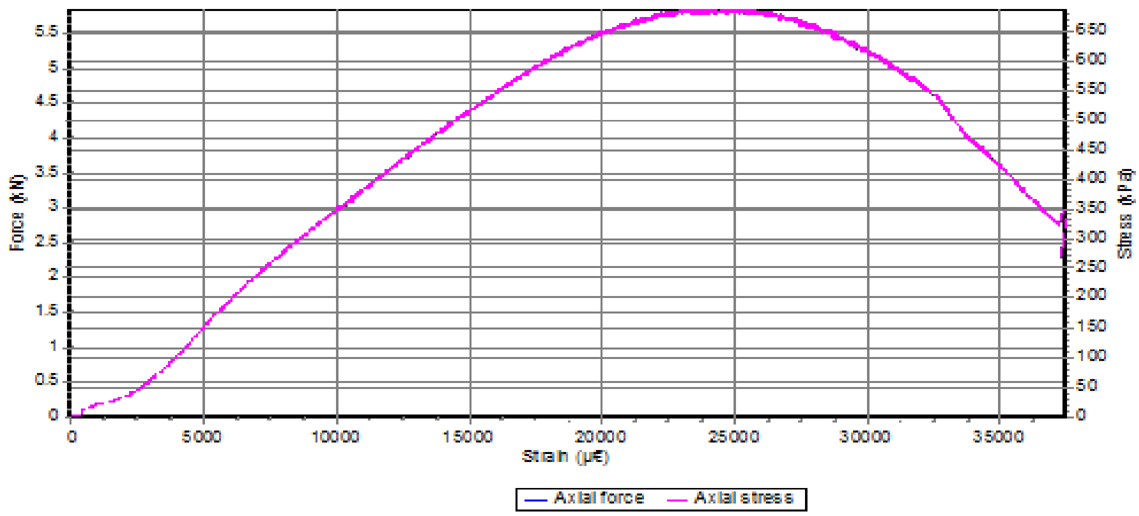
Unconfined compressive test (UCS) 100% recycled crushed concrete-core K



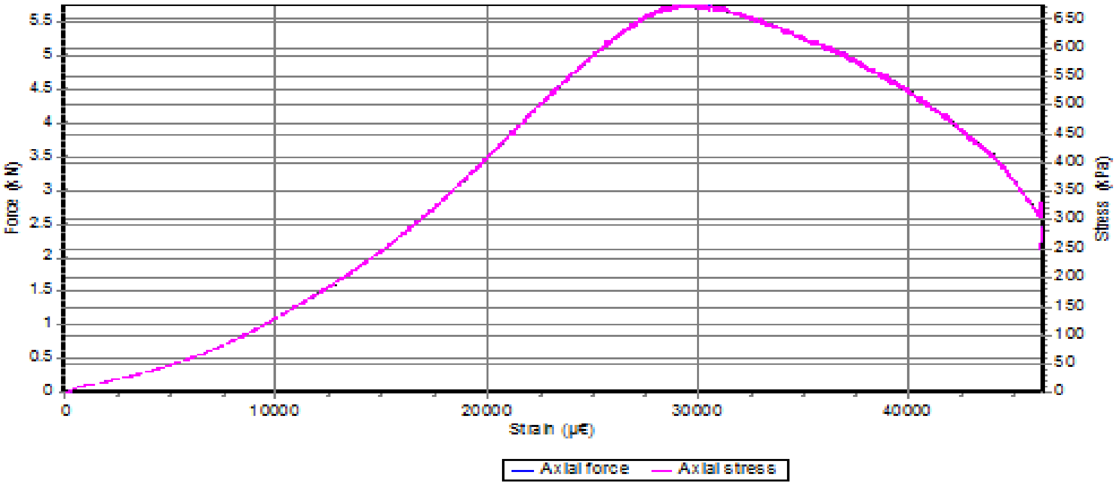
Unconfined compressive test (UCS) 100% recycled crushed concrete-core J



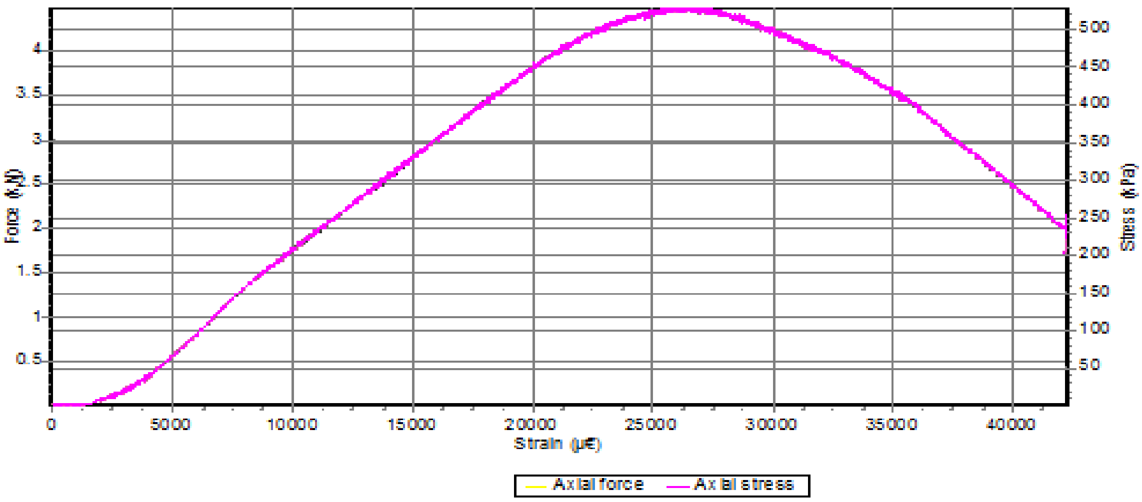
Unconfined compressive test (UCS) 10% crushed brick and tile with RCC-core Y



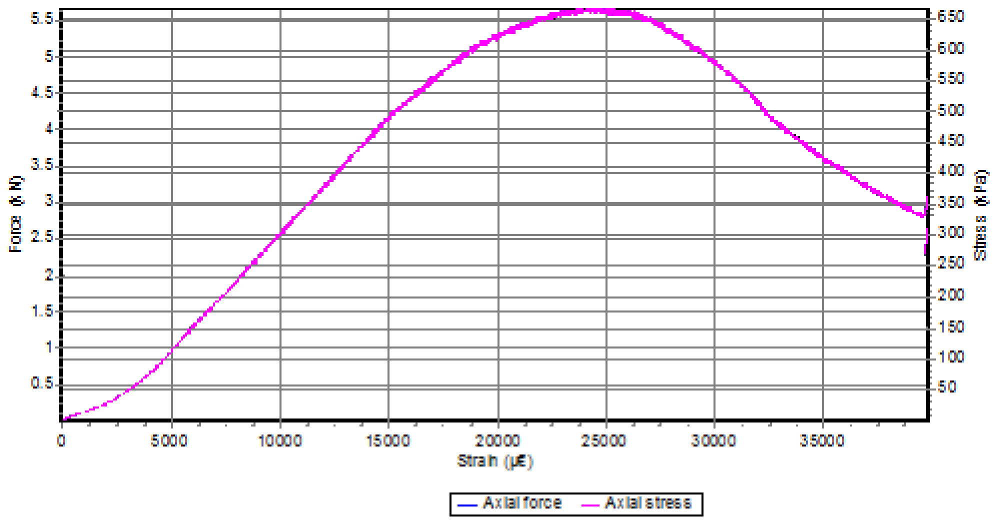
Unconfined compressive test (UCS) 10% crushed brick and tile with RCC-core U



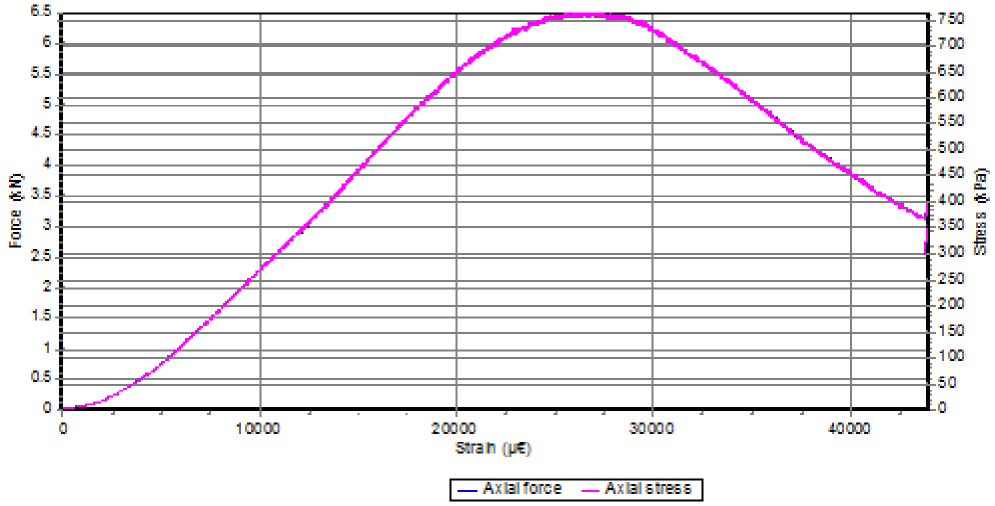
Unconfined compressive test (UCS) 10% crushed brick and tile with RCC-core test 1



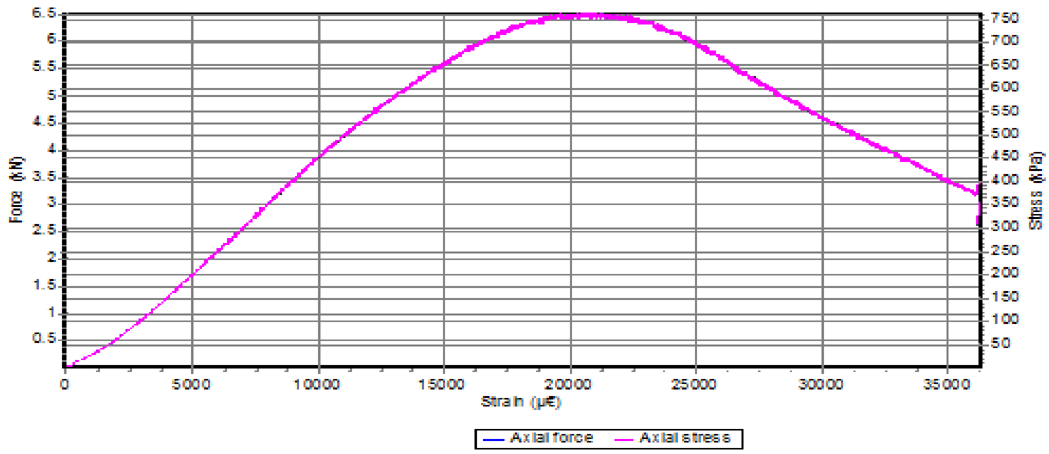
Unconfined compressive test (UCS) 10% crushed brick and tile with RCC-core T



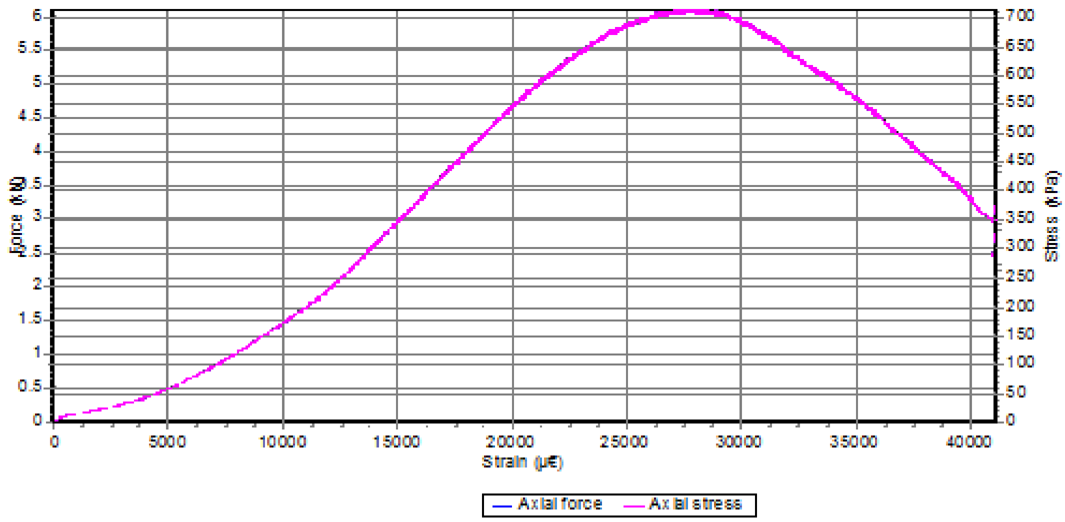
Unconfined compressive test (UCS) 10% crushed brick and tile with RCC-core S



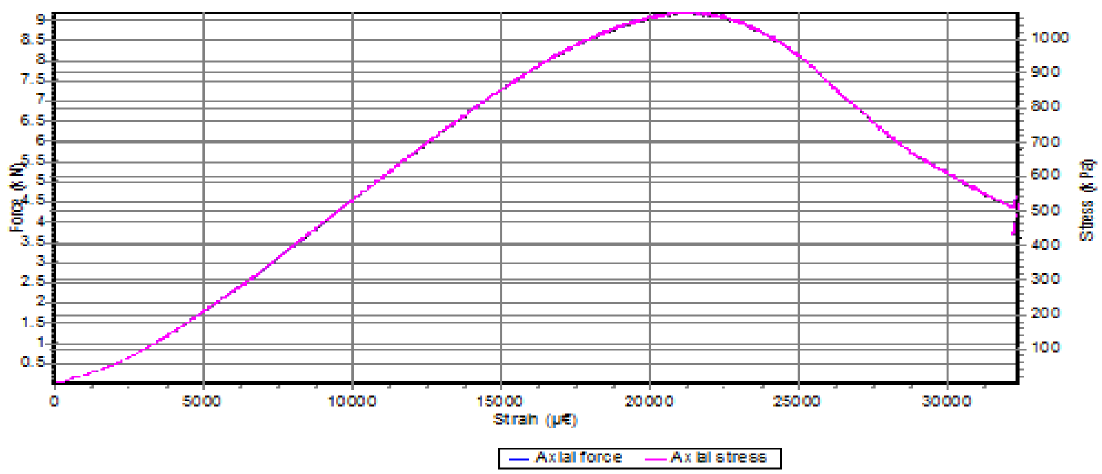
Unconfined compressive test (UCS) 10% crushed brick and tile with RCC-core R



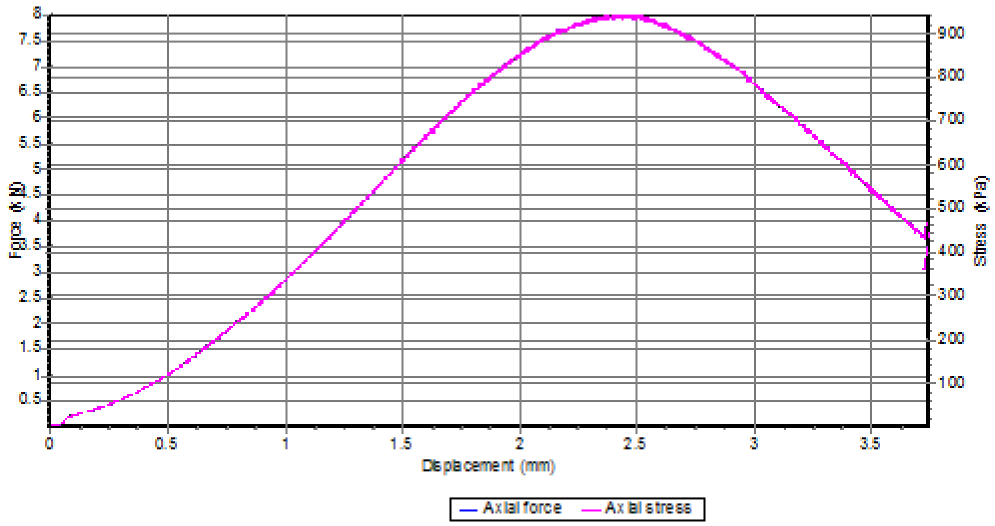
Unconfined compressive test (UCS) 10% crushed brick and tile with RCC-core Q



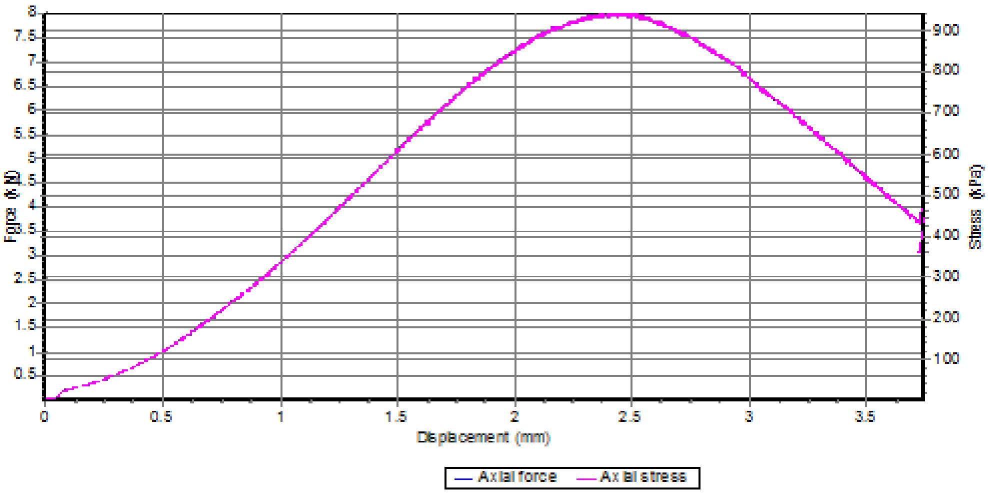
Unconfined compressive test (UCS) 30% crushed brick and tile with RCC-core X



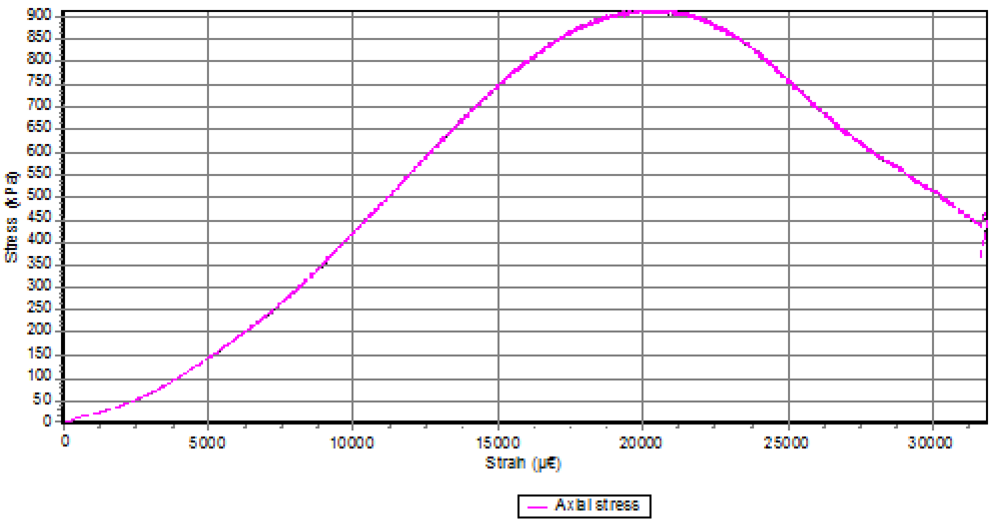
Unconfined compressive test (UCS) 30% crushed brick and tile with RCC-core L



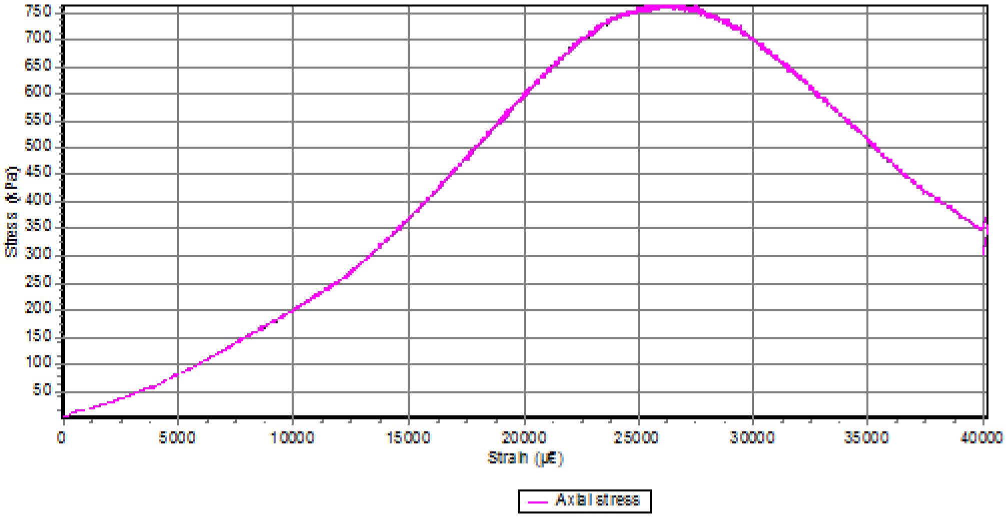
Unconfined compressive test (UCS) 30% crushed brick and tile with RCC-core I



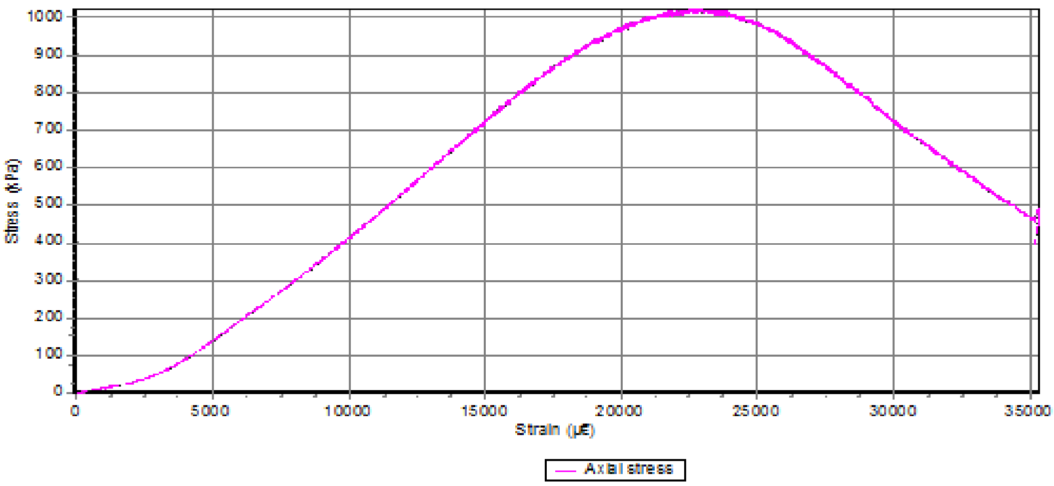
Unconfined compressive test (UCS) 30% crushed brick and tile with RCC-core H



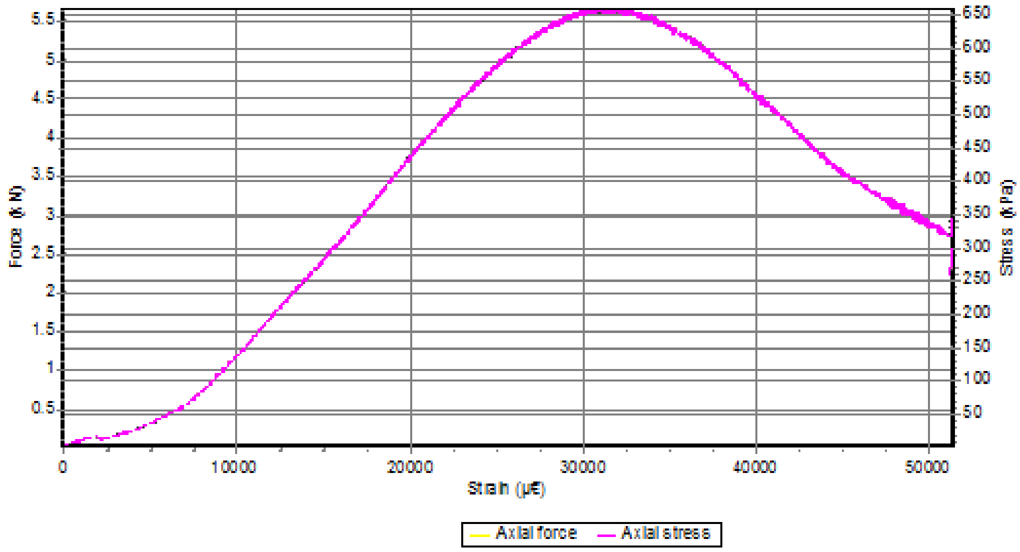
Unconfined compressive test (UCS) 30% crushed brick and tile with RCC-core G



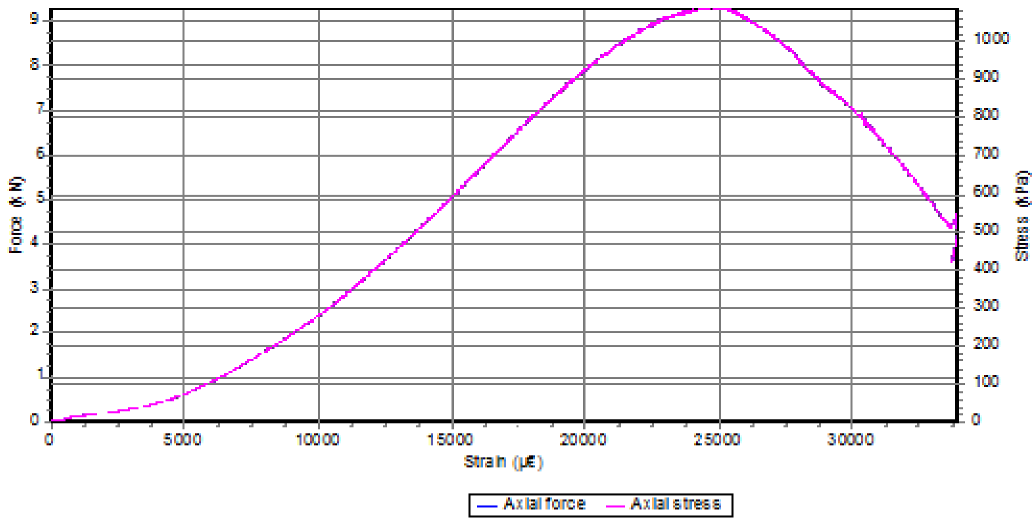
Unconfined compressive test (UCS) 30% crushed brick and tile with RCC-core F



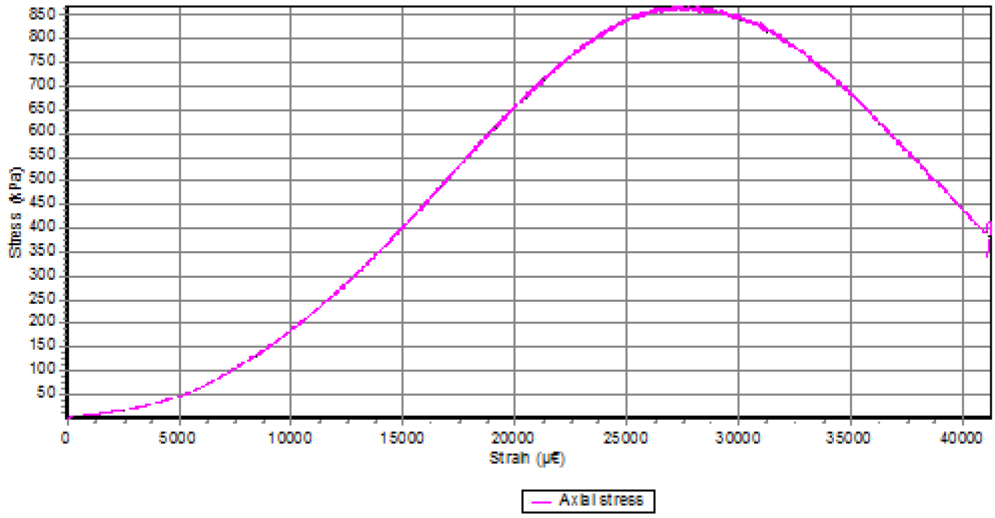
Unconfined compressive test (UCS) 30% crushed brick and tile with RCC-core 6



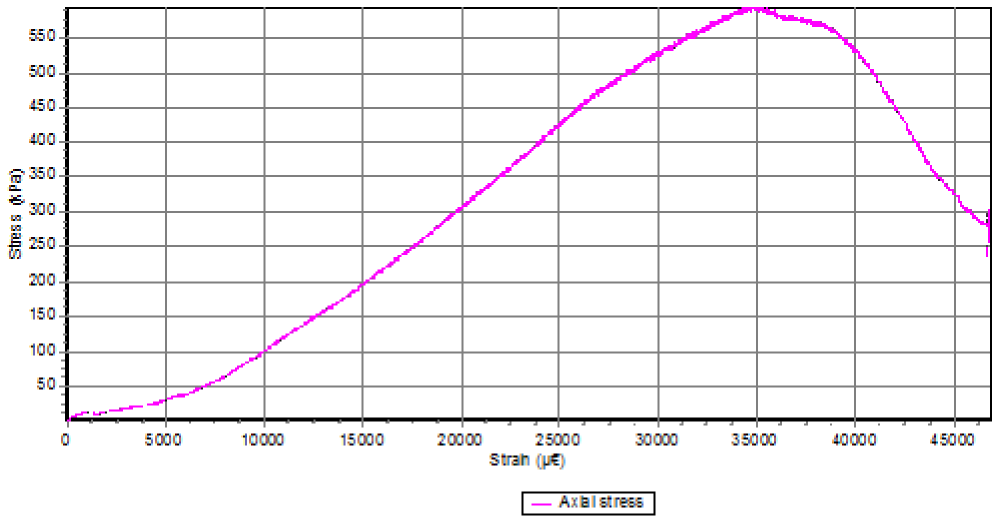
Unconfined compressive test (UCS) 50% crushed brick and tile with RCC-core W



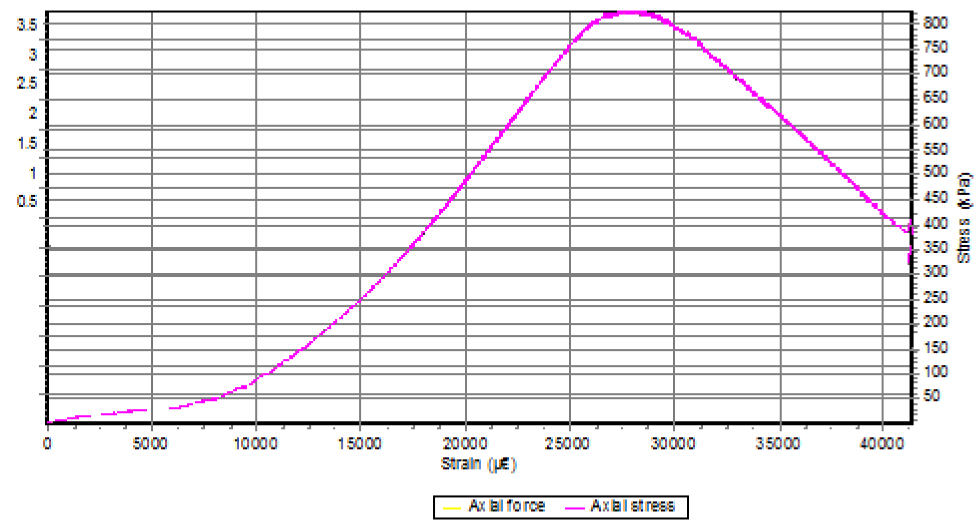
Unconfined compressive test (UCS) 50% crushed brick and tile with RCC-core E



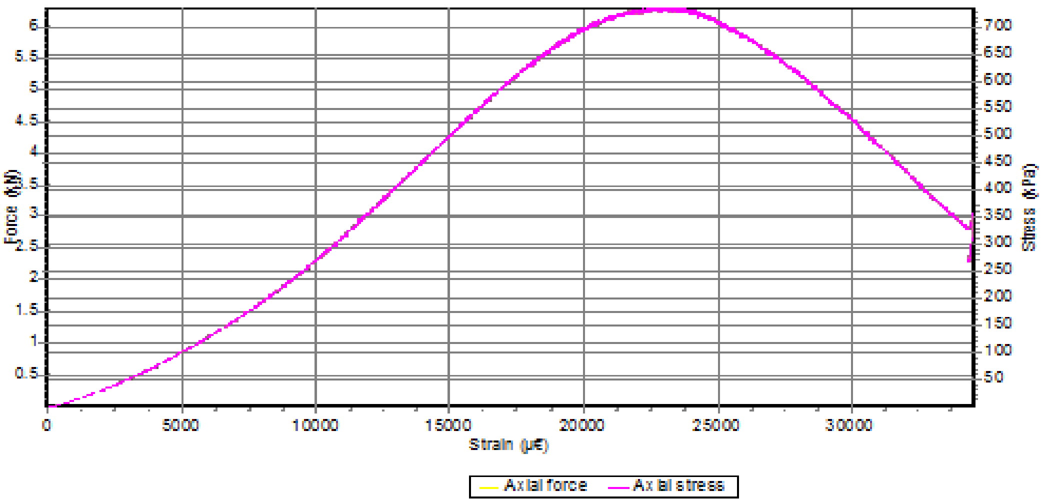
Unconfined compressive test (UCS) 50% crushed brick and tile with RCC-core D



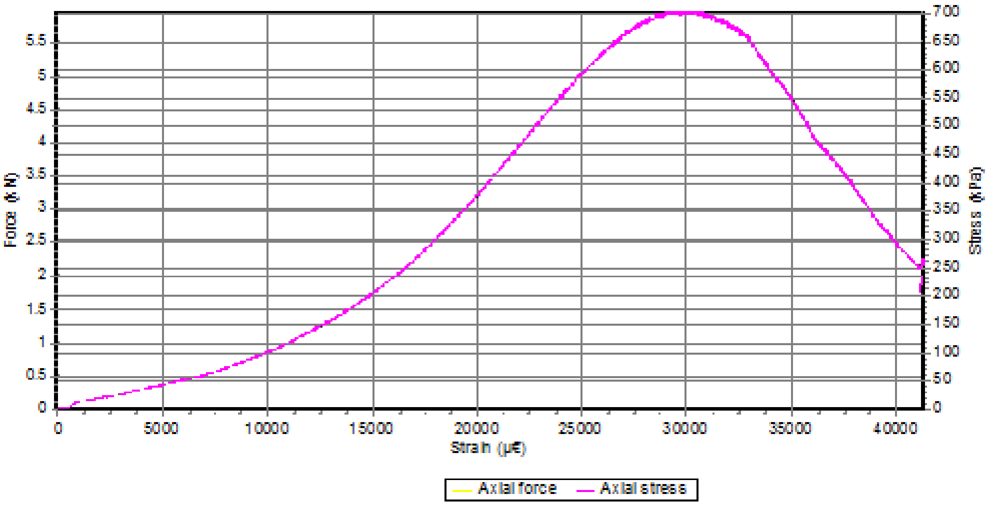
Unconfined compressive test (UCS) 50% crushed brick and tile with RCC-core C



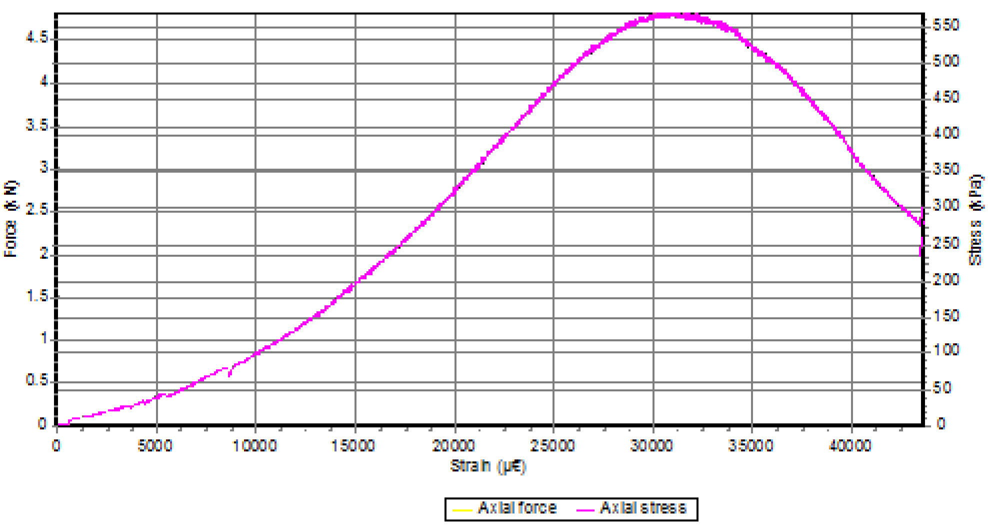
Unconfined compressive test (UCS) 50% crushed brick and tile with RCC-core B



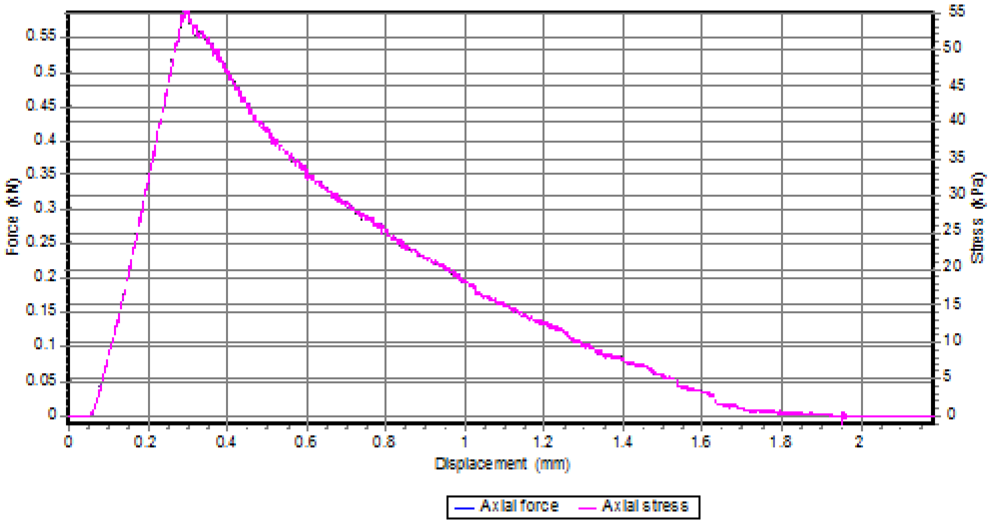
Unconfined compressive test (UCS) 50% crushed brick and tile with RCC-core A



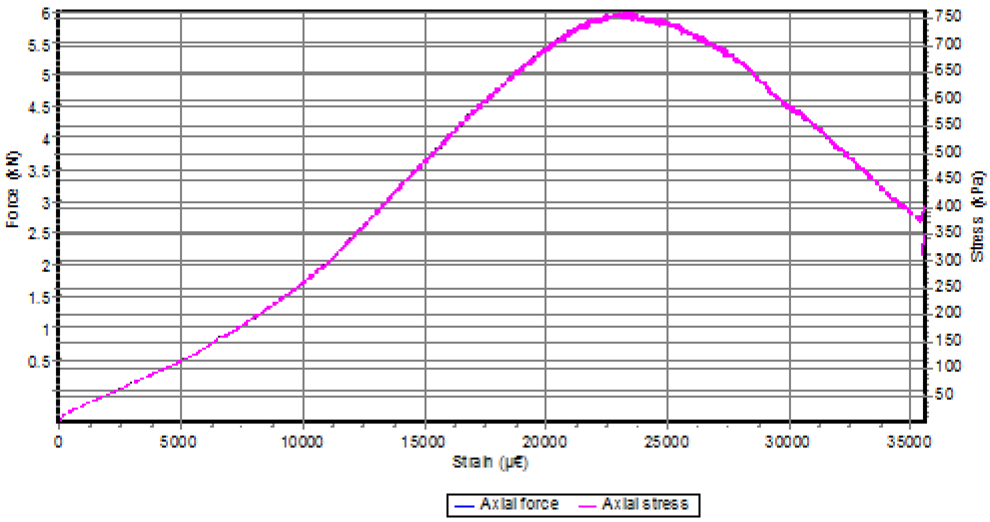
Unconfined compressive test (UCS) 50% crushed brick and tile with RCC-core 7



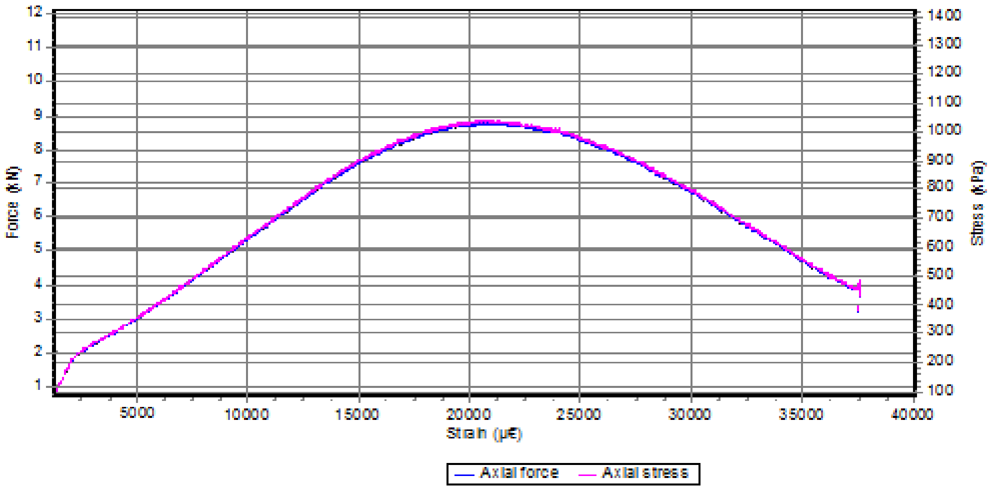
Unconfined compressive test (UCS) 10% Ferricrete with RCC-core Z17



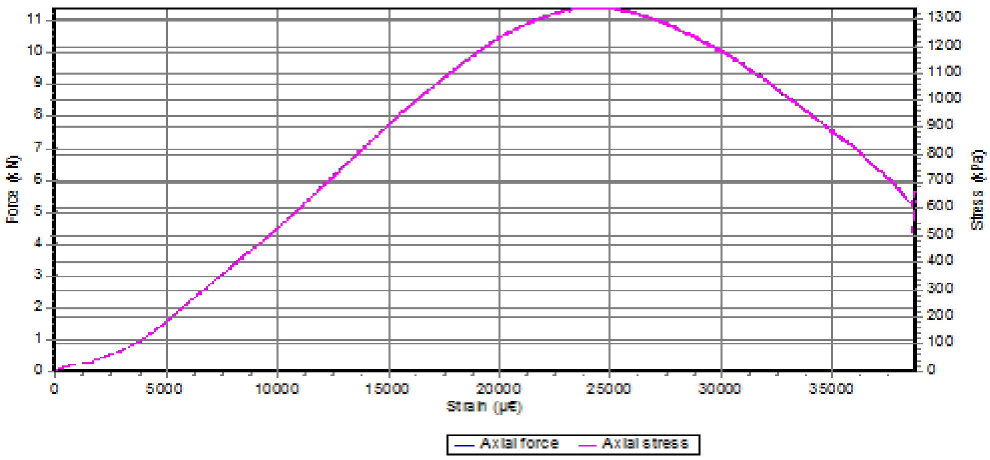
Unconfined compressive test (UCS) 10% Ferricrete with RCC-core Z5



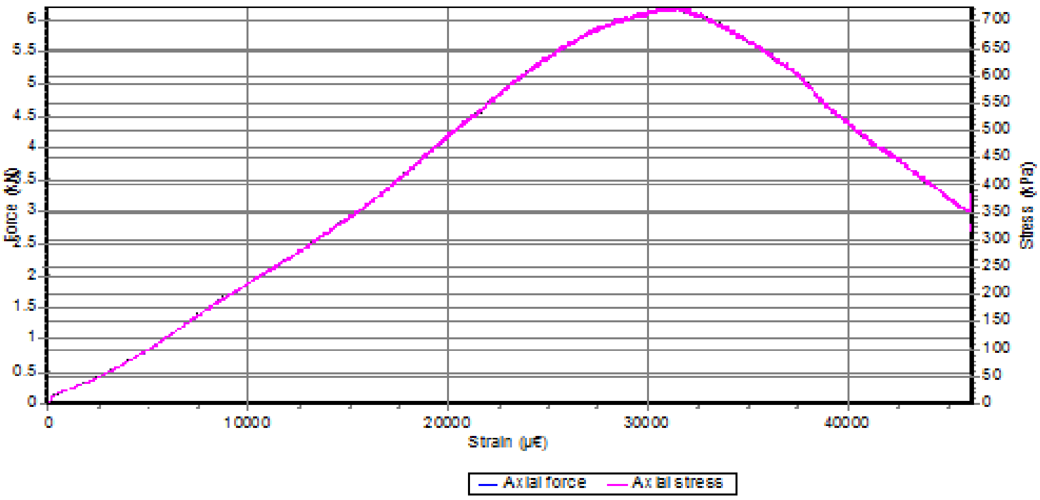
Unconfined compressive test (UCS) 10% Ferricrete with RCC-core Z3



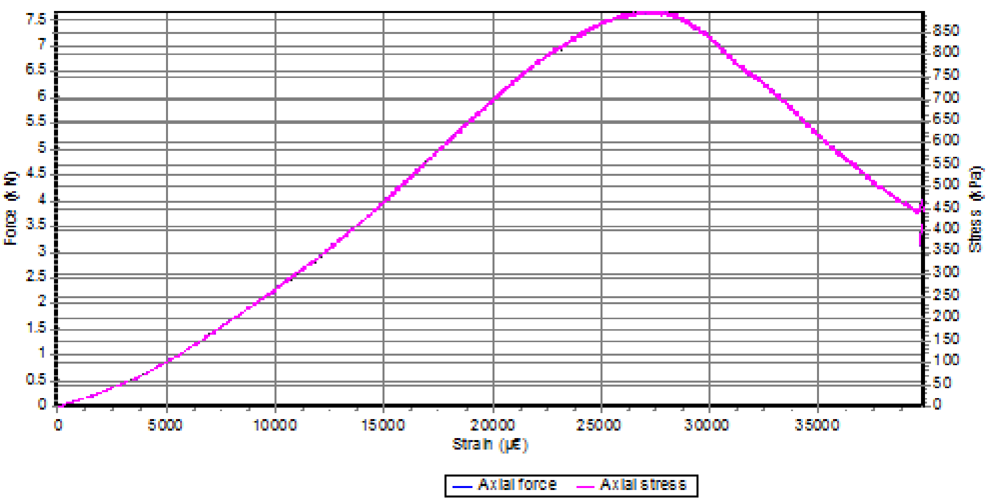
Unconfined compressive test (UCS) 10% Ferricrete with RCC-core Z2



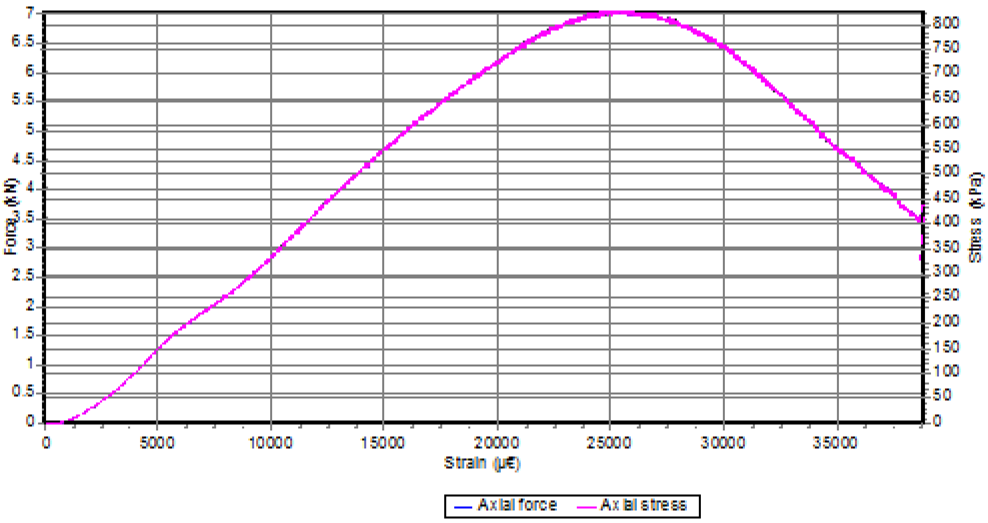
Unconfined compressive test (UCS) 10% Ferricrete with RCC-core Z1



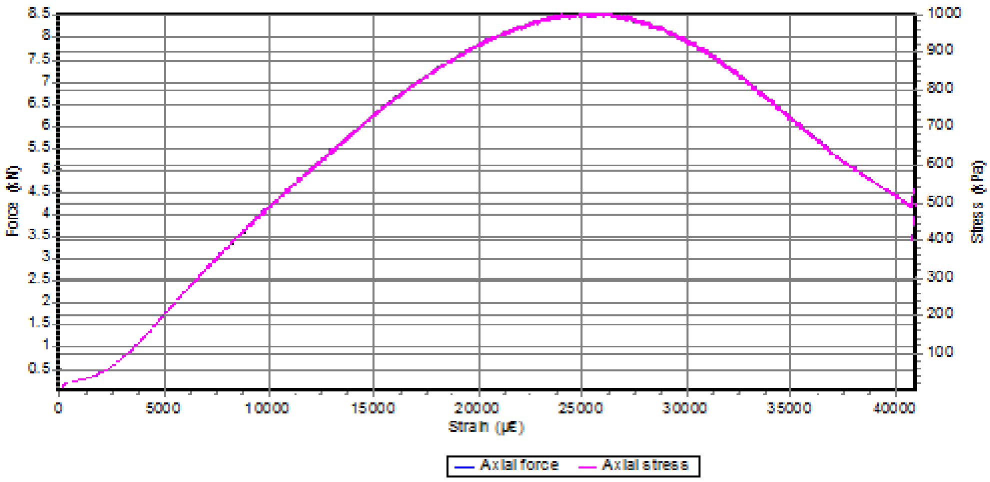
Unconfined compressive test (UCS) 10% Ferricrete with RCC-core 00



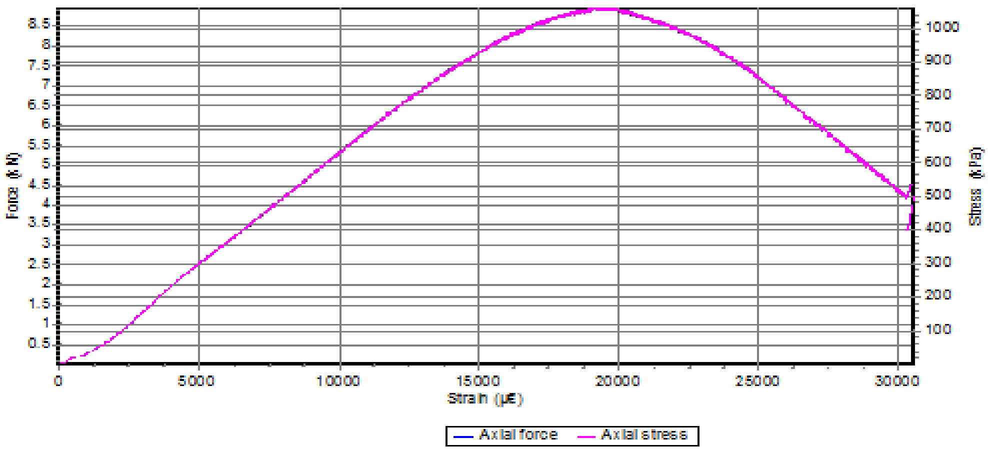
Unconfined compressive test (UCS) 30% Ferricrete with RCC-core Z18



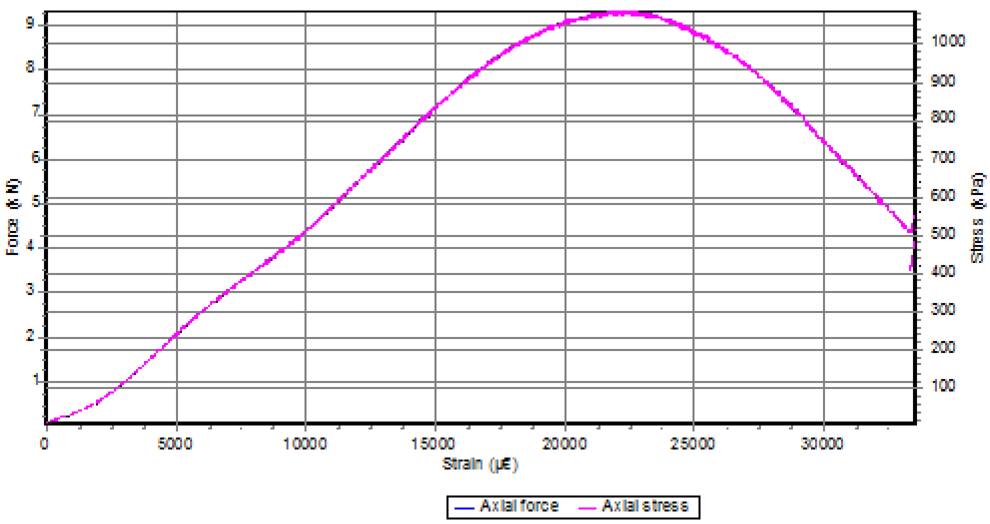
Unconfined compressive test (UCS) 30% Ferricrete with RCC-core Z10



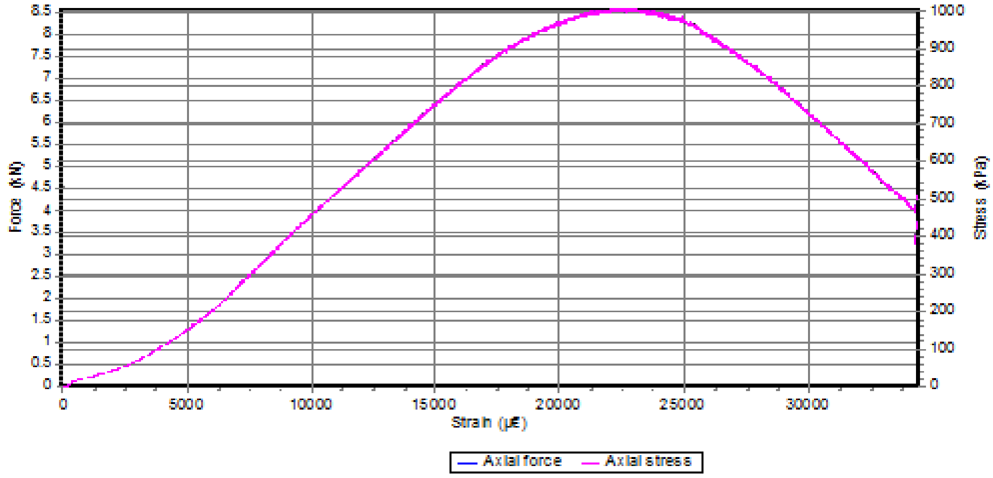
Unconfined compressive test (UCS) 30% Ferricrete with RCC-core Z9



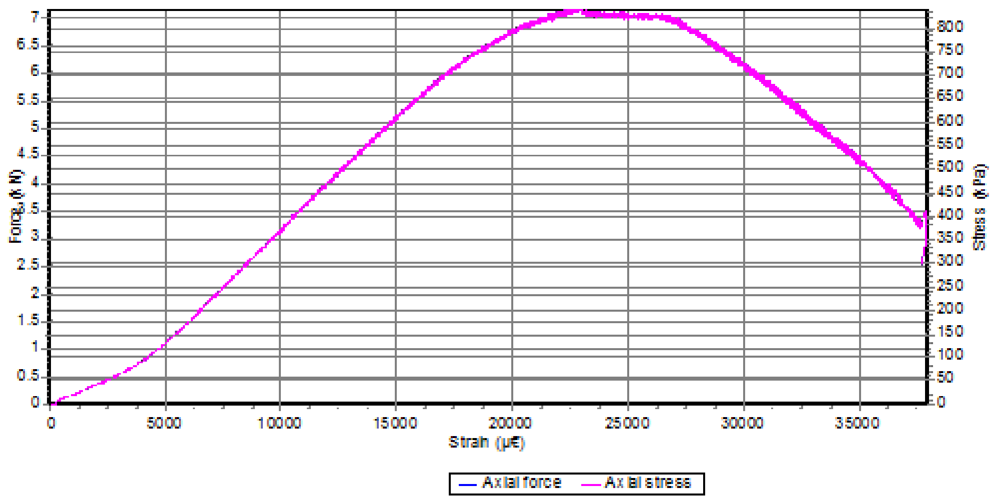
Unconfined compressive test (UCS) 30% Ferricrete with RCC-core Z8



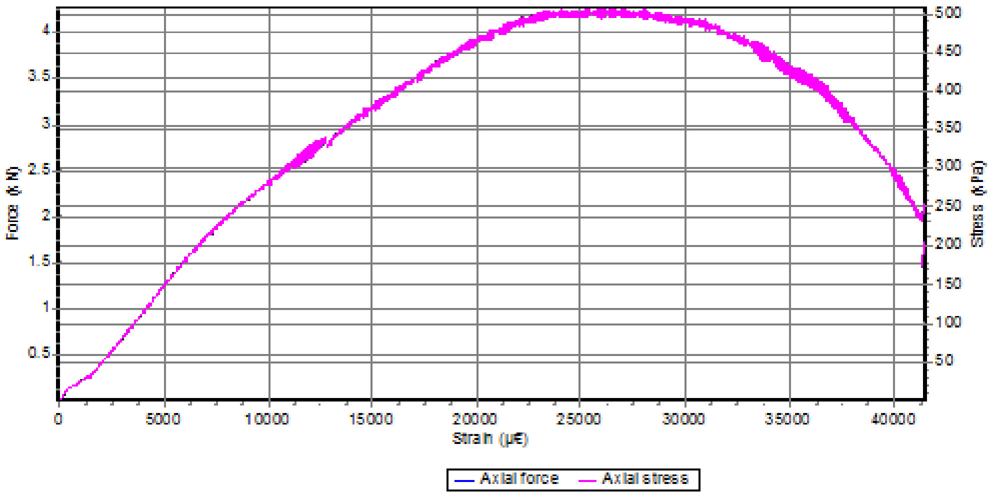
Unconfined compressive test (UCS) 30% Ferricrete with RCC-core Z7



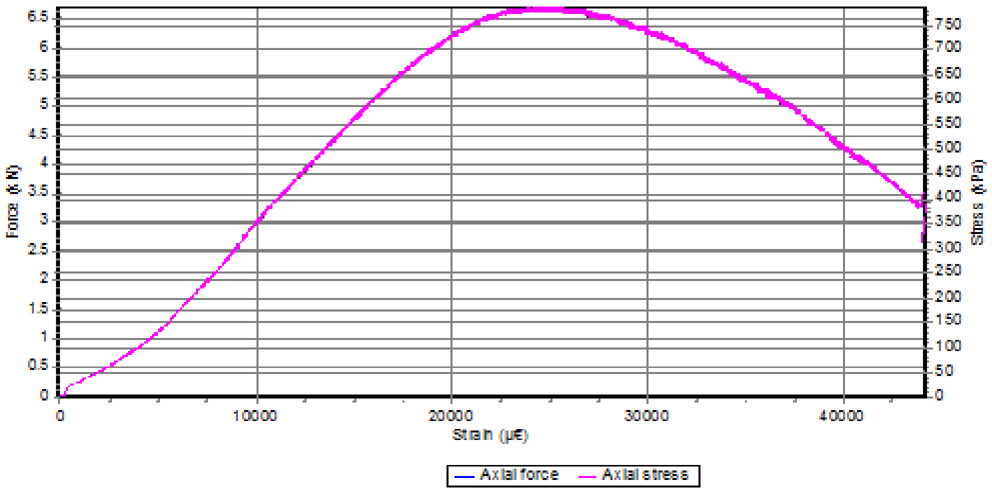
Unconfined compressive test (UCS) 30% Ferricrete with RCC-core Z6



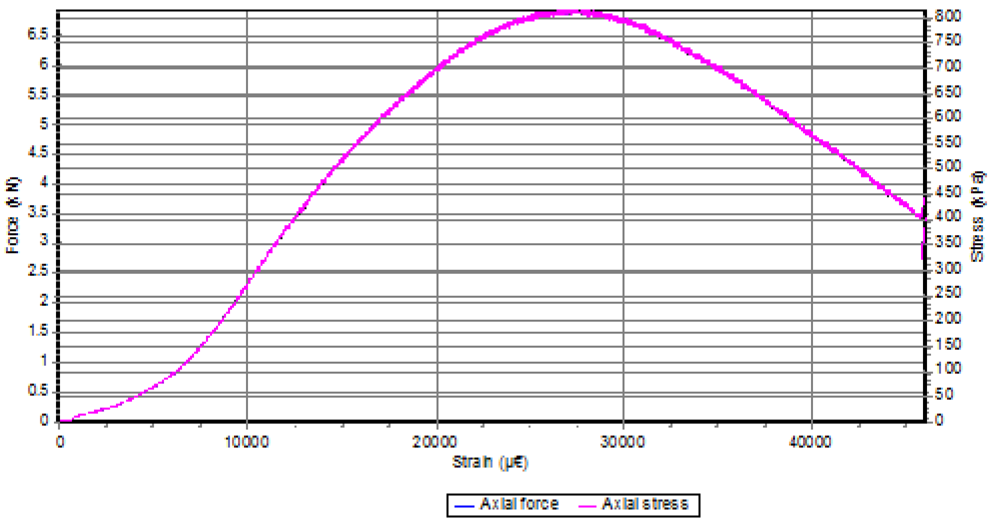
Unconfined compressive test (UCS) 30% Ferricrete with RCC-core OO2



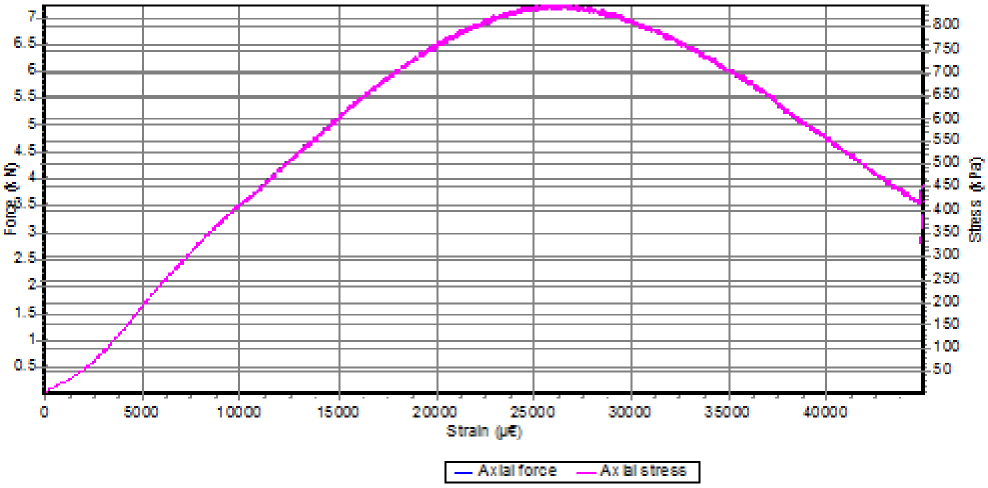
Unconfined compressive test (UCS) 50% Ferricrete with RCC-core Z11



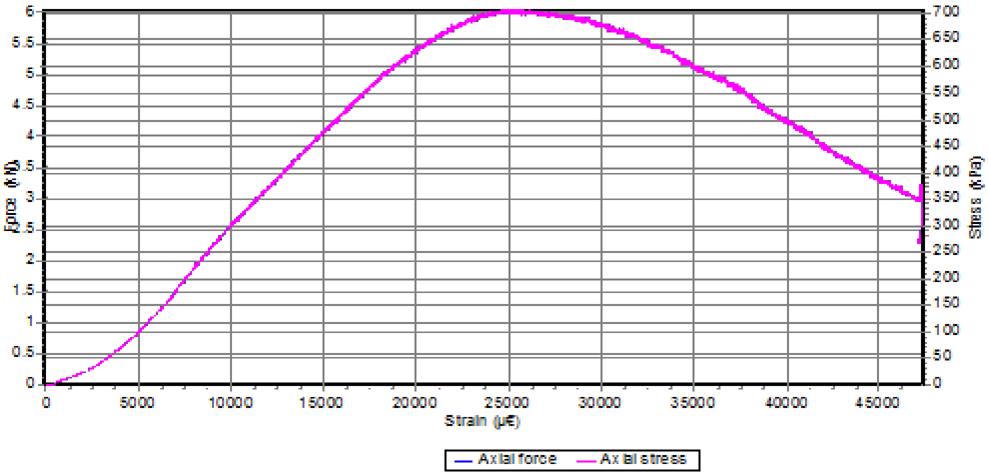
Unconfined compressive test (UCS) 50% Ferricrete with RCC-core Z12



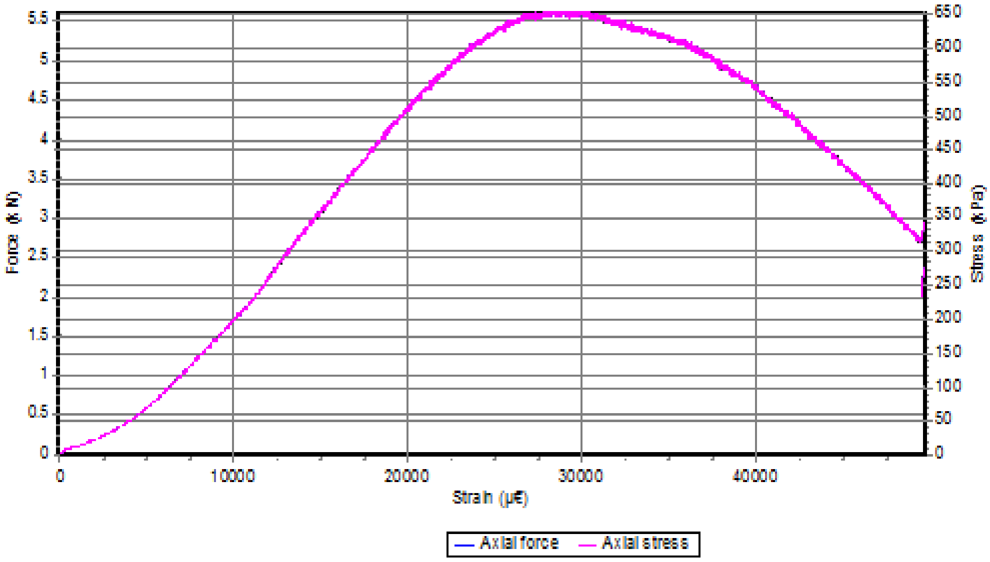
Unconfined compressive test (UCS) 50% Ferricrete with RCC-core Z13



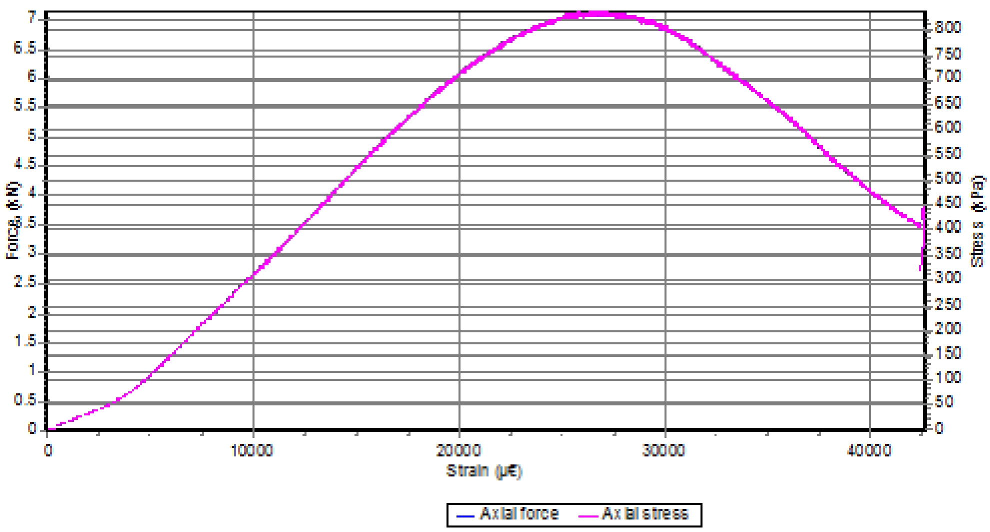
Unconfined compressive test (UCS) 50% Ferricrete with RCC-core Z14



Unconfined compressive test (UCS) 50% Ferricrete with RCC-core Z15



Unconfined compressive test (UCS) 50% Ferricrete with RCC-core Z16



Appendix G Statistical Analysis

Appendix G1: Statistical analysis of RCC cores with different blends of Brick& Tile(UCS test)

ANOVA: Single factor				
SUMMARY	Statistical analysis of UCS tests. RCC cores with different mixture of Brick &Tile			
	Count	Sum	Average UCS value	Variance
100%RCC	5	3.87	0.774	0.0088
10%Brick&tile+90%RCC	8	5.47	0.684	0.0053
30%Brick&tile+70%RCC	7	6.19	0.884	0.0214
50%Brick&tile+50%RCC	7	5.34	0.763	0.0316

ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	0.15	3	0.05	2.95	0.054	3.028
Within Groups	0.39	23	0.017			
Total	0.54	26				

P-value>0.05

F<Fcrit

Appendix G2: Statistical analysis of RCC cores with different blends of Ferricrete(UCS test)

ANOVA: Single factor				
SUMMARY	Statistical analysis of UCS tests. RCC cores with different mixture of Brick &Tile			
	Count	Sum	Average UCS value	Variance
100% RCC	5	3.87	0.774	0.0087
10%Ferricrete+90%RCC	7	6.1	0.871	0.0857
30%Ferricrete+70%RCC	7	6.31	0.901	0.0423
50%Ferricrete+50%RCC	6	4.61	0.768	0.0058

ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	0.085	3	0.028	0.71	0.55	3.07
Within Groups	0.833	21	0.040			
Total	0.918	24				

P-value>0.05

F<Fcrit

Appendix G3: Statistical analysis of RCC beams (Maximum axial force)

ANOVA: Single factor				
SUMMARY	Statistical analysis of UCS tests. RCC cores with different mixture of Brick &Tile			
Groups-curing(hr)	*Count*	*Sum*	*Average Maximum axial force(KN)*	*Variance*
0hr	8	4.024	0.503	0.014857
24hr	5	3.8	0.76	0.0267135
48hr	5	2.617	0.5234	0.006538

ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	0.225	2	0.113	7.134	0.007	3.682
Within Groups	0.237	15	0.016			
Total	0.462	17				

P-value<0.05

F>Fcrit

It is not acceptable, and then the T-test was considered to compare two compaction conditions

Appendix G4: T-Test of RCC beams condition 0hr and 24hr (Maximum axial force)

T-test: RCC beam condition 0hr and 24hr		
T-Test: Two-Sample Assuming Unequal Variances	*0hr*	*24hr*
Mean	0.503	0.76
Variance	0.0149	0.02671
Observations	8	5
Hypothesized Mean Difference	0	
df	7	
t Stat	-3.028	
P(T<=t) one-tail	0.00957	
t Critical one-tail	1.8945	
P(T<=t) two-tail	0.0191	
t Critical two-tail	2.3646	

Appendix G5: T-Test of RCC beams condition 0hr and 48hr (Maximum axial force)

T-test: RCC beam condition 0hr and 48hr		
T-Test: Two-Sample Assuming Unequal Variances	*0hr*	*48hr*
Mean	0.503	0.5234
Variance	0.015	0.0065
Observations	8	5
Hypothesized Mean Difference	0	
df	11	
t Stat	-0.363	
P(T<=t) one-tail	0.362	
t Critical one-tail	1.796	
P(T<=t) two-tail	0.724	
t Critical two-tail	2.201	

Appendix G6: T-Test of RCC beams condition 24hr and 48hr (Maximum axial force)

T-test: RCC beam condition 24hr and 48hr		
T-Test: Two-Sample Assuming Unequal Variances	*24hr*	*48hr*
Mean	0.76	0.5234
Variance	0.02671	0.0065
Observations	5	5
Hypothesized Mean Difference	0	
df	6	
t Stat	2.901	
P(T<=t) one-tail	0.013	
t Critical one-tail	1.943	
P(T<=t) two-tail	0.0273	
t Critical two-tail	2.447	