

PHYSICAL MODELLING OF VIBRO STONE COLUMN USING RECYCLED AGGREGATES

by

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Abstract

Vibro-stone column (VSC) is one of the most commonly used ground improvement techniques worldwide. It provides a column-soil composite to reinforce soft ground; increasing the bearing capacity and improving the settlement characteristics.

The performance of the VSC depends on the quality of aggregates used and the interaction with the surrounding soil. The overall mechanism is understood. However, the impact of installation methods used and the choice of aggregates to form the columns are still unknown which can result in short and long-term failures of the columns. This is further hampered by the use of aggregate index tests that do not represent the actual environment of the installation process.

As opposed to previous research where only sand, gravel and primary aggregates were used in the unit cell modelling of the VSCs, in this research a selection of primary (granite) and three recycled aggregates (crushed concrete and brick, incinerator bottom ash aggregate types 1 and 2) which are commonly used in the practice of VSCs were compared in the actual context of the installation and loading of a single stone column in soft clay.

The aggregate index tests recommended by the standards were performed on all of the primary (PA) and the recycled aggregates (RA). The results showed that in most of the index tests, the RAs performed poorly compared to the granite and based on these criteria they could not be used for the construction of VSCs.

However, in this research the aggregates were modelled in two sets of the large and the small unit cell tests (LUC and SUC) which were designed for the study of the behaviour of a single column in the short-term in which the dry top feed method of installation

was used on the actual PA and the RAs, despite their unacceptable aggregate index tests results.

In both of the unit cell tests, the RAs behaved comparable to the PA in terms of the load carrying capacity and showed that the aggregate index tests results alone should not be considered for the selection of the materials for the use in the context of the VSC. The particle size distribution (PSD) and well-graded or uniformly graded range of the aggregates were found to be one of the most important factors affecting the column density and formation and ultimately its load carrying capacity.

In the LUC tests it was concluded that the existence of the VSC increased the load carrying capacity of the host ground by approximately 60% regardless of the type of the aggregates used. Despite the unacceptable results in the index tests, the RAs performed satisfactorily in the unit cell tests and improved the load carrying capacity of the ground by up to 190% and also, due to their well-graded PSD and the level of packing achieved in the column outperformed the PA in the stress-strain comparison under similar installation and loading conditions.

The condition of the aggregates (wet/dry) was an important factor in terms of the performance. The columns of wet aggregates performed between 10 to 15% poorer in the LUC compared to the columns of the dry aggregates under the loading, especially when the wet recycled material was loaded.

In the SUC, three series of tests were performed to understand: 1) the effect of installation versus the loading on the crushing of both the PA and the RAs, 2) the effect of the time (energy) of compacting of each layer of the PA during installation on the load carrying capacity and 3) the effect of contamination of the PA with fine material on the load-settlement behaviour of the VSC.

In the first series of the SUC tests the RAs were crushed up to 5% more than the PA during the installation. The level of crushing of the RAs was up to 2% during the loading and the crushing of the PA was minimal during both the installation and loading stages. It was concluded that the installation forces can cause more change in the PSD of the materials whereas, during the loading the nature of the RAs can hold the particles together and prevent any further crushing.

In the second series of the small unit cell tests it was observed that 50% reduction in the duration (energy) of installation resulted in 10% reduction in the density of the column and ultimately 40% reduction in the load carrying capacity of the composite (column of the PA and the soft clay); whereas an increase of three times in the time of vibrations increased the bearing capacity by almost 35%. The time of installation per layer of aggregates should be sufficient enough for the column formation (proper diameter and length should be achieved) to carry the loads and over-treatment should be avoided due to ground heave and a less cost-effective project.

In the third series of the SUC tests the addition of fines to the column of granite reduced the bearing capacity by approximately 40% when 10 and 20% fines were added compared to the column which was free from fines. During the storage, transportation and the installation process fines might be introduced to the column material that can affect the performance of the VSCs in the short-term.

Dedication

For my dearest parents Azita and Bahram

And my beloved brother Khashayar

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Abbreviations

AA	Alternative aggregate
ACV	Aggregate crushing value
AIV	Aggregate impact value
CC/CB	Crushed concrete and crushed brick
GWL	Ground water level
IBAA	Incinerator bottom ash aggregate
LA	Los Angeles
LUC	Large unit cell
PA	Primary aggregate
PSD	Particle size distribution
RA	Recycled aggregate
SA	Secondary aggregate
SUC	Small unit cell
TFV	Ten percent fines value
VSC	Vibro stone column

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CHAPTER ONE

INTRODUCTION

1. INTRODUCTION

In this chapter the concept of using vibro stone column (VSC) was briefly introduced as one of most commonly used ground improvement methods worldwide.

The gaps in the knowledge have been highlighted which indicated the necessity of the study of the installation and the use of alternative aggregates in the context of VSC.

The aim of this research is presented, followed by the objectives to achieve this aim via the laboratory testing designed in this research.

1.1 Background

Ground improvement methods are widely used to improve the ground condition and the sustainability of the projects (Mitchell and Jardine, 2002). In the UK, these methods are used to treat fills, alluvial soils and many other problematic grounds to improve the bearing capacity and the settlement behaviour (McKelvey and Sivakumar, 2000).

The design is mostly empirical or semi-empirical; thus field trials, laboratory tests and numerical models are constantly used to assist in evaluation of the design theories and the assumptions used (Weber *et al.*, 2006).

VSC is currently the most common ground improvement method used in the UK (Serridge, 2006). This method is economical and is used for light structural foundations, embankment stability and controlling the liquefaction potential in seismic areas (McKelvey and Sivakumar, 2000). It is suitable for soft cohesive soils both economically and technically (McCabe *et al.*, 2009).

VSC is a replacement method; the vibro-flot (poker) penetrates the ground and the cohesive material is replaced with granular, hard and inert aggregates. The column-soil composite is formed which improves the stiffness, the bearing capacity and the settlement characteristics of the weak ground (Charles and Watts, 2002).

By loading the column, bulging happens and causes lateral deformations and stress changes in the surrounding soil after the initial vertical settlements, which is followed by the resistance from the ground due to lateral restraint developed in it.

Ultimately the system reaches equilibrium. As a result, the VSC acts as a reinforcement element in the ground. The column (as a granular material) acts as a vertical drain, which increases the consolidation rate and therefore reduces the post-construction

settlements (Charles and Watts, 2002). Best results were observed when the column were loaded over a bearing stratum (Barksdale and Bachus, 1983).

For many years primary or natural aggregates have been used in the construction of VSCs (Jefferson *et al.*, 2010); however, these sources are becoming more and more scarce. On the other hand, new legislations regarding no waste policies emphasise the use of alternative sources in various industries (Schouenborg, 2005).

During the installation process of the stone column, the aggregates are charged and compacted at stages (BRE, 2000). After the installation and during the loading, the lateral restrains and shearing forces are carried through these aggregates. Therefore, there are certain requirements for the use of aggregates, regardless of their source (primary or alternative), such as being hard, inert, stable and having proper grading (BRE, 2000). Whatever the source of the aggregate is, it should be 'fit-for-purpose' (Serridge, 2006). Lots of factors such as the grading, the grading compatibility with the installation method, contamination with fines and the condition (wet or dry) may affect the performance of VSCs both in the short and the long term (Serridge, 2006).

Despite clear understanding of the mechanism of the VSC and defined criteria for the use of material in its context, in terms of the performance, there have been number of failures both in the short and the long term (Bell, 2004).

The impact of installation methods used on various aggregates during construction is still unknown. Using the index tests on these aggregate sources to evaluate their suitability for the use in the stone column construction may not be the best and the only indicator to reflect their behaviour under the installation and loading of columns.

1.2 The use of alternative aggregates

Use of the alternative aggregates in the construction of VSC is becoming more popular; it is recommended that the material should be fit for purpose (Serridge, 2005) and there are several laboratory index tests required to ensure the properties of the material selected such as the strength and crushability meet the design and performance requirements of the VSCs (ICE, 1987).

There are many uncertainties and barriers against the use of alternative sources especially in the context of VSC such as:

- Firstly, it should be evaluated whether the aggregate index tests recommended by various standards (ICE, 1987; BRE, 2000) are representative of the condition of aggregates in the context of VSC both during installation and loading;
- Secondly, whether the different types of aggregates (primary and alternative) should be assessed using the same criteria (index tests) for the context of VSC; the standards recommend the same evaluation methods for all the aggregate types (ICE, 1987);
- And thirdly, would primary and alternative aggregates behave differently under the same installation effect? *i.e.*, the performance of VSC under a combination of use of alternative aggregates and installation effects is still unknown.

1.3 Research aim

In previous research, the aggregate index tests were used on various primary and alternative aggregates to understand the aggregate properties such as the hardness, the angle of shearing resistance and the porosity (Chidiroglou *et al.*, 2009; McKelvey *et al.*, 2004; Steele, 2004; Schouenborg, 2005).

The index tests did not consider the unique conditions of the installation process and loading of the aggregates in the context of the VSCs.

Other researchers tested a single or column groups under various installation and loading conditions. However, in most of these tests the actual aggregates were not used. Sand or gravel or in fewer cases only primary aggregates were modelled in the installation and loading of the VSCs (Hughes and Withers, 1974; Barksdale and Bachus, 1983; Black *et al.*, 2007).

In this research, three recycled (CC/CB, IBAA (1) and IBAA (2)) and one primary (granite) aggregates were selected for the laboratory testing. The index tests were performed on all the aggregates, however, **the aim was that instead of sand or gravel or only PAs, for the first time the actual recycled sources should be used in the installation and loading of a single stone column and the behavior of these aggregates should be compared with the PA, despite the results of the aggregate index tests.**

In this research the validity and the relevance of the aggregate index tests regarding the performance of the VSC was studied via two sets of the large and the small unit cell tests.

In these tests the short-term behaviour of a single stone column was compared for the primary and the three recycled aggregates under dry top-feed installation and in the short-term.

1.4 Research objectives

According to the aim, the objectives of this research were as follows:

- 1) To study the current state-of-the-art in the area of VSC, in which the mechanism, failures, limitations, and aspects of design, construction, material and loading affecting the short-term performance of the VSC was understood.
- 2) The critical review of the literature was narrowed to concentrate on the various aspects of installation and material in the short-term. This methodology did not consider the long-term behaviour of the columns and the aggregate deterioration due to the time limitations. The short-term duration was broken into during installation and during loading of the columns.

The next stage was to use a set of laboratory tests to model the critical factors affecting the material and the installation in the context of VSCs. The laboratory modelling assisted in creating controllable conditions under which various factors were studied separately or simultaneously.

- 3) The materials were tested for their basic properties. These included Kaolin (China clay) as the host ground and 4 types of aggregates to be used in the installation of the columns. Granite as a primary aggregate was used as a benchmark to compare the behaviour of the recycled aggregates against a primary source. Three types of recycled aggregates were studied which were a mixture of crushed concrete and brick (CC/CB), and two forms of incinerator bottom ash aggregates (IBAA), unprocessed and burnt, IBAA (1) and IBAA (2), respectively. Full description of the aggregate sources and the reasons they were selected for this research were presented in chapter 4, section 4.5.1. However, these aggregates were initially selected as they are commonly used in practice but not enough data is available regarding their performance. The aggregate index tests were performed on all the aggregates.

- 4) Factors studied in this research were categorized for the purpose of the unit cell testing. These categories covered aspects of installation such as various installation times (or energy) and crushing of aggregates due to the installation. Also, regarding the use of the materials, the conditions such as wet or dry and the contamination with fines (due to the installation process) were studied via various series of tests.
- 5) Two unit cells (referred to as the large and the small) were designed and developed in order to study the short-term behaviour of a single stone column in the soft clay using the material described in the previous objectives. In the smaller unit cell tests, the behaviour of the columns during the installation and loading were compared by the use of measurement of the crushing of the aggregates at each stage. This effect was compared for both the primary and the recycled aggregates.
- 6) In the small unit cell, for the primary aggregate various installation times were tried to observe the effect of the installation energy on the overall behaviour of the VSC in the small unit cell tests. Also, on this material, the effect of the addition of fines to the source was studied by adding crushed granite. Not enough material was available from the RA sources to study this effect and only the granite was tested. This was performed in the small unit cell tests in which the columns were loaded and compared to the columns constructed with no fines in the material.
- 7) In the large unit cell, various aggregates were tested under static loads after the installation. The columns were compared for their load-settlement behaviour under the same installation and loading conditions. Also, the water levels were measured at various depths and radii from the column in order to study the

behaviour of the surrounding ground under the installation and loading of the stone columns. Also, in this unit cell, the wet and dry aggregates were compared in the large unit cell, in which the granite and the crushed concrete and brick were soaked and loaded to be compared with each other and with the dry aggregates. Finally, a long-term test was performed in the large unit cell on the granite in which the load was applied to the column 3 months after it was constructed (refer to section 6.7 in chapter 6). The loading was the same as the other large unit cell tests; it indicated the difference of quickly loading the column after the construction versus leaving the column in the ground before the loading commenced.

- 8) The results of the large and the small unit cell tests were compared and analysed and relevant published work was used to evaluate the findings.

In order to cover the aim of this research, various recycled materials were tested for their index properties and also in the context of VSC under static loads. The analysis demonstrated whether the index tests predicted the behaviour of the aggregates for the purpose of VSCs. On the other hand the results were used to find other important factors such as the particle size distribution and the angle of shearing resistance of the material as well as the density of column constructed that affect the short-term behaviour of the VSCs.

1.5 Thesis outline

The review of the literature is presented in two chapters of two and three. In chapter two, the general background on the performance of the VSC is presented. The VSC mechanism and failures is explained which leads to three aspects of the design, the construction and the material. Each of these aspects was briefly introduced and important factors affecting each were highlighted using various cases and studies.

Chapter three covers the aspects of the performance which are important regarding the aim of this research. Therefore, the important factors affecting the performance of VSC were divided not only in categories of the installation and the material but also in the durations of before installation, during installation, during loading and in the long-term. As the aim of this research is to study various recycled aggregates in the short-term, only the factors affecting during installation and loading of the columns were further discussed.

In chapter four, the methodology used in this research is explained. It is stated why the laboratory modelling is a useful method in assessing the performance of the VSC in a unit cell under static loading. The unit cell tests designed required the host ground and the column material to form the single column.

This chapter deals with the material tests, both on the clay as the host ground and on the aggregates as the column material. The index tests performed on the China clay used as the host ground are quality control tests to check that it has the required properties such as the moisture content and the undrained shear strength for the column installation.

The aggregate index tests were performed to compare the primary and the recycled aggregates and to assess their suitability for the use in the context of VSCs. Regardless of the results of the aggregate index tests, various primary and alternative aggregates are used in the construction of VSC. The index tests can assist in analysing the behaviour of the material under specific loads. Evaluation of these tests is also explained in chapter 4 which is further completed in the results and discussions.

Chapter five presents the results and discussions of the material tests. Results of the clay tests are provided followed by the discussions. In case of the aggregates, the results are presented and discussion includes comparison of the results with other published

research. According to the results of the aggregate index tests, some of the material sources used in this research may have unpredictable behaviour in context of VSC.

Chapter six is the unit cell testing in which the materials tested is used to form the stone columns in the two small and large cells. Assumptions, limitations, measurements, instrumentations and preparations of columns constructed are fully explained for both of the unit cell tests. In the large cell, 15 tests were performed where various primary and recycled aggregates were compared for their load-settlement behaviour.

In the small unit cell, three series of tests were performed; in the first series various recycled and primary aggregates were compared during installation and loading. Crushing of the aggregates was measured at each stage for these materials (i.e., objective 6).

In series two and three the primary aggregate was used to form the column and in second series the time of installation was varied to study the effect of installation on the performance of columns. In the last series of the small unit cell tests, fines were added to primary aggregate to form the column and the effect of the contamination with fines was studied when the column was loaded (*i.e.*, objective 6).

Chapter six includes tables of all the tests performed both in the large and small unit cells, followed by the explanation and differences of each of the tests.

In chapter seven, the results of the large unit cell tests are presented, followed by the discussions in which the aggregate index tests, the column density, the particle size distribution and the angle of shearing resistance of the material were used in the interpretation and comparison with other published work. Comparison of the large unit

cell results was used to assess the performance of the recycled aggregates in the VSCs in the short-term.

In chapter eight, three series of the small unit cell results were presented and compared. Various primary and recycled aggregates were compared during the installation and loading in terms of crushability. The effect of the installation time or energy on the column formation and load carrying capacity on the primary aggregates was discussed and compared to the published work; and finally, the contamination of the primary sources with fines was analysed in the small unit cell tests. The shape of the columns constructed under installation or loading was compared for the small unit cell tests for further analysis of the behaviour of the columns in the short-term.

Chapter nine summarizes the conclusions of the research, in which the performance of recycled aggregates was studied under controlled installation and loading conditions. Conclusions cover the aggregate index tests, their relation with the unit cell tests, performance of the columns under static loading in the unit cell tests and comparison of the various columns constructed using various materials. Also, the effect of the condition of the aggregates (wet/dry and contamination with fines) on the performance of a single stone column under static loading was described.

In this chapter recommendations are made for future research in this area, using other sources of alternative aggregates and adding more factors to the study in the unit cell testing such as the effects of the contamination with fines in the recycled aggregates and the long-term performance of the VSC in unit cell testing.

1.6 Summary

This chapter summarized the background on the performance of the VSCs, where the unknown areas were discussed. The aim and the objectives were explained followed by the stages of the laboratory programme.

CHAPTER 2

LITERATURE REVIEW ON PERFORMANCE OF VIBRO STONE COLUMN

2. LITERATURE REVIEW ON PERFORMANCE OF VIBRO STONE COLUMN

In this chapter the background information on the vibro stone column (VSC) as a common ground improvement technique has been provided. The behaviour and the failure mechanisms together with the discussion of the impacts of installation, design and materials on the performance of VSCs are critically reviewed.

This chapter provides a general review on the current state-of-the-art of VSC technique highlighting the most important factors affecting the performance in the short and long term to be further discussed in chapter 3.

2.1 Ground improvement and vibro techniques

2.1.1 Introduction to ground improvement

As suitable construction area is not always available, engineers need to modify the ground based on the technical requirements of each project (Zomorodian and Eslami, 2005). In addition, environmental issues are becoming more important on all aspects of construction and in turn geotechnical engineering (Mitchell and Jardine, 2002).

Egan and Slocombe, (2010) and Rogers *et al.* (2012) captured the essence of ground improvement in terms of improving the ground condition and to control the cost, social and environmental aspects (*i.e.* sustainability) of the projects. In the UK these methods are used to treat a range of different ground conditions such as fills, alluvial and other weak soils and problematic ground conditions to improve the stability, bearing capacity and settlement behaviour of the ground (McKelvey and Sivakumar, 2000).

Ground Improvement methods include a variety of treatments such as vertical drains, jet grouting, and vibro techniques (Woodward, 2005). The technique used can be selected according to the project requirements to increase the bearing capacity and the overall stability and reduce settlement and/or to control ground water (Woodward, 2005; Raju and Valluri, 2008).

Ground improvement methods were divided into four main categories of mechanical (modifying and altering the soil by changing the stress and loading conditions), chemical (changing the chemical composition of the soil and therefore its characteristics), hydraulic (by improving the drainage and the permeability of the soil) and reinforcement (improving the tensile and compressive strength of the ground through its structural form) based on the nature of modification (Mitchell and Jardine, 2002).

Most of the design is empirically or semi-empirically based (Kirsch and Sondermann, 2003). Thus, usually field trials are used to either evaluate the method adopted or to achieve more accurate quality assurance (BRE, 2000).

In addition, there are several laboratory tests and finite element based packages that can be used to improve the design analysis (Kirsch and Sondermann, 2003). There are several assumptions used in the design and the construction of various ground improvement methods, which generalize the field conditions and therefore, there is the constant need for re-evaluation of the design theories (Weber *et al.*, 2006).

2.1.2 Vibro stone column

Vibro techniques were first used in France by the military engineers in the nineteenth century and was forgotten until the 1930s where it was used again for the construction of autobahns in Germany (McKelvey and Sivakumar, 2000). Since then vibro stone column (VSC) has become one of the most globally used deep compaction methods (McCabe *et al.*, 2007). This method is currently the most common ground improvement method used in the UK (Serridge, 2006) which is a relatively economical alternative to the conventional piling methods for less settlement sensitive structures (Weber *et al.*, 2006).

VSC is used for many foundation situations (ICE, 1987); such as light structural foundations, embankment stability and controlling the liquefaction potential in seismic areas (McKelvey and Sivakumar, 2000). This method is also suitable for soft cohesive soils both economically and technically (McCabe *et al.*, 2009).

2.2 Vibro compaction and vibro replacement

2.2.1 Vibro compaction

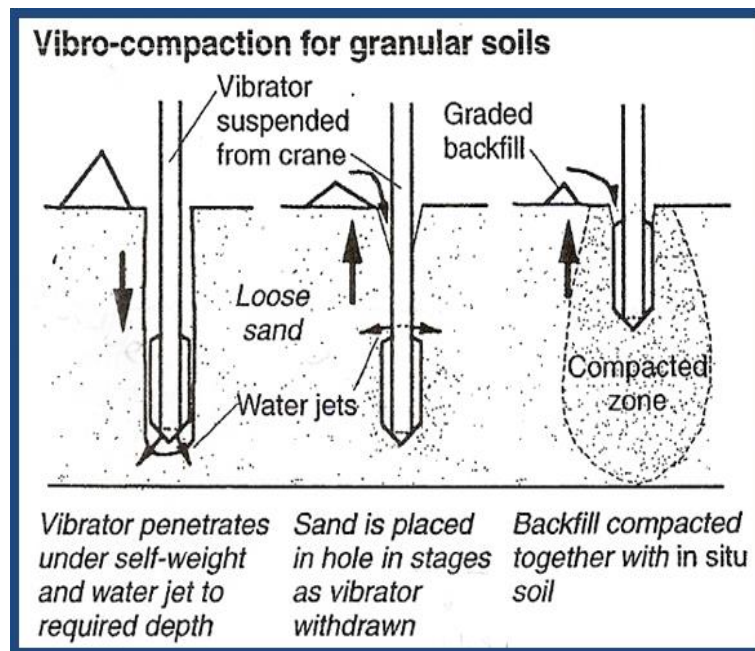
As illustrated in Figure 2.1, in vibro compaction method a vibro-float or poker penetrates the ground through its self-weight and via the air or water jet (Woodward, 2005). The vibrations and penetration shake the soil grains into a denser position (Raju and Sondermann, 2005). As a result, the compressibility (Van Impe *et al.*, 1997) and the density of the ground is improved (McKelvey and Sivakumar, 2000). This method provides immediate drainage for granular soils and dissipates the excess pore water pressure quickly (Raju and Sondermann, 2005).

For granular soils, the vibro compaction densifies the ground and therefore reduces its settlement and liquefaction potential (Adalier and Elgamal, 2004), and consequently increases the bearing capacity and the stability of the ground. However, Mitchell and Jardine (2002) reported that when the percentage of the fines present in the soil is more than approximately 15 to 20 percent (which is estimated based on several case studies), compaction becomes more difficult and limited improvement is achieved and can generate significant excess pore water pressures (Mitchell and Jardine, 2002). In practice quality control tests are usually conducted one week after the compaction process has finished as the soil gains higher strength with time due to excess pore water pressure dissipation (Schmertmann, 1993).

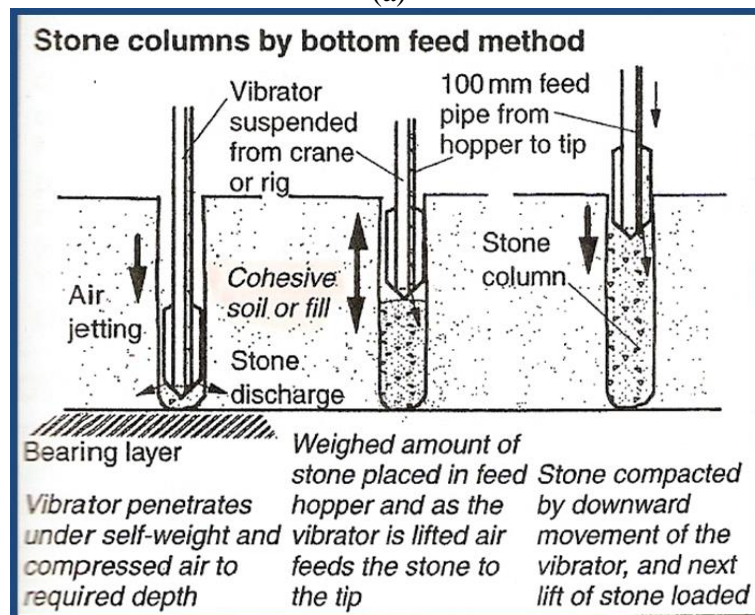
2.2.2 Vibro replacement

When fines content in the soil exceeds 15 to 20%, the soil is replaced by stones or gravel which is poured in stages; a process called vibro-replacement. At each stage the aggregates are vibrated into a dense state. The column-soil composite formed reduces

the settlement and the compressibility of the ground as well as increasing the bearing capacity, the stiffness and the shear strength of the soil (Charles and Watts, 2002). The ductility of the column material makes the application of higher loading possible (Raju and Sondermann, 2005). The best results are usually achieved where a bearing stratum exists (Barksdale and Bachus, 1983); (refer to Figure 2.1 (b)).



(a)



(b)

Figure 2.1: Vibro techniques: (a) vibro compaction and (b) vibro replacement, VSC (Woodward, 2005)

Another form of the vibro replacement techniques used is the vibro concrete column; however this is not the subject of this research. This method is an adaptation of the VSC and more details can be found in Charles and Watts (2002).

2.3 Applications and limitations of VSC

2.3.1 Applications

Based on several case studies the vibro techniques can be used for a wide range of soils (refer to Figure 2.2) and for various projects and applications such as landfills, embankments, highways, airports, railways, slope stability and bridge abutments (McKelvey and Sivakumar, 2000). VSC can be cautiously used in very soft marine clays, thin layers of peaty clay and clays from mine tailings (Raju and Sondermann, 2005).

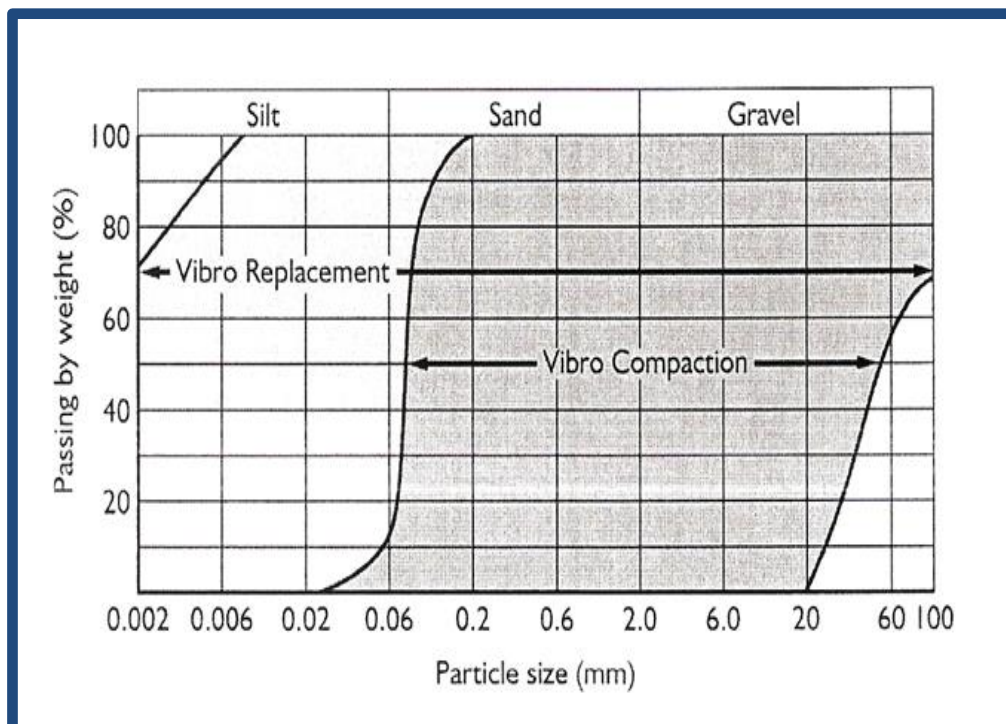


Figure 2.2: Range of soils suitable for vibro compaction and vibro replacement methods (Mitchell and Jardine, 2002)

2.3.2 Limitations

Use of the VSC is limited in a number of situations including:

- A soil with organic content; for instance a soil containing peat layer with a thickness more than the column diameter which can shrink in the long term due to storing the moisture content ten times its weight, and therefore causing excessive settlements due to the long term excess pore water pressure dissipation (Waltham, 2009).
- Also, it is not recommended to use the VSCs where the soil has undrained shear strength (c_u) values less than 15kPa as it may not provide the sufficient strength for the process of the installation of the columns (Priebe, 2005). Although in some cases the VSC has been successfully used for the undrained shear strength values as low as 5kPa (Priebe, 2005). Raju (1997) reports the construction of VSCs in very soft soils with the undrained shear strengths of less than 10kPa; although it is emphasized that the quality control and constant monitoring are keys for the success in such conditions (Raju, 1997).
- If the plasticity index (PI) is low, the soil is sensitive due to large strength changes with a small change in the moisture content. Therefore, PI values of 40% or higher are recommended for the soils in which the VSCs are to be designed and constructed (McCabe *et al.*, 2007).
- Clay fills or loose fills cause extra settlements which are not desirable in the long term, therefore the long term settlements should be considered in the design in such conditions to avoid unpredictable long-term failures (McCabe *et al.*, 2007).

2.4 Mechanism and failures of VSC

2.4.1 Mechanism

As shown in Figure 2.3, when the stone column is loaded, this load is transferred to the column material. With VSC, the controlling mechanism that achieves the improvement is primarily the column bulging (Barksdale and Bachus, 1983) which causes lateral deformations into the surrounding soil after the initial vertical deformations have taken place. After a small amount of movement the soil resists the bulging in the lateral direction through the lateral restraint that is developed in the ground.

In order to achieve the resistance, the column material should have appropriate shear resistance and the particles must bear stress concentrations in the column (Jefferson *et al.*, 2010). The stiffening of the ground due to the bulging occurs up to the critical length (Hughes and Withers, 1974; Wood *et al.*, 2000) that is defined as the length up to six times the diameter of the column (refer to Figure 2.4) (McKelvey *et al.*, 2004). Consequently, consolidation takes place, followed by further small movements until the system reaches an equilibrium condition (Barksdale and Bachus, 1983).

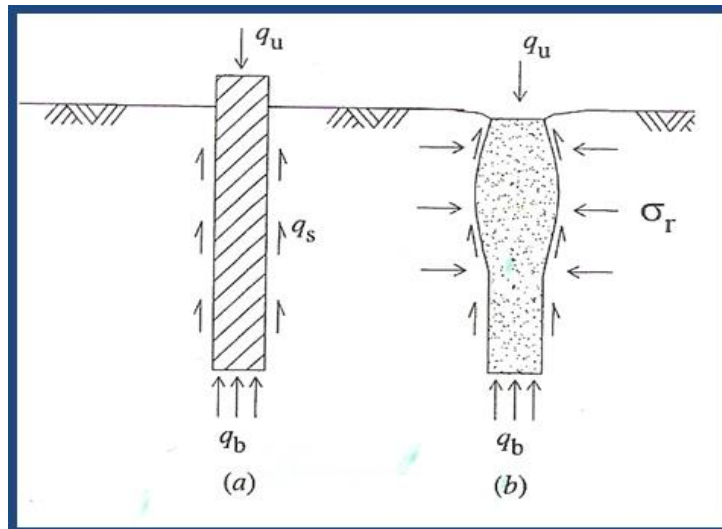


Figure 2.3: (a) rigid pile and its reactions to the loading, (b) the bulging and loads equilibrium on stone column and soil composite (Hughes and Withers, 1974) (σ_r is the radial stress on column)

2.4.2 Failure modes

There are two types of columns constructed based on the length of the column and resistance forces developed in them (Barksdale and Bachus, 1983):

- End-bearing (full depth) which reaches a firm, supporting stratum and
- Floating (partial depth) which will resist the forces with side friction

As shown in Figure 2.4 the columns can be short or long, and based on their slenderness ratio which is defined as the ratio of the column diameter to the column length (McKelvey *et al.*, 2004), the following types of failures may occur:

- a) Bulging failure; in which the column is overlying a bearing stratum. When the column is loaded, the column bulges and the lateral stresses in the ground increase and eventually reach equilibrium
- b) Short columns ($L/D \leq 6$, where L is the column length and D is the column diameter (McKelvey *et al.*, 2004)) overlying a bearing stratum may undergo local shear failure
- c) Short columns on a weak stratum may fail in the end bearing or the punching failure before the bulging happens

Both the end-bearing and the floating columns may fail in bulging within the critical length (Hughes and Withers, 1974). For the short end-bearing type, if the column is bearing on a weak strata, the local bearing capacity failure may occur (before the bulging happens) which should be considered in the design process. If the columns are not taken to a sufficient depth, the punching shear failure may also occur (Barksdale and Bachus, 1983).

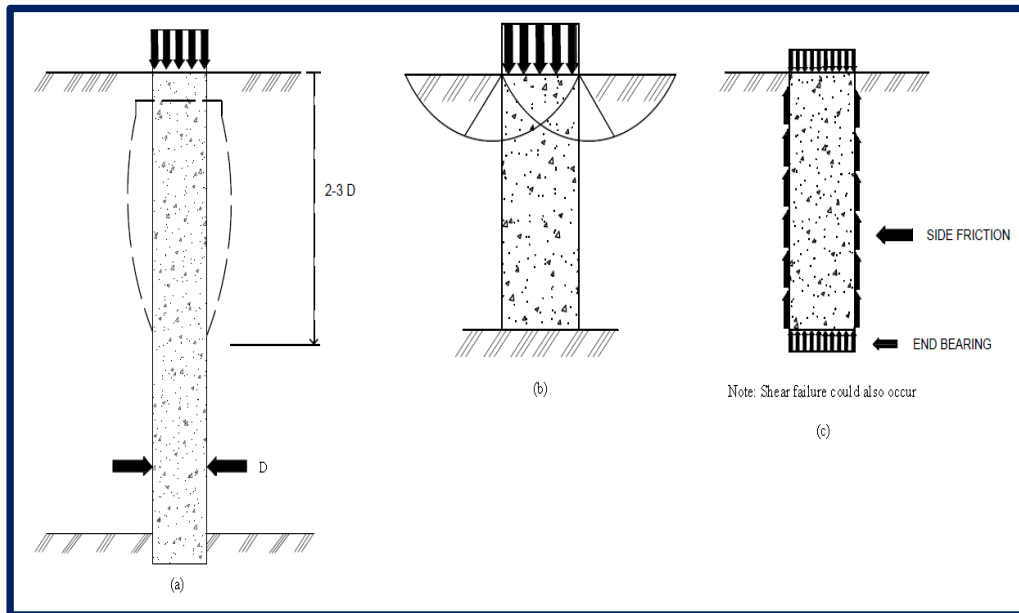


Figure 2.4: Types of column failure (Barksdale and Bachus, 1983)
 (a) Long stone column with firm or floating support-Bulging failure, (b) Short column with rigid base-Shear failure, (c) Short floating column-Punching failure

Laboratory modelling and research on the single and group of columns have shown that a single column has lower ultimate load capacity than a column in a group; as the neighbouring columns have effects on the bulging and enhance the lateral restrains and the equilibrium of each other (McKelvey *et al.*, 2004). There have been several studies on the behaviour and failure mechanisms of a single or group of columns via physical modelling by Wood *et al.* (2000), McKelvey *et al.* (2004) and Black *et al.* (2007a) which are discussed in chapter 3 (refer to sections 3.3.3.1 and 3.3.3.2).

2.5 Construction of vibro stone columns

2.5.1 Types of installation

There are 3 main types of VSC installation: the dry top feed, the dry bottom feed and the wet method (top feed) (BRE, 2000). The dry or wet methods are defined with respect to the air or water being used in installation process. The top feed and the bottom feed methods are demonstrated in Figure 2.5, where the aggregates are charged into the ground from the top or from the base of the vibro-float, respectively.

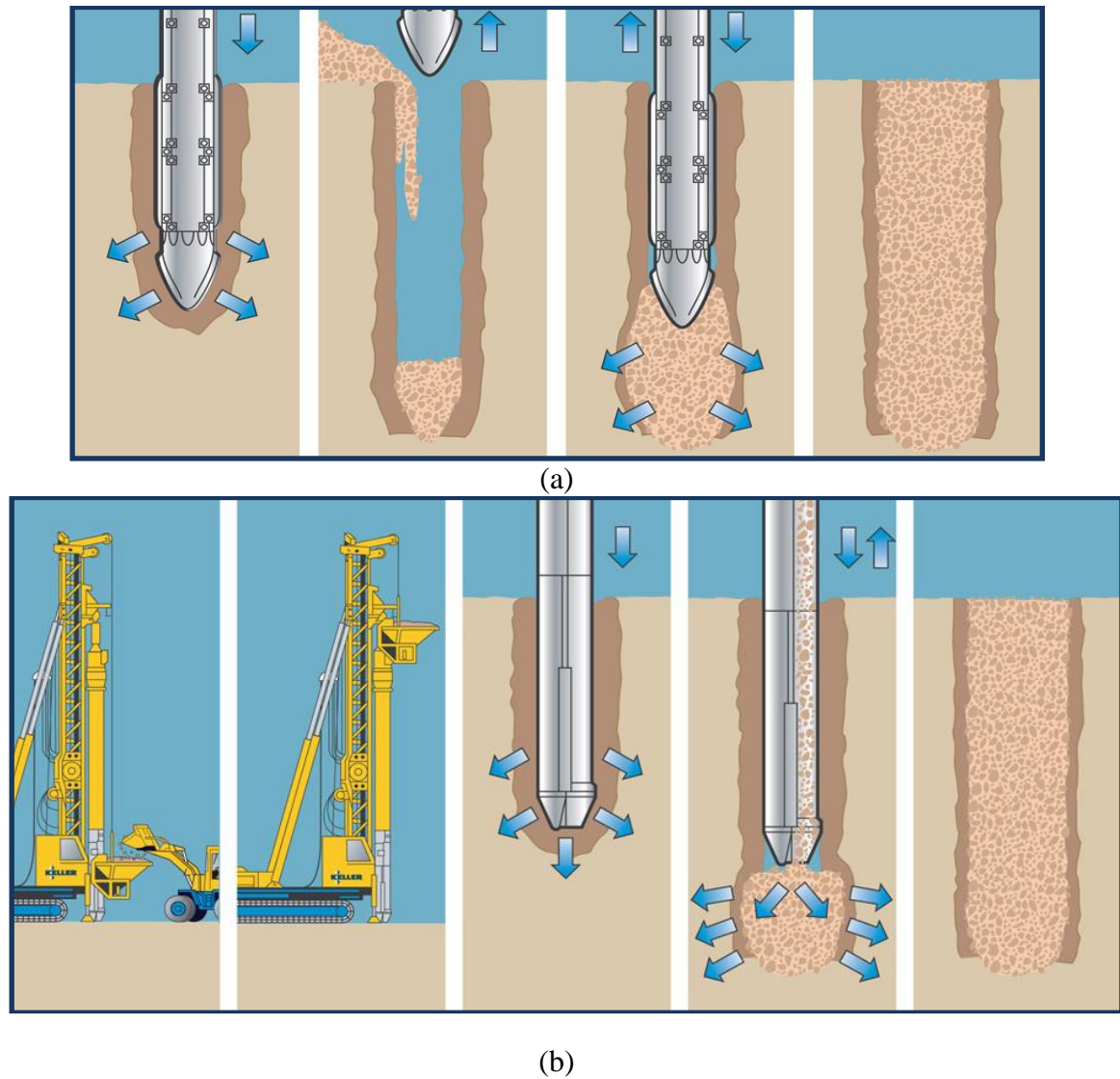


Figure 2.5: (a) Top feed and (b) bottom feed methods of VSC construction
www.keller.co.uk

2.5.2 Vibro-float

The installation process is carried out by the means of a large vibrating poker which consists of an eccentric weight causing vibrations in the lateral direction as illustrated in Figure 2.6. The poker itself consists of a horizontally oscillating base called the ‘vibrator’, attached to an isolator and extension tubes (BRE, 2000).

Contractors use various types of vibro-floats with different sizes and powers. The weight of the vibrator can vary between 15 to 40 kN. The motor can operate electrically

or hydraulically with a typical power range of 50 to 150 kW, up to 200 kW (www.penninevibro-piling.com). Due to the power and frequency of the vibro-float, a load of around 150 to 700 kN can be transferred into the ground depending on the system used (Raju and Sondermann, 2005).

These typical values are only measured when the vibro-float is suspended in the air. The performance can differ depending on the type of the soil the vibro-float is exerting its forces to. There are various parts of the vibro-float such as the extension tubes and the water or air jet pipes that can be different for various machines. But the mechanism is the same (Raju and Sondermann, 2005). The water or air jet creates radial forces to assist the penetration and in practice it is observed that the fluid flow rate is a more important factor than the fluid pressure (Raju and Sondermann, 2005). Also, it is observed in many cases that the water assists stronger penetration for the vibro-float resulting in a larger column diameter (Hughes and Withers, 1974); on the other hand, the dry method has the advantage of not requiring supply and disposal of the water and therefore, can be easily used on sites with limited access (McCabe *et al.*, 2009).

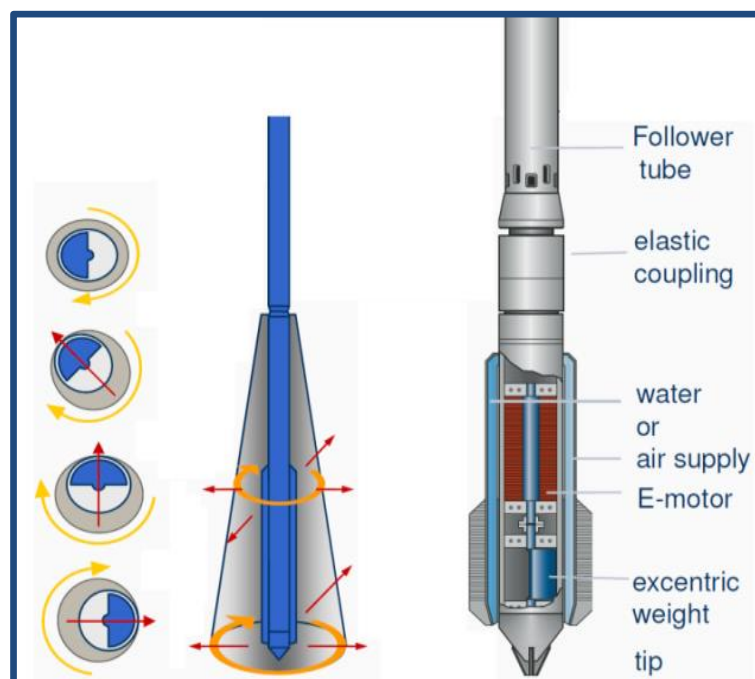


Figure 2.6: Deep vibrator movements and its various elements (www.keller.co.uk)

2.5.3 Column formation

When the vibro-float penetrates the ground to the required depth; due to the poker penetration a cylindrical hole is created in the soil (BRE, 2000) which is backfilled with the material at stages, usually at intervals of 300 mm (BRE, 2000). Each stage is compacted for 30 to 60 seconds or until the pre-defined amperage of the vibro-float presenting the level of densification is achieved (Raju and Sondermann, 2005). The vibro-float is inserted and retracted at these stages to achieve the design requirements for the column diameter, depth and density (Priebe, 1995).

The column constructed has a diameter range of 0.7 to 1.1 metres and the centre to centre spacing of the columns is usually between 1.5 to 2.5 metres. The designed depth can vary between 6 to 20 metres, but greater depths have also been constructed (Raju *et al.*, 1997). (McKelvey *et al.*, 2004) suggest that increasing the column length to more than six times its diameter will not increase the load carrying capacity of the column and therefore, an optimum design depth exists.

2.5.4 Installation effects

The stone columns formed should provide sufficient interaction with the surrounding soil (BRE, 2000). The three installation methods create different columns in the ground. Based on the studies by (McCabe *et al.*, 2009), the improvement factor defined as the ratio of the unimproved soil settlement to the settlement of the improved ground (Priebe, 1995) calculated or predicted is different from the improvement values measured in the field.

Based on the database provided for widespread loadings on foundation, the settlement predicted and measured was calculated and the results were presented in Figure 2.7 (McCabe *et al.*, 2009). These cases used different installation methods. When the

improvement factor measured in the field is more than the value predicted, it means that the settlement has been improved more than Priebe's method predictions.

The bottom feed method shows more improvement in practice and the theoretical calculations. The problem of this graph is that the results are produced for the widespread loading and footings on VSC only, where similar analysis is required for the columns under pad or strip foundations (McCabe *et al.*, 2009). Also, the database is limited to a few cases available in the study; however, the results obtained show close predictions by Priebe's method.

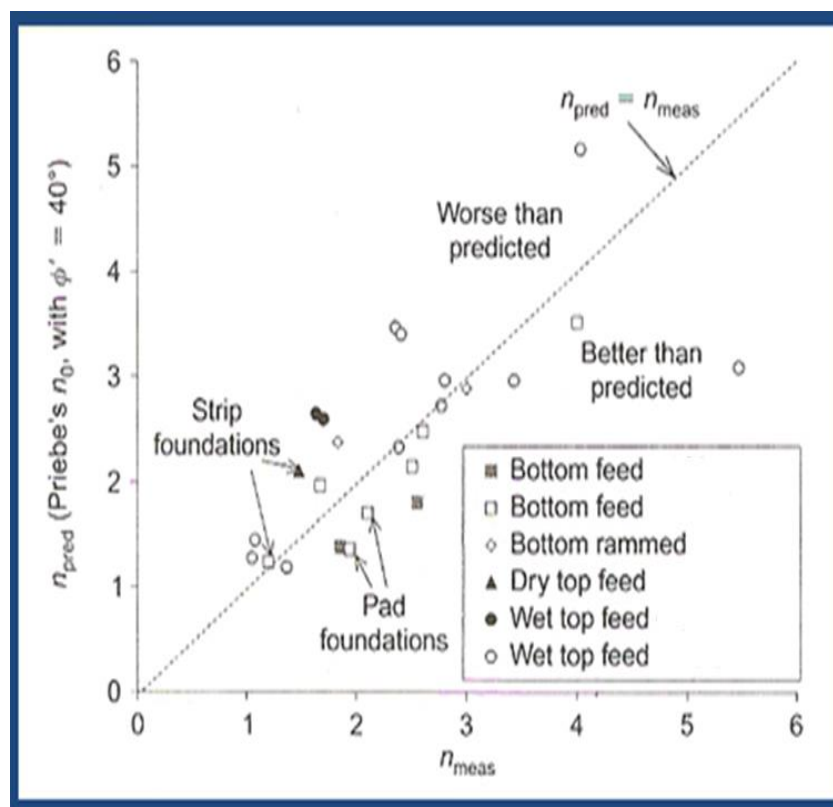


Figure 2.7: Predictions and measured settlement improvement factors for widespread loading and footings, with different installation methods used (McCabe *et al.*, 2009)

In another study by Douglas and Schaefer (2012), a bigger database of 250 cases was used to evaluate the reliability of Priebe's method of settlement prediction based on the

actual measurements of the settlements on field. It was concluded that in the various cases studied, Priebe's method is 89% conservative for the settlements of up to 80mm. However, there are cases where this method underestimates the values of the settlement and it is suggested that proper site investigation and consideration of unique response of the ground to the installation equipment are the critical factors in the prediction of the settlement behaviour of the ground treated by VSC.

Table 2.1 summarizes the different installation methods and their applications and limitations:

Table 2.1: Different installation methods and their specifications (BRE, 2000; Raju and Sondermann, 2005; Serridge, 2006)

Method	Ground conditions	Depth of the column constructed up to	Diameter of the column constructed	Suitability for different GW* condition	Material (Stone) properties for column	Advantages and disadvantages
Dry top feed	<ul style="list-style-type: none"> -Not suitable for cohesive soils -Suitable for insensitive and stable soils -Shear strength should be more than 30kPa 	-10 metres is typical (could be extended to 20-30 metres)	0.4-0.8 metre	No recommendations are provided	<ul style="list-style-type: none"> -Grading: 40-75 mm -Angle of shearing resistance: 40-45 degrees is recommended in the UK -More angular particles are also applicable in top feed method 	<ul style="list-style-type: none"> -Hole remains open during construction -Air improves stability
Dry bottom feed	<ul style="list-style-type: none"> -Suitable for soft cohesive soils -Shear strength between 15 to 50kPa is acceptable 	Exceeding 15 metres	No specific diameter suggested	Suitable for layers below ground water level (GWL)	<ul style="list-style-type: none"> -Grading: 10-50 mm -Angle of shearing resistance: 40-45 degrees is recommended in the UK -Rounder and smaller particles are recommended to ease the feeding through the bottom of poker 	<ul style="list-style-type: none"> -Hole stability is assured -Assures that column diameter is being constructed particularly at each depth -Air improves stability
Wet method	<ul style="list-style-type: none"> -Suitable for soft cohesive soils -also suitable for fully saturated soils -Suitable when hole is unstable in the usual ranges of undrained strength of 15 to 25kPa (Priebe, 1995) 	10 metres typical (could be extended to 20-30 metres)	0.5-1.0 metre	Suitable for layers below GWL	<ul style="list-style-type: none"> -Grading: 25-75mm -Angle of shearing resistance: 40-45 degrees 	<ul style="list-style-type: none"> -Water maintains the annulus and the hole stable (water flow rate is important) -Poker hangs freely, therefore, diameter bigger than designed is achieved -Not sustainable when water supply and disposal is not available -Nowadays only used for very weak soils -Compared to the other two methods is not environmentally preferable

* Ground water (GW)

2.6 Design of vibro stone column

2.6.1 Unit cell concept

The design philosophy of the VSC is related to its bearing capacity, settlement and also the key failure mode of bulging (Baumann and Bauer, 1974). The concept of unit cell idealization was developed (Barksdale and Bachus, 1983) to define the area that the stress concentrations can be calculated for (McKelvey and Sivakumar, 2000). The stone column and the equivalent area of the soil around it form the unit cell are shown in Figure 2.8 (a). The diameter of the unit cell (D_e) is defined for two common grids of VSC construction (triangular and square). According to Figure 2.8 (b) based on the geometry and the influence of the column; D_e is defined as 1.05 and 1.13 times centre to centre spacing (S) of the columns for the triangular and the square grids, respectively.

Both arrangements can be used for the design of the VSCs depending on the foundations layout and the loads applied; however, using a simple analysis of applying the same loads over both areas of the triangle and the square in the same ground conditions can reveal that the triangular arrangement might provide a more stable pattern compared to the squared one for the construction of VSCs.

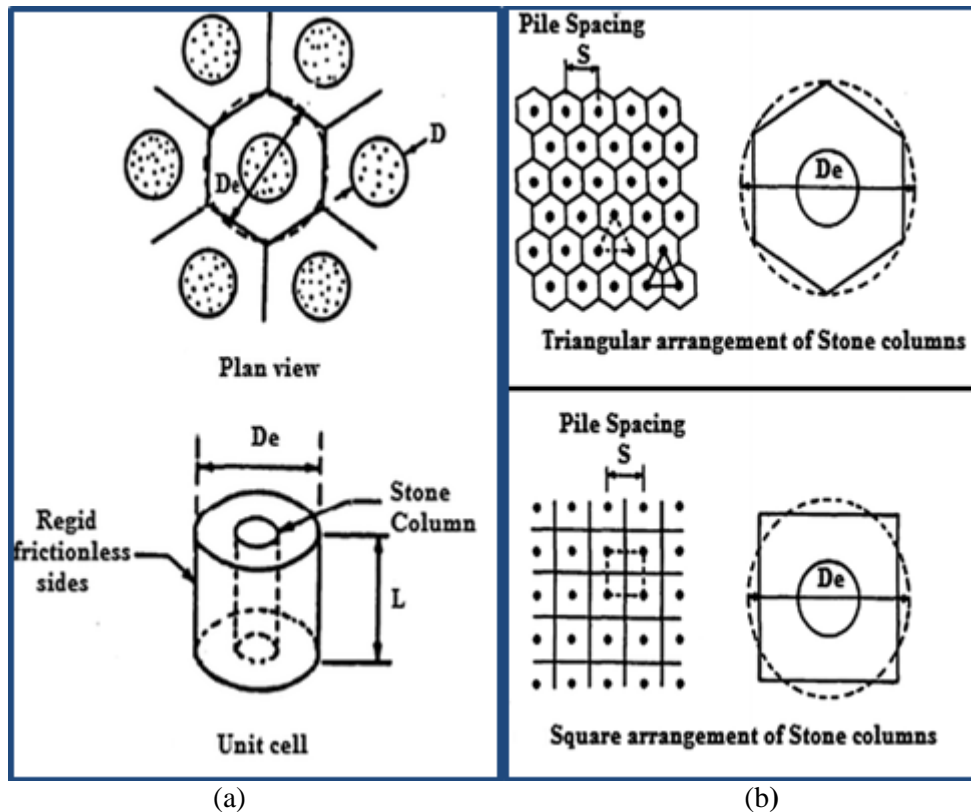


Figure 2.8: (a) unit cell concept (b) unit cell diameter for triangular and square grids of column installation (Barksdale and Bachus, 1983)

The three following stages are commonly used for the design of VSCs in the UK:

2.6.2 Bearing capacity of single column

Hughes and Withers (1974) developed the basic approach to the design based on the laboratory testing of a series of Leighton Buzzard sand columns in Kaolin clay, under a uniform anisotropic stress field. The vertical distortion upon loading was expanded up to 4 times the column diameter, therefore, if the column length is less than $4d$ (d is the column diameter), then it will fail due to the end-bearing rather than the bulging.

The horizontal distortion expands up to 2.5 times the column diameter; therefore the neighbouring columns may affect the horizontal distortions of the other columns.

The ultimate strength of the column and the surrounding soil is a function of the aggregates used (as the column material) and the maximum lateral restraint of the soil around the bulging zone (McKelvey and Sivakumar, 2000).

Based on this approach (Hughes and Withers, 1974) the key factors affecting the load carrying capacity of a single column are the angle of shearing resistance of the aggregates and the lateral confinement pressure exerted by the surrounding soil.

2.6.3 Factor of safety against bulging failure

In the UK the bulging is calculated according to Bauman and Bauer (1974) method. For the bulging failure, the important factor is the ratio of the stress distribution between the column and the soil; and relates to the A_0 (area of influence) and the centre to centre spacing of the columns. The area of influence can be defined using the unit cell idealization concept (Barksdale and Bachus, 1983) in which the column and the soil surrounding it are considered as a composite element (refer to Figure 2.8).

2.6.4 Settlement reduction factor

Priebe's method is a most commonly used analysis for the settlement predictions of VSCs (Serridge, 2007). There are three main assumptions in Priebe's method in order to calculate the settlement of VSCs:

Firstly, the column is assumed to be overlying a rigid layer and therefore no end-bearing failure occurs.

Secondly, the column material is assumed to be incompressible;

Finally, the bulk density of the column and the soil are neglected. Based on these unrealistic assumptions it can be concluded that the column does not fail due to end bearing, and therefore, the settlement of the column is due to bulging only and is constant over the length. The surrounding soil is elastic when the column shears.

The modified Priebe's method produces a settlement reduction factor which is related to the angle of shearing resistance and the compressibility of the column material and the area replacement ratio. Priebe's method has been known to be too conservative in many studies (McCabe *et al.*, 2009; Douglas and Schaefer, 2012), which were presented in detail in section 2.5.4.

According to the stages of the design of VSC, the angle of shearing resistance of the column material is a key factor in the behaviour and the performance of stone columns.

2.6.5 Modifications of Priebe's method

After the first publication of Priebe's method in 1976, the improvement factor was modified several times.

At first, the effect of the compressibility of the column material was considered (Priebe, 1995). Accordingly, the curves showing the factors affecting the settlement were modified (Priebe, 1988; Priebe, 1990; Priebe, 1991). In the later years the depth factor was added to the calculations to allow the effects of the unit weights of the soil and the column to be taken into account (Priebe, 1995).

In the year 2005 the end bearing column assumption (section 2.6.4) was modified (Priebe, 2005). Based on this modification the floating column does not act like a floating pile where the load might cause the punching failure (Barksdale and Bachus, 1983). Some of the load is transferred through the column length and therefore, values of the punching settlement caused by the load are a lot less compared with those of associated with a pile (Priebe, 2005).

The overall settlement of the floating column is calculated based on the settlement of the treated area plus the settlement in the form of punching and the settlement of layers below the column (Priebe, 1995).

2.6.6 Critical reviews on Priebe's method

Based on (Ellouze *et al.*, 2010), the Priebe's method has limitations in the settlement calculation. They showed that the assumptions made and used in Priebe's method are not clearly defined. Also, the different publications have used their own interpretation of the formula (Ellouze *et al.*, 2010), which has led to confusion and incorrect calculations. In several studies by Weber (2004) and Weber *et al.* (2006), these aspects have been modelled using a series of laboratory tests. The installation effect and uneven settlement of embankments on the column grids were added to the Priebe's method (Weber *et al.*, 2006).

Other settlement calculation methods have been used (Ellouze *et al.*, 2010) to evaluate Priebe's method in other studies (Dhouib A *et al.*, 2004; Dhouib and Blondeau, 2005). The results demonstrate different values from Priebe's method (Ellouze *et al.*, 2010). In most cases the various methods are in general agreement with Priebe's results, although it is observed that Priebe's method might provide slightly conservative values of settlement (Elshazly *et al.*, 2007).

2.6.7 Other design methods

There are several alternative empirical, semi-empirical, analytical, numerical and composite cell theories that can be used for the different aspects of design (Bouassida *et al.*, 2009). Empirical or semi-empirical methods are widely used. For instance Hughes and Withers (1975) method is based on the plasticity theory. Therefore, field trials can assist for the site specific design (BRE, 2000); also, appropriate site investigation may

assist in the more accurate observation of the ground profile and relatively more accurate design (Charles and Watts, 2002). A few other methods are briefly introduced in Table 2.2:

Table 2.2: Alternative bearing capacity design methods

Name	Method	Basis*	comments
(Thorburn and MacVicar, 1968)	empirical	Relates undrained shear strength of soil to allowable working load	Results are in agreement with Hughes and Withers (1974)
Barksdale and Bachus (1983)	empirical	Cavity expansion theory	Used for ultimate bearing capacity of a single column
Priebe (1995)	empirical	Load carrying capacity is a function of area replacement ratio (which is the area of columns to the area of treated ground)	-
Greenwood (1970)	empirical	Graphically relates the consolidation settlement to column spacing and undrained shear strength of clay	-
Aboshi <i>et al.</i> , (1979)	Equilibrium method	Uses one dimensional consolidation theory	Is not recommended for settlement calculations in soft clays
Goughnour and Bayuk (1979)	Incremental method	Load is applied to column constructed using wet method in the field as well as using incremental modelling	-Predicted stress and settlement values agree with field results -Used for embankment type loading conditions

* These methods cannot be directly compared to each other. The methods are assessing the other existing design methods and each has specific assumptions and analysis; therefore, direct comparison of the factors studied and the results obtained is not possible as each case is unique.

2.6.8 Critical factors in design

1) Angle of shearing resistance

The angle of shearing resistance of the column material is an important factor in the design for the bearing capacity (Hughes and Withers, 1974) and the settlement calculations (Priebe, 1995). In the UK based on the specified range of the materials used, quality of workmanship, capacities and particle natures, the values of angle of

shearing resistance considered are between 40 to 45 degrees (Serridge, 2006), and as the fine percentage increases, the SRF (settlement reduction factor) reduces (explained in section 2.6.4).

2) Condition of column material

McKelvey *et al.* (2002) studied the effect of the condition of the aggregates (dry, wet, 10 and 20 percent fines) on the performance of VSC in a shear box test. The materials tested were crushed basalt (a primary aggregate), crushed concrete, building debris, and quarry waste (recycled aggregates). The results show that the recycled aggregates have lower shear strength than the virgin aggregates; also, their volume is reduced during the shear test at the high pressures due to the crushability of the material and the reduction in the angle of shearing resistance.

3) Host ground limitations

It should be noted that in soft soils, the settlement criterion is more critical than the bearing capacity of VSC (McCabe *et al.*, 2009). If the grid of the columns designed is non-uniform, differential settlements can occur (Al-Khafaji and Craig, 2000).

4) Geometry and loading of columns

Geometrical characteristics, such as the column length, the centre to centre spacing, and the column designed in a group or a single column, the foundation layout and the loading type, the floating or end bearing design and several other factors affect the design process (Priebe, 1995; Al-Khafaji and Craig, 2000; Wood *et al.*, 2000; McCabe *et al.*, 2007) and ultimately the performance of VSCs. It has been observed in the various cases that the wide loading such as embankments provides better performance compared with the strip or pad foundations (Wood *et al.*, 2000).

5) Site investigation and quality assurance

The design should include a review of all the factors likely to influence the performance starting with proper site investigation (Raju and Sondermann, 2005). Several design assumptions, such as the level of improvement achieved on site, can often be verified only during or after the construction (BRE, 2000), and this needs to be reflected in the approach to work by constant monitoring and quality control (Bell, 2004) .

2.7 Material used for vibro stone column

VSC improves the ground due to its composite nature (Charles and Watts, 2002). VSC materials need to meet several specifications to provide the support and the reinforcement in the ground (BRE, 2000) and also, provide the drainage path for the surrounding soil, which accelerates the consolidation rate (Schmertmann, 1993).

During the column installation the aggregates are charged at stages, and compacted (BRE, 2000). When the installation is completed the lateral restrains and the shearing forces are carried through these aggregates. Therefore, whatever the source of the aggregate is, lots of aspects, from the storage and supply, the grading, the grading compatibility with the installation method used, the contamination and smearing with the fines due to the storage or the installation process, the condition (wet or dry) and the hardness may affect the performance of VSCs both in the short and the long term (Serridge, 2006).

2.7.1 Primary and alternative aggregates

In general the source of the aggregates used for VSC may be either of the following categories:

- 1) Primary aggregates (PA), traditionally used in the construction of vibro stone columns, a natural material that has not been processed except for the crushing

and or the grading for its intended purpose (Tranter *et al.*, 2008). This includes quarried aggregates such as granite, basalt and also gravel.

- 2) Recycled aggregates (RA) are the material provided from previously used sources in construction and therefore have been subjected to reprocessing (Steele, 2004). Examples are recycled concrete and old railway ballast (Serridge, 2006).
- 3) Secondary aggregates (SA) can be defined as by-products of industrial processes that have not previously been used in construction (Steele, 2004); more accurately these are divided into two categories: 1) from manufactured sources, e.g. PFA: Pulverized Fuel Ash and metallurgical slags and 2) SA from natural sources, e.g. China clay, sand or slate aggregate (Jefferson *et al.*, 2010).

For many years the PA or natural or virgin aggregates have been used in the construction of VSC (Jefferson *et al.*, 2010), but nowadays due to the importance of sustainable construction, there are clear legislations regarding no waste policies in industries around the globe (Schouenborg, 2005). In addition, the natural sources like sand and gravel are becoming scarcer (Jefferson *et al.*, 2010). Therefore, as geotechnical and ground engineering is an initial phase of almost every civil engineering project; it is necessary to study and consider the more sustainable options in the design and construction (Chidioglou *et al.*, 2008).

For installation process of VSC, the primary sources such as sand, gravel and crushed rock have been used for several years (Chidioglou *et al.*, 2009), but alternative aggregates may provide more sustainable choices (in terms of three pillars of environment, economy and social) (Jefferson *et al.*, 2010).

2.7.2 Guidelines on use of materials for VSC

2.7.2.1 General criteria

Regardless of the source used; there are several basic requirements for the material which are mentioned in various standards related to VSC such as BRE (2000), ICE (1987) and BSI (2005):

The material should be hard, stable and inert with proper grading, nominal single size of 20 to 75 mm (BRE, 2000); with specific shape, flakiness, interlocking and drainage effect (Jefferson *et al.*, 2010). The material should be “fit for purpose” (Serridge, 2005) and be able to withstand the long term static loads, the impact forces of the vibro-float and retain the long term integrity under the applied foundation loads (BRE, 2000).

For vibro stone columns, as the column material act as vertical drains, the nominal size of aggregates and the lack of fines improves the performance by accelerating the consolidation process (Charles and Watts, 2002).

2.7.2.2 Specific aggregate tests

The most important tests recommended by the standards are the aggregate impact value (AIV) (BSI, 1990e), the aggregate crushing value (ACV) (BSI, 1990f), the Los Angeles (LA) test (BSI, 2010) and the ten percent fines value (TFV) (BSI, 1990g). In the standards such as BRE and ICE, there are several criteria that are recommended when using aggregates; these are summarized in Table 2.3:

Table 2.3: Recommended tests for aggregates by BRE and ICE

Standards	Maximum fines by mass	AIV (BSI, 1990e)	ACV (BSI, 1990b)	LA (BSI, 2010)	TFV (BSI, 1990c)
BRE	5%	<30%	<30%	Not required	Test suggested but a specific value is not given
ICE	9%	Not required	Not required	≤ 50%	≥ 50kN (only if LA is 50%-60%)

It should be noted that the TFV test is withdrawn from the ICE (1987); and only the ACV, AIV and LA tests are recommended by this standard. The aggregates tested according to the above table should be nominal single size of between 20 to 75 mm (BRE, 2000).

Apart from the aggregates, the structure or the source which aggregates are provided from is critical in terms of the quality (Schouenborg, 2005) and the strength (Chidioglou *et al.*, 2009), but as it will be costly and time consuming to test the source thoroughly, it is vital to have appropriate quality control in sorting and testing of aggregates used instead (Schouenborg, 2005).

2.7.2.3 Comparing the standards

Generally the aggregates should have the appropriate grading (BRE, 2000), therefore, the particle size distribution (PSD) is one of the initial tests required for the use of aggregates in VSC suggested by both ICE and BRE, but the sieving method itself may affect the grading of the aggregates and the results may show more fine percentage than the actual percentage of fines in the source. Also, the sieving of large quantities is costly and time consuming (Steele, 2004).

In BRE (2000), the main hardness tests introduced are the AIV and the ACV; these tests do not take into account the effects of the porosity, the water absorption and the moisture content (Schouenborg, 2005) but are flexible tests regarding the crushing of aggregates during the construction (BRE, 2000).

In ICE (1987), for the purpose of determining the aggregate hardness, the Los Angeles (LA) test is mentioned which does not provide a representation of the actual

environment of the stone column; however can be related to the aggregate environment during the installation (Tranter *et al.*, 2008).

The standards state the grading with less than 10 percent fines in both the dry and wet conditions (ICE, 1987; BRE, 2000). Based on the design of VSC, one of the most important factors affecting the performance is the angle of shearing resistance of the aggregates (Serridge, 2006), which even 10 degrees reduction in its value, causes the reduction in bearing capacity and the settlement improvement values by 50 and 30 percent, respectively (Priebe, 2005). The crushing happening during the construction might also reduce the angle of shearing resistance value by crushing the aggregates and smearing them with fines which are reflected by the TFV test (McKelvey *et al.*, 2004).

As opposed to BRE (2000), the whole process of use of aggregates from the storage and supply, the testing, the site investigation and contamination with fines, is not considered in ICE (1987). The storage of the aggregates should be controlled as aggregates should not be subject to fine material (such as clay or dust); the percentage of fines in the source can result in a lower angle of shearing resistance of the material used and subsequently more settlement in the columns (Serridge, 2006).

The TFV test is common between ICE and BRE, but in BRE no specific value is suggested as the limiting criteria, while in ICE, the 10% fines value of $\geq 50kN$ is required for the soaked condition. Also, the TFV considers the long term impacts of the moisture content on the durability of the material if it is carried out on the saturated samples (Schouenborg, 2005). Due to the high porosity of the alternative aggregates, the short term tests may not be suitable to assess the water absorption (Schouenborg, 2005); the stone material may degrade or weaken when saturated (Steele, 2004).

According to McKelvey *et al.* (2002), the condition is very important regarding the angle of shearing resistance and ultimately the performance of the columns; the condition (wet or dry) and 10 or 20 percent smearing of the aggregates with the clay can change the angle of shearing resistance of the column material by 5 to 10%. Also, the long term performance on field could be affected by the deterioration of the aggregates (McKelvey *et al.*, 2002).

The large shear box test ($305 \times 305 \text{ mm}$) is recommended by various researchers in the area of alternative aggregates (Steele, 2004; Chidirolou *et al.*, 2008). This test can provide information such as the angle of shearing resistance and the angle of dilation which is the ratio of the plastic volumetric strain to the shear strain (Head and Epps, 2011). However, the shear box test does not reflect the context of VSC installation, loading and shearing of aggregates throughout these stages. Also, due to the size limitations of the large shear box, the real aggregate sizes used for VSC may not be used in the testing (Steele, 2004).

2.7.3 Alternative aggregates and barriers

The main problem regarding the use of the alternative aggregates is that the tests introduced in the standards do not represent the actual installation impacts and the loading of VSCs.

During construction, the fines might be added to the aggregate charges (especially in the top feed method), or fines might be introduced due to repeated movement of the vibro float (which is less in the bottom feed method compared with the top feed as the shaft movements are minimal in the bottom feed method) and also, the crushing usually occurs during the compaction of the aggregates (Jefferson *et al.*, 2010).

Another problem is that the tests consider individual behaviour of the particles rather than the interaction of the layers of aggregates in the field; therefore some other testing methods such as dynamic triaxial loading might be a better indication of the aggregates behaviour (Schouenborg, 2005).

During the site investigation, detection of the chemical composition of the ground is important for the selection of appropriate type of aggregate to avoid contamination and deterioration. For instance, the crushed concrete deteriorates in the long term when the ground has alkali nature, but has enough strength for the treatment below the ground water level (Slocombe, 2003). On the other hand, the slag waste is another form of alternative aggregate that is relatively heavier but also, weaker in terms of strength and therefore is not suitable for below the water level (Slocombe, 2003).

In general, the strength of the alternative aggregate must be sufficient if the column is installed below the water table as the aggregate must withstand the water pressure (Slocombe, 2003).

There are various types of load in static or cyclic form that can be applied to VSC in the long-term and the recommended tests do not always reflect these loads (Chidirolou *et al.*, 2009).

The shape is another important factor in the selection of appropriate type of aggregate for VSC as most alternative aggregates are angular and do not have free flow in the vibro-float and may damage the equipment during construction (Slocombe, 2003).

Reclaimed railway ballast is widely used in the UK (Serridge, 2005) which has high potential of fines contamination and therefore, must be washed thoroughly before use.

Based on Priebe (1995), decrease in the angle of shearing resistance from 45 to 39 degrees can cause a 25 percent reduction in the improvement achieved (Priebe, 2005).

Although it is recommended to use the alternative aggregates (Serridge, 2006), they might not always be the most sustainable option and the engineer should consider several factors such as the geographical availability of aggregate source, the cost of alternative aggregates production (Slocombe, 2003), the transportation, the storage, the supply and basically all aspects of sustainability, in other words the whole life cycle (Schouenborg, 2005), not just accepting that the alternative aggregates are better than the virgin aggregates (Jefferson *et al.*, 2010).

To summarize, the barriers against using the alternative aggregates are either:

- Environmental; such as noise and dust generated during the processing, transportation, storage, space required and the contamination of the aggregates (Serridge, 2005)
- Or regarding their performance, such as the quality and their compatibility for the design and the installation method used (Slocombe, 2003).

When the alternative aggregates (RA and SA) are used, the quality of the source is very critical regarding their short and long term behaviour (Chidiroglou *et al.*, 2009). Sometimes the records regarding the quality are not reliable or even in some cases not enough data is available (Schouenborg, 2005). But the quality control is the key in the proper use of material for the construction of VSC (Steele, 2004).

2.8 Summary of factors affecting performance of VSC

Based on the review of literature, there are various factors affecting the performance of VSC in the short and long term, they can be categorized into:

- (1) Material factors such as the grading, the percentage of fines, the shape of aggregates, the strength, the internal angle of shearing resistance, the crushability during the installation process, the crushability during the column loading and the condition (wet or dry).
- (2) Installation factors such as the installation energy (or time), the stress and excess pore water pressure changes in the ground and the column.
- (3) Design factors such as the internal angle of shearing resistance, the design method assumptions, the geometry of the columns and the loading type.
- (4) Pre-treatment assessment of the ground such as the site investigation approach and the host ground properties.
- (5) Post-treatment assessment of the ground; the assessment of improvement achieved in terms of the bearing capacity and the settlement and also the drainage and the consolidation rate acceleration.

Based on these factors, chapter 3 discusses their influence on the performance of VSC in the short and long term and how these factors have been addressed in the literature through numerical and physical modelling and also field testing.

CHAPTER THREE

ASSESSING THE PERFORMANCE OF VIBRO STONE COLUMNS

3. ASSESSING THE PERFORMANCE OF VIBRO STONE COLUMNS

In this chapter the important factors affecting the performance of VSCs are highlighted from the design, the material, the installation process and the loading from the current state-of-the-art literature. The methods of the assessment of the performance of VSCs are discussed in terms of numerical, laboratory and field investigations.

The assessment of the performance is broken down into three stages: during installation, during loading and over the long-term. Factors related to the installation, the material and the quality control are further discussed across these three stages for the purpose of comparison and assessment in the following chapters.

3.1 Factors affecting the performance of vibro stone columns

Various factors affect the performance of VSCs, but in this research the categories summarized in Tables 3.1 to 3.4 were studied.

3.1.1 Material

Table 3.1 summarizes the material factors and how they can affect the performance of the VSCs. The range or the recommending comments on their properties has also been presented in Table 3.1. Other factors such as porosity and water absorption are among other material factors that can also affect the performance of VSCs, however, Table 3.1 only mentions the factors that have been tested and investigated in the unit cell tests of this research.

Table 3.1: Material factors affecting the performance of VSC

Factor	Comment	Range of values/recommendations	References
Shape	Angularity of material affects the installation. Also post-construction, angular or round particles can affect the performance via interlocking and strength properties	Round particles are more suitable for bottom feed installation	Chidioglou <i>et al.</i> , 2009
Size (grading)	Size of aggregates can affect installation and long term performance of VSC by being single size aggregates or an aggregate range. A range of aggregate sizes can affect packing and better densification and ultimately better load carrying capacity and performance	Generally 20 to 75mm; Refer to Table 2.1	Charles and Watts, 2002
Angle of shearing resistance	A crucial factor in terms of compressibility and therefore bearing capacity and strength. Reduction in internal angle of shearing resistance can mean addition of fines and blockage of drainage path, which leads to slower excess pore water pressure dissipation and more settlements	40 to 45 degrees	Priebe, 1995
Type of aggregate	Aggregates can have various sources and therefore be categorized as primary, recycled or secondary aggregates. The type is not important if the aggregate is “fit for purpose”. It should have the strength and properties to withstand the loads in context of VSC	Should be fit for purpose	BRE, 2000; Serridge, 2006
Condition of aggregate	Aggregates can be dry or partially soaked or completely soaked when they are used to form the columns. The effect of moisture should be considered in loss of strength of material and long term performance of VSC	-	McKelvey <i>et al.</i> , 2002; Steele, 2004
Contamination with fines	Smearing of aggregates with fines: this can happen in storage, transportation, during installation or after the column is loaded. The introduction of fines in VSC can reduce shear strength and pore water pressure dissipation rate	Less than 10% fines are allowed	McKelvey <i>et al.</i> , 2002
Storage	Can affect the condition of aggregates. Rainfall, freezing and thawing can affect the strength and other properties of material. Also, during this time fines might be added by dust or due to crushing of material under heavy loads.	Should be free from dust and water	Steele, 2004
Crushability	Aggregates can be crushed while they are transferred to the site or storage, also during installation due to vibrational forces of the vibro-float. When the column is loaded aggregates can crush and internal angle of shearing resistance can change. Also addition of fines can affect consolidation rate.	Aggregate index tests are recommended	McKelvey <i>et al.</i> , 2002; BRE, 2000 and ICE, 1987
Durability	Durability and deterioration: these properties affect long-term performance of VSC. When material used is not durable; during installation or loading of VSC, aggregates lose their strength and therefore, the bearing capacity and settlement designed for the column will not be achieved.	Durability tests such as AIV and ACV should be performed	Steele, 2004

3.1.2 Installation

The installation factors affecting the performance of VSCs have been summarized in

Table 3.2:

Table 3.2: Installation factors affecting performance of VSC

Factor	Comment	Range of values/ recommendations	References
Equipment	Different contractors have various vibro-floats with different energy and power. The vibrational forces exerted can affect the aggregates poured and also the hole formed in installation. Different installation methods create various diameters	Table 2.1	Hughes and Withers, 1974
Method of installation	Top and bottom feed methods can affect the performance of VSC. The column formation, diameter achieved, crushing of aggregates are some of the most consequences of installation method used.	Table 2.1	McCabe <i>et al.</i> , 2009
Installation energy/ time	Layers of aggregates are compacted by the vibro-float and this time can vary between 30 to 60 seconds or until a predefined amperage is achieved. When time of compaction increases, the possibility of having a bigger column and crushed aggregates increases.	Controlled using amperage or time-controlled	Raju and Sondermann, 2005
Wet or dry method	The method of installation using air or water can affect the performance. The wet method usually has higher power and creates bigger column.	Table 2.1	Hughes and Withers, 1974

3.1.3 Loading

Loading factors that affect the performance of VSCs have been summarized in Table 3.3:

Table 3.3: Loading factors affecting performance of VSC

Factor	Comment	Range of values/ recommendations	Reference
Load type	VSC is designed for various applications to improve the ground under impacts of static or cyclic loads. It can be designed for instantaneous dynamic load application such as earthquake to reduce liquefaction hazard.	-	Adalier and Elgamal, 2004
Foundation type	Various foundations such as strips, pads or mat foundations can be constructed over VSC. The type of foundation affects the eccentricity of the loads applied and can cause differential settlements.	Not suitable for settlement sensitive structures	BRE, 2000
Rapid loading	During an earthquake or any other rapid application of loads on the stone columns, pore water pressure cannot dissipate efficiently and therefore, due to pore water pressure build up unpredicted settlements can occur.	-	Mitchell and Jardine, 2002

3.1.4 Design

The design factors affecting the performance of VSCs have been presented in Table 3.4:

Table 3.4: Design factors affecting performance of VSC

Factor	Comment	Range of values/ recommendations	Reference
Column length	The length of the column is designed according to ground condition and ultimately an end-bearing or floating column can be constructed. Different failure modes are dominant in these two different types.	Up to 30 m; Table 2.1	Barksdale and Bachus, 1983
Column diameter	Variations in column diameter can cover different percentage of the ground. Area replacement ratio is an important factor in design that can change bearing capacity and bulging failure of the column.	0.7 to 1.1 metres; Table 2.1	Baumann and Bauer, 1974
Centre to centre spacing of columns (group layout and geometry)	The area replacement ratio and unit cell concept depend on this parameter, which consequently affects the bearing capacity, bulging and settlement designed.	1.5 to 2.5 m	Raju <i>et al.</i> , 1997

Continued on
next page

Slenderness ratio	Column slenderness affects the failure mode and behaviour of the column in short and long term.	-	McKelvey <i>et al.</i> , 2004
Single or group of columns	The performance of any of the VSCs in a group is affected by the neighbouring columns. Each column installation and loading affects the neighbouring columns. The failures, stress changes in the surrounding soil and pore water pressure dissipation in and surrounding each column are all affected by the other columns during installation, when columns are loaded and in long-term.	-	Castro and Sagaseta, 2012

3.2 Assessment of performance of vibro stone column

Based on the current review of the literature, there are three main methods of assessment of the performance of the stone columns (McKelvey and Sivakumar, 2000):

- 1) Numerical methods (finite element analysis)
- 2) Field testing and measurements
- 3) Laboratory modelling

3.2.1 Numerical analysis of vibro stone columns

In numerical methods, mathematical models are used to study the settlement of the ground reinforced by VSCs (Mitchell and Huber, 1985; McKelvey and Sivakumar, 2000). Two main methods of unit cell idealization and homogenization can be used to study the behaviour of the foundations over VSCs (Gerrard *et al.*, 1984); also, the failure modes and the column-soil behaviour during bulging (Lee and Pande, 1998). Numerical modelling is not the subject of this research, and therefore, is not further elaborated in this thesis.

3.2.2 Field testing and measurements of vibro stone columns

Field testing can be used as a form of assessment of the performance before and after the column construction.

3.2.2.1 Pre-construction

Before the column construction, site investigation is used to provide the ground properties and the geological hazards (Waltham, 2009). VSCs are designed based on the ground properties, the material properties and the loading requirements (Baumann and Bauer, 1974; Hughes and Withers, 1974; Priebe, 1995). Via the field testing a column can be constructed and loaded in the appropriate scale to confirm the values and the assumptions of the design (BRE, 2000). Where the design agrees with the field measurements (especially in terms of the settlement improvement), the construction of the rest of the columns continues or otherwise the design can be reviewed. Large scale tests such as the plate load and the large zone tests are among the common tests to evaluate the design of VSCs (BRE, 2000) which are often costly and time consuming. Proper ground investigation before the design is the key in providing as much information as possible regarding the ground conditions.

3.2.2.2 Post-construction

Field testing and measurement have been used on many cases to assess VSCs' post-construction behaviour. Excess pore water pressure dissipation measurements by Castro and Sagaseta (2012) and the heave induced in the surrounding area of the vibro stone column construction (McCabe *et al.*, 2013) are examples of the field assessment post-construction. The measurements can be carried out in the long-term for the purpose of monitoring even after the column construction and loading have finished.

Various case studies mention the methods of field assessment to address the behaviour of VSCs and the surrounding ground (McKelvey and Sivakumar, 2000). The assessments have been carried out on either single or group of columns.

Hughes *et al.* (1975) used a series of large plate load test to compare the field settlement and the bulging behaviour of a real stone column to theories proposed earlier by Hughes and Withers (1974). A single 10m long column with the diameter of 0.73m was loaded by a circular plate with the diameter of 0.66m. The settlement and deformations measured were in agreement with the laboratory tests (Hughes *et al.*, 1975). Later on, the plate load test studied by Greenwood (1991) confirmed the theories of Hughes and Withers (1974).

On the assessment of group of columns, the study by Engelhardt and Golding (1975) considered the application of seismic loads on the column and the column-soil composite (Engelhardt and Golding, 1975). It was observed that due to the reinforcement of the ground via VSC, the liquefaction potential reduces and the shear strength of the ground increases significantly (Adalier and Elgamal, 2004).

Goughnour and Bayuk (1979) simulated a field study where the vertical load tests were applied on groups of 45 columns under an embankment. The columns were installed using the wet method with the diameter of 1.1m. It was observed that the settlement behaviour was improved; although the actual settlements of the columns located at the corner of the arrangement were lower than the settlements estimated. This was attributed to the wrong assumptions regarding the horizontal coefficient of earth pressure (Goughnour and Bayuk, 1979).

3.2.2.3 Geophysical assessment

Geophysical methods such as continuous surface wave (CSW) have recently been used in the field measurements and assessment of the settlement improvement of the ground (Madun *et al.*, 2012). These methods can be used in the site investigation to obtain the ground properties and stratification. Also, they can be used post-construction to assess the improvement achieved. A few of the advantages and disadvantages regarding the use of geophysical methods compared to conventional investigations are summarized in Table 3.5.

Table 3.5: Advantages and disadvantages of geophysical methods of investigations

Advantages	Disadvantages
There are non-invasive where physical tests are usually destructive	-
No sampling or drilling is required	-
Geophysical methods can cover a large area of treatment (Butcher and Powell, 1996)	However, cannot visualize the three dimensions of the ground and require other tests and methods to provide both horizontal and vertical profiles (McDowell <i>et al.</i> , 2002)
Mostly very fast methods of investigation, therefore are cost effective	However, various methods and equipment might be required to investigate different properties of the ground and therefore, increase the costs of investigations (McDowell <i>et al.</i> , 2002)
Measurements are in-situ and the values measured are close to operationally determined ones	Not enough data and accurate data with high resolution is available in many cases to evaluate the data collected from the geophysical investigation and also, the data processing and analysis can cause many inaccuracies (Madun <i>et al.</i> , 2012)
Laboratory and numerical models usually deal with well graded, idealized conditions, where most sites treated by ground improvement methods are brownfield sites, filled ground and alluvial deposited sites (Sivakumar <i>et al.</i> , 2004). Consequently, geophysical methods can measure the performance regardless of idealizations and assumptions for various sites.	-
Most physical tests do not take into account the long term performance of VSC (for instance the pore water pressure dissipation after treatment is finished); where geophysical methods could be used to study these effects in long term (Redgers <i>et al.</i> , 2008).	-

Based on the VSC case studies presented in Redgers *et al.* (2008), the settlement estimations are carried out based on Priebe's method, the continuous surface wave (CSW) and the load test measurements. The results are compared and the values of CSW and the load tests are in more agreement compared with Priebe's method. Priebe's calculations are too conservative, comparatively. This might be due to the assumptions considered in the theories behind Priebe's formula (Priebe, 1995) and the generalization of the site conditions as opposed to sites being highly heterogeneous.

3.2.3 Laboratory modelling of vibro stone columns

Laboratory modelling is another method of assessment which has been performed on single or group of VSCs. A summary of the methods used are presented:

3.2.3.1 Single column

Hughes and Withers (1974), Barksdale and Bachus (1983) and Charles and Watts (2002) tested single columns. Hughes and Withers' tests were on a sand column in clay surrounding tested in a triaxial cell (Hughes and Withers, 1974). Various diameters were tested and using radiography displacement, the clay was monitored during the loading. It was concluded that an area of 2.5 times the column diameter was affected by the column installation. The settlement rate and its magnitudes were reduced by 4 and 6 times, respectively. The critical length in these tests was defined based on the column bulging up to a depth of 4 times the column diameter.

Charles and Watts (2002) confirmed these findings via a series of laboratory tests on 1m diameter oedometer samples. Various column diameters of gravel in clay surrounding were tested and it was concluded that for a vertical load, the surrounding clay is 10 times more compressible than the columns constructed. The study does not consider the effects of various materials used as stone columns (McKelvey and Sivakumar, 2000).

Charles and Watts (2002) also found out that with increase in the area ratio, the vertical compression of the composite would decrease. Similarly, Barksdale and Bachus (1983) used various columns of gravel in clay to form the physical unit cell tests and studied the effect of different diameters (or area replacement ratios) on the bulging. As opposed to Hughes and Withers (1974), the lateral bulging was insignificant during loading. Also, it was concluded that increase in the column diameter improves the settlement behaviour of the model under vertical loads. In this study, an area replacement ratio of 40% is recommended.

McKelvey *et al.* (2002) studied the undrained strength of single columns where three types of recycled materials were used in the construction. The tests were carried out in a large shear box and it was observed that the smearing of aggregates with fines and the wet or dry condition of the aggregates affect the angle of shearing resistance by magnitudes of up to 10 degrees (McKelvey *et al.*, 2002).

In a triaxial modelling by Sivakumar *et al.* (2004), a series of single wet sand columns were installed via compaction and were compared with frozen columns installed in pre-bored holes in the surrounding clay. The columns were constructed with various lengths to form partial and full-depth penetrations. Two forms of uniform loading and foundation type loads were applied on the samples. It was concluded that the full-length columns under the uniform loading outperform other columns in terms of the bearing capacity.

Under foundation type loading, the increase in the column length improved the bearing capacity but beyond the column lengths 5 times the diameter, the bearing capacity improvement was not significant, therefore, VSCs might be more suitable for shallow improvements. The addition of geogrids in VSCs can increase the bearing capacity even

further to twice the values obtained without the reinforcement. In this study the optimum length of the column is not mentioned (Sivakumar *et al.*, 2004).

Sivakumar *et al.* (2007) studied the effect of the length of the column in the failure under similar modelling of sand in soft clay. Transparent clay-like material was used to examine the columns in groups, visually. It was observed that in the longer columns, the bulging and in the shorter ones the punching and the bulging occur under similar loading conditions. The optimum length of $6d$ (d is the diameter of the column) was concluded to provide the best results in terms of the bearing capacity under rigid footing.

Black *et al.* (2007a) used a series of single columns of basalt in peat and studied the behaviour of the ground where three series of no column, soil improved by VSC and soil improved by VSC and mesh reinforcement were tested. The peat layer had significant depth compared to the columns constructed in full and partial lengths. It was concluded that in the full-length column the load-deformation behaviour of the ground improved by over 2 and 1.5 times in case of the reinforcement and VSC compared to the no column, respectively. When the ratio of the column length to the diameter was less than 6, the punching was expected in the partial depth columns, whereas, in the longer columns the bulging was more significant.

3.2.3.2 Column groups

Black *et al.* (2007b) used a series of triaxial testing to compare single and column groups. The single column of sand with the diameter of 32mm was installed in full and partial-lengths. Also, three columns of 20mm diameter were constructed in the same cell with the diameter and height of 100 and 200mm, respectively. Both the drained and undrained conditions were tested. It was observed that a 33% increase in the undrained

strength occurred in the full-length column compared to the no column condition. Also, the drained tests showed better undrained strength results compared to the undrained tests. It was also observed that even with high area replacement ratios, the single column in the drained condition can outperform the group of three columns.

This research was further elaborated by Black *et al.*, (2011) where the settlement behaviour of the single and group of columns was compared in a large triaxial cell of the diameter and height of 300 and 400mm, respectively. It was concluded that a proper balance between the column length and the area replacement ratio can produce improved settlement. The short columns with the higher area replacement ratio can improve the settlements in similar magnitudes to the long columns with the lower area replacement ratio. The optimum values of the area replacement ratios are recommended to be between 30 to 40% which agree with the findings of Barksdale and Bachus (1983). However, the settlement behaviour of the treated ground by VSCs can be a function of various factors such as the column length, diameter, area replacement ratio and the footing properties.

A column in a group has been modelled by Barksdale and Bachus (1983), Hu (1995), McKelvey *et al.* (2004), Black *et al.* (2011). Also, Wood *et al.* (2000) tested large groups of columns and their deformation patterns, where McKelvey *et al.* (2004) tested short and slender columns in transparent clay-like material (McCabe *et al.*, 2007). It was confirmed that similar to a single column, in a group of columns, for shorter columns the punching and for longer columns the bulging were the dominant failure modes (McKelvey *et al.*, 2004).

In the laboratory models, the bearing capacity and the failure modes have been studied several times. There were fewer cases where the settlement was physically modelled.

Black *et al.* (2009) studied the settlement of a small group of columns under the large triaxial apparatus. The slenderness and the area replacement ratios were studied. It was concluded that if the length of the column increases, with the lower area replacement ratio, the settlements can still be controlled.

On the other hand, for the shorter columns, the increase of the area replacement ratio was crucial to control the settlement improvement. Based on these tests, the optimum area replacement ratio of 30 to 40 percent was recommended (Black *et al.*, 2009).

3.3 Shortcomings of laboratory studies

In previous laboratory studies the actual aggregates used in the construction of VSCs were not used in the laboratory modelling, and the column materials were scaled to sand or gravel size. In the construction of stone columns, the aggregates provide better densified columns and faster drainage. The aggregates are also better packed using the vibro-float (Bell, 2004). In few other cases where the actual aggregates were tested, for instance the shear strength tests of the recycled aggregates by McKelvey *et al.* (2002), the aggregates were not tested in the actual environment of VSC where the clay and aggregates interactions are important in terms of the performance assessment. On the other hand, in the study by Black *et al.* (2007a), 6 mm single sized basalt (primary aggregate) was used to form the columns in peat and a row of columns was studied under the strain controlled loading; however, the alternative aggregate sources were not tested in this research in the context of stone columns.

Apart from the aggregate sizes, the boundary conditions and the scaling effects of the tests were limited to apparatus used; for instance the size of the triaxial or the large shear box containers.

3.4 Validation and comparison of assessment methods

Various assessment methods of laboratory modelling, numerical analysis and field testing are usually compared to each other.

For instance, in the research by Pongsivasathit *et al.* (2012) the settlement of floating columns was studied via all the three assessment methods. The laboratory model was a large scale test on a single column of cement mix in soils with the undrained shear strengths of around 10 to 13kPa.

The aim was to determine the factors affecting the punching of the column. Apart from the area ratio (area of the column divided by the area of the unit cell) and the depth improvement ratio (the column length divided by the thickness of the soft clay layer); the load intensity and the undrained strength of the soft clay were found to be important factors in terms of the punching behaviour of the floating column.

The physical model was evaluated via four case studies in Japan and also an axisymmetric 15 node triangular mesh analysis of the column. It was concluded that the punching estimated should consider all the factors contributing to its value, otherwise the estimation is less than the actual punching values recorded (Pongsivasathit *et al.*, 2012).

There are several issues regarding the modelling and comparison of the three assessment methods. For instance, the design assumptions such as Poisson's ratio, the column depth and diameter, the centre to centre spacing, and the excess pore water pressure used in the numerical modelling may not represent the actual field conditions. Also, the construction quality and the energy of vibro-float are not considered in the numerical modelling and many laboratory investigations. Although others such as Weber (2006) and Wehr (2006) have modelled the installation and studied its effects on

the laboratory models (Weber, 2006; Wehr, 2006) via penetration and withdrawal simulation of the vibro-float.

The long-term investigations are usually time consuming and expensive and therefore, have not been fully utilized for the assessment of performance of VSCs. There are specific cases where the long-term field assessments have been used without disturbing the ongoing project and the results of the long-term settlement and consolidation of the ground post-treatment have been analysed (Raju *et al.*, 2004).

The material properties are another aspect that is not fully investigated via the modelling. The field investigation cannot reveal direct information on the condition of the aggregates post-treatment and the numerical modelling is limited in only using a few material properties such as the angle of shearing resistance as an input in analysis.

3.5 Short and long term assessment of performance of vibro stone columns

The performance of VSC is a complicated criteria to be assessed and can mean general stability of the ground treated, the bearing capacity improvement, the settlement reduction, the drainage improvement and the improvement in the consolidation rate (Charles and Watts, 2002) and in some cases mitigation of liquefaction hazard (McKelvey and Sivakumar, 2000; Raju *et al.*, 2004).

Total stress and excess pore water pressure are two factors that undergo changes during the installation process, during the loading and in the long-term post-treatment (McKelvey *et al.*, 2004). The tools to study the performance of VSC have been introduced in section 3.2; however, it is important to define the durations in which certain factors become critical in terms of affecting the performance of VSC for the purpose of this research. In previous studies the factors were discussed at two main time limits of the short and long-term.

The short-term assessment itself comprises of during installation of VSC and post-installation (or during loading); where the long-term assessment refers to the stage that the construction and loading are finished and most of the immediate and secondary settlements have occurred. The performance of columns at this stage can be the long-term load carrying capacity and the long-term settlements and drainage role of the stone columns in the ground.

Table 3.6 summarizes the factors affecting the performance which have been considered in previous research, their category (installation, material and quality control) and the duration that these factors are critical in terms of the performance.

Table 3.6: Important factors affecting the performance of VSC, the duration in which the factors affect the performance and relevant categories in which these factors can be observed

Factors affecting performance of VSC		Impact of the factor on performance	Duration at which the factor is affecting the performance			Category of the factor affecting the performance		
			During installation	During loading	Long-term	Installation	Material	Quality control
Geometry	Centre to centre spacing (layout)	The effect of neighboring columns would be affected		*	*	*		*
	Column diameter	The bearing capacity, settlement and general stability would be affected		*	*	*		*
	Column depth	Stability, bearing capacity, settlement and failure mode would be affected.		*	*	*		*
	Column position and deviation	The neighboring columns would be affected		*	*	*		*
Column properties	Column density	Bearing capacities can be affected and differential settlements and ground heave might happen		*	*	*		*
	Contamination migration via the column	Columns provides a drainage path since installation starts; proper site investigation and monitoring are key	*	*	*			*
Permeability	Smearing zone	The permeability of remolded area is affected by installation (Weber, 2010) which can affect the performance since installation starts and also during loading and carry on for long-term and therefore, affect the consolidation rate of the treated area.	*	*	*	*	*	
Undrained shear strength	Undrained shear strength of the host ground	The installation process can affect the undrained shear strength of the surrounding soil and ultimately affect the bulging and failure of the ground. Installation process and the host ground are important for this aspect. Quality control in the form of site investigation pre-treatment can identify the values of undrained shear strength	*	*	*	*	*	*
Unforeseen ground conditions	Ground cavities	Are sometimes unavoidable during installation. More material might be required and quality control means that these details should be recorded and site investigation data should be updated	*			*	*	*

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Material for the column*	Aggregate type (primary or alternative)	Load carrying capacity, settlement behaviour, drainage and consolidation due to pwp dissipation could be affected	*	*	*		*	
	Aggregate size (grading) (mm)	Damage to the vibro-float can happen and then during loading different results might be produced due to degree of packing of aggregates and load carrying capacity (Charles and Watts, 2002)	*	*			*	
	Aggregate shape (round or angular)	Possible damage to the apparatus during installation. Angle of shearing resistance can be variable and the loading and ultimately bearing capacity and stability would be affected	*	*			*	
	Angle of shearing resistance	This is one of the most important factors in terms of load carrying capacity and long-term behaviour of the column (priebe,1995)		*	*		*	
	Aggregate crushability	Can affect the angle of shearing resistance and ultimately bearing capacity and failure of the column It can happen both due to installation forces and loading, but would affect the installation by showing false feedback regarding the amount of material needed to be compacted and the behaviour of the column in loading and long-term will suffer consequently	*	*	*	*	*	
	Aggregate condition (wet or dry)	Aggregates might become wet at storage, also the wet installation method might change the condition of aggregates that would affect the load carrying capacity and long-term deteriorations can affect the overall stability of the treated area		*	*	*	*	*
	Contamination of aggregates with fines	This can happen at storage, during transfer and also during installation. The rate of pwp dissipation since installation would be reduced if aggregates are contaminated with fines; during loading and specifically rapid loading the fines can further reduce the drainage and cause more settlements than estimated	*	*	*			

Continued on next page

Vibro-float	Bottom-feed or top-feed	Column diameter is affected Also selection of aggregates would be affected by this choice as aggregates should have free flow during installation	*			*	*	
	Wet or dry method	Column formation, diameter and loss of stability in the surrounding soil during installation are affected	*			*		
	Vibro-float energy	Can affect the installation by crushing aggregates and also reduce the load carrying capacity of the material.	*	*		*		*
	Level of compaction of each layer of aggregates in the column	Column density achieved and also the crushing of aggregates is affected.	*	*		*		*
Loading and foundation layout	Static loading	Can affect the failure and settlement behaviour of the column both during loading and in long-term. It can affect the material used in the column by excessive crushing.		*	*		*	
	Cyclic loading	Can affect the failure of the column and settlement. Material could undergo fragmentation and abrasion. Installation forces can also exert repetitive forces over aggregates		*	*	*	*	
	Rapid loading	Does not provide the opportunity for pwp dissipation. Monitoring of loading stage is key for this aspect		*				*
	Foundation layout	Can induce differential settlements in case of eccentric loading		*	*			*

* Other material factors such as porosity and water absorption also affect the performance of VSCs; however, these factors are not investigated in this research and therefore, have not been presented in Table 3.6.

3.6 Assessment of effects of installation on the performance of vibro stone columns

The installation process can affect many aspects of the performance of VSCs such as material selection, material crushing and column formation. Some of the important factors are elaborated at three stages of during installation, after installation (when column is loaded) and in the long-term.

3.6.1 During installation

Factor 1: Geometry and vibro-float

Firstly, prior to the installation, as the ground to be treated by VSC may not provide an appropriate working area; a suitable platform is required for the poker and its crane (BRE, 2000). The platform material should be granular, suitable for the ground condition and not prevent the vibro-float penetration.

The vibro-float deviation during the installation is important for accurate column formation. Based on previous case studies, in order to achieve successful construction of the columns, the deviation should not to be more than 1 to 20 (BRE, 2000). The column position should be as accurate as stated in the design details; the reduction or increase in the centre to centre spacing of the columns might affect the neighbouring columns in a column group (McKelvey *et al.*, 2004).

The vibro-float penetration should be controlled to ensure the design depth is achieved (Bell, 2004). During the installation, unforeseen ground condition such as obstructions need to be removed and recorded which may delay the installation process (BRE, 2000). It should be noted that this might damage the vibro-float (Slocombe, 2003).

The method of installation selected (top-feed or bottom-feed and wet or dry) can affect the surrounding clay and also the column formation. The top feed wet method creates a larger diameter compared to the dry method. The shape of the vibro-float and its fins can also slightly increase the diameter of the hole formed (Hughes *et al.*, 1975).

In the bottom-feed method there is more control over the charges of aggregate and therefore the volume of the aggregates and the column can be more accurately estimated which ultimately results in more accurate column formation in terms of the diameter (McCabe *et al.*, 2009). The method of installation can also affect the crushing and the behaviour of the aggregates. Reduction in the shear strength of the surrounding soil occurs during the vibro-float penetration especially in the wet method (Kirsch, 2006).

Various types of vibro-float are used for each method of VSC construction. Contractors use different apparatus for the penetration and compaction of the columns. The energy consumed may show the stiffness of the ground and also the level of compaction achieved at each layer of aggregates which are charged and compacted (Raju and Sondermann, 2005). But this is not always a reliable criterion to assess the level of compaction achieved in the column. Also, the surrounding soil might have obstructions and variable lateral pressures at each stage (Bell, 2004) which show false feedback regarding the strength and stiffness of the host ground and the level of compaction achieved on the aggregate charges.

Figure 3.1 shows a soil profile in the UK which was reinforced by VSC technique and the poor in situ test results post-construction triggered further investigations and excavations (Bell, 2004). The results confirmed that the designed values of the column diameter and the depth of treatment have not been achieved in several columns.

Although some variations in the diameter of the column is to be expected at different

depths (due to different lateral resistance of the different layers), the investigations showed that many columns were not even formed in the top few metres of the length and the vibro-float had not reached the ultimate required depth. Also, based on the records, the amount of aggregates consumed was a lot less than the mass required based on the volume and the density of the columns designed.

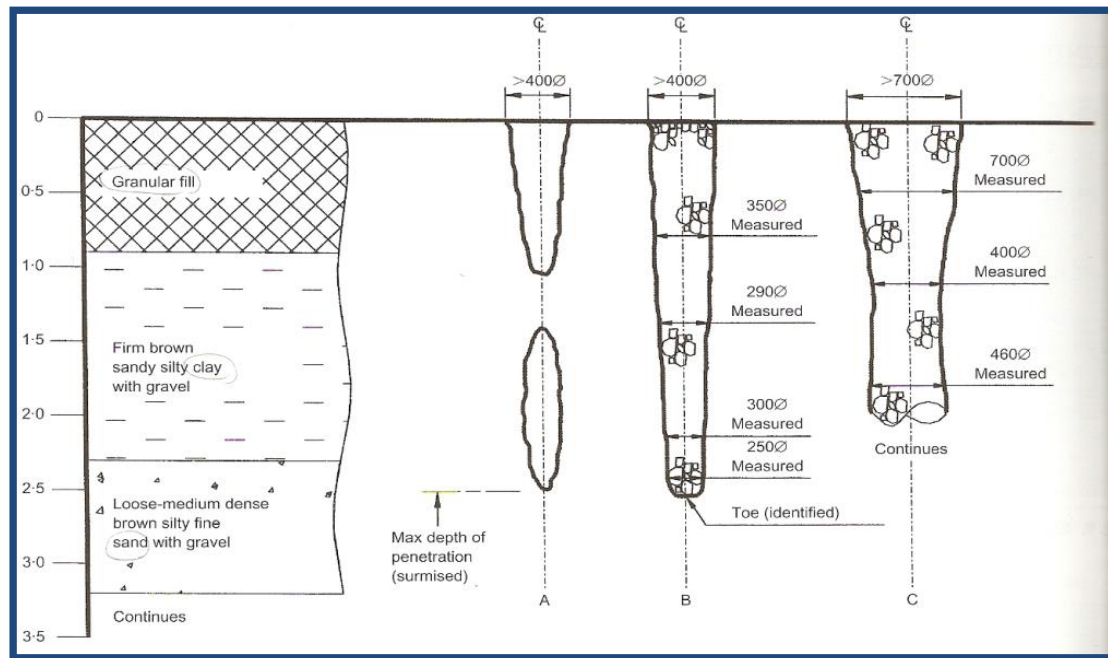


Figure 3.1: Poor stone column construction, case study (Bell, 2004)

According to this study the key factors affecting the formation of the columns are 1) compacting each layer sufficiently before charging and compacting the next level of aggregates and 2) the amount of aggregates used for each stage should be recorded accurately to assess the density of the column achieved (Bell, 2004). Therefore, the quality of workmanship and constant monitoring are important.

Factor 2: Ground movements: installation induced heave and settlements

During the installation, poor compaction or over-compaction of the aggregates may cause immediate settlements or heave, respectively (Kirsch, 2006). Heave in the

surrounding area of construction may cause damage to the adjacent structures and services (McCabe *et al.*, 2013).

There have been a few cases that the ground heave was recorded and based on the studies the amount of the heave is a function of the diameter and length of the column, the centre to centre spacing, the extent of the treated area and more importantly the quality and method of construction (Egan *et al.*, 2009).

Other cases where the heave is measured during construction for different arrangements of columns such as Castro (2007), Watts *et al.*, (2000) and case studies presented by Egan *et al.*, (2009), show that the heave was significantly related to the arrangement of the columns and columns in large arrays have more vertical heave than other patterns studied. Although the database was very limited for the heave measurements, the finite element analysis on few cases showed similar behaviour regarding the heave for VSCs as driven piles (McCabe *et al.*, 2013).

It can be concluded that the installation is key in achieving the proper column density in order to prevent the ground movements either during the installation or later on when the columns are loaded.

Factor 3: Stress and pore water pressure

Another parameter which varies during the installation of VSCs is the in situ stress of the ground. In some cases, up to 60 kPa increase in the total stress was observed during the installation in saturated soils (Watts *et al.*, 2000).

As the column installation is a fast process, the undrained cavity expansion theory could be used to calculate the stresses for the elastic and plastic zones surrounding the column. Based on the calculations, at a specific depth, the stresses decrease with an

increase in the radius of the area surrounding the column, but after a specific point, the stresses are constant (Egan *et al.*, 2009).

The effect of the centre to centre spacing should not be ignored in changing the stresses in the surrounding soil for the group of columns.

The column installation is a fast process that also affects the excess pore water pressure build-up in the ground. Based on Castro's investigations (Castro, 2007), used as a general trend, the excess pore water pressure changes measured via field piezometers can be observed at various stages for different cases. The measurements show that the excess pore pressure increases dramatically in the beginning of the installation (vibro-float penetration) and reaches the maximum value when the vibro-float is at the same depth as the piezometer used for measurement. While the vibro-float is lowered and raised in several stages, the excess pore water pressures fluctuate. The excess pore water pressure reaches equilibrium after the installation is completed and again increases as other adjacent columns are constructed.

There are no available field observations regarding the dissipation of the ground water after the columns are installed, but based on the finite element analysis, columns working as drainage path; increase the dissipation rate and therefore consolidation rate is higher compared to ground with no VSCs (Egan *et al.*, 2009).

One of the most problematic soils is peat which contains a lot of ground water and also shrinks under loading (Waltham, 2009), which may lead to false feedback regarding the pore water pressure changes and dissipation during the construction.

In the laboratory investigations by Weber *et al.* (2010), one of the important effects of the installation was the permeability of the host ground. In this study columns of sand

were modelled in clay using a centrifuge apparatus. The bottom-feed installation was simulated using withdrawal and reinsertion of a tube that poured measured quantities of sand in clay (Weber, 2004). Via mercury intrusion and x-ray tomography, the intersection between the column and clay was studied.

The influenced area was divided into three zones of 1) penetration; where the column materials penetrated into the clay, 2) smearing; where the clay particles were reoriented due to the column installation and 3) densification; where the structure of the clay was the same, but the column had only compacted the clay (Weber *et al.*, 2010).

The smearing area had a radius of around 2.5 times the column diameter. This area was remoulded during the installation and was therefore strongly sheared. In this area the permeability of the clay was affected. Horizontal permeability was observed to have reduced and therefore, it was recommended to consider the time factor for the settlement and consolidation calculations (Weber *et al.*, 2010).

In addition, the vibration of the ground was observed up to the distance of five times the radius of the column from the column centre during the installation of VSC (Kirsch, 2006), therefore, a safe working distance of 10 metres was recommended for practice of VSCs (Raju and Sondermann, 2005).

3.6.2 During loading

As column construction is a fast process; after the installation, the columns are usually quickly loaded. The installation factors affecting the performance during the installation, could also affect the post- treatment behaviour of the columns shortly after the installation has finished and while the columns are being loaded. These factors can reduce the bearing capacity and the overall stability of the ground or induce differential settlements and movements once the columns are loaded.

Effect 1: Column bulging

In studies by Sivakumar *et al.* (2007) the column length affects the failure mode of the VSCs. In shorter columns, the punching and in longer columns, the bulging have been observed under various loads in the triaxial tests on columns of sand in clay (Sivakumar *et al.*, 2007). The bulging itself can be affected by the centre to centre spacing, the pattern and neighbouring effects of the other columns. The bulging causes further stress and excess pore water pressure changes in the surrounding soil (Hughes and Withers, 1974). Quick loading can cause high excess pore water pressure build up and unforeseen total or differential settlements as the excess pore water pressure does not have the time to dissipate.

Effect 2: Excess pore water pressure

During the installation of VSCs, after the initial vibro-float penetration, the excess pore water pressure rises rapidly and then fluctuates through compaction stages of the aggregates and then becomes steady. Cases show that its value rose up to 100kPa and then returned to the initial values of pre-treatment after two months (Watts *et al.*, 2001 and Egan, 2009). As VSC acts as a drainage path, the water pressure might decrease in the longer duration after the construction (Castro and Sagaseta, 2012).

Figure 3.2 shows the approximate trend of the excess pore water pressure changes during the installation, and shortly after the installation when the adjacent columns are constructed (according to studies by Castro and Sagaseta (2012)). The columns continue to act as drainage path during the loading.

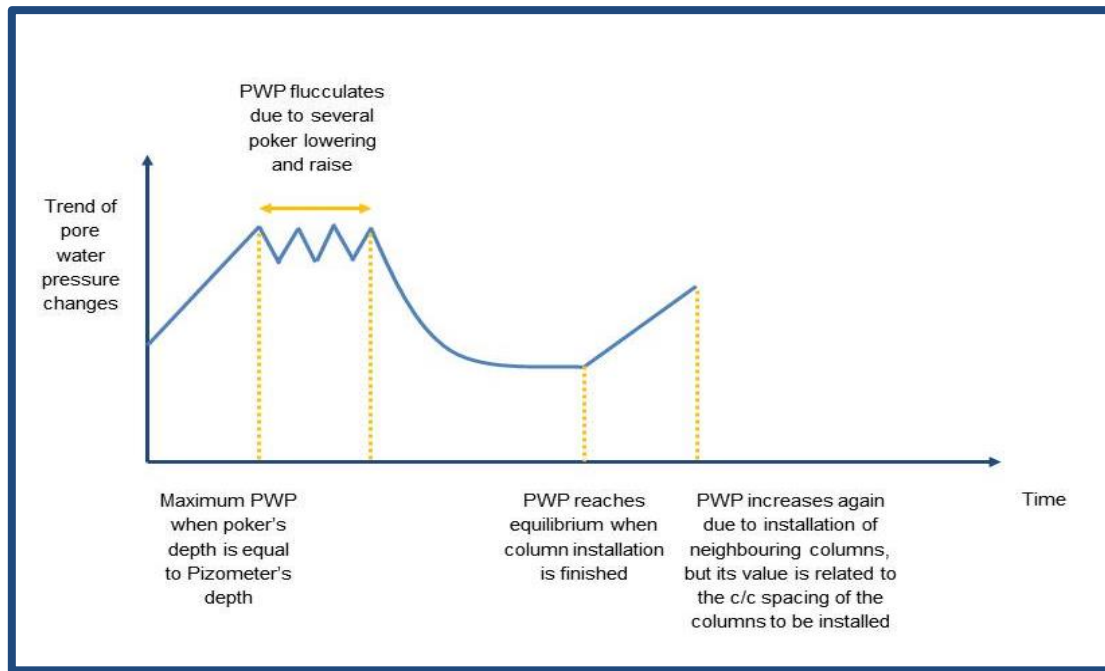


Figure 3.2: Pore water pressure changes due to column installation (after Castro and Sagaseta, 2012)

Behaviour of the VSCs after the column construction could be related to the factors mentioned before which affect the column performance during the installation. For instance if proper length, diameter and centre to centre spacing are achieved during the installation process; the behaviour of the columns during the loading can be positively affected, consequently (BRE, 2000).

The column density achieved, the aggregate condition (wet or dry) and crushability and the properties of host ground directly affect the load carrying capacity of the columns during the loading stage (McKelvey *et al.*, 2004).

The type of load and foundation constructed over the columns can also affect the behaviour of the VSCs during the loading. For instance, in case of eccentric loading, the columns may undergo differential settlements (McCabe *et al.*, 2009).

3.6.3 Long-term effects of installation

Geometry (column depth and diameter, centre to centre spacing *i.e.*, group and layout), column density, aggregate crushability and the conditions, loading and host ground properties are among the factors which can affect the long-term behaviour of VSCs. In the long-term the column works as a drainage path and therefore, it is expected to accelerate the consolidation rate (Raju and Valluri, 2008).

Effect 1: ground movements

Total and differential settlements in the long-term and the continuous heave are examples of the long-term effects that may be caused by improper installation (McCabe *et al.*, 2013). If the aggregate charges are not properly compacted at each stage during the installation, not only will the column not perform as expected under the applied loads, but also in the long term unpredicted ground movements may occur.

On the other hand, over-treatment causes heave and may induce movements in the ground after loading. Compaction of the aggregates via the vibro-float may crush aggregates during the installation and therefore produce blocked drainage path in the column; this may lead to further long-term settlements and prolonged consolidations.

Effect 2: Foundation layout and loading

Foundation layout and loading can also affect the settlements and the bearing capacity failures of the columns in the long term. Unsymmetrical foundation layouts may lead to differential settlements over the columns (McCabe and McNeill, 2006).

Also, the installation should be performed in a controlled way in order to have similar column densities across a field to prevent uneven ground movements and differential settlements (BRE, 2000).

Usually monitoring and the quality of workmanship are key factors in successful VSC practice in the long-term (Bell, 2004), but the long term effects are not the subject of this research.

3.7 Assessment of effects of material properties on performance of vibro stone column

In the aggregate selection process for the VSCs the most important concept is being “fit for purpose” (Serridge, 2005); as an inappropriate primary aggregate can also result in poor performance of the columns if the source does not have the requirements for the performance (Jefferson *et al.*, 2010).

3.7.1 During installation

Effect 1: Aggregate crushing and the angle of shearing resistance

The crushing of aggregates means more fines are introduced and therefore, the angle of shearing resistance decreases and causes less drainage and reduction in the bearing capacity and the settlement improvement of the system (Charles and Watts, 2002).

Effect 2: Column density

Based on previous experience on similar projects and also the volume of the stones required for each of the columns, the amount of aggregates required should be calculated and considered during the construction in order to achieve the proper column densities (Priebe, 1995). In case a cavity exists in the ground, more material might be required to complete the column installation (BRE, 2000).

Effect 3: Vibro-float and material

Apart from the need for a free flowing material in the vibro-float during the installation; material compatibility with the method of installation is crucial in terms of aggregate

size and shape (BRE, 2000). The angular materials are more suitable for the top feed method as the charges are from top of the bore excavated, while for the bottom feed method, smaller and rounder aggregates are required not to damage the poker and to have free flow as they are charged through the tip of the poker in the hole. Aggregates might be crushed due to the poker compaction.

3.7.2 During loading

The properties of the material can affect the load carrying capacity of the column and affect the bulging and the failure mode. On the other hand, the type of load applied to the column (static or cyclic) can affect the column behaviour (McKelvey *et al.*, 2004).

The application of repetitive loads can cause deterioration in the column material by crushing them as soon as the loads are applied, therefore, more investigation and assessment of the behaviour is required for the material under cyclic loads (Chidirolou *et al.*, 2009). Not only the loading process itself, but also installation of the columns could cause breakage and change in the physical properties of the material.

3.7.3 Long term

Material properties are extremely sensitive in terms of the long-term behaviour of the VSCs.

Firstly, the columns act as vertical drains due to their granular nature, and therefore, should provide proper drainage path to improve the consolidation behaviour of the ground (Barksdale and Bachus, 1983). Apart from the excess pore water pressure dissipation, columns can transfer contamination to the surface or foundations (Serridge, 2006). This can be mitigated by proper site investigation pre-treatment (BRE, 2000).

3.8 Assessment of effects of quality control on the performance of vibro stone columns

Since the ground improvement methods have been developed, the quality control has gained more importance to evaluate the performance of the treated area (Mitchell and Jardine, 2002). The quality control can be divided into pre-treatment (referred to as site investigation) and post-treatment (monitoring) phases. The settlement control and excess pore water pressure monitoring are among the common controlling measures for VSCs post-treatment (Chu and Yan, 2005; Silva, 2005).

Successful VSC practice requires thorough site investigation pre-treatment in order to identify the soil strata and the undrained strength of the ground at each layer; the ground water level to assist in the installation method selection and the material choice, possible contamination in the ground, the density and compressibility of the ground and the existence of cavities and their size (BRE, 2000). Site investigation can assist in the design assumptions, construction planning, risk assessment and mitigation of the potential hazards.

During the installation, the vibro-float energy and the level of compaction of each layer of the material are important factors for the monitoring and analysis of the performance of VSCs (Raju *et al.*, 2004).

Also, aggregates selected for the construction should be properly stored and no fines should be added to them during the storage or delivery to the site (BRE, 2000). The quality control and records on the aggregate properties and condition are key elements in interpretation of the behaviour of the material used in the columns.

3.8.1 During installation

During construction, the site investigation could be updated as there might be unforeseen ground conditions such as cavities. The contractor and designer should cooperate to modify the design and installation if required (BRE, 2000). It is important to utilize an efficient recording method for the unforeseen ground conditions, the aggregate consumption (to avoid over-treatment and ground heave or under-treatment and failure) and the vibro-float energy (Raju *et al.*, 2004).

During installation, the geometry *i.e.*, centre to centre spacing and the column diameter and depth should be monitored to achieve the designed requirements.

3.8.2 During loading

The factors mentioned during installation of VSCs can also affect the performance during the loading. If the columns are not formed properly and the host ground condition are unknown or the aggregates are crushed due to over-treatment by the vibro-float; the loading procedure may lead to failures and reduction in the bearing capacity and the settlement improvement factor (BRE, 2000).

3.8.3 Long-term

Monitoring the ground post-treatment can be most illuminating regarding the assessment of the level of improvement achieved. In order to investigate the improved properties of the host ground, the standard penetration test (SPT), the cone penetration test (CPT) and the dynamic penetrometer test (DPT) can be used (Raju *et al.*, 1997).

Also, large zone tests or plate load tests on one or more columns and their surrounding soil can show the level of improvement achieved post-treatment (BRE, 2000). A rigid or cast in-situ plate can be used to load the column parallel to settlement gauges and

piezometers to measure the settlement reduction factor and the excess pore water pressure dissipation, respectively.

In practice quality control tests are usually performed a week after the columns construction in order to record the long-term consolidation behaviour versus the short-term settlements (Raju and Sondermann, 2005).

There are several cases where the appropriate installation method and the quality control have resulted in excellent performance of the VSCs in the long-term. An example is the hydraulically placed fill in Bahrain which was modified by VSCs instead of bored piles. The results of the performance were based on the cone penetration test (CPT) carried out pre and post construction combined with the large zone tests. Monitoring and measurements proved that the design method was acceptable and only underestimated the improvement achieved. Only in silty layers of the soil profile, the excess pore water pressure dissipation required more time. The pre and post treatment CPT results indicated a high improvement factor. Based on the zone tests, the Priebe's method of settlement estimation had slight over-estimation compared to the actual settlement values measured (Renton-Rose *et al.*, 2000).

In other cases reported by Mitchell and Huber and Munfakh *et al.* presented in McCabe *et al.*, (2009), the wet top feed method has been used in soft cohesive soils and has shown successful performance based on the field test results (McCabe *et al.*, 2009). Also, Venmans (1998) reported successful performance of the dry bottom feed method for a clay embankment of $c_u = 15 - 20 \text{ kN/m}^2$ (McCabe *et al.*, 2009).

Raju *et al.* (2004) reported the use of VSCs on a soil with the undrained shear strength of between 5 to 15 kPa to improve a 15 metre-high highway embankment over a mining pond in Malaysia. The long-term monitoring showed improvement in the consolidation

time and the settlement of the treated area via VSC method, even for the undrained shear strength of less than 10kPa; although as it is not commonly practiced, it is recommended to have a lot of monitoring and quality control (Raju *et al.*, 2004).

In this case study the consolidation time measured was reduced to 90 days after the treatment compared to the initial estimated values of 6 months and most of the settlements were recorded during the embankment construction at an early stage. The strength of the treated area was measured via the vane shear test (VST) and was improved three times; which was in agreement with Priebe's theory that the load is shared by both the ground and the column post-treatment (Priebe, 1995).

The vibrations of the ground induced by the vibro-float during installation were also monitored, and the peak vibration was recorded as 20mm/sec at one metre distance from the vibro-float (Raju *et al.*, 2004). This value is within the acceptable vibration range of between 20 to 50mm/sec recommended by the British Standard (BSI, 2014).

To summarize, visual monitoring of various stages of the improvement such as the column location and the diameter, and collecting and analysing data during the installation and observational methods such as field testing can assist in successful execution of VSCs. Previous experience on similar projects helps in identifying the critical factors regarding the performance of VSCs in the short and long-term.

3.9 Summary of assessing the performance of vibro stone columns

Various factors related to the design, the installation process, the materials selection and the loading of the VSCs affect their performance in the short and long-term.

In previous research, many of these factors have been assessed using the numerical, laboratory and field investigations. There are certain limitations for each of these assessment methods.

The laboratory modelling has the advantage of producing repeatable tests where certain factors can be varied and studied in a carefully controlled environment. On the other hand, in modelling the VSCs in soft clay, the scaling and the use of sand and gravel instead of the actual aggregates has previously limited the interpretation of the results when recycled sources of aggregates were used in actual context of the VSCs.

For the purpose of this research, in order to assess the performance of RAs in the context of the stone columns, important factors related to the materials and the installation which have been rarely considered in previous research were highlighted in this chapter at various stages of the installation and the loading to be further considered for the laboratory modelling.

Based on the gaps in the knowledge mentioned in this chapter regarding the installation effects and the materials selections for the construction of VSCs, it is necessary to model the columns of actual RAs and apply the static loads from the foundation on the columns in the short-term to study the load-deformation behaviour of various single columns when the RAs are compared against a commonly used PA. The effects of the installation process on the materials should also be considered.

CHAPTER FOUR

METHODOLOGY- PART 1: MATERIAL TESTING

4 METHODOLOGY- PART 1: MATERIAL TESTING

The laboratory testing designed for this research is modelling of a single stone column in soft clay to be loaded statically for the study of its short-term behaviour.

This chapter explains the importance of the index tests on the host ground (clay) and the aggregates (column material) in order to be used in the laboratory unit cell tests (full details can be found in chapter 6). The standards and methods of evaluating the results have been briefly presented for the tests on Kaolin clay and the various natural and recycled aggregates used in this research.

4.1 Research philosophy

VSC is a commonly used method all over the world; especially in the UK to improve the properties and the behaviour of the host ground (McCabe *et al.*, 2007; Serridge, 2005). Based on the review of the literature presented in chapters 2 and 3, there are factors related to the design, material selection, the installation process and the quality control that can influence the behaviour and the performance of VSCs both in the short and long-term.

Despite the shortcomings of the laboratory modelling (refer to section 3.3), the unit cell modelling of a single stone column constructed using various primary and recycled aggregates can assist in understanding the short-term behaviour of the columns under carefully controlled installation and static loading conditions.

The main advantage of a large scale unit cell test is that the actual aggregates (the PA and the RA) can be used in the VSC construction without being scaled down to sand or gravel particles (Sivakumar *et al.*, 2004; Black *et al.*, (2007a)); therefore, comparing the aggregates against each other in the context of VSC becomes possible.

On the other hand, the installation process of the VSCs can be simulated in the laboratory to enable the researcher in understanding the effects of the installation forces on the different sources of the aggregates used. There are only a few cases where the installation method using a vibro-float has been simulated in the laboratory such as the research by Weber *et al.* (2006) which was explained in chapter 3, sections 3.4 and 3.6.1 (Factor 3).

4.2 Research question

The research question is to compare the use of the various RAs with a commonly used PA source for the construction of VSCs where the context of installation and loading of a single column can be simulated using laboratory unit cells.

The investigation can reveal which column can perform better in the short-term in terms of the load carrying capacity, the settlement behaviour, aggregate crushability and the excess pore water pressure dissipation.

Using the index properties of the aggregates is the only recommendation for the assessment of the materials to be used in the construction of VSCs. This research aimed to assess whether the aggregate index tests can be solely trusted in the suitability assessment and selection of materials for use in VSCs.

4.3 Methodology outline

In this chapter the materials used for the unit cell testing have been introduced and the index tests are presented for each material before they can be used in the actual environment of VSCs.

Figures 4.1 and 4.2 are schematic representations of the large and small unit cell (LUC and SUC) tests which have been fully described in chapter 6.

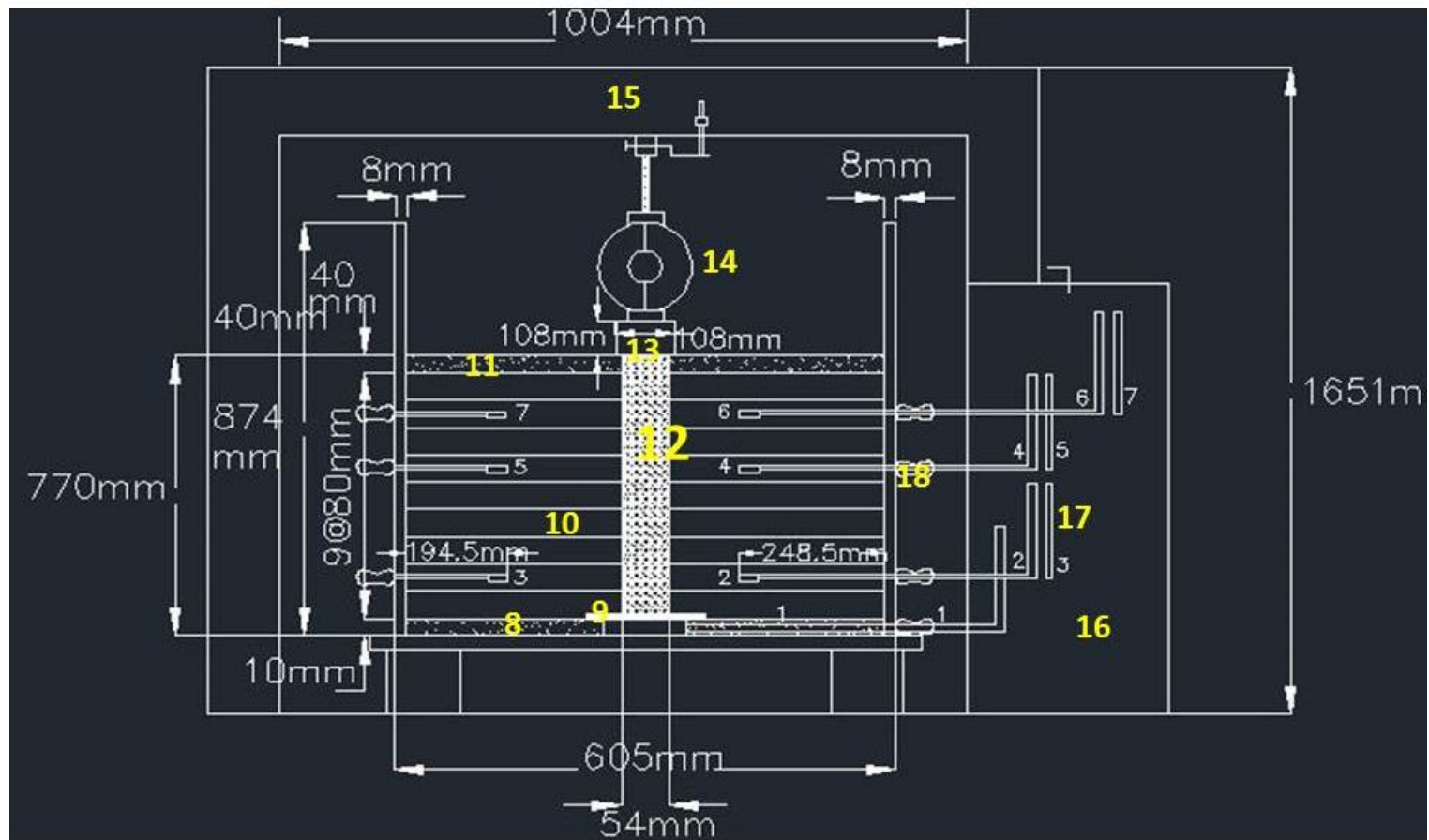


Figure 4.1: Schematic side section of the large unit cell tests

- ¹ Porous stone at the base of the cell, ²⁻⁷ Piezometers in the partially saturated clay, ⁸ Layer of saturated Leighton Buzzard sand at the base, ⁹ Filter paper, ¹⁰ Kaolin clay; compacted in layers, ¹¹ Layer of saturated Leighton Buzzard sand on the top, ¹² The column of aggregate, ¹³ The foundation type loading plate, ¹⁴ The loading ring, ¹⁵ The loading frame, ¹⁶ Wooden board to read the water levels, ¹⁷ Water level pipettes, ¹⁸ Water level taps

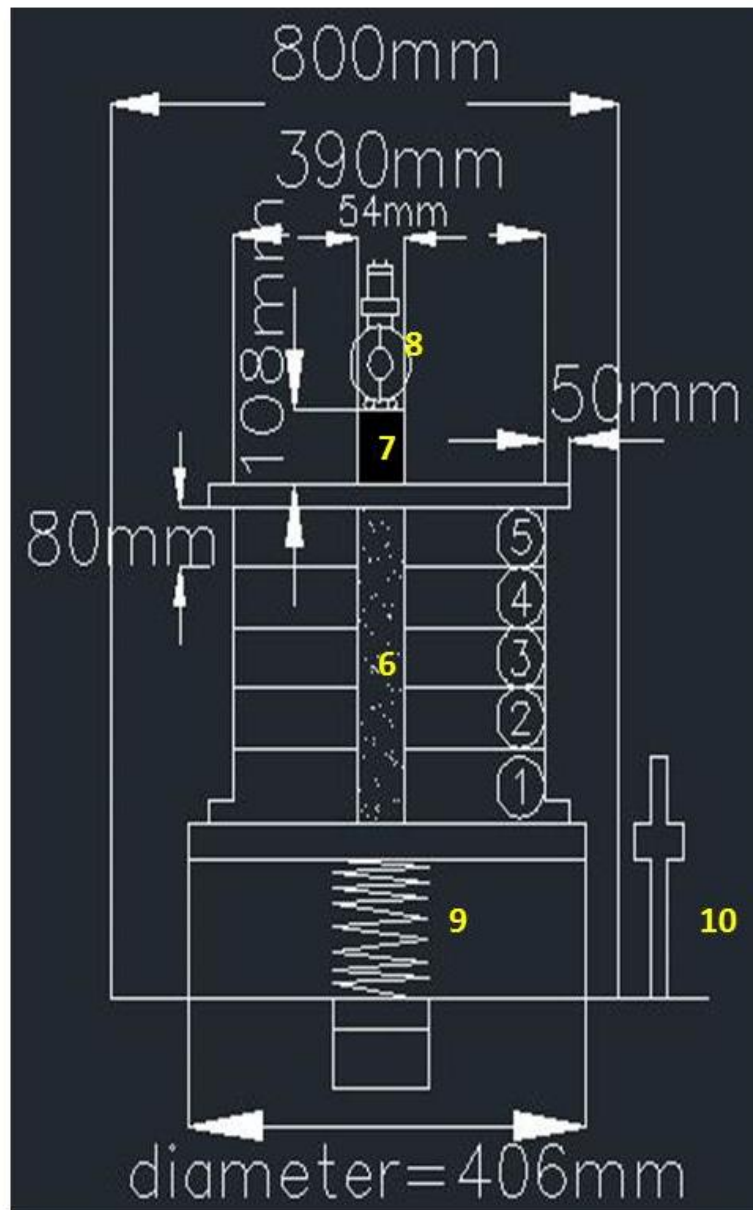


Figure 4.2: Schematic side view of the set up of the small unit cell tests
¹⁻⁵ Kaolin clay; compacted in layers, ⁶ The column of aggregate, ⁷ The axial loading plate, ⁸ The loading ring, ⁹ The loading apparatus, ¹⁰ Displacement measurement Vernier

As shown in the schematic cross sections of the LUC and the SUC tests, a stone column was constructed in the soft clay, where the actual scaled and crushed primary (granite) or recycled aggregates (crushed concrete and brick and two types of incinerator bottom ash aggregates) have been placed in the unit cells which have been designed and

developed by the researcher based on the boundary conditions. According to the set ups the most important aspects of methodology are:

- 1) The host ground: Kaolin (China clay); the source, the reason for using this material, the tests required for Kaolin according to the unit cell concept and the evaluation of its use were described in sections 4.4.1 and 4.4.2.
- 2) Stone column material: the granite (primary aggregate) and the three recycled aggregates (CC/CB, IBAA (1) and IBAA (2)) are chosen for these tests. The sources, the reason behind the selection, the index tests and the requirements for use in VSC are explained in sections 4.5.1 to 4.5.3.
- 3) Loading equipment; including the frames, the proving rings, the load plates and the rate of the loading have been explained in chapter 6 (sections 6.4 and 6.6), for the unit cells.
- 4) Various measurements such as the load-deformation behaviour and the water levels have been explained in chapter 6 (section 6.5). For the small unit cell tests the other measurements include the column formation and the study of the shape which have been explained in section 6.5.5.

Therefore, the materials used in the unit cell tests should be properly studied for their properties and behaviour. In section 4.4, the host ground material testing has been described, followed by section 4.5 for the column materials (*i.e.*, aggregates).

4.4 Material testing-Host ground

4.4.1 Kaolin

Kaolin or China clay is a form of industrial mineral with the chemical composition of $Al_2Si_2O_5(OH)_4$ (Waltham, 2009). It has low shrinkage and swelling capacity, is inert and easy to mix and therefore is a widely established material used in the laboratory

modelling (Weber, 2004). Using the China clay makes repeating and reproducing of samples with similar properties possible.

The Kaolin used in this research was English China clay of type Puroflo 50 (from WBB Devon clays Ltd). Its chemical analysis, mineralogical composition, particle size distribution (PSD), PH value and surface area were provided by the manufacturer. The data has been presented in chapter 5 (section 5.2.1).

Kaolin was also been tested for its index properties. Natural moisture content, plastic and liquid limits, specific gravity and compaction tests were performed on the China clay used in the modelling in this research. The index tests have been explained briefly:

1) Moisture content test (BSI, 1990a 3.2):

The equipment and the procedure of the natural moisture content using the oven drying method is fully explained in the British standard (BSI, 1990a 3.2).

The test was repeated three times, each time on three samples to ensure that the results represent the clay samples used in the modelling. The results have been presented in chapter 5 (section 5.2.2).

2) Plasticity index:

The plasticity index is the range between the liquid and the plastic limits, *i.e.*:

$$PI = LL - PL \quad \text{Equation 4.1}$$

Where PI is the plasticity index (%)

LL is the liquid limit and

PL is the plastic limit

In order to calculate the plasticity index for the Kaolin used in the modelling, the liquid limit and the plastic limit tests were performed using the following tests:

Liquid limit test (BSI, 1990a 4.3):

Two series of tests were carried out, using the electric cone penetrometer apparatus according to the procedure described in (BSI, 1990a 4.3). A part of the sample was kept for the plastic limit test to be performed on the same sample later. The details of the measurements and the graph have been presented in Appendix 1.

Plastic limit test (BSI, 1990a 5.3):

The sample kept from the liquid limit test which was left overnight for homogenization, was used for the plastic limit tests. Similar to the liquid limit test, two sets of tests were performed on the Kaolin. The details have been presented in Appendix 1. Plasticity index was calculated based on the liquid and plastic limit values and was reported in percentage in chapter 5 (5.2.3).

3) Plasticity index using tap water:

As in unit cell testing (both the small and large cells), large quantities of China clay were used (approximately 225kg and 62.5kg for each of the large and the small unit cells, respectively); a lot of distilled water would be required to mix the clay for the preparation. It is very costly and time-consuming to provide 100 litres of distilled water in the laboratory for each of the large unit cell tests. Using the tap water was the proposed solution for the unit cell tests; therefore, the plasticity index was measured again for the China clay where the tap water was mixed with the clay instead of the distilled water. The same procedures mentioned above for the liquid and plastic limit tests were repeated (BSI, 1990a 4.3) and (BSI, 1990a 5.3); only the tap water was used

throughout the entire process. The results have been reported in chapter 5 (5.2.3) and the details have been presented in Appendix 1.

4) Specific gravity test (BSI, 1990a 8.3):

The equipment and procedure are fully explained in (BSI, 1990a 8.3) in order to measure the specific gravity of the China clay using the density bottles method.

The result of the density bottle test has been presented in chapter 5 (section 5.2.4) and the detailed measurements can be found in Appendix 1.

5) Standard compaction test (BSI, 1990b 3.3):

The standard compaction test was performed on the Kaolin clay according to (BSI, 1990b 3.3). The aim was to obtain the compaction curve and to obtain the optimum moisture content and the maximum dry density.

In the standard compaction test usually five moisture contents and dry densities are sufficient to form the compaction curve (BSI, 1990b 3.3). However, in this research further points were tested in order to achieve low shear strengths of below 25 kPa in the sample.

This is fully explained in the unit cell testing concept (refer to section 6.2.6), as the shear strength was chosen as the most important criteria in the host ground preparation. As a single stone column was constructed in the soft clay, an undrained shear strength of lower than 25kPa was required for all the layers in the unit cell tests; therefore, the compaction tests were continued at higher moisture contents to achieve low shear strengths. The results of the compaction curve with the air void lines and the undrained strengths have been presented in chapter 5 (section 5.2.5), and the details in Appendix 1.

6) Vane shear test (BSI, 1990d):

The vane shear test has been used for the soft fine-grained soils where the shear strength was needed to be measured in the field. The value of the shear strength obtained is the undrained value as the test is performed very quickly (Head, 2006). The hand vane shear apparatus was used in the laboratory tests in this research to determine the undrained shear strength of the various layers of soil in the unit cell tests.

The higher the strength of the soil is, the vane would show more resistance to the rotation of the blades in the soil. This test is very quick and easy to perform in the laboratory to control the shear strength of the Kaolin used only if it is done accurately and correctly, otherwise, the error created can result in invalid numbers.

As well as creating low quality results in case of poor execution of the test, another disadvantage of this test is that the data collected is at specific points in the soil and does not represent all the points and layers (*i.e.*, data is discrete and not continuous) (Head, 2006).

This test was used in this research parallel with the compaction tests; performed at each layer of the compacted soil after the compaction test was finished and while the soil was cleaned out of the compaction mould. Also, in the unit cell tests (both the large and the small), one of most important controlling measures for the uniformity of prepared soil was the undrained shear strength which was measured using the hand vane. This test was repeated accurately on each of the compaction test samples or each of the unit cell tests.

7) Variations of the compaction test

For the purpose of this research two variations of the compaction test were performed:

- Standard compaction test performed according to (BSI, 1990b 3.3).
- Compaction test using the vibrating (Kango) hammer in the standard compaction mould where each layer was compacted for 15 or 10 seconds.

BSI (1990b 3.7) describes the equipment and the procedure for the compaction test via the vibrating hammer. This test is suitable for granular material and a bigger mould than the standard compaction mould should be used (Head, 2006); however, in this research the same mould as the standard compaction test was used to test the cohesive material (China clay) using the vibrating hammer.

The aim was to apply the results of the compaction in the standard mould in estimation of the energy required for the compaction of large quantities of clay in the unit cell tests. The energy estimation and calculations have been presented in Appendix 2.

The first attempt of using the vibrating hammer was to compact each layer of the clay for 10 seconds, where 5 layers of material were filled in the standard compaction mould. It was observed from this test that 10 seconds was a very short time for the compaction and there was so much error in the time of the compaction due to the time consumed for switching the apparatus on and off and moving it around in the mould.

The second two tests used the same equipment, but the vibrating hammer was used for 15 seconds per layer on 5 layers of China clay in the standard compaction mould. 10 second compaction results were not used in the evaluation of the compaction time required for the unit cell tests. The results of 15 second compaction and its repeat test have been presented in chapter 5 (section 5.2.6).

An important part of these tests was the graph where the compaction curve (dry density versus the moisture content) and the undrained strength values versus the moisture

content were combined to achieve the range of the moisture contents at which the required undrained strength for the unit cell tests was achieved.

4.4.2 Evaluation of Kaolin index tests

4.4.2.1 Errors in the laboratory tests

The results of the laboratory tests were not valid unless the errors embedded were described. The errors are inevitable and even the most accurate testing conditions create some degree of error. Firstly, errors should be identified and then reduced as much as possible and also, the results should be reported using the calculated values of error (Taylor, 1982).

In the laboratory testing, several factors can contribute to the existing errors such as poor lighting while reading results, errors in the measurement equipment such as tape measures, measurements that depend on other factors such as dust, temperature and finally human errors or mistakes. Most of these errors can be controlled and reduced using better lighting and more accurate equipment.

The most common errors in the laboratory testing could be related to the inaccuracies in the test set up as well as reading scales or equipment where some degree of estimation exists in the reading values. Repeatable measurements assist in obtaining the values closest to reality.

Sometimes due to systematic errors even the repeats cannot help in identification of the source of errors; for instance if a stop watch is not working properly, repeating the tests cannot reduce the element of error; in such cases the instruments should be calibrated and checked against another one (Taylor, 1982).

Test set ups were explained for the materials (refer to section 4.4) and the unit cell tests (refer to chapter 6, section 6.3). The same method of preparation was followed for each of the test set ups to avoid and reduce these sources of errors as much as possible.

The index tests performed on the China clay were conducted according to the British standards mentioned in the previous sections (Head, 2006). The standards mention possible mistakes and sources of errors and the guidelines give clear instructions on reporting the results. Where variable results are obtained from the similar samples, repeats are suggested to make sure values obtained represent the samples in the best possible way. The tests have been repeated in this research to increase the accuracy of the results.

4.4.2.2 Comparison and repeats

The results of the clay tests have been presented in chapter 5, (section 5.2) of this research. The reported values were checked against the British standard guidelines on the typical values where errors were considered. In this research, the results matched the estimated ranges reported in the standards (Head, 2006).

The procedures of the standards were precisely followed using the clear guidelines in all the tests to avoid the mistakes and errors as much as possible.

In case of the tests performed differently to the standards, the clear instructions were provided by the researcher to enable the reproducing of the tests using similar material, apparatus and conditions.

4.4.3 Leighton Buzzard sand

Uniform Leighton Buzzard sand was used in this research in the large unit cell container as a firm layer at the base to construct the column over it. It was also used as a platform

on top of the host ground in the LUC in order to level the host ground surface and keep the moisture of the Kaolin in the layers below for a longer duration for the tests. These two layers have been shown in the large unit cell cross section in Figure 4.1.

Both the layers were soaked in tap water and then put in the cell and lightly compacted using a hammer. Water was constantly sprayed over the top layer of the sand during installation and the testing to maintain the moisture of the sand and the layers below (refer to section 6.7.1).

In the pilot test for the large unit cell, Leighton Buzzard sand was used as the column material to install the column for the first time. This was performed in order to test the possibility of the column construction in the LUC and therefore the properties of Leighton Buzzard sand and the column constructed were not important in terms of the analysis and comparisons. The properties of the Leighton Buzzard sand were not tested using the index tests as the sand was not a material affecting the test results and was only used as a granular material where required.

4.5 Material testing-Stone column

4.5.1 Material source

Various aggregates have been used for years as column materials for the VSCs (Jefferson *et al.*, 2010). Primary aggregates (PA) such as granite have been used for many years. Use of the alternative aggregates has always been limited compared to the PA as alternative aggregates usually yield poorer results in the laboratory index tests. However, this gap in the performance may be insignificant for the purpose of VSCs in terms of the potential benefits such as cost reduction, environmental advantages and the performance criteria regarding the load carrying capacity and the settlement of the stone columns.

For the column material in this research four aggregates were used: Granite (PA), crushed concrete and brick (CC/CB) and incinerator bottom ash aggregate (IBAA) types 1 and 2. The three later aggregates were provided from the recycled sources. The CC/CB is one of the most commonly used recycled aggregates in the UK (Serridge, 2006). The IBAA is a type of RA with a high potential for the use in construction of VSC but with rare previous published data on its properties and the behaviour in the context of VSC (Hasan *et al.*, 2011).

1) Granite

The granite (PA) used in this research was sourced from a housing development construction site in Tipton, in the West Midlands for a VSC project. The samples taken were hand-filled in bags to represent the material used on site in terms of the size and the shape. Also, observation concluded that the material on site was quite uniform in terms of the crystal size and the mineral composition and was probably sourced from one rock unit.

The majority of the aggregates were sized between 20 to 50 mm, which was in accordance with the requirements for VSC construction. A small percentage was below 20mm which has been explained in the particle size distribution (PSD) test results in section 5.5.1. The granite has been used as a bench mark in comparisons of the primary and the recycled aggregates in this research.



Figure 4.3: Granite (PA) from Tipton site in a large shear box

2) CC/CB

The crushed concrete and brick used in this research was also provided from a housing development in Bilston in the West Midlands. The samples were hand-filled in bags and this was relatively difficult as the source was a combination of red brick, concrete and round pebbles. After observation of the source, materials were selected with 40% crushed brick, 40% crushed concrete and 20% rounded pebbles to represent the source used in the field in terms of the composition, the fragment size and the shape.

The brick fragments were red and round; the concrete was grey and included small clasts of 10 to 20mm diameter which were held together in a sandy matrix. The pebbles on site seemed to be from a different source and were only selected for the samples used in the research to represent what was present in the housing development site. The pebbles were sized between 20 to 60mm. The PSD of the CC/CB has been explained in section 5.5.1. The source had a higher proportion of larger aggregate sizes compared to the granite.



Figure 4.4: Crushed concrete and brick (recycled aggregate) from Bilston site

3) IBAA

Incinerator bottom ash aggregate (IBAA) can be a new source of recycled material for the use in the VSCs. The IBAA used in this research was supplied by the Keller Ground Engineering and was sourced from Ballast Phoenix, a company that processes and sells IBAA across the UK. The IBAA used was initially taken from Ballast Phoenix's Ridham Dock site in the southeast of England. The material collected was not sufficient for all the aggregate index tests and the unit cell testing, therefore, additional material was collected from the Ballast Phoenix's plant in the Castle Bromwich, in Birmingham.

The aggregates were expected to differ and the two batches collected were different in size, shape, composition and the physical appearance. The index properties and the differences of the two IBAA's used in this research have been fully explained in chapter 5 (refer to section 5.5).



(a)

(b)

Figure 4.5: (a) IBAA (1) from Ridham Dock, (b) IBAA (2) from Castle Bromwich

Both the batches of the IBAAAs were used in this research for the index tests and in the column construction for the unit cell testing, the results of which have been presented in chapter 5 (refer to section 5.5). The descriptions in this chapter only refer to the visual observations before any index tests were performed on both types of the IBAAAs.

The first type of the IBAAAs was collected from Ridham Dock and was called IBAA (1) and the second type was collected from Castle Bromwich and was called IBAA (2).

IBAA (1) was highly variable in the nature and contained a mixture of angular glass fragments and ceramics as well as metals such as springs, ball bearings and AAA batteries which were separated from the source before use in any of the tests on VSCs (ICE, 1987).

The glass and the ceramic bits observed in the samples were large in length and small in thickness, and gave the impression of brittleness and crushability. The particles were mainly between 10 to 20mm in size and were mostly finer than 10mm rather than above 20mm. The PSD has been further discussed in chapter 5 (section 5.5.1) and Appendix 3.

The material was not in the usual range of 20 to 75 mm recommended for the use in the VSCs (Serridge, 2006); however, other properties such as the degree of packing in the column, the angle of internal friction of the aggregates and the crushability resulted in unexpected behaviour of this material in the context of VSC which has been fully discussed in chapters 7 and 8.

The IBAA (2) was sourced from Castle Bromwich and its appearance was completely different from the IBAA (1). The colour was grey; and pieces of glass, ceramics and metals were covered in ash dust. Metal elements were separated from the source before being used in the tests. As the material was covered in ash its plate like feature was not apparent. As opposed to the IBAA (1), the aggregate sizes were mostly above 20mm

and below 5mm which confirmed high dust content and clumped pieces of material by ash as opposed to loose material visible in the IBAA (1).

In order to use the aggregates in the unit cell tests; the aggregate sizes smaller than 9.5mm were required for the scaling and the boundary conditions of the unit cells.

The CC/CB was crushed to produce particles with the required sizes for the index tests such as the AIV, ACV and TFV tests. If a smaller size range of the CC/CB aggregates were to be sourced to be suitable for the unit cell testing, the source might have been significantly different in the properties compared to the original aggregates obtained and tested; therefore, the same aggregates were crushed and used both for the index and the unit cell tests.

4) Small granite

In case of the IBAA (1) and (2), the sizes available were already suitable both for the index tests and the unit cell testing. Only the granite was different in the case of the index tests and the unit cell testing. The granite used as a source of the primary aggregates in the index tests was considered as a bench mark to compare the recycled aggregates with. This granite was too big to be used for the unit cell testing and instead of crushing the aggregates similar to the CC/CB; the granite was only crushed for the index tests. For the purpose of using the granite in the unit cell testing, a smaller size of the same type of the granite was ordered from an online distributor. This aggregate was produced for decorative purposes and gardening but was the same type as the original granite used in the index tests as well as having similar colour and structure.

The size of the second batch of the granite was between 3 to 8mm and was uniformly distributed. Observation showed more round edges rather than Sharpe ones and the

PSD, the shear box test, the AIV, ACV and TFV tests were performed on both types of the granite (original and small).



Figure 4.6: Small granite used for the unit cell testing

4.5.2 Aggregate tests

The following tests are among the standard aggregate tests and recommendations for the use of aggregates in VSC (ICE, 1987; BRE, 2000). The tests were performed on all the four aggregates (granite, CC/CB, IBAA (1) and IBAA (2)) and the results have been compared in chapter 5 (refer to section 5.6); also, the interpretation relevant to the VSCs has been provided.

1) Particle size distribution test (BSI, 2012):

As the aggregates used in this research needed to be granular and free from fines, they were properly washed before use in any of the aggregate index or the unit cell tests. For the PSD, the dry sieving method was suitable which was performed using the procedure described in BSI (2012).

The sieve sizes used for the different tests were variable. Aperture sizes of 50, 37.5, 31.5, 20mm and pan were used for the original granite. For the CC/CB, the IBAA (1) and (2) the sizes of 20, 14, 10, 5mm and pan were used.

The aggregate crushing procedure has been explained in section 8; in order to prepare the aggregates for the unit cell testing. The crushed aggregates as well as the small size granite were all sieved in sizes of 9.5, 6.3, 5, 3.35, 2.36, 2 mm and the pan for the purpose of the modelling in VSC in the unit cell tests.

The sieving method can contribute to some degree of crushing of the material itself and may not always be the most accurate representation of the sizes; however, for the purpose of many tests, distribution of the sizes was more important than the actual particle sizes recorded (Head, 2006).

2) Aggregate impact value test (BSI, 1990e):

In this test only particles between 10 to 14mm were subject to the impact forces according to the (BSI, 1990e). Therefore, the brick crusher was used on the big (original) granite and the CC/CB to crush the particles into the appropriate size required. Use of the brick crusher has been fully explained in section 8.

The aggregate impact value (AIV) can be obtained from equation 4.2:

$$AIV = \frac{M_2}{M_1} \quad \text{Equation 4.2}$$

Where M_1 is the total mass of the sample in grams; and M_2 is the mass of the material passing 2.36 mm sieve in grams.

Results of the AIV have been presented in chapter 5 (section 5.5.2).

3) Aggregate crushing value test (BSI, 1990f):

The equipment and the procedure are fully explained in the BSI (1990f). After the sample is prepared and it is ensured that it has a smooth surface in the mould, it should

be placed at the centre of the aggregate crushing machine to be loaded. Load is applied from the top at a stationary rate to reach 400kN in 10 minutes (± 30 seconds).

The apparatus used in this research was computer controlled but due to technical problems, the loading had to be adjusted manually. A screen existed on the machine showing the load applied. Using a stopwatch and an estimation of 10kN increase in load at every 15 seconds, the proper load was applied.

It was essential to apply the load steadily and dials and switches were available to control the load application which was successful in all the tests. The results have been presented in chapter 5 (5.5.3) where the ACV was calculated via equation 4.3:

$$ACV = \frac{M_2}{M_1} \quad \text{Equation 4.3}$$

Where M_1 is the total mass of the sample in grams; and M_2 is the mass of the material passing 2.36 mm sieve in grams.

4) Ten percent fines value test (BSI, 1990c):

The procedure and the equipment used for the TFV test was exactly the same as descriptions in the BSI (1990c). The TFV can be calculated via equations 4.4 and 4.5:

$$F = \frac{14f}{m + 4} \quad \text{Equation 4.4}$$

$$m = \frac{M_2}{M_1} \times 100 \quad \text{Equation 4.5}$$

Where F is the force in kN, required for 10% fines to be produced for each specimen,
f, is the maximum force applied in kN,
m, is the percentage of the material passing the 2.36mm sieve at the maximum force
 M_1 is the total mass of the sample (grams)

M_2 is the mass of the material passing 2.36 mm sieve (grams)

The results should represent the load at which 10% fines are produced in the sample. It is not possible to find the load at which the exact 10% value is obtained, however, the tests were repeated several times and the closest values to the 10% fines were considered the values at which the load was considered as the best result. The details have been presented in chapter 5 (5.5.4) and Appendix 3.

5) Los Angeles test (BSI, 2010):

The LA test was performed based on the procedure described in the BSI (2010). However, the condition in which the force was applied to aggregates under the rotational movements in the LA drum was far from the condition that aggregates experience in the context of VSCs. This test was performed as part of the index tests recommended by the standards on both the primary and the three recycled aggregates (ICE, 1987). As this test was not used in the interpretation of the behaviour of materials in the unit cell tests, it was not repeated on the small granite used in the unit cell testing.

The LA value is calculated via equation 4.6:

$$LA = \frac{5000 - m}{50} \quad \text{Equation 4.6}$$

Where m is the mass of the material retained on the 1.6mm sieve (grams).

The results have been presented in chapter 5 (section 5.5.5) and Appendix 3.

6) Large shear box test (BSI, 1990c):

The large shear box apparatus is used for the measurement of the angle of shearing resistance of the granular material. The large shear box allows testing of the larger particles which are more representative of the aggregate size range used for the VSC.

The full procedure of performing the shear box test is presented in BSI, (1990c). The exact procedure was planned to be followed for all the aggregates (the two primary and the three recycled sources).

In order to find out about crushing of the aggregates during the shearing process, the PSD tests were to be performed before and after the shearing of each material and it was originally planned to apply normal pressures of 60, 120, 180, 240 and 300kPa on each sample. The tests were planned to be repeated once for each material at each normal pressure.

The speed of the shearing was adjusted using the gear box to shear the samples with a constant rate suitable for the drained condition which was not too slow or too fast (BSI, 1990c). At the shearing speed of 0.71 mm/min, the readings should be taken for every 0.25 mm of the horizontal displacement. The readings were taken from the proving ring to show the shear stress and also, the vertical movements over the lid of the sample.

The apparatus used in this research had limited travel due to the partly broken thread between the driving shaft and the gear box. It was controlled throughout the test that the travel was not beyond the maximum travel available, otherwise the thread would have been more damaged and inaccurate results were produced. Therefore, the test should have been stopped either when the shear strength started to reduce or when the maximum travel was achieved.

It became apparent during the first test that the damaged thread was affecting the data. For a few minutes no horizontal movement was observed and the test had to be stopped. In this test the normal pressure of 60kPa was applied on the big granite. After unloading and further inspection, it was observed that the thread was completely warped and had

to be taken out. The thread was replaced in a few days with a new one, but the second test showed that the same problem was repeated.

As each time stopping the test made the results inaccurate, it was decided that the use of the large shear box was not feasible for this research. Many other researchers have performed the large shear box test on various primary and recycled aggregates (Chidiroglou *et al.*, 2008; McKelvey *et al.*, 2002; Tranter *et al.*, 2008). The results can be used in interpretation when similar PA or RAs were tested.

It was finally decided to perform the small shear box test on the small size granite purchased later, the crushed CC/CB and both the IBAs. Although the small shear box test is not a good representative of the behaviour of the aggregate sizes for the use in VSCs; it can be an indicator and the results can be compared to the available data in the literature regarding the estimation of the angle of internal friction for the various materials.

7) Small shear box test (BSI, 1990c):

The aggregate sizes used in the unit cell testing were between 2 to 9.5 mm and were too big for the small shear box test; however due to the damage of the large shear box apparatus, the small shear box was conducted on the small granite and the three recycled aggregates.

Proper loading discs were chosen for the application of normal pressures of 60, 120 and 180kPa. Each load was applied two times on each material. The maximum travel of 16 mm with the shearing rate of 1.2mm/min was selected. Readings were taken at every 0.20mm of the horizontal displacement, where the shear strength and the vertical displacements were recorded. After the maximum travel was achieved, the test was

stopped and unloaded and the PSD of the material was performed to be compared to the PSD results before the shearing.

It was noted that in a few tests, the lid of the box tilted over the aggregates and the pressure was not applied vertically over the sample. However, this should not affect the results as it happened towards the very end of the test and beyond the failure point.

The failure envelope and the angles of internal friction were the crucial findings of the small shear box test. The results have been presented in chapter 5 (section 5.5.6) and the details in Appendix 4 (refer to the attached CD).

8) Aggregate preparation

According to the standards, the aggregates were prepared before each test.

The process of washing and drying was performed for each test. The important aspect was to make sure the dust and fines were removed from the aggregates. The dust might have been introduced to the aggregates during the storage, transportation or crushing.

The big granite and the crushed concrete and brick were crushed via a brick crusher to produce the sizes required for the aggregate index tests. The crushing of aggregates produced fines and sharper edged aggregate fragments. The fines were removed in a second washing and drying process. The sharper edges of the aggregates were affected by the sieving procedure. Each time the aggregates were sieved it was noted that the particles became rounder. However, the distribution of the aggregate ranges was more important than the size or the angularity for the purpose of this research.

Sieving for 10 minutes might also affect the breakage of the aggregates. Especially in the tests that the same set of sieves were used before and after the test, some addition of fines might be due to the sieving action and several impact forces applied to the material

from the metal sieves and other aggregates (Ashton, 2008); however, the same method was used for all the tests and the results were consistent (BSI, 2012).

For the crushed material that the washing and drying process was repeated several times, the addition of water and the oven drying might affect the properties of the material. This can also happen while the aggregates are stored if they are subject to several rain and sunshine or freeze and thaw cycles.

4.5.3 Evaluation of aggregate index tests

4.5.3.1 Errors in the laboratory tests

Similar sources of errors mentioned in section 4.4.2.1 for the clay tests, can also cause errors in the aggregate tests. Poor lighting, measurement equipment errors, systematic errors and human mistakes can contribute to inaccurate results. However, the aggregate index tests were performed following the exact procedures described in the British standards and in case of mistakes, tests were repeated.

According to the standards, the index tests have to be repeated several times and the values reported as final results are average values of several tests. Where two results are different, a third one is recommended to make sure the average value is a proper representation of the aggregate properties. The detailed results have been attached in Appendix 3.

The process of washing, drying, sieving and the sample preparation was performed with care to avoid damage to the samples. Washing was handled with care to avoid the particle breakage. The sieving procedure was repeated several times before and after all the aggregate index tests and can be a source of particle breakage and inaccurate results.

Some of the fines present in the material after sieving can be contributed to the sieve shaker apparatus and its exerted vibrational forces.

When the samples are being loaded in the ACV and the TFV, the sample was adjusted at a position that the load would be applied at the centre. The apparatus used for these two tests was controlled manually in this research which could result in inaccurate load application and its rate. The problem was tackled with care and the rate was accurately controlled and adjusted every 15 seconds to create the required loading rate.

Due to the damage to the large shear box apparatus, the shearing of the aggregates had to be performed in the small shear box, which created inaccuracy due to the size limitations of the box and the aggregate sizes tested. The values obtained and reported are only used as guidelines and were checked against other sources (Chidiroglou *et al.*, 2008; McKelvey *et al.*, 2002; Tranter *et al.*, 2008).

4.5.3.2 Comparison and repeats

If the procedures described in the British Standards on the aggregate index tests are followed precisely, the tests can be easily reproduced.

The problem with aggregate testing is that the material is sourced from variable primary or recycled sources and the comparison of results requires a lot of information on the original source, its structure and the geological background (in case of the PAs).

The values reported can only be compared for the specific sources tested. All the primary and the recycled aggregates cannot be compared in the way the material used in this research has been compared. The reason for the aggregate index tests and the comparison of the primary and the recycled aggregates in this research was to be able to analyse the material behaviour in the context of VSC via the unit cell tests.

4.6 Summary of the material tests

In this chapter the materials used in the unit cell modelling of the VSCs were divided into two main categories of the host ground and the column material.

The host ground was Kaolin clay, which should be prepared to represent a soft host for the construction of a single column in the unit cell tests. Therefore, the compaction test with the specific moisture content and dry density at which the undrained shear strength of between 10 to 25 kPa could be gained was a necessary test for this material parallel to other basic tests of the PI and the specific gravity.

The column materials include 1 primary (granite) and 3 recycled aggregates (CC/CB, IBAA (1) and (2)). The aggregate index tests are recommended by the standards for these materials to be used in the VSC construction.

The aggregates were crushed (when necessary), washed and dried and the AIV, ACV, TFV, LA and the shear box tests were performed on them. The results of these tests define whether these materials are suitable for the use in the VSCs or not. Chapter 5 presents the material index test results.

CHAPTER FIVE

RESULTS AND DISCUSSIONS- PART 1: MATERIAL TESTS

5 RESULTS AND DISCUSSIONS- PART 1: MATERIAL TESTS

In this chapter the results of the material index tests have been presented. The results basic clay properties and their connection with the requirements for the unit cell testing.

The aggregate index test results have also been presented, and the discussion has been provided specifically for the use of various aggregates in the context of vibro stone columns. The results and discussions of the aggregate tests show that most of the materials tested were not suitable for the use in the VSC modelling; however, the materials were used in the modelling to assess the validity of the aggregate index tests.

5.1 Introduction to material results and discussions

In chapter 4, the tests performed on the materials were fully explained. In this chapter the results of the material tests have been reported followed by the discussions and comparisons. The details of the measurements and calculations have been presented in Appendices 1 and 3. This chapter only presents the final results obtained.

Comparisons of the results can be with standards, other authors and published works, comparisons with other research by postgraduate students at the University of Birmingham and comparing the behaviour of the various aggregates used in this research with each other.

5.2 Clay results and discussions

As mentioned in chapter 4, Kaolin or China clay was used as the host ground in all laboratory tests on the performance of VSC in this research. Therefore, its properties should be defined before use as a host material. The criteria defined were the moisture content of 41% and the undrained strength of between 10 to 25 kPa (± 2) to provide a soft host ground for the columns to be installed and loaded (refer to section 5.3). In order to achieve this, the soil should be mixed with water and compacted to certain level of densification. In order to predict the behaviour of the host ground under these conditions, its basic properties such as the natural moisture content, the plasticity index, the specific gravity, and the compaction behaviour should be identified.

The process of each of these tests was explained in chapter 4, section 4.4.1. The details of the laboratory readings and the graphs have been presented in Appendix 1. The final results followed by their discussions have been presented in this chapter.

5.2.1 Clay composition and its technical data

Table 5.1 summarizes the important characteristics of the clay used in the laboratory tests which was provided by the manufacturer. More details have been presented in Appendix 1.

Table 5.1: Highlights of the technical data of the English China clay of type Puroflo 50, provided by WBB Devon Clays Ltd

Analysis	Results
Particle size distribution	Equivalent spherical diameter
	Microns: 1 2 5 10 20
	% passing: 37 49 76 94 99
PH value	5.1
Mineralogical composition	Composition
	Kaolinite
	Potash Mica
	Soda Mica
	Quartz
	Rational analysis
	64
	24
	2
	6

As observed in Table 5.1, the host ground used was acidic, and mostly consisted of Kaolinite. Also, due to the other components it was expected to have slightly higher permeability compared to other clayey soils in general (Head, 2006). The clay was used in all the unit cell tests, and therefore, in comparison of the behaviour of the various stone columns, the soil composition was not one of the factors considered in the stone column performance in the short-term and had a fairly constant condition in all the tests.

5.2.2 Natural moisture content

The natural moisture content of the clay in the laboratory was measured three times. Each series had three samples. The three samples of each series were taken from one bag of Kaolin, therefore, the various range might be representative of the different storage conditions of the bags and the various moisture contents in the laboratory at different seasons. The detailed results are presented in Appendix 1.

Table 5.2: Results of the natural moisture content on clay, repeated three times

Series	Average value of the three samples (%)	Value reported (%)
1	0.54	0.5
2	0.87	0.8
3	0.84	0.8

The natural moisture content considered for the China clay was reported as 0.7% which is the average of the three values reported with accuracy of 0.1% (BSI, 1990a 3.2). This value was negligible for the purpose of mixing the soil with tap water for the unit cell tests. As the moisture content of 41% is to be achieved, it is assumed that the clay used was originally dry and a moisture content equivalent to 41% of the clay mass was added for the unit cell tests.

5.2.3 Plasticity index

The liquid and plastic limit tests were performed on the clay using both the distilled and tap water. The details have been presented in Appendix 1, and Table 5.3.

Table 5.3: Plasticity index of the clay with distilled and tap water

Test	Sample	Result (%)	Plasticity index (%)	Average (%)
LL with distilled water	1	56	26	26
PL with distilled water		30		
LL with distilled water	2	56	25	
PL with distilled water		31		
LL with tap water	3	54	20	20
PL with tap water		34		
LL with tap water	4	54	20	
PL with tap water		34		

The value of 20% was considered as the plasticity index of the China clay with the tap water, as the tap water was used in all the unit cell tests to be mixed with the clay. The results showed that the distilled and tap water affect the liquid and plastic limits of the China clay, especially in the plastic limit tests. This was due to the existence of the

minerals and salts in the tap water which affected the properties of the soil (Head, 2006).

Despite using the tap water, consistent results were produced in the layers of clay in the unit cell tests. The typical range for the liquid limit was between 40 to 60 % for the Kaolinite and in both cases of the distilled and tap water; the results were in the acceptable range. In case of the plasticity index test the acceptable range for the Kaolinite was between 10 to 25 %. In case of the distilled water, the result was slightly higher than the acceptable values as opposed to the plasticity index measured with the tap water, where the results were acceptable (Head, 2006).

5.2.4 Specific gravity

The details of the SG results have been attached in Appendix 1. The result of the SG obtained in the laboratory using the density bottles was 2.6353 which was reported as 2.63 or 2.6 (BSI, 1990a 8.3) that is in the usual range mentioned for clays (Head, 2006). This value was used in the calculations of the degree of saturation of the clay for the large unit cell tests (refer to section 6.5.4).

5.2.5 Standard compaction test

The standard compaction test in which three layers of soil are compacted via a standard hammer was performed to obtain the optimum condition of the Kaolin used. The maximum dry density was in the range of 1.48 to 1.51 kg/m^3 , with the optimum moisture content of 27 to 29 %.

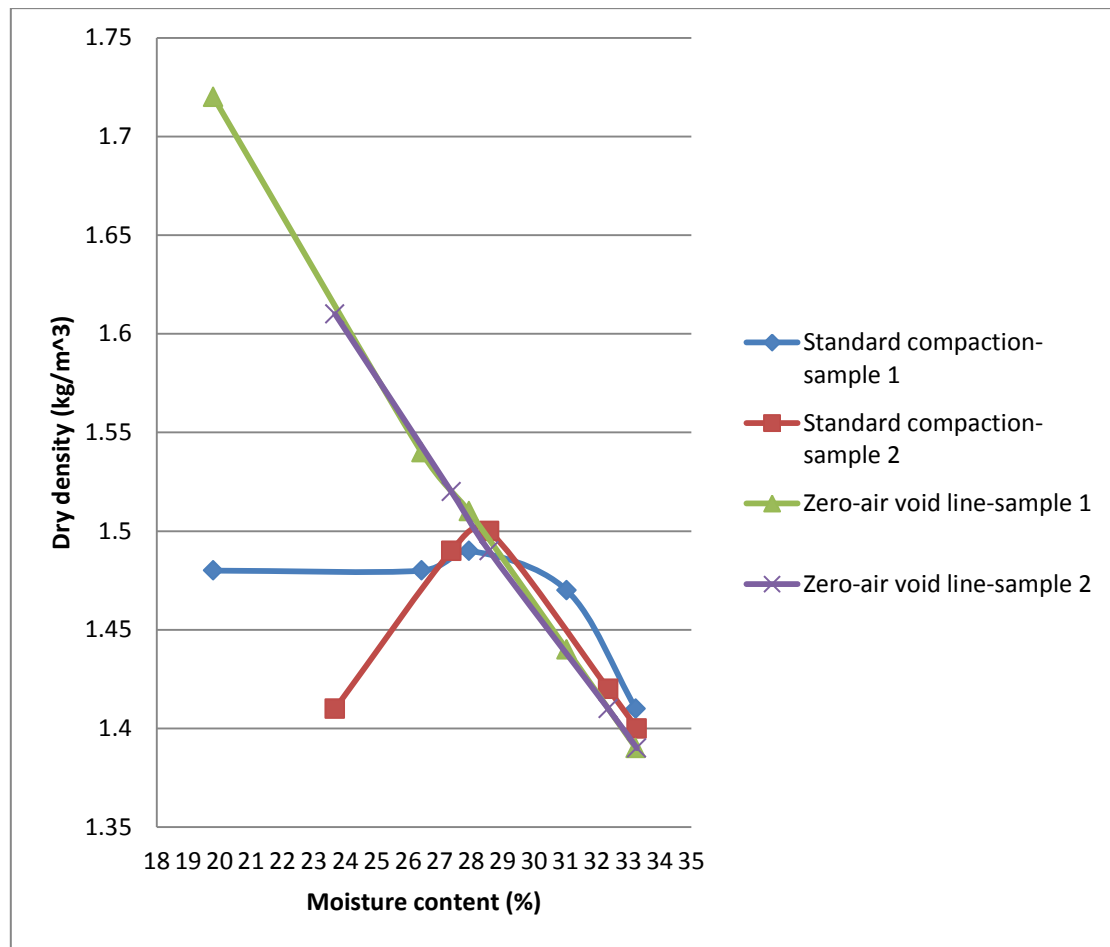


Figure 5.1: Standard compaction test and repeat, with zero-air void line

As this type of compaction was not used in this research, the results were not used as guidelines in the preparations of the Kaolinite for the unit cell tests. Sample 2 was the repeat test for sample 1. It should be noted that the first point in sample 1 in Figure 6.1 was an error of compaction by the researcher which was modified in the test procedure for the sample 2 and therefore, sample 1 should have a similar trend to sample 2 when test is performed correctly from the beginning. At the final points, the samples were very close to the zero-air void line which was due to the errors involved in the procedure of the compaction test. The undrained strength of soil was also measured and the details of the results of the standard compaction tests have been provided in

Appendix 1. Figures 5.2 and 5.3 show the 100%, 95% and 90% saturation for both the samples.

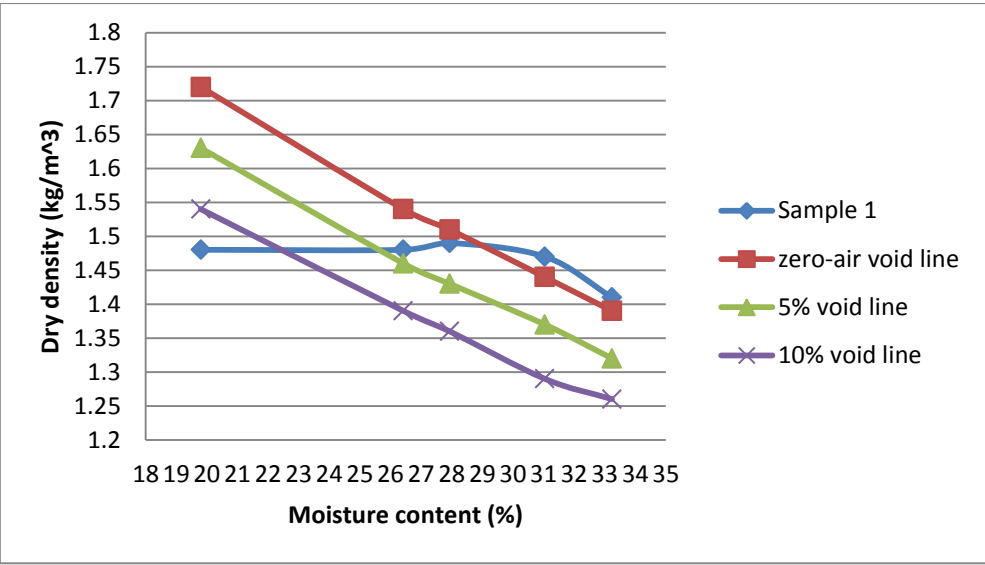


Figure 5.2: Standard compaction test on sample 1 with 0, 5 and 10% air void lines

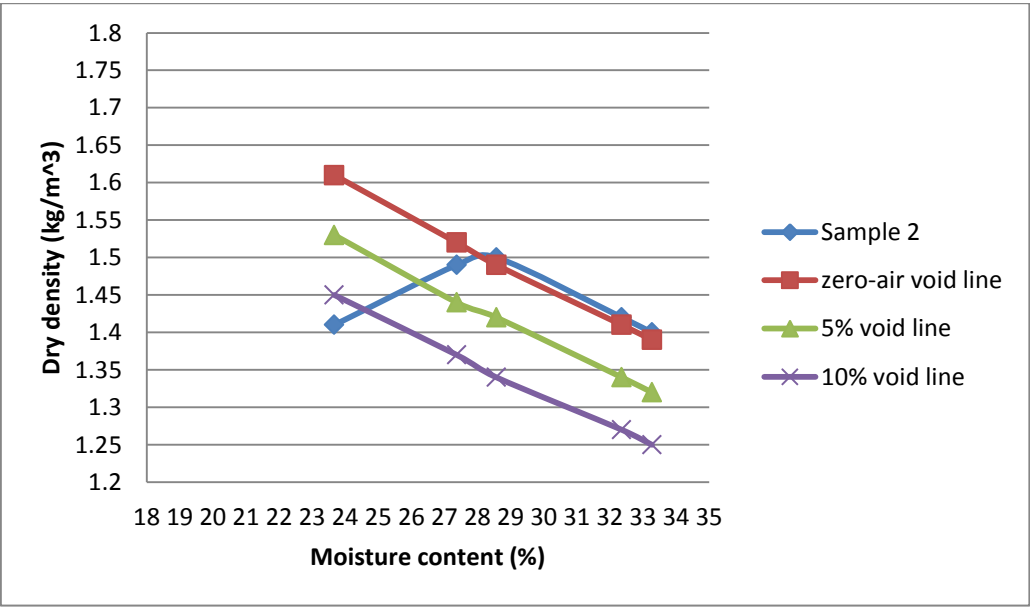


Figure 5.3: Standard compaction test on sample 2 with 0, 5 and 10% air void lines

5.2.6 Compaction via the vibrating hammer

Due to the requirements of this research for the unit cell testing, the Kango hammer was used to compact the samples. This was first tried using 10 and 15 seconds of compaction per layer. Due to significant error of the 10 seconds compaction per layer, it was abandoned after the first trial. Instead, three samples were tested with 5 layers of the China clay being compacted for 15 seconds per layer. The results have been presented in Figures 5.4 to 5.8:

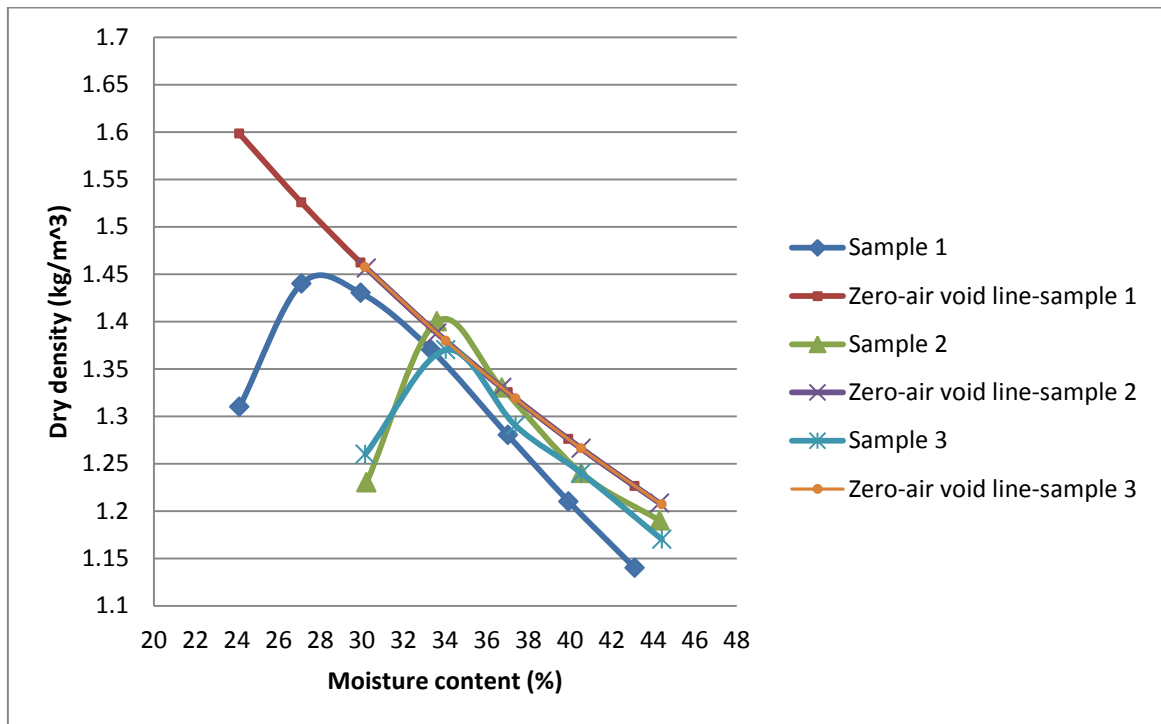


Figure 5.4: Compaction results via vibrating hammer-15 seconds compaction per layer

According to Figure 5.4, sample 1 was inconsistent compared with the other two samples, and showed the optimum dry density of approximately 1.45 kg/m^3 at the optimum moisture content of around 28%. Sample 2 was compacted and as the results of samples 1 and 2 were different, the compaction was repeated on the third sample. Samples 2 and 3 showed the maximum dry density to be between 1.37 and 1.41 kg/m^3 with the error margin of between 1.35 and 1.45 kg/m^3 .

These values were obtained at the optimum moisture content of between 33 to 35%.

These graphs showed that the moisture content requirement for the unit cell test, which was 41%, was beyond the optimum dry density of the China clay. At this moisture content, the dry density observed in samples 2 and 3 was around $1.24 \text{ kg/m}^3 (\pm 0.05)$.

Figures 5.5 to 5.7 demonstrate each of the dry density curves for the three samples including the 100, 95, and 90% saturation curves. It was observed that the density curves in all the three cases mostly fell between the 0 and 5% air void lines, very close to the saturation condition in the range of the moisture contents for which the compaction tests were performed.

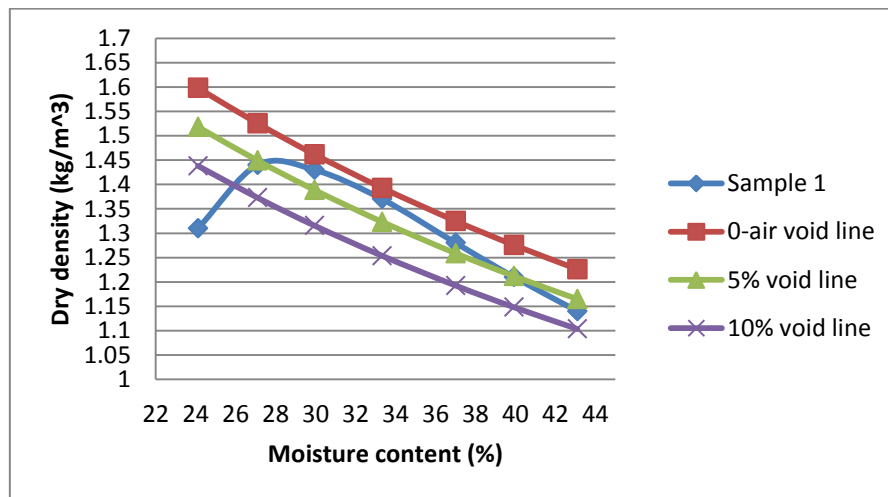


Figure 5.5: Compaction via vibrating hammer-sample 1; 0, 5 and 10% air void lines

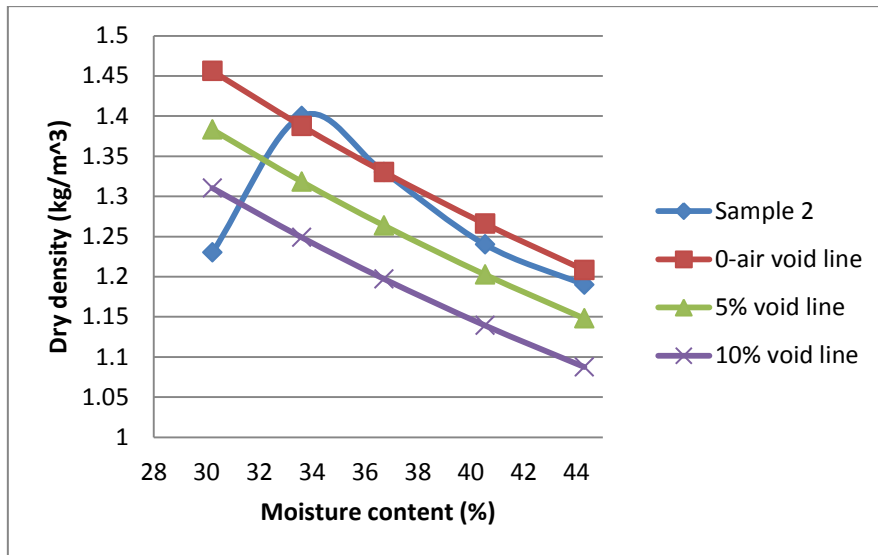


Figure 5.6: Compaction via vibrating hammer-sample 2; 0, 5 and 10% air void lines

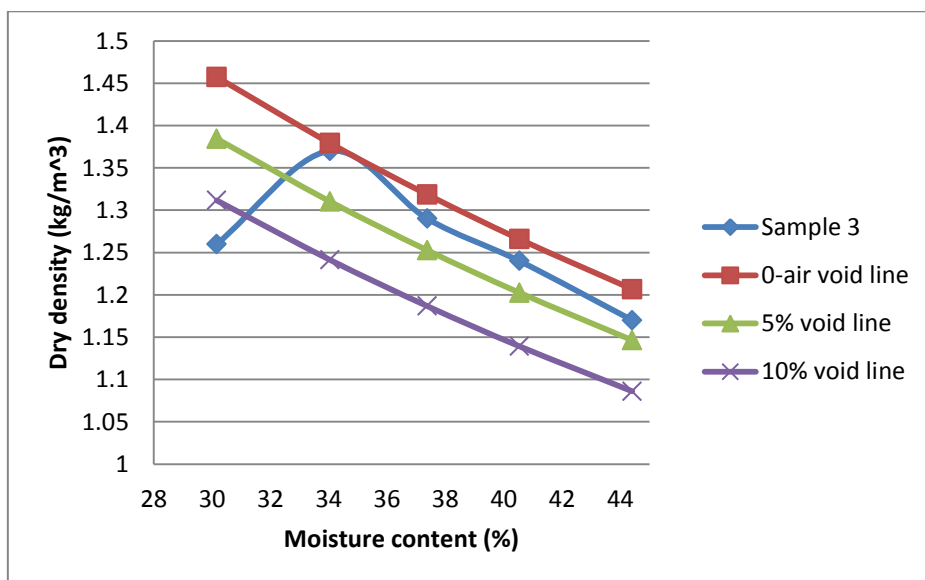


Figure 5.7: Compaction via vibrating hammer-sample 3; 0, 5 and 10% air void lines

Figure 5.8 shows the interaction of the dry density and the undrained strength of the three samples tested. The vertical axis on the left is the dry density and the one on the right shows the undrained strength values measured via the hand vane shear apparatus presented in kPa. As observed, the increase in the moisture content results in rapid reduction in the undrained strength of the soil. The initial criteria to prepare the host

ground for VSC testing was defined as very soft soil with the undrained strength of between 10 to 25kPa. According to this graph, these values required a moisture content range of between 38 to 44%. The average value of the moisture content was 41% which was considered as the aiming value in the host ground mixes. However, the range of between 38 to 44% was acceptable as it should still provide the undrained strength suitable for the VSC testing.

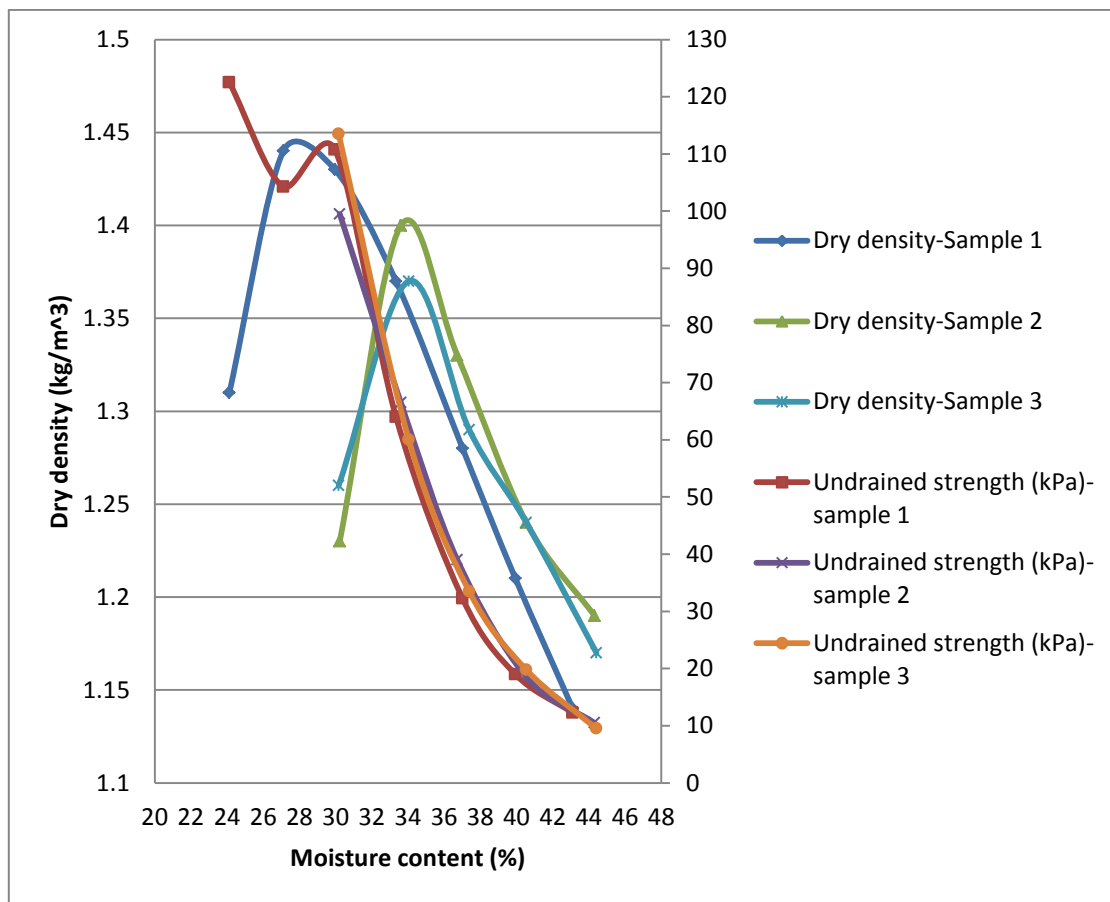


Figure 5.8: Compaction via the vibrating hammer, the dry density and the undrained strength on the three Kaolin samples-15 seconds of compaction per layer

5.3 Host ground requirements for the unit cell testing

In order to assess the required properties of the host ground in the unit cell tests, after performing the index tests on the Kaolinite, the small unit cell was used to control the undrained strength and the moisture content of the samples in trials.

Two tests were performed where the container was only filled with the China clay. In the first attempt, there were three layers, each layer having thickness of 130 mm; and a total depth of 390 mm. In the second test, clay was filled in 5 layers, each having thickness of 80mm, reaching a total depth of 400mm. Both the tests were compacted for 4 minutes per layer, which was the time estimated and tried for the compaction of the small unit cell tests (refer to Appendix 2).

The clay was left in the container overnight and the next day, samples of moisture content and the undrained strength were taken from each layer.

In order to take the moisture content samples, 5 holes were drilled in the clay using the installation tube and the auger used for all the unit cell tests. From each of the cores 10 samples were collected for the moisture content. After sampling, the clay left which did not collapsed into the holes was cleaned out in layers and the values of the undrained strength were recorded via the hand vane apparatus.

As well as the moisture content and the undrained strength, in the second test, the dry density range of the clay was measured via four samples taken from each layer in the container with pre-measured volumes.

The details of the results have been presented in Appendix 1, and the summary of the results has been presented in Table 5.4:

Table 5.4: Quality control of the host ground in the small unit cell container

Test	Number of layers	Range of average undrained strength values (kPa)	Range of moisture content values (%)	Range of average of dry densities (kg/m^3)
Test 1	3	14 to 18 (± 2)	38 to 43 (± 0.1)	Not measured for test 1
Test 2	5	14 to 17 (± 2)	39 to 43 (± 0.1)	1.25 to 1.28 (± 0.05)

The moisture content was in the range to provide the undrained strength of between 10 to 25 kPa which was required in this research. Despite the number of layers, similar

results were obtained. The sample in the second test was subject to higher degree of compaction as depths of the layers were smaller and each layer was also compacted for 4 minutes similar to the first test.

According to Figure 5.8, the obtained range of the dry density from these tests, agrees with the values shown on the dry density curve for the moisture content value of 41%.

5.4 Evaluation of the host ground results

The results of the clay index tests were compared to the standard ranges available for similar materials (Head, 2006). The most important factor was the moisture content and the energy of compaction in the unit cell tests to provide the undrained shear strength of below 25kPa. The values were checked both in the standard compaction mould and in the small container. The level of compaction and depths of the layers provided the strength required for column the installation in the unit cell tests.

5.5 Aggregates-results and discussions

Five aggregates were used in this research: two forms of granite, CC/CB, IBAA (1) and (2). The materials were tested for their index properties via the PSD, AIV, ACV, TFV, LA and the shear box tests.

In cases of IBAA's, there are no published data to compare the results with. Some of the other results were compared to previous research by the postgraduate students at the University of Birmingham such as Tetteh (2007) and Ashton (2008). Direct comparison was not possible for many tests and certain assumptions had to be used to allow the comparison of the results. For instance, neither of previous researchers used the mixed CC/CB and crushed concrete and crushed brick were tested separately, therefore, in order to allow comparison, the average values of the index tests results were used for

crushed concrete and crushed brick to be compared with the mixture of both tested in this research.

The big granite was compared to the previous results on basalt which was referred to as the natural aggregate by Tetteh (2007) and Ashton (2008). Due to the ambiguity in the description of this source of aggregates in the previous research, direct comparison between the granite and ballast was not possible. The main form of comparison was their behaviour in the unit cell tests and the index properties against each other.

As the small granite was smaller than 9.5 mm in size, it was not suitable for most of the index tests; however, the tests had to be altered in order to achieve an estimation of the material behaviour.

The index tests provide an understanding of the behaviour of the materials to some extent; however, the question was whether these were suitable criteria regarding the VSC construction. Many of these materials show unacceptable results in the index tests, however, the results were completely analyzed in chapters 7 and 8 in the context of VSC installation and loading.

5.5.1 Particle size distribution

Figure 5.9 shows the PSD of the various aggregates used in this study as supplied. The graph represents the PSD before the aggregates were crushed for the purpose of the index tests and the unit cell testing and therefore, the diversity in the ranges of the PSD was observed. After the original PSD, the particles above 50 mm were separated and not used for any of the tests.

The big Granite, the CC/CB, the IBAA (1) and (2) were subject to the particle size analysis in their original state with their initial particle sizes before being crushed for the

other tests. As observed in Figure 6.9, the big granite and the CC/CB were originally much bigger in size than the IBAA's. The small Granite was purchased with the sizes of less than 10 mm which was the size required for the use in the unit cell testing. All the other aggregates were crushed before being used in the tests.

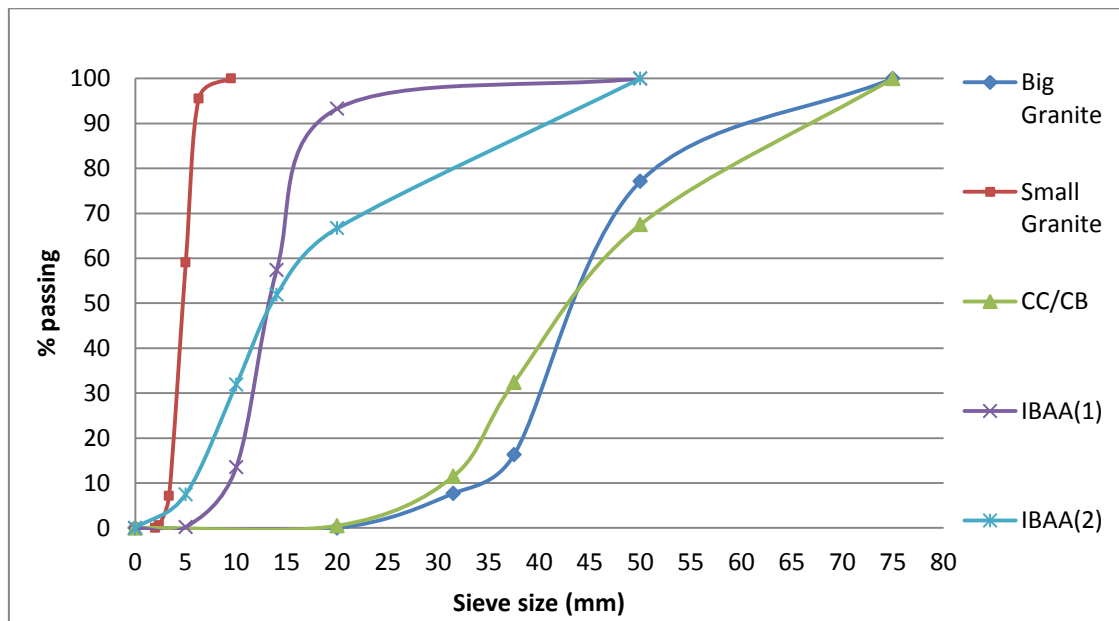


Figure 5.9: Particle size distribution curves for the aggregates as supplied

The PSD analysis showed that the majority of the big granite fragments were sized between 20 to 50mm, with very low percentage below 20mm. This is the typical aggregate size used in the real VSC construction; however, this size was not used for the unit cell tests due to the scaling limitations.

The crushed concrete and brick was also similar to the big granite in terms of the PSD, where most particles fell above 20 and below 50mm in size, however, a higher proportion of aggregates above 50mm in size were observed in the original sample, which was not used in the sieve analysis. Comparing the big granite with the CC/CB concluded that a higher percentage of the material fell between 32 to 46mm in case of the CC/CB compared to the granite.

A similar trend was observed for the two types of IBAAAs. IBAA (1) mainly consisted of particles between 10 to 20mm, with a higher percentage above 13mm compared to the IBAA (2). It was observed that only 13.4% of the material was smaller than 10mm in IBAA (1) and even a smaller percentage of 6.6% above 20mm. The material was subject to the PSD in its original state and it was not the recommended range of 20 to 75mm for the VSC purposes. The IBAA (2) had a higher percentage above 20mm compared to the IBAA (1), and also, a higher percentage below 5mm. This represented the high dust content in the source.

The small granite which was ordered with a specific size limitation to be used for the unit cell testing was 100% below 9.5 mm in size. The material was mostly between 6.3 to 9.5mm, with a lower percentage between 5 and 6.3 mm. There were fines in the source which fell below 2mm, however, each time before the unit cell tests, the aggregates were sieved and only the sizes above 2mm were used in the unit cell tests.

5.5.2 Aggregate impact value

The procedure for this test was explained in chapter 4 (section 4.5.2) and the details of the calculations of the AIV have been presented in Appendix 3.

The mean value of the three tests performed on each material has been presented in Table 5.5. The exception was the IBAA (2), where due to the limitation in the source availability; the mean value was the average of the two tests performed.

Tetteh (2007) and Ashton (2008) did not perform these tests on the IBAAAs and therefore, no results were available by these authors for the comparison and the actual results were only compared to the BRE (2000) in case of the IBAAAs.

Table 5.5: Aggregate impact values, actual results and comparisons

Material	Actual AIV (%)	BRE (2000) recommended value (%)	Expected value by Tetteh (2007)	Expected value by Ashton (2008)
Big Granite	4.1	<30	20	11.4
Small Granite	12.7		-*	-
Crushed concrete and brick	17.3		30.3	36.6
IBAA (1)	27.8		-	-
IBAA (2)	22		-	-

*For these materials no previous results were published to be compared to the actual results obtained in this research

The AIV is an indicator of the behaviour of the material under impact forces. Higher percentage of the AIV shows higher susceptibility for the breakage of the particles under static impact loads. For this test all the materials except for the small granite were graded to sizes between 10 to 14mm and then tested. The available range of the small granite was used; therefore, the comparison of the results of the other material with the small granite was not accurate.

The results showed that all the materials (the primary and the recycled), had an AIV below 30% which was the recommended value by (ICE, 1987; BRE, 2000); with the IBAA (1) showing very close value to 30%; although the granite and the CC/CB showed much better results compared to the IBAA's.

The AIV of the CC/CB and granite was also much lower than the previous findings of Tetteh (2007) and Ashton (2008). On the other hand, direct comparison with these research was not possible, as the type of the primary aggregates used was different from the granite and the CC/CB used in this research which was a mixture as opposed to the other research where the crushed concrete and the crushed brick were tested separately and the values shown in the table are the average values of the two separate materials.

The results of the AIV tests in research showed approximately a 50% lower AIV for the granite and CC/CB compared to the previous data obtained.

In comparison, the granite performed better than all the recycled aggregates by a large margin; which was in accordance with the previous theories that the primary aggregates perform better than the recycled ones. After the granite, the CC/CB outperformed IBAA's; and among the two types of the IBAA's, the first type showed poorer results than type 2. The composition and the plate like shape of the particles might cause more crushing in the IBAA (1), as opposed to the IBAA (2) where the ash and dust covered and held the particles together under the impact forces.

5.5.3 Aggregate crushing value

The procedure for this test was explained in chapter 4 (section 4.5.2). The detailed calculations have been presented in Appendix 3.

Similar to AIV, due to the limited quantity of IBAA (2) available, the test could only be performed once on this material, whereas other results shown in Table 5.6 are the average values of the tests repeated on each material.

Table 5.6: Aggregate crushing values, actual results and comparisons

Material	Actual ACV (%)	BRE (2000) recommended value (%)	Expected value by Tetteh (2007)	Expected value by Ashton (2008)
Big Granite	24.8	<30	25.4	14.9
Small Granite	40.2		.*	-
Crushed concrete and brick	33.9		29	29
IBAA (1)	47.6		-	-
IBAA (2)	41.1		-	-

*For these materials no previous results were published to be compared to the actual results obtained in this research

This test is an indication of the aggregates behaviour under prolonged loading. The higher the percentage of the ACV is means that more fines are produced under the loading, which is not favorable for the purpose of VSC construction.

Similar to the AIV, the results of the ACV showed that the actual values obtained in this research were slightly different from the previous research. This could be contributed to the nature of the material used, and also the difference in the apparatus used for the loading. In case of the small granite, this material was only tested to be used in the scaled unit cell tests and no previous results exist in the mentioned previous research (Tetteh, 2007; Ashton, 2008) to be compared to the actual results of this material.

All the ACVs were above the recommended values (ICE, 1987; BRE, 2000), except for the big granite. The IBAA's were the extreme case where approximately 50% more fines were produced than the recommended values. Even the big granite showed values very close to 30% that was recommended. This might indicate that the recycled aggregate are not appropriate compared to the granite to be used under prolonged loads.

The general trend indicates that the big granite performed better than the small granite, CC/CB and IBAA's. Only the big granite can be accepted based on the BRE (2000) recommendations. The results of the small granite and the three RAs were similar and the CC/CB outperformed the other types of the RAs.

In case of the IBAA's, IBAA (2) was better than IBAA (1) under prolonged loading. This might be due to the clumped nature of the IBAA (2) that not only held the particles together under the impact forces of the AIV, but also keeps the matrix intact under the static loading of the ACV.

According to the standards certain sizes of the aggregates should be used in the construction of VSC (BRE, 2000). The small granite was ordered based on the requirements of the aggregate sizes to be used in the construction of VSCs which in this research was scaled for the unit cell modelling and therefore, was not graded according

to the standards for the aggregate index tests, and the results were not accurate representation of this material's behaviour due to the error of the size.

5.5.4 Ten percent fines value

The procedure of the TFV test was explained in chapter 4 (refer to section 4.5.2) and the detailed calculations have been presented in Appendix 3.

Based on the experience on the ACV test, the load to produce 10% of fines in the material was estimated and the three tests performed on each sample were loaded to provide close percentage to the 10% fines being produced. In all the tests, values of between 7.5 to 12.5% of fines passed 2.36mm sieve and the closest value to 10% was considered as the final result. Due to the limitation of the sources used, the tests could not be repeated.

Table 5.7 presents the summary of the results of the TFV test. Other researchers at the University of Birmingham only performed this test on the primary aggregate (basalt) and CC/CB. The only available data was the typical values given by Ballast Phoenix and the recommendation by Keller Ground Engineering.

Table 5.7: Ten percent fines value results for aggregates

Material	Actual TFV (kN)	Recommended value by Keller (kN)	Expected value by Ballast Phoenix (kN)
Big Granite	124	>60	-*
Small Granite	83		-
Crushed concrete and brick	49		-
IBAA (1)	41		50
IBAA (2)	38		50

*For these materials no previous results were published to be compared to the actual results obtained in this research

In this test if the load required to produce 10% fines, is higher, it means that the aggregates are less prone to fragmentation and therefore, might be more suitable for the use in VSCs.

According to the index tests results the RAs used in this research were not suitable to be used in the construction of the VSCs according to the recommendations presented in Table 6.7. However, the aim was to use the materials in the unit cell modelling in the condition of VSC installation and loading rather than relying on the aggregate index tests alone when material is being assessed.

Similar to the AIV and the ACV tests, a higher percentage of fines can be used as an indicator of higher probability of crushing under the vibro-float and column loading in the VSC context; although, it should be considered that the installation of the VSC is not well represented in the form of impact and prolonged loads applied in the index tests.

The results of the TFV tests showed the largest gap between the primary and the recycled aggregates among all the index tests. The big granite produced 10% of fines at more than twice the recommended load by Keller Ground Engineering. This test also showed the largest gap in the results between the granite and the CC/CB. The CC/CB showed a result of 11kN below the recommended value, which makes it unsuitable for the VSC construction.

The IBAA's were also not fit-for-purpose as the results showed that the 10% of fines were produced at loads 20kN below the recommendations. As opposed to the AIV and ACV, the IBAA (1) showed better performance in this test compared to the IBAA (2), which could mean that although under higher values of loads in the ACV, the composition of the IBAA (2) held the particles together and prevented them from

crushing, in lower values of loads, the ash matrix broke initially and produced more fines in the beginning of the loading. In case of the ACV the load increased to almost four times the values of the TFV test, the initially crushed matrix prevented further crushing.

5.5.5 Los Angeles test

The procedure for the LA test was explained in chapter 4 (refer to section 4.5.2), and the detailed calculations have been presented in Appendix 3. Due to the source limitation and the large quantities needed for each test, only one sample was tested from each material. The small granite was not tested as the size available did not fall in the aggregate size range suitable for this test.

Table 5.8 summarizes the results of the LA values and the recommendations and expectations by the standards and other research.

Table 5.8: Los Angeles test results

Material	Actual LA (%)	Recommended value by ICE (%)	Expected value by Ballast Phoenix (%)	Expected value by Ashton (2008) (%)
Big Granite	14	≤ 50	.*	13.1
Small Granite	-		-	-
Crushed concrete and brick	31		-	32
IBAA (1)	43		38-44	-
IBAA (2)	44		38-44	-

*For these materials no previous results were published to be compared to the actual results obtained in this research

The LA results indicate how the aggregates behave under sustained loads. A higher percentage in the results shows more tendencies of the aggregates to crush under loading which is not favorable for the use of aggregates in the context of VSC. The requirement explained in the ICE standard is less than 50 % fines being produced in this

test and all the materials fell under this category, which meant the both the primary and the recycled aggregates tested in this research were suitable for VSCs according to this recommendation (ICE, 1987).

Similar to the other aggregate index tests, the granite outperformed all the recycled aggregates by a great margin. Close to the granite, CC/CB performed better than the IBAA. The two IBAA materials showed very similar results and were the weakest among the material tested.

In the previous research by Ashton (2008), the primary aggregate (ballast) and the CC/CB were tested and the results of the current research were close to the previous results obtained. In case of the IBAA the expected values presented by Ballast Phoenix showed a range and the results obtained in this research fell within the range and very close to the higher end values.

Although these results were satisfactory and may indicate suitability of the aggregates in terms of strength, the conditions of the LA test, in which material was rotated and crushed using balls in a drum, is far from the condition the aggregates experience in the context of VSC installation and loading. Also, the duration of the LA test is much longer than the duration of the aggregate vibration during each stage of the VSC installation.

5.5.6 Small shear box test

The small shear box test was used to obtain the internal angle of shearing resistance of the aggregates used in this research. Due to the small box and the large aggregate sizes the results could not be confidently used for the interpretation of the behaviour of the material; however, the typical values for the granite and the crushed concrete have been

presented in various published work such as McKelvey *et al.*, (2002) which can be used to evaluate the results obtained in this research.

In case of the IBAs there was no published data on the shear box test and the results obtained here can only be used as an indication to compare these materials with against each other.

The material tested was subject to the PSD before each test and only sizes between 2 to 9.5mm were used in the shear box tests as this was the size used in the unit cell testing.

The procedure of the shear box test and the details of the calculations have been presented in chapter 4 (refer to section 4.5.2) and Appendix 4 (refer to CD), respectively. Summary of the results has been presented here:

5.5.6.1 Particle Size Distribution

The PSD of all four materials were tested before and after each shear box test. For each material three normal pressures of 60, 120 and 240kPa were applied. Each pressure was repeated once. The results presented in section 5.5.6 are the average values of the two results obtained for each test on each material. The amount of particle crushing due to the shearing forces can be an indicator of the strength and the behaviour of the material and can be linked to other aggregate index test results.

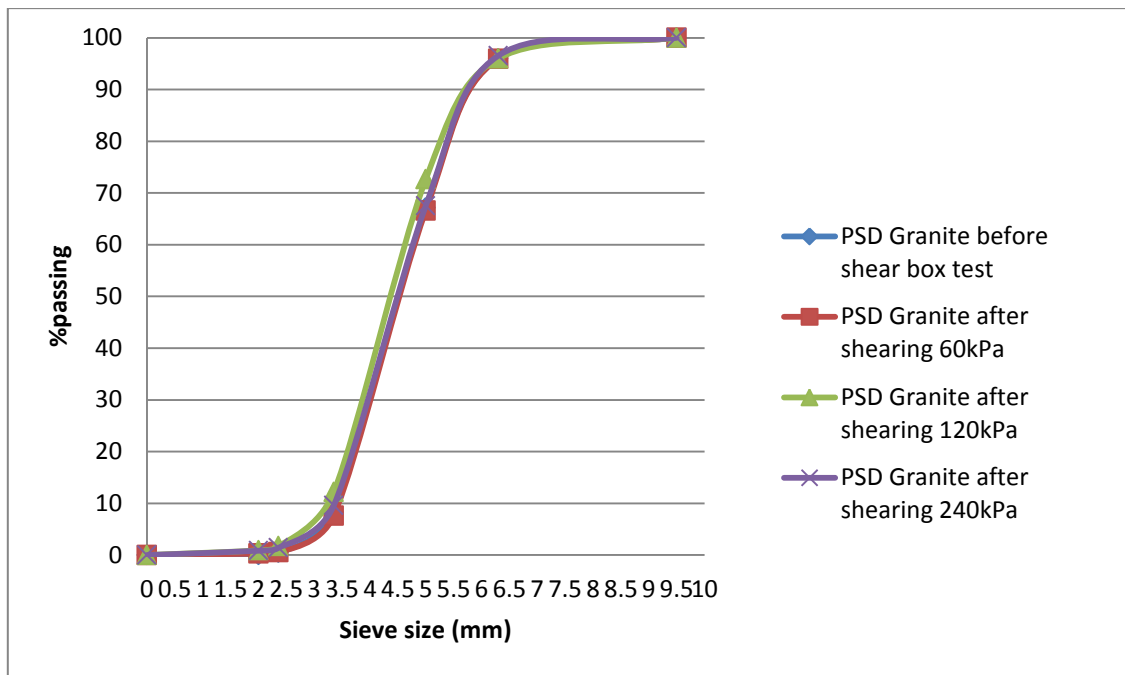


Figure 5.10: PSD before and after shearing-Granite

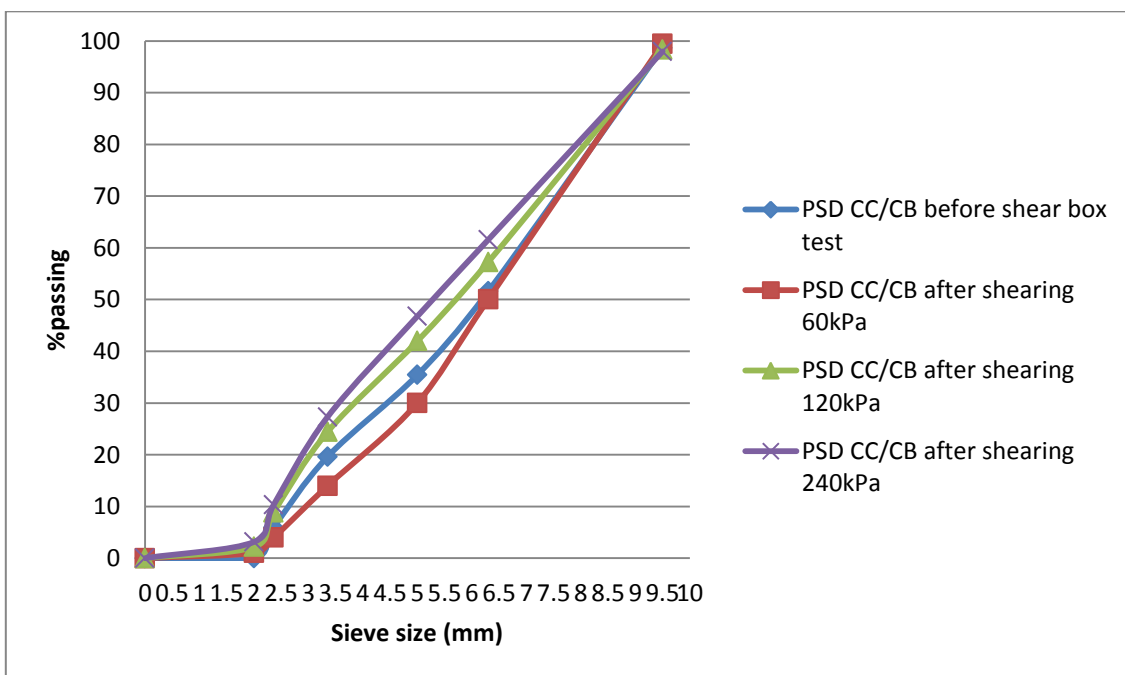


Figure 5.11: PSD before and after shearing-CC/CB

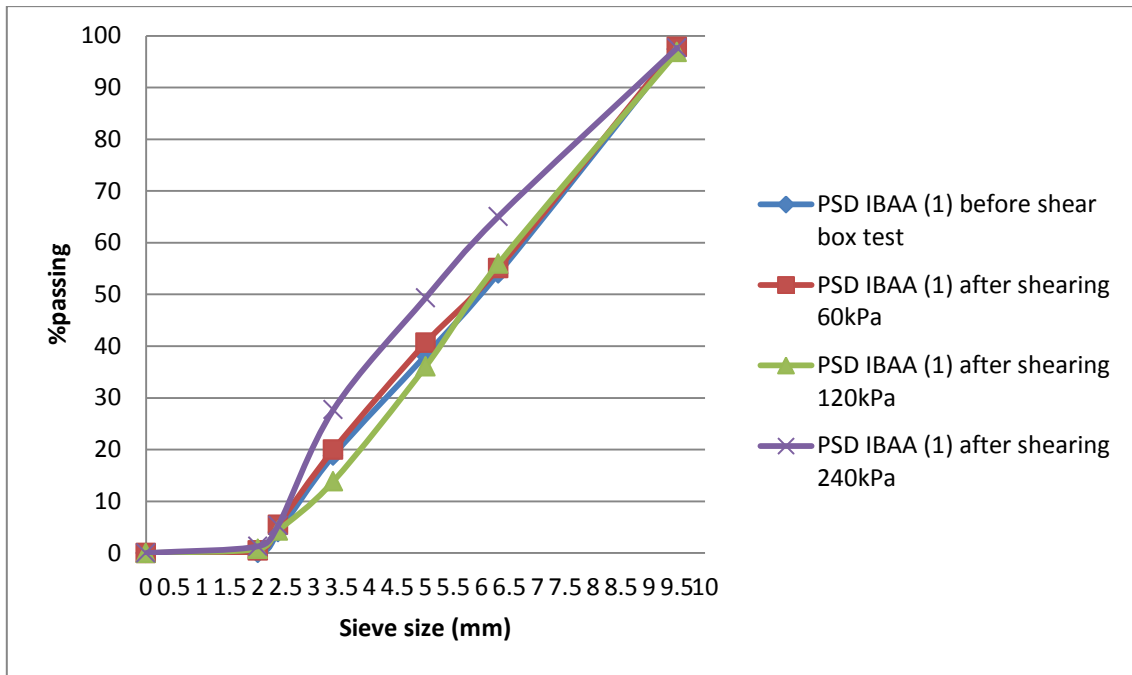


Figure 5.12: PSD before and after shearing-IBAA (1)

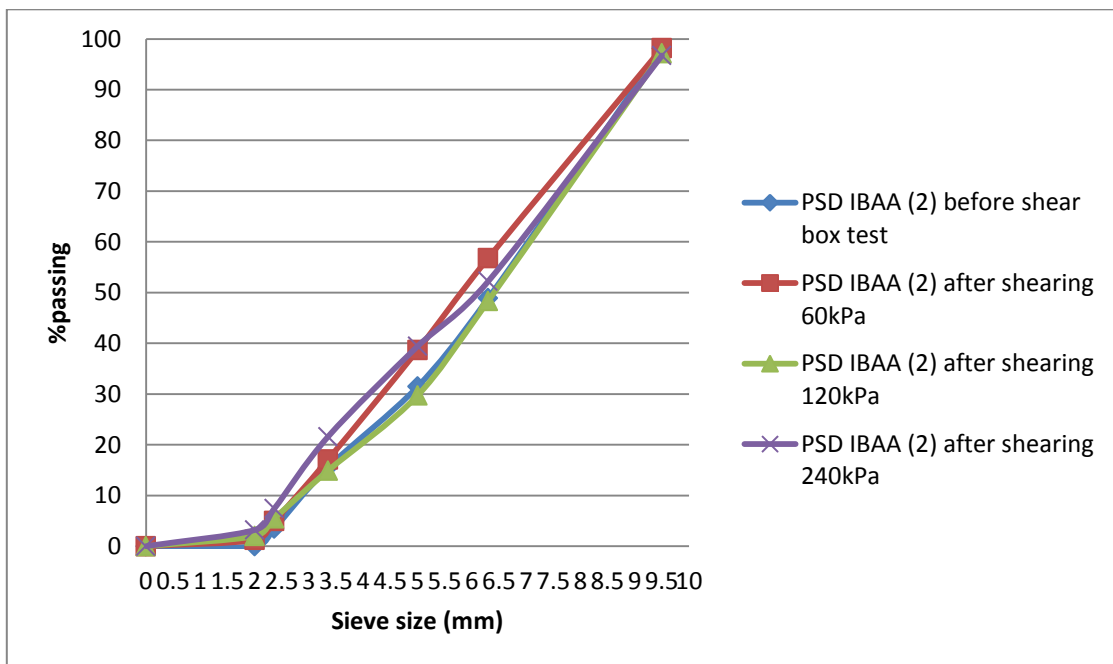


Figure 5.13: PSD before and after shearing-IBAA (2)

In granite, the difference before and after the shear tests in the particle size distribution of the material was minimal compared to all the RAs. A very small change was

observed after each process of loading and shearing. After the first test under a normal load of 60kPa, the PSD was almost the same as before the test and approximately 0.3% fines were produced. It was observed that as opposed to the expectation, the 120kPa pressure caused more fines to be produced compared to the 240kPa pressure. This could be due to the error in the collection of fines after the test from the container into the sieve. Some of the fines in the form of powder could be lost during the transfer of material from the shear box into the sieves. Approximately 1% fines were produced in the second test, and 0.9% under the 240kPa normal pressure. The values presented are the average values of the tests and the repeats and the error observed between the test and repeat was negligible.

For the CC/CB, a very logical trend was observed, where all the materials were crushed to a certain extent after the shearing. The amount of crushing was more than the granite and the predicted trend of more crushing in the 240 than the 120 and 60kPa was observed.

The crushing was observed in all the sizes and the highest level of crushing seemed to occur between sizes of 3.5 to 6.5mm. Although the CC/CB is a recycled aggregate and more crushing compared to the granite was expected, due to the initial PSD which covered a wider range of aggregate sizes compared to the granite, a well-graded trend was observed after shearing.

The IBAA (1) showed more breakage compared to the other material during the shear box tests. As expected a lot of fines were produced at the maximum normal pressure of 240kPa applied. Due to the nature of this material and the high glass content at the higher normal loads, the breakage started rapidly when the normal load was applied even before the shearing started.

The unexpected value was the lack of crushing due to the 120kPa normal pressure application, but this could also be contributed to the error in the collection of fines after the test for the sieving. This was more problematic in case of the IBAA (1), as the glass was crushed a lot and its collection was difficult.

The same error existed in the IBAA (2) under the 120kPa normal pressure. The fines produced seemed to have been lost as the values of the fines produced should be higher than the original material before the shearing.

The interesting change was observed between the two tests of 60 and 240kPa, where at a higher normal load, more of the small aggregate sizes were crushed compared to the larger sizes, and this could be contributed to the aggregates being held together by the ash matrix when a high normal load was applied.

In the lower normal pressure of 60kPa, a steady trend was observed where all the aggregate sizes were crushed with a similar trend. It seemed that similar to the AIV, ACV and TFV; the IBAA (2) was performing better than the IBAA (1) in terms of the crushing which made it more suitable for the purpose of VSC construction; however this should be evaluated using the unit cell loading of these two materials.

5.5.6.2 Shear strength versus horizontal displacement

The shear strength versus the horizontal displacement or the strain was measured for all the materials at all the three normal pressures. The details have been attached in Appendix 4 (refer to CD).

It was expected that the shear strength would increase initially and then after reaching the peak values, be leveled out. The initial build up was due to the particle resistance to the shearing until the peak value (Powrie, 2013).

Figure 5.14 presents the shear strength versus the horizontal displacement (strain) for all the four materials, and the values shown are the average of the initial and the repeat tests.

The trends were as expected for the PA and the RA sources. Similar to the other index tests, the granite outperformed the recycled aggregates. IBAA (1) shows more zigzag movement due to its breakable nature.

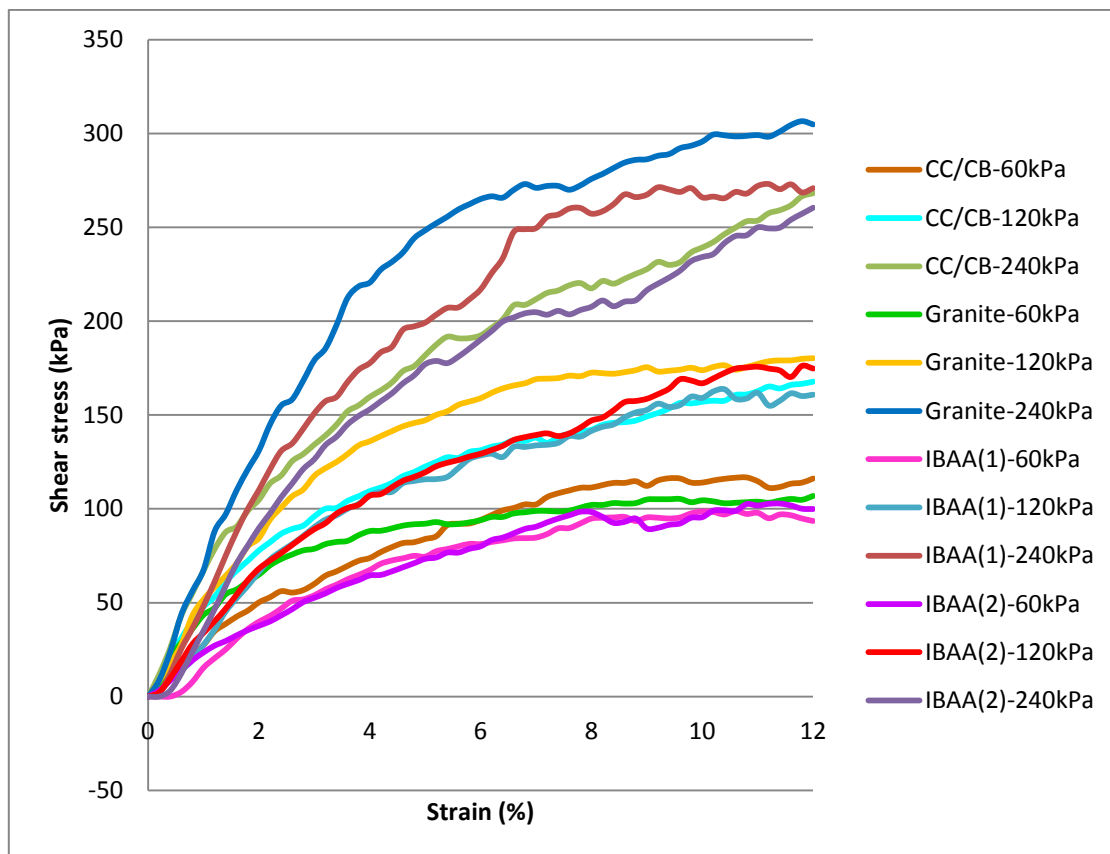


Figure 5.14: Shear strength versus strain

5.5.6.3 Vertical versus horizontal displacement

The vertical displacements indicate the volume changes during the shearing. A lot of change was observed initially due to the pressure being applied to the material. The change was due to the rearrangement of the aggregates under loading and shearing. Initially decrease in the vertical movement was observed (settlement) as the load was compressing the material; after no more compressing was possible, the vertical

movements increased in the form of swelling. As this information was not directly used in the context of VSCs, the data has only been presented in Appendix 4 (refer to CD).

5.5.6.4 Internal angle of shearing resistance

The tests were carried on for a maximum travel of 16mm which was the equivalent to 16% strain. The values of the internal angle of shearing resistance were obtained from the failure envelope, where the shear strength versus the three normal pressures of 60, 120 and 240kPa were drawn for each material. Figure 5.15 shows the failure envelope for all the materials at a typical strain of 10%. The peak values were also very close to the values at 10% strain.

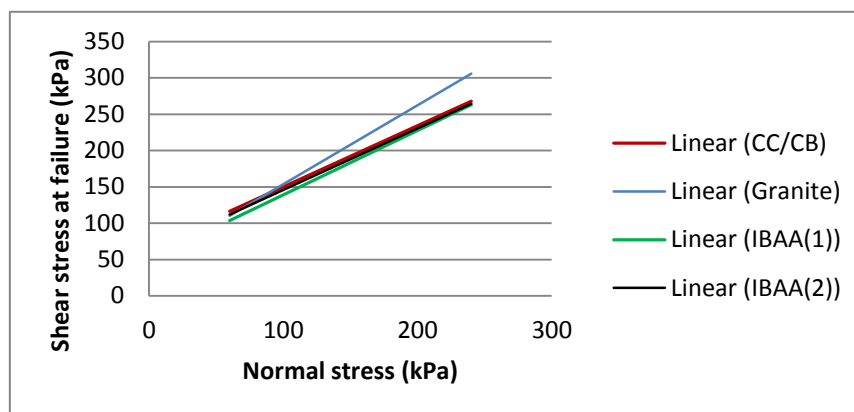


Figure 5.15: Failure envelope for the primary and the recycled aggregates

Table 5.9 shows the values of the internal angle of shearing resistance obtained for the four materials tested in this research.

Table 5.9: Internal angle of shearing resistance obtained from the small shear box test

Material	Internal friction angle (degrees)
Granite	47
CC/CB	40.2
IBAA (1)	41.5
IBAA (2)	40.2

According to the failure envelopes, the granite had the highest angle of shearing resistance, followed by the CC/CB and the IBAA. As expected from previous research (Ashton, 2008; McKelvey *et al.*, 2002) and the other index tests, the value of internal angle of shearing resistance of the granite was expected to be much higher than the recycled materials.

The CC/CB showed a slightly lower value compared to the IBAA (1), however the results were very close in this test as opposed to the other aggregate index tests. the IBAA (1) outperformed IBAA (2) by a small amount, and all the four materials seemed suitable for the use in construction of VSC as the internal angle of shearing resistance of 40 to 45° is recommended for the various methods of VSC installation (Serridge, 2006).

On the other hand it should be noted that this criteria is one of the most important factors in the design and performance of VSC and as this test was not performed on the proper size material, the results can be misleading in the judgment of suitability of these aggregates for the VSC construction. The results can only be used as an indication to compare the various materials with each other, and it was observed that although the difference in the behaviour of the primary and the recycled aggregates was significant in the other index tests, in the shear box results, the internal angle of shearing resistance was not very different especially for the three types of the recycled materials.

The difference can be significant in terms of the design and performance of VSC as even 10 degrees reduction in the internal angle of shearing resistance can reduce the bearing capacity and the settlement reduction factor by 50 and 30%, respectively (Priebe, 1995; Serridge, 2006).

In the study by McKelvey *et al.* (2002) in which the effects of 10 and 20% fines in the shearing behaviour of ballast and crushed concrete were compared, the results agree with the findings of this research where the recycled aggregates show a lower shear strength compared to the primary source (McKelvey *et al.*, 2002).

5.6 Evaluation of the aggregates tests results

Summary of the aggregate index tests results has been presented in Table 5.10.

Unacceptable results based on the recommendations (ICE, 1987; BRE, 2000) were highlighted in the orange coloured cells.

Table 5.10: Summary of the aggregate index tests

Material	AIV (<30%, BRE, 2000)	ACV (<30%, BRE, 2000)	TFV (>60kN, Keller)	LA (≤ 50%, ICE, 1987)	Internal friction angle (40- 45° , Serridge, 2005)
Big granite	4.1%	24.8%	124kN	14%	-
Small granite	12.7%	40.2%*	83kN	-	47°
CC/CB	17.3%	33.9%*	49kN*	31%	40.2°
IBAA (1)	27.8%	47.6%*	41kN*	43%	41.5°
IBAA (2)	22%	41.1%*	38kN*	44%	40.2°

*Orange cells represent the results which were unacceptable based on the recommended target values

The results are an indication of the hardness of the materials used in this research. In reality during the VSC installation, high vibrational forces are applied from the vibro-float to the aggregates; therefore hardness is an important factor to predict the material behaviour during the installation (BRE, 2000). Lower hardness means more crushing and reduction in the internal angle of shearing resistance that leads to poor bearing capacity and settlement reduction factors (Priebe, 1995). Also, crushing and the addition of fines results in the reduction in the angle of shearing resistance which ultimately reduces the drainage and the consolidation rate of the ground (Schmertmann, 1993).

It was observed that the granite was the best material in terms of the performance under the impact and continuous loads of the index tests, followed by the CC/CB and IBAs. The results agreed with the predictions of the general behaviour of the natural aggregate sources compared to the recycled ones. Among the recycled aggregates, the CC/CB was performing better than the IBAs.

Despite having a different appearance and structure, both the IBAs performed poorly in all the tests and their results were fairly similar in most cases. The results obtained can be used as an indication of the behaviour of the material. Direct comparison of the results was not possible with any other research due to the errors such as the limitation of sources available, the different machinery, the different aggregates sizes used in the tests and the fact that each source can be different due to its structure and composition. However, the general patterns observed and comparison of the material used in this research with each other was possible using this data.

The next stage was to analyze these materials in the context of installation and loading of VSCs which has been discussed in chapters 6, 7 and 8. Although materials such as IBAs were not acceptable in the tests such as ACV and TFV, they are used in practice and therefore their performance in VSC can be more illuminating of their behaviour rather than the index tests.

In chapter 2, section 2.7.2.3 the two main standards of ICE and BRE were compared in terms of the requirements for the use of aggregates in the VSC construction. The first important criteria were the PSD and the maximum percentage of fines allowed. In the ICE, the internal angle of shearing resistance was introduced as one of the most important factors. All the recommended tests by both the standards were carried out on

the material used in this research to comply with the unit cell results (ICE, 1987; BRE, 2000).

5.7 Summary of the results and discussions of the material tests

In this chapter the index test results of the Kaolin clay and the aggregates used in this research were presented followed by the discussions. An important part of the discussions was to understand the aspects of the results which can assist in interpretation of the behaviour of the aggregates in the context of VSC construction and loading. In terms of hardness, the PA was proved better than all the RAs tested in this research. However, these materials were all used in modelling of a single stone column in the unit cell tests and the index tests can be used parallel to the unit cell results presented in chapters 7 and 8.

The results obtained from this study suggest the following order of the aggregate index tests to be performed on the materials which are considered for the use in the construction of VSCs:

1. PSD range (well-graded versus uniformly graded material) and the maximum percentage of fines
2. Large shear box test (for obtaining the internal angle of shearing resistance)
3. AIV and ACV to consider the material hardness during the loading of the VSCs (Serridge, 2014)
4. The LA and the TFV tests to consider the effects of the installation on the addition of fines and the performance of the VSCs after the angle of shearing resistance is reduced (Serridge, 2014)

It should be noted that performing all of these tests before selecting the source of the material for the use in the construction of VSCs can be costly and time-consuming and

the most appropriate tests should be selected based on the unique specifications of the design and construction of each project.

For instance, the PSD can be avoided when the source of the material is within the acceptable range of 20 to 75 mm (Serridge, 2006); although the uniform or well-graded aggregate ranges can affect the performance of the VSCs and the effects should be considered in the selection of the installation method and the design of VSCs.

CHAPTER SIX

METHODOLOGY-PART 2: UNIT CELL TESTING

6 METHODOLOGY-PART 2-UNIT CELL TESTING

In this chapter the two unit cell tests used in this research were explained. The aim was to construct a single stone column using various primary and recycled aggregates in soft clay. The unit cell test set ups were explained starting by the assumptions used, the factors studied, the measurements and instrumentation.

15 tests were conducted in the large cell and 27 tests were performed in the small cell. The procedure and specific factors studied in each of the tests was explained. The various series of tests enable comparison of the behaviour of the columns of recycled and primary aggregates in the unit cell tests designed.

6.1 Unit cell testing

The unit cell concept was explained in chapter 2 (refer to section 2.6.1) and the unit cell idealization is a method in which defined geometry and boundary conditions are used to study the stone column in clay (McKelvey and Sivakumar, 2000).

Balaam (1978) first used the unit cell method on a group of columns to study the effect of loading on the column and its surrounding soil. Since then, the unit cell testing has been used in research to study the behaviour of a single or a group of columns under various conditions (Sivakumar *et al.*, 2004; Black *et al.*, 2007a).

In this research the unit cell idealization (refer to section 2.6.1) was adopted for the laboratory testing of a single column constructed with various aggregates in the soft clay. The column materials used were granite, crushed concrete and brick and IBAA (1) and (2).

The soft clay was Kaolin with a moisture content of 41% and the undrained strength of between 10 and 25kPa to represent the weak soil condition that requires improvement by construction of VSCs (Priebe, 2005). For detailed results of Kaolin properties and the criteria of the soft clay chosen for this research refer to sections 5.2 and 5.3.

In order to assess the performance of a VSC in a unit cell test, two types of containers were used. The large unit cell (LUC) and the small unit cell (SUC) containers. Both of the models were used to study the short-term behaviour of the stone column under static loading when various installation and material factors were implied.

The outcome was the comparison of the behaviour of the four types of aggregates used as the column material under controlled installation conditions. Load-deformation

behaviour, water level changes, installation and loading effect and the column shape were among the most important findings of the unit cell tests results.

6.2 Simplifying assumptions

6.2.1 Single column

A single column was modelled in order to study the effects of the material choice and the installation method on the VSC behaviour. In reality columns are constructed in a group and the neighbouring columns affect each other (McKelvey *et al.*, 2004); however, it was vital to study the effects of the recycled material on a single column before other factors due to the neighbouring columns made the analysis more complicated.

6.2.2 Short-term behaviour

Due to the time limits of this research, only the short-term performance was studied. This was divided into two time frames of during installation and during loading of the columns.

It is possible to study the performance in the long-term after the columns are loaded, however, many important changes such as the pore water pressure and the column bulging start from the time of installation and loading of the columns and these changes carry on after the loading with a relatively slower rate (Weber *et al.*, 2006), therefore the short-term observation of the VSC behaviour can be very useful in the analysis of its overall behaviour in the long term.

6.2.3 Static loading

Based on the unit cell concept it was assumed that the static load applied to the column was only carried by the column and the surrounding soil in an area which has a diameter

equivalent to 1.05 or 1.13 times the centre to centre spacing of the columns for the various column grids (refer to chapter 2, section 2.6.1) (Barksdale and Bachus, 1983). Therefore, the containers used were made as frictionless as possible by the application of grease to the internal sides and walls to avoid the load being transferred to the container instead of the unit cell area.

Use of nylon/plastic sheets would have been more accurate as the grease can affect the adjacent clay, however, due to the existence of the piezometers and taps (for water level measurements) on the sides of the large container, the application of grease was practical.

Also, the unit cells tests were quickly performed to study the short-term behaviour of the columns and the possible effects of the grease were minimal. More importantly, the unit cells were designed and developed in sizes where the sides of the containers were beyond the boundary conditions of the single stone columns (an area with a diameter of 2.5 times the column diameter is the estimated boundary condition (Hughes and Withers, 1974)) and would not affect the load carrying capacity results (refer to section 6.2.2).

6.2.4 Scaling effects

The scaling of the columns constructed had two main components of diameter and length. In the LUC, the columns had the diameter of 54 mm and the length of approximately 760 mm. This was adopted similar to the laboratory research concepts on the VSC by Black *et al.*, (2007a). In the laboratory modelling by Black *et al.*, (2007a), the aggregate sizes of 8 mm were selected which were approximately 6 times smaller than the diameter of the column.

In this research, a range of aggregate sizes were selected between 2 to 9.5 mm which provided a more realistic range similar to the real aggregates being used in practice which are not always single sized (refer to section 6.2.5).

The size of the LUC container was selected in a way that the diameter of the container which was 605mm was approximately 11.2 times the column diameter. In the studies by Hughes and Withers (1974), the unit cell diameter was 2.5 times the column diameter. Also, in other research by Black *et al.*, (2007a); Black *et al.*, (2007b); Black *et al.*, (2011); and Sivakumar *et al.*, (2004), columns were constructed in the clay and were loaded in a triaxial apparatus where smaller boundary conditions were used.

Single columns of 32mm diameter were constructed in a container with the diameter of 100mm. Therefore, the model used in the LUC tests had the advantage of more accurate boundary conditions compared to the previous research and eliminates the possibility of transfer of the load to the container instead of the column-soil composite. The column constructed in the LUC was an end-bearing column which sat on a hard porous stone at the base of the metal container.

During the development of the methodology, a few factors were tested in the smaller container before being used in the LUC. For instance, the standard installation method used on columns in the LUC was first tried in the SUC. Other examples include shape of the column after installation prior to loading and also after installation and loading which have all been explained in section 6.5.5.

The small container available for these factors to be tested had a diameter of 390 and length of 420 mm. The column diameter of 54 mm was too big for this container compared to the LUC tests; however, the tests were performed in the SUC to provide a better understanding of the specific factors (refer to Tables 6.2, 6.3 and 6.4) studied in

the LUC regardless of the size limitations and the boundary conditions. The columns constructed in the SUC were also end-bearing resting on the plastic base of the container.

6.2.5 Aggregate sizes

Four types of aggregates were used for the modelling of the single stone column, one primary (granite) and three recycled. All these aggregates were sieved to a range from 2 to 9.5mm in size. The small granite used in the unit cell tests was supplied with the range required. But the crushed concrete and brick were first crushed using a brick crusher and the IBAs were sieved to provide the range needed. The maximum size of the aggregates was almost $1/6^{\text{th}}$ of the column diameter. However, the aggregates used had higher percentage of finer particles and fewer particles above 6mm in size.

In most of the unit cell tests, the aggregates were scaled down and sand or gravel were used as a representative of the aggregates in terms of the scaled sizes (Hughes and Withers, 1974; McKelvey and Sivakumar, 2000), whereas the aim of this research was to study the load-deformation behaviour of the actual recycled aggregates in the context of VSC to compare with a natural aggregate source. The aggregate fragments could have been replaced by other material of the same size such as gravel. However, the crushability under the installation and the load carrying capacity were the focus for the specific recycled aggregates considered for this research.

6.2.6 Host ground

The columns were constructed in the soft Kaolin clay which provided repeatable and similar host ground conditions for these tests. The clay was mixed with 41% tap water and had the undrained shear strength of between 10 to 25 kPa to represent the weak soil condition in which the VSC might be used for the ground improvement purposes

(Priebe, 2005). Clay was placed in the container in 9 layers after being mixed with the tap water and each layer was compacted. The details of the mixing and compaction of the clay were fully explained in section 6.6. The preparation of the clay specifically for the LUC tests was explained in section 6.7.

6.2.7 Axial versus foundation loading

In order to apply the static load on the columns, two types of cylindrical plates were placed on the columns in different tests as model foundations. The smaller plate had a diameter and a height of 54 and 108 mm, respectively. This plate allowed the load to be applied axially over the column.

A bigger plate with an equal diameter and height of 108 mm was used in the LUC tests to apply the load on the column and an area around the column. In the LUC, both the plates were used. This enabled analysis and comparison of the behaviour of the column in condition of axially applied load (the small plate) versus foundation load (the large plate). In the SUC, due to the boundary conditions (refer to section 6.2.4) only the smaller plate was used to apply axial loads to the columns (refer to Figures 4.1 and 4.2).

6.3 The Large and small unit cell tests

Tables 6.1 to 6.4 show the details of the LUC and the SUC tests with the most important factors studied in each of them.

Refer to Figures 4.1 and 4.2 in chapter 4 which showed the cross-sections of the large and the small unit cell tests, respectively with all the equipment and components of the tests annotated.

6.3.1 Large unit cell tests

Table 6.1 summarizes the 15 LUC tests and the specific factors studied in each of the tests designed:

Table 6.1: Large unit cell tests

Test number	Test name	Host ground	Column material	Material range	Material condition	Load plate	Measurements
1	Pilot test	Saturated sand and partially saturated clay	Leighton Buzzard Sand	Up to 2 mm	Dry	Small plate	-load-deformation
2	No column-axial	Standard design: Clay (41% moisture content, $10 < C_u < 25 kPa$)	-	-	-	Small plate	-Load-deformation -water pressure during loading
3	Primary aggregate	Standard design	Granite	2-8 mm	Dry	Big Plate	-Load-deformation -water pressure during loading -column density
4	CC/CB	Standard design	CC/CB	2-8 mm	Dry	Big Plate	-Load-deformation -water pressure during loading -column density
5	IBAA(1)	Standard design	IBAA(1)	2-8 mm	Dry	Big Plate	-Load-deformation -water pressure during loading -column density
6	No column	Standard design	-	-	-	Big Plate	-Load-deformation -water pressure during loading
7	IBAA(2)	Standard design	IBAA(2)	2-8 mm	Dry	Big Plate	-Load-deformation -water pressure during loading -column density
8	Primary aggregate-repeat	Standard design	Granite	2-8 mm	Dry	Big Plate	-Load-deformation -water pressure during loading -column density

Continued on next page

9	CC/CB-repeat	Standard design	CC/CB	2-8 mm	Dry	Big Plate	-Load-deformation - water pressure during installation -water pressure during loading -column density
10	IBAA(1)-repeat	Standard design	IBAA(1)	2-8 mm	Dry	Big Plate	-Load-deformation -water pressure during loading -column density
11	Wet recycled aggregate	Standard design	CC/CB	2-8 mm	Wet	Big Plate	-Load-deformation -water pressure during installation -water pressure during loading
12	Wet recycled aggregate-repeat	Standard design	CC/CB	2-8 mm	Wet	Big Plate	-Load-deformation -water pressure during installation -water pressure during loading -column density
13	Wet Primary aggregate	Standard design	Granite	2-8 mm	Wet	Big Plate	-Load-deformation -water pressure during installation -water pressure during loading -column density
14	Wet primary aggregate-repeat	Standard design	Granite	2-8 mm	Wet	Big Plate	-Load-deformation -water pressure during installation -water pressure during loading -column density
15	Long-term primary aggregate	Standard design	Granite	2-8 mm	Dry	Big Plate	-Load-deformation -water pressure during installation -water pressure during loading -column density

6.3.2 Small unit cell tests

Tables 6.2 to 6.4 summarize the three series of the SUC tests and the specific factors studied in each of the tests designed:

Table 6.2: Small unit cell tests-Series 1

Test number	Test name	Column material	Installation type	Installation time	Installation only	Installation and loading	PSD before installation	PSD after installation	PSD after loading	Measurements
1	Aggregate crushing and column shape due to loading	Granite	Compaction by standard compaction hammer	- 10 blows per aggregate layer		x	-		-	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test -column shape after loading
2	Aggregate crushing and column shape due to loading	Granite	Vibrations by concrete poker	20 seconds/layer		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
3	Aggregate crushing and column shape due to installation	Granite	Vibrations by concrete poker	20 seconds/layer	x		x	x		-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
4 (repeat)	Aggregate crushing and column shape due to installation	Granite	Vibrations by concrete poker	20 seconds/layer	x		x	x		-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
5 (repeat)	Aggregate crushing and column shape due to loading	Granite	Vibrations by concrete poker	20 seconds/layer		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
6	Aggregate crushing and column shape due to installation	CC/CB	Vibrations by concrete poker	20 seconds/layer	x		x	x		-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
7	Aggregate crushing and column shape due to loading	CC/CB	Vibrations by concrete poker	20 seconds/layer		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test

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8 (repeat)	Aggregate crushing and column shape due to installation	CC/CB	Vibrations by concrete poker	20 seconds/lay er	x		x	x		-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
9 (repeat)	Aggregate crushing and column shape due to loading	CC/CB	Vibrations by concrete poker	20 seconds/lay er		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
10 (repeat)	Aggregate crushing and column shape due to loading	CC/CB	Vibrations by concrete poker	20 seconds/lay er		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
11	Aggregate crushing and column shape due to installation	IBAA(1)	Vibrations by concrete poker	20 seconds/lay er	x		x	x		-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
12	Aggregate crushing and column shape due to loading	IBAA(1)	Vibrations by concrete poker	20 seconds/lay er		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
13 (repeat)	Aggregate crushing and column shape due to installation	IBAA(1)	Vibrations by concrete poker	20 seconds/lay er	x		x	x		-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
14 (repeat)	Aggregate crushing and column shape due to loading	IBAA(1)	Vibrations by concrete poker	20 seconds/lay er		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
15	No column	-	-	-		x	-		-	-load-deformation behaviour -moisture content of core -moisture content and VST after the test
16 (repeat)	No column-repeat	-	-	-		x	-		-	-load-deformation behaviour -moisture content of core -moisture content and VST after the test

Table 6.3: Small unit cell tests-Series 2

Test number	Test name	Column material	Installation type	Installation time	Installation only	Installation and loading	PSD before installation	PSD after installation	PSD after loading	Measurements
17	Effect of installation time on crushing and column shape	Granite	Vibrations by concrete poker	20 seconds/layer		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
18	Effect of installation time on crushing and column shape	Granite	Vibrations by concrete poker	30 seconds/layer		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
19	Effect of installation time on crushing and column shape	Granite	Vibrations by concrete poker	10 seconds/layer		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
20	Effect of installation time on crushing and column shape	Granite	Vibrations by concrete poker	90 seconds/layer		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
21 (repeat)	Effect of installation time on crushing and column shape	Granite	Vibrations by concrete poker	90 seconds/layer		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
22 (repeat)	Effect of installation time on crushing and column shape	Granite	Vibrations by concrete poker	10 seconds/layer		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
23 (repeat)	Effect of installation time on crushing and column shape	Granite	Vibrations by concrete poker	30 seconds/layer		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test

Table 6.4: Small unit cell tests-Series 3

Test number	Test name	Column material	Installation type	Installation time	Installation only	Installation and loading	PSD before installation	PSD after installation	PSD after loading	Measurements
24	Effect of fines in column aggregates on load carrying capacity	Granite (10% fines)	Vibrations by concrete poker	20 seconds/layer		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
25	Effect of fines in column aggregates on load carrying capacity	Granite (20% fines)	Vibrations by concrete poker	20 seconds/layer		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
26 (repeat)	Effect of fines in column aggregates on load carrying capacity	Granite (10% fines)	Vibrations by concrete poker	20 seconds/layer		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test
27 (repeat)	Effect of fines in column aggregates on load carrying capacity	Granite (20% fine s)	Vibrations by concrete poker	20 seconds/layer		x	x		x	-Column density -load-deformation behaviour -moisture content of core -moisture content and VST after the test

6.4 Factors studied in the large and the small unit cell tests

6.4.1 Material factors

Column of the primary aggregate (granite) was compared to the columns of the RAs. Also, the condition of wet or dry was compared in the LUC tests for the granite and the CC/CB. The results of the load carrying capacity of the wet aggregate columns were compared to the dry columns. Also, the results of the performance of the wet granite and the wet CC/CB were compared with each other.

In the SUC, four tests were run on the aggregates mixed with powdered granite to represent a material contaminated by fines. These four tests were performed on the granite only, as enough material was not available for the RAs.

The results of the effect of contamination with fines on the performance of VSC when a PA is used can be very useful in predicting the column behaviour when the material is contaminated during the storage, transportation or installation of the columns.

6.4.2 Installation factors

The energy of the model vibro-float was varied by means of increasing the time of compaction on each stage of the aggregate compaction during the construction. Tests were performed on the granite in the SUC where times of 10, 20, 30 and 90 seconds were used separately on each test to study the effect of the energy of compaction on the material.

The installation apparatus was a concrete poker which has been explained in the instrumentation section (refer to section 6.6). The PSDs before and after the loading were used as an indicator to study the crushability of the aggregates under the installation and loading impacts.

Also, the shape of the column due to installation alone versus installation and loading was investigated in the SUC. In all the tests the top-feed installation method was modelled where the installation condition was dry (dry top-feed installation).

6.4.3 Loading

The columns constructed were rapidly loaded in the laboratory as in reality the process of the column installation and loading is a fast process where at least approximately 300 meters of columns can be constructed per day depending on the columns length, soil strata and the method of installation used (Raju and Sondermann, 2005).

The exception in this research was the final LUC test, in which the column was constructed and left for the duration of 3 months to represent the estimated time required for the consolidation of the host ground in the LUC container (as opposed to other tests where the clay was only compacted in layers); however, the consolidation did not take place in any of the tests and this duration only represented the estimated time required for the consolidation process in case it was done.

In this test the host ground was compacted and this process was shortly followed by the construction of the column and then the column and the host ground were left for three months before the loading commenced. The column was made of the dry granite and the results were compared with the other columns of granite which were rapidly loaded.

6.5 Measurements for the unit cell tests

6.5.1 Moisture content and the undrained strength of the soft clay

Moisture content was one of the key parameters measured for the host ground in order to make sure the Kaolin used provided the condition (Moisture content and undrained strength) required for the construction of VSCs.

As shown in the clay results (refer to section 5.2.5), an increase in the moisture content reduces the undrained strength of the Kaolin. Therefore, the clay was mixed with 41% of tap water to provide the required range of the undrained strength.

After mixing, two samples of moisture content were taken to make sure the 41% moisture content was achieved. This controlling measure was performed on the clay used in the LUC. For the SUC, the clay was reused from the LUC tests. The process has been explained in section 6.8. Therefore, for the SUC no moisture content samples were taken before the test.

The quality control tests after each of the unit cell tests included the vane shear measurement of the actual range of the undrained strength of the clay and the moisture content (refer to sections 7.2, 8.2.1, 8.3.1 and 8.4.1). These measurements were taken in each of the 9 layers of the Kaolin which were placed and compacted and the readings were at 4 points across each layer.

As the undrained shear strength measurement was destructive of the host ground, it was only performed after each test. The vane shear apparatus was used. The points where the measurements were taken were located at a radius of 135 mm from the centre of the column which was 2.5 times the column diameter and was the boundary condition of the unit cell (refer to section 2.6.2) (Hughes and Withers, 1974).

The measurements after the unit cell tests were taken at least one week after the test to represent the long-term assessment of the host ground condition (Raju and Sondermann, 2005).

The moisture content of the host ground was also controlled during the installation of the columns. The installation process has been fully explained in the preparation of the columns in section 6.6.5. To summarize the installation process, at the centre of the unit cell a hole was first formed using a tube and an auger. When the core was extruded to be replaced by the aggregates to form the column, the Kaolin material of the core was used to provide three moisture content samples at three depths of the top, the middle and the bottom of the column.

6.5.2 Particle size distribution and the density of column

PSD is a key controlling measure for the aggregates used in the unit cell tests. In the LUC, the aggregates were graded before the installation, to make sure the required range of 2 to 9.5 mm was used in the modelling (refer to section 6.2.5). After the LUC tests the aggregates were not subject to the PSD as the test aim did not include an estimation of the crushing of the aggregates during loading in the LUC tests.

On the other hand, the crushing of the aggregates during installation and loading was the aim of the first series of tests in the SUC. In these tests, the aggregate was graded before the installation. After the installation aggregates were vacuumed out and were subject to the PSD again.

The density of the columns constructed was estimated in both the LUC and the SUC tests. The column diameter and length were known for the both cells and the volume of the columns was estimated.

For each test, the amount of aggregates used for the installation was recorded.

According to the volumes estimated and the amount of aggregates used, the density of each column was estimated. The densities were compared in the results for the various columns constructed (refer to Table 7.3). The results of the densities can be related to

the aggregate range used for each column and the level of packing achieved during the installation. The comparisons have been fully explained in chapter 7 (refer to section 7.3.2).

6.5.3 Load-deformation

Both the unit cell tests were subject to loading once columns were constructed. This was achieved via loading frames and the axial and foundation plates. The aim was to apply the load and observe the bulging and the failure of the column and also, measure deformations. In the LUC, columns were loaded and the load carrying capacity of the columns of various aggregates was compared.

In the SUC, the small loading plate was used to apply the axial load in order to assess other factors such as crushing of material under the loading, and the load carrying capacity of the columns contaminated by fines. Also, various times of installation were used in the SUC which created different column densities and load carrying capacities.

6.5.4 Water level measurements

The excess pore water pressure changes during the installation, during the loading and in the long-term were among the important field measurements in recent researches on the performance of VSCs (Castro and Sagaseta, 2012). The changes in the excess pore water pressure can indicate the behaviour of the surrounding soil and also, how the column acts as a drainage path for the host ground.

In this research the Kaolin used was only compacted and not consolidated, therefore it was not fully saturated and the measurement of the excess pore water pressure was not possible. The consolidation of the Kaolin in the unit cell for numerous numbers of tests and in the scale designed could take a long time and was not feasible for the purpose of

this research. Therefore, the degree of saturation for the clay compacted was estimated to be 78% based on Equation 6.1(Barnes, 2010):

$$S_r = \frac{\rho_b w G_s}{\rho_w G_s (1 + w) - \rho_b} \times 100 \quad \text{Equation 6.1}$$

Where ρ_b is bulk density (for partially saturated soils);

W is the water content

G_s , is the specific gravity

and ρ_w is the water density

In the partially saturated soft clay used, 6 model piezometers were used at various depths and radii from the centre of the column to measure the changes in the water level during the installation and loading in the LUC tests.

Three of the model piezometers were located at a distance equivalent to the column diameter (54 mm) from the centre of the stone column. Three others were located at a distance twice the diameter of the column (108 mm) from the centre of the stone column. These distances represented radial water level changes in the model.

These 6 piezometers were located at depths of 160, 320 and 640 mm from the top of the stone column constructed. This enabled the study of the effect of bulging and the stress transfer through the column to be studied via the water levels. The two piezometers at the distances of 54 and 108 mm from the centre of the column were located at the same level. The piezometers were located in the host ground and due to the pressure changes in the system when the load was applied to the column; water was transferred through the piezometers to the measurement tubes shown in Figure 6.1.

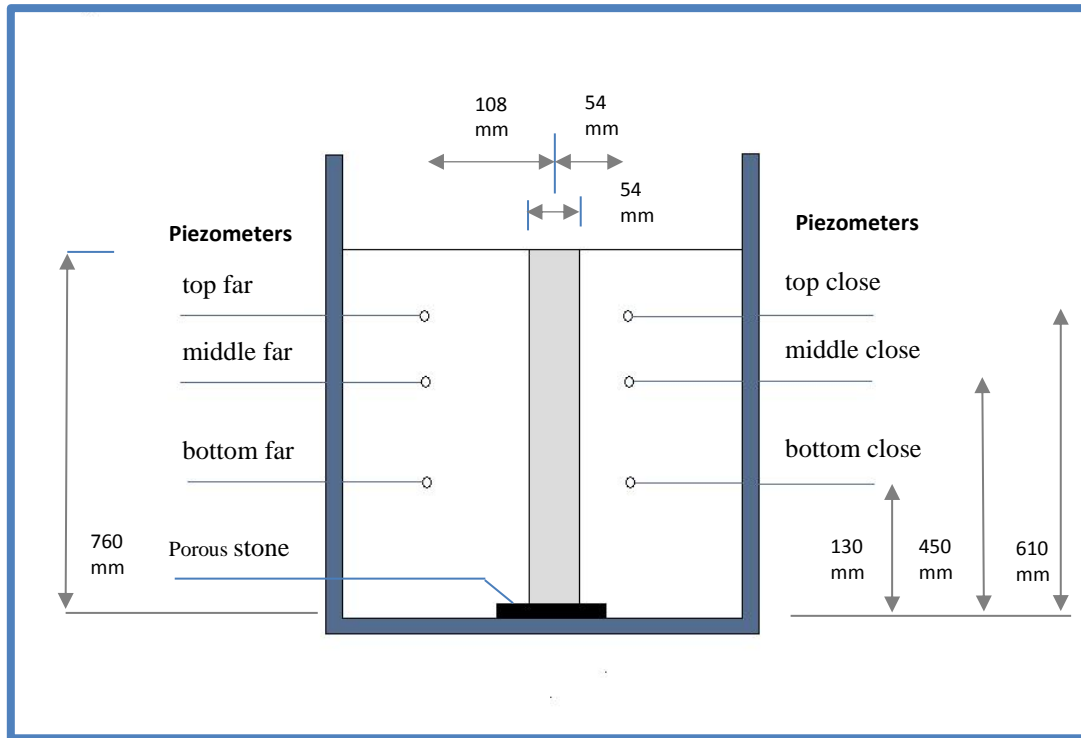


Figure 6.1: The porous stone and the piezometers and their locations

For the purpose of this research Table 6.5 was used to refer to the porous stone and the piezometers with specific numbers. The same numbers were used in the results (refer to sections 7.7.3 to 7.7.6).

Table 6.5: The porous stone and piezometers and the numbers used for the results interpretation

The instrument name for the water level measurement	Number
The porous stone	1
The bottom close piezometer	2
The bottom far piezometer	3
The middle close piezometer	4
The middle far piezometer	5
The top close piezometer	6
The top far piezometer	7

At the base of the system, the end-bearing column constructed sat on a porous stone.

The porous stone enabled the measurement of water transferred from the system directly through the column. The amount of water measured in the porous stone was expected to be much higher than the model piezometers as water can travel faster and easier through

the granular materials of the column. Also, the piezometers were located in the partially saturated clay as opposed to the porous stone which was located at the base of the container and only the granular column (with higher permeability than the clay) sat on it.

In the LUC tests the measurements via the porous stone and the model piezometers started after the host ground was prepared, as the piezometers were placed in the clay during the preparation stage. After the layers of clay were completely prepared, the measurements of water changes were recorded, however insignificant, for 48 hours at every 12 hours. The measurements were carried on during the column installation for a number of LUC test after each stage of the aggregate pouring and vibration was finished until the column was completely installed.

The water level measurements were initially designed for during loading of the columns, where most pressure changes were expected. During the loading all the 7 water level values were recorded at every 0.50 mm of penetration of the foundation into the column.

After the column was unloaded, the water level changes were recorded for the duration of 48 hours, at 2, 4, 16, 24, 40 and 48 hours after the test was finished. In the last LUC test (test 15), the column was constructed but not loaded for 3 months and the water levels were measured once every day. The water level measurement was not the objective of the SUC tests and was only recorded in the LUC tests.



Figure 6.2: Water level measurement tubes and board

6.5.5 Column shape

In the SUC tests, the column shape was investigated after each test. The first series of the SUC tests studied the effect of installation versus installation and loading combined; on the primary and the recycled aggregates.

In these tests the granite, the crushed concrete and brick, and the IBAA (1) were used. The IBAA (2) could not be used as not enough material was available for these tests. The three materials were once used in the installation of column under 20-second compaction per layer. After the installation, the aggregates were vacuumed out and subjected to PSD. The grading was compared before and after the installation to investigate the level of crushing achieved.

The same test under the exact same conditions was repeated with the three aggregates where after the installation of the column, it was loaded. After loading, the aggregates were vacuumed out and subject to PSD. The level of crushing contributed to the loading process was estimated for the various materials through these tests.

After the vacuuming of the loose aggregates that did not penetrate into the surrounding clay, what was left in the cell were the host ground and the outer part of the column which was emptied of inside material using the vacuum. Only the aggregates that penetrated into the clay during the installation and loading could not be vacuumed out. These aggregates showed the shape of the column after either the installation or after the loading.

Cement grout was used with a water cement ratio of 50% to be poured in the column which was empty inside. After 24 hours when the grout was set, the surrounding clay was cleaned out and the side aggregates attached to the cement grout in the middle remained in the cell. The column shape was studied.

For the installation only tests (refer to Table 6.2), the steps of installation were observed. In case of loading, the bulging and the column deformations were studied. The difference in the shape (after installation only and after the loading) was compared for the various aggregates.

Before the cement grout was used, epoxy resin was tried in a few tests to glue the column aggregates together and enable the study of the shape of the columns. In these trial tests, the aggregate was not vacuumed out and instead the epoxy was poured into the entire column under a fume cabinet.

The glue was left to set and then the surrounding clay was removed and the shapes of trial columns were observed. However, this method could not be used as the epoxy resin was expensive regarding the size and volume of columns constructed and also, due to health and safety reasons the epoxy had to be poured over the columns under the fume cabinet and the LUC and the SUC containers could not be transferred under the fume

cabinet. Due to these reasons this method was not feasible and was abandoned for this research.

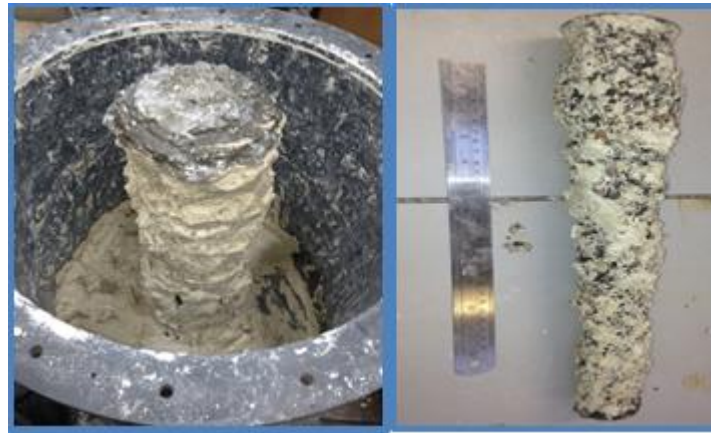


Figure 6.3: Column shape after the grout was set and surrounding soil was cleaned out

6.6 Instrumentation for the unit cell tests

6.6.1 Porous stone

A porous stone was used in the LUC at the base of the tank to measure the water transferred through the column during the installation, loading and in the long duration. The stone had a diameter of 100 and thickness of 10mm. There was a tube attached to the stone to transfer the water out to the side of the LUC.

At the side of the LUC, the tube was attached to a tap on the outer face, which was connected to a pipette fixed to a wooden board in a way that the pipette's tip is at the same height of the tap attached to the porous stone. Therefore, the water coming out of the stone was directly measured without significant height difference. The pipette used had a capacity of 25mL. It was expected before the tests that the porous stone would collect more water than the other model piezometers due to the granular nature of the column (refer to Figure 6.2).

6.6.2 Model piezometers

As explained previously, 6 model piezometers were constructed in the laboratory to be used in the LUC. The concept was to use a porous material to collect the water and transfer it through the tubes to the outer side of the tank to the reading board. The challenge was to use a filter material to stop the clay from penetrating into the tubes and allow the water to travel easily. Any filtering material could not be 100% efficient and some clay particles were inevitably transferred through the tubes. However, the measurements showed successful readings of the water levels in the tests.



Figure 6.4: Model piezometers used in the large unit cell tests

The tip of the model piezometer was punched at several points to allow the water to be drained. The filter paper covered the punched tube. All parts were sealed using the hot glue and left to dry. After the piezometers were prepared and completely dried, their performance was tested under running water. It was observed that water was easily transferred to the tube through the tip.

As these piezometers were reused for all the LUC tests, before use for each test, they were properly washed and left to soak in tap water overnight. The piezometers were attached to the pipettes on the reading board to record the water levels during the installation and the loading of the stone columns.

6.6.3 Mixer

An electric mixer was used for the preparation of the LUC samples, in which the clay and tap water were mixed. In each mix one bag of the Kaolin weighing approximately

25 kg was mixed with 10.250 Lit of tap water to achieve 41% moisture content. The time of mixing was 10 minutes for each bag of clay where the water was added gradually to ensure a uniform mix was achieved. For the SUC tests, the clay was reused from the LUC.

6.6.4 Vibrating hammer

In the process of host ground preparation, a vibrating hammer (Kango hammer) was used to compact the clay in layers in both the unit cells. For each unit cell a wooden plate was placed over the clay during compaction. The advantage of using the plate was that the hammer would not stick to the clay. On the other hand, some of the energy of the hammer was being transferred to the plate. The energy transferred from the hammer to the clay could not be easily calculated using the properties of the hammer provided by the manufacturer, however, trial tests were used to make sure the properties of the host ground (i.e., the moisture content and the undrained strength) were consistent in the layers.

6.6.5 Concrete poker

A concrete poker was adapted to model the installation of the VSC in the laboratory unit cell tests. The poker comprised of an electric motor, connection cables and a vibrating rod. The rod had a diameter of 25 and length of 300mm.

The poker was used to model the top-feed method of installation under dry condition. In the SUC tests, the second series of the tests was performed to compact each layer of aggregates in installation with a specific time of vibration per layer (refer to Table 6.3). These included 10, 20, 30 and 90 seconds. It was observed that the 20 second compaction and vibration of the aggregates for each aggregate charge during the installation produced uniform installation for all the LUC and SUC tests. This was the

standard time used in all the LUC tests to compact each charge of the aggregates during installation.

The concrete poker was properly washed and dried before each test. The concrete poker was not forcefully pushed into the hole to compact the aggregates, as the aim was only light compaction and better packing of the aggregates. In the case of forceful compaction, the aggregates were pushed into the clay and more material than estimated were required for the column formation; which ultimately led to variable column diameter and densities.



Figure 6.5: Concrete poker used for the compaction of the aggregates during the installation of VSCs

6.6.6 Loading frames

Two loading frames were used for the unit cell testing in this research. The LUC tests were loaded in an assembled loading frame and a reverse triaxial gearbox. The gearbox provided the rate of 1.2 mm/min for the loading. The maximum travel available was 110 mm. The gearbox was connected to the calibrated proving ring. The maximum travel considered for the LUC tests was 80mm which assured failures in the column before the test was stopped.

According to the maximum travel and the loading rate, the entire loading took approximately 67 minutes. During this time at every 0.50 mm of deformation the load applied was recorded from the proving ring. At the same time the 7 values of the water levels from the porous stone and the model piezometers were recorded.



Figure 6.6: Set up for the large unit cell tests

In the case of the small unit cell, a manufactured loading frame was used. The gearbox had a loading rate of 3.2×10^{-3} mm/min. At this rate the small cell was gradually pushed upwards for the maximum travel of 30 mm. There was maximum travel of 300 mm available on this apparatus; however, 30 mm was beyond the failure of the columns in the SUC. At every 0.50 mm of deformation, the load was recorded from the proving ring. In the case of the SUC tests this was the only measurement taken during the loading of the columns.



Figure 6.7: Set up for the small unit cell tests

6.7 Preparations for the large unit cell tests

6.7.1 The host ground

For both the large and the small unit cells, the cell was properly cleaned and dried. Grease was applied to the sides of the containers. For the LUC, the porous stone was properly washed and placed in the centre of the container and was saturated before each test.

Then a thin layer of saturated Leighton Buzzard sand was placed at the base to be leveled with the porous stone. The sand was soaked in the tap water. It was then gently tapped into a level position via a tamping rod. A filter paper was placed over the porous stone after the saturation to prevent the clay in the upper layer penetrating into the stone. No Leighton Buzzard sand was used for the base of the SUC tests.

In the LUC, the Kaolin was mixed and two samples of moisture content were taken from two different parts of the clay to control the consistency of the mix. The results of the moisture content tests before each of the LUC tests were presented in Appendix 5 (refer to CD).

Due to the large surface area and the thick layers of the clay in both the cells (each layer was 80 mm in thickness); the vibrating hammer needed to be used instead of the standard compaction hammer. Calculation was tried initially to find the energy of compaction transferred from the hammer to the layers of the clay. The energy calculated was compared to the standard compaction mould results, however, due to several properties of the vibrating hammer such as variable frequency; the energy calculation was not straight forward. This was further complicated by the fact that the energy transferred to the layers by the vibrating hammer could not be easily scaled and compared to either of the unit cells (refer to Appendix 2).

Instead of calculations, trials were used in the SUC container based on the calculations and estimations. Finally 10 and 4 minutes of compaction per layer in the LUC and the SUC were performed, respectively. These times of compaction provided the host ground with the required undrained strength for the column installation. After the compaction of each layer, the surface was leveled using a pallet knife. This process was repeated nine times to complete 9 layers of the host ground for the LUC and 5 layers in the SUC.

In the large container, before the clay in layers 2, 6 and 8 from the base were compacted (refer to Figure 4.1); the model piezometers were placed at the two opposite sides of the layer. In order to cover the tip of the piezometers, Leighton Buzzard sand was used to cover the piezometer. The piezometers were then saturated from the tubes on the outer side of the container.

When the clay was compacted in 9 layers and all the 6 model piezometers were placed; the water level taps were opened and the water level was recorded even before the installation and loading. The details of these measurements were attached in Appendix 5 (refer to CD).

The soil properties (the moisture content and the undrained strength) for the LUC and the SUC are presented in chapters 7 (refer to section 7.2, Table 7.1) and 8 (refer to sections 8.2.1, 8.3.1 and 8.4.1).

6.7.2 Column installation

Before the installation started, Leighton Buzzard sand was soaked in tap water and poured over the clay in a layer with a thickness of 40mm. This created a platform for the installation and as the sand was saturated, it helped keeping the moisture content of the Kaolin below.

The aggregates were washed and dried in the oven for a maximum duration of 4 hours at $110 \pm 5^{\circ}C$. This process did not represent the actual procedure in the field for the materials used in the construction of VSCs; however, as the columns constructed in the LUC tests and most of the SUC tests needed to be compared against each other, materials were initially washed from the dust and fines to be able to compare the material behaviour of different columns against each other.

The aggregates were then placed to cool before sieving for the PSD. The mass of the aggregate required for each test was estimated according to the expected density of the column. Approximately 3500 g of aggregates were prepared for each test.

Before the aggregates were poured into the column, the hole was formed. A steel tube with an outer diameter of 54mm (the same as the column diameter) was pushed into the centre of the unit cell.

The clay that was mixed with water and compacted made the downward movement of the tube very difficult. In order to push the tube vertically and exactly at the centre of the cell, a cross was used with a hole inside to adjust the tube in (Figure 6.8). Once the steel tube reached the base of the cell; an auger was pushed into the tube and the clay inside was taken out at various stages. Three moisture content samples were taken from the top, the middle and the base of the core extruded. The tube was then pulled out gradually. There were small amounts of deformations observed (using a torch) towards the centre of the column area near the base of the cavity formed.



Figure 6.8: shows the cross and the auger used for the column formation

Figure 6.8: Cross and auger used to form the column in the centre of the unit cells

After the hole was formed and tube was taken out, the aggregates were poured from the top in layers with approximate depth of 30 to 50mm. Each layer was then vibrated and compacted for 20 seconds, until the column reached the surface of the top sand layer. When the column was formed, the mass of the remaining aggregates was recorded and according to the volume estimation of the hole, the density of the column constructed was calculated. In a few tests (9, 11, 12, 13, 14 and 15; refer to Table 6.1) the water level changes were recorded during the installation. In these tests after each layer of the aggregate was vibrated the changes of the water level were recorded. At the same time during vibrations the fluctuations of the water levels were monitored. These measurements indicated where more changes in the system were occurring at each level of column installation. The results were presented in Appendix 5 (refer to CD).

Smearing of the surrounding clay with the aggregates starts during the installation (Weber, 2004); this effect and the shape of the column were studied in the SUC tests (refer to Tables 6.2 to 6.4 and chapter 8 for the results).

6.7.3 Loading and unloading

After the installation process was complete, the column constructed was rapidly loaded. The cell prepared was placed under the loading frame; the proving ring and the loading plate were located over the column. After the maximum travel required was achieved, the column was unloaded, however, in case of the LUC tests; the water levels were recorded for 24 hours after the tests.

After one week, the clay was cleaned out and the quality control measures were performed. The cleaning started by using a vacuum cleaner to take the aggregates out as much as possible; the aggregates on the side of the column formed penetrated into the surrounding clay. The cleaning was carried out in stages where at each layer four moisture content samples and the hand vane data were collected. These measurements assisted in controlling the consistency of the layers; also, as the clay was reused for the SUC tests, the properties were important for the quality control.

6.8 Preparations for the small unit cell tests

6.8.1 The host ground

Similar to the LUC tests, the container was cleaned and dried. Grease was applied at the sides of the cell before the clay was placed. As opposed to the LUC where the clay was mixed and prepared fresh for each test, in the SUC tests, the Kaolin was reused as large quantities were cleaned from each of the LUC tests.

In order to make sure the host ground reused in the SUC tests was suitable to be compacted again after each of the LUC tests, the moisture content and the vane shear tests were performed at each layer of the LUC tests after the tests were finished.

Despite of the slight loss of the moisture content after each test the undrained shear strength values were still mostly below 25 (± 2) kPa; therefore, reusing the soil seemed practical for the SUC tests. The soil was only reused once and was disposed after each of the SUC tests.

Four minutes of compaction per layer via the vibrating hammer provided the required properties of host ground in the small container. The clay was compacted in 5 layers, each having a thickness of 80 mm. The thickness was the same as the LUC tests. In this container no water level was measured and also, no saturated sand layer was placed at either the top or the bottom of the container.

6.8.2 Column installation

Similar procedure described for the LUC tests was repeated on the small container for the column installation, where the density of column constructed was roughly calculated using the aggregates used and the volume of the column formed.

6.8.3 Column loading

Load was applied rapidly after the installation procedure was completed in the SUC. The container was transferred under the loading frame. The small loading plate (the diameter of 54 mm and the height of 108 mm) was used to apply the load over the stone column. After the maximum travel was achieved; the test was stopped and unloaded. The container was then removed from under the frame and the column shape was studied.

In the study of column shape either after the installation alone or after the loading, at first the aggregates in the column were vacuumed out and subject to the PSD. This showed the crushability of the aggregates under either of the installation or loading.

This process was only performed on the SUC tests as opposed to the LUC tests where the aggregates were only graded before the installation.

6.9 The LUC tests procedures

All the assumptions, the instruments, the measurements and preparations of the LUC tests were explained in sections 6.2 to 6.7. In this section, the specific details of each of the LUC tests in Table 6.1 were explained briefly.

Test 1-The pilot test

In this test, the unit cell was filled with sand and compacted clay. The column of the sand was installed and loaded; therefore, the preparations, the column installation and loading were practiced to make sure the set up ran smoothly for all the LUC tests.

Instead of the 9 layers of clay, only three layers were used. The base was filled with soaked Leighton Buzzard sand for a depth of 240 mm (equivalent to 3 layers) and gently compacted via a tamping rod. Above the sand, three layers of the Kaolin (with the moisture content of 41%) were compacted and covered with another layer of soaked sand with the depth of 240mm. No water level was measured in this test.

The material used for the column construction was dry Leighton Buzzard sand. The sand was washed and dried and used in stages to form the column via the top-feed method. The concrete poker was used to compact each layer for 20 seconds.

The small plate was used to apply the axial load on the column. After unloading and the removal of the top sand layer, the hand vane shear test was used to check the undrained strength of the clay.

The procedure confirmed the column installation and the loading method could be used for all the LUC tests.

Tests 2 and 6-No column

In these tests the load was applied on the host ground alone to assist in comparison of the no column versus various types of stone column constructed. Test 2 was loaded via the small plate and test 6 was loaded via the big plate. Therefore, the effect of the axially loading and the foundation load were compared in these two tests.

No water level was recorded after the preparation or after the unloading.

Tests 3 and 8-Primary aggregate

In these tests the column of granite was constructed to study the effect of PA column versus no column. Also, the granite was used as a bench mark to compare the columns of primary and recycled aggregates with each other. Both the tests were loaded with the big plate. Test 8 was a repeat test for test 2.

Tests 4 and 9-CC/CB

In these tests, the behaviour of the CC/CB as a RA was studied in the unit cell. The results were compared to a cell with no column, also, with the column of granite and against the other RAs. Test 9 was a repeat test.

Tests 5 and 10-IBAA (1)

Similar to tests 4 and 9, another type of the RA (IBAA (1)) was used to construct the VSC. The results of load carrying capacity were compared to a container with no column, the column of primary aggregate and the columns of other recycled aggregates. The water levels were measured which were compared to the other types of the columns to study the drainage and the behaviour of the ground during the loading of the column.

Test 10 was a repeat test.

Test 7-IBAA (2)

Similar to the other RAs, the results of load carrying capacity and the water level changes in the system were compared to the no column, the column of PA and the other RAs. This test was not repeated as the quantity of the material used for column construction was limited and only the trend of the load carrying capacity was considered as the important factor to study and compare with the other columns.

Tests 11 and 12-Wet recycled aggregate

In these tests, the aggregates were soaked in distilled water. In reality during the storage and transportation, the aggregates might be subject to water and rainfall and temperature changes. These conditions might change the aggregate properties in the short and the long-term.

In this research the effects of the condition of the aggregates (dry or wet) were compared for the primary and the recycled aggregates. The only RA used was the CC/CB as there was not enough material available from the other sources. The results of the load carrying capacity and the water level changes in the system were compared to the dry CC/CB (Tests 4 and 9), and also, the wet PA (tests 13 and 14).

After the unloading, as opposed to the other dry tests, the aggregates were cleaned out gradually and simultaneously with the Kaolin. The reason is that the wet aggregates might damage the vacuum cleaner during this process.

Tests 13 and 14-Wet primary aggregate

In these tests the results of the load carrying capacity and the water level changes were compared to the dry PA and RA and the wet CC/CB.

Test 15- The long-term primary aggregate

The only long-term aspect considered was the loading of the column long after it was installed. The behaviour of the column when loaded after a long duration that the column was constructed was compared to the rapidly loaded column after the installation performed on the primary and the three recycled aggregates.

After the installation the container was completely covered using plastic sheets to avoid the loss of moisture as much as possible during the three months before the loading.

6.10 Evaluation of the large unit cell tests

6.10.1 Errors in the laboratory tests

Similar to all the laboratory experiments, temperature changes, equipment and system can create errors for the LUC tests (Taylor, 1982). The assumptions considered in the design of the large cell tests created degrees of uncertainty and specially scaling of the column and the aggregates created variations from the practice of the VSC using the primary and the recycled aggregates.

The preparation process in which the clay was mixed with the tap water instead of the distilled water created errors due to the existing chemicals in the tap water which may affect the properties of the clay. The mix itself should be uniform and the clay was mixed for 10 minutes and left overnight after all the 9 layers were compacted in the LUC for homogenization (Head, 2006). This process was performed on all the LUC tests.

The installation process used in the unit cell tests created errors in the results and affected the density of the columns achieved. The process was performed accurately however, human mistakes via the exertion of pressure to the material during vibrations

was unavoidable. This caused various columns to be formed using the same material but with different densities.

The material source was another important factor affecting the accuracy and the analysis of the results. The sources were unique and could not be directly compared to other sources of primary or recycled aggregates. The strength, the PSD, the degree of packing and the density in the column significantly depended on the material used in the modelling which cannot be reproduced using various sources.

The measurements such as the water level changes were recorded from the pipettes that were numbered and the values read were not always accurate.

In the LUC, most of the tests on the columns of aggregates were repeated once. The exception is the IBAA (2), where enough material was not available for the repeat test. It was better to repeat the tests more times; however, the results of the repeats were used in calculations of the mean load-settlement values and error bars (refer to section 7.5). The deviations were mainly due to the various densities achieved in the columns due to the installation method and the energy applied to the aggregates.

6.10.2 Comparison and repeats

In order to reproduce and repeat the tests, clear instructions were provided by the researcher to make the repeat models of the LUC tests possible. However, as mentioned before different sources of aggregates and installation method can cause errors and variations in the results obtained. It has been discussed in chapter 7 that the densities of columns constructed using the same material was variable in the LUC tests due to the nature of the materials and the errors of the installation method used (refer to section 7.3.2).

The load-settlement measurements could not be directly compared to the other columns of different aggregates, as the PSD, the degree of packing, the density of the column and the angle of internal friction could be different and create variable results. The results of these tests could only be used as guidelines on how the specific sources of the recycled and the primary aggregates used in this research behaved in the context of VSCs.

The water level changes could be used to identify and interpret the behaviour of the surrounding soil in the unit cell, however, the results could not be directly compared to the measurements of the excess pore water pressure dissipation in the previous published work (Weber *et al.*, 2006; Cimentada and Da Costa, 2009; Castro and Sagaseta, 2012) as the soil was not consolidated in the LUC tests.

Each of the LUC tests takes approximately between two weeks to one month to prepare, load and clean depending on the availability and smooth performance of the equipment.

6.11 The SUC tests procedures

Test 1

The aim of this test was to try the procedure of the series 1 of the tests in the SUC. This was the only test in which the column was compacted by the standard compaction hammer. As this was not a regular procedure in any other tests performed in this research and also does not represent the installation of the VSCs via the vibro-float, the method of using the compaction hammer for the installation was abandoned after this test. However, the experience was used as a pilot test.

In series 1 of the tests in the SUC (refer to Table 6.2), the crushing of aggregates due to the installation and loading of VSC was studied. Also, the shape of the column constructed could be observed.

In test 1, the granite was used to model the column. The process started by preparation of the clay, which was compaction for 4 minutes per layer in 5 layers of 80 mm depth on the reused clay from the LUC tests.

As this was a trial test, no PSD was performed on the granite before or after the installation; neither after the loading. The granite was compacted in layers of 30 to 50 mm height for 10 blows per layers. The amount of the aggregates used in the installation was recorded for the column density estimation.

It was observed that during the installation of some of the columns the material of the column was slightly pushed into the surrounding clay due to the vibrational forces of the installation equipment and therefore, more material was required for the installation. The densities of the columns may vary due to this reason and the results of the column densities for all the LUC and the SUC tests are presented in chapters 7 (refer to section 7.3) and 8 (refer to sections 8.2.2, 8.3.2 and 8.4.2).

After the installation was finished, the cell was moved under the loading frame and the load was applied to the column over the small plate. After the unloading the shape of the column was investigated.

It was noted that due to the method of installation, large quantities of granite were used. Also, the compaction caused material to penetrate into the clay during the installation. This was further increased by the loading of the column and resulted in inaccuracy.

The method of installation was abandoned and the shape of the column was not used as an indicator of the behaviour of the stone column under installation and loading. After the test, the soil was cleaned from the container and the moisture content and the hand vane shear tests were performed at every layer of the soil.

Tests 2, 3, 4 and 5

In tests 2 and 3 the effects of installation and loading of the VSCs were compared. Tests 4 and 5 are repeat tests. Column shape was also studied under the 1 installation via the concrete poker.

The clay was prepared and the column was installed and vibrated using the concrete poker for 20 seconds per layer. The aggregate used in these tests was the dry granite and was sieved before all the tests.

In test 3, when the installation was complete, the column was not loaded. A clean and dry vacuum cleaner was used to take the aggregates out of the column. The material extracted was subject to the PSD after the installation. The comparison of the PSD before and after the installation indicated the level of aggregate crushing by the concrete poker.

The empty column was then filled with the cement grout and left for 24 hours to set. The shape of the column represented the effect of the installation.

Test 2, was the same as test 3, where after the installation of column, the aggregates were not vacuumed out. The column was loaded and after the test aggregates were vacuumed out and subject to the PSD.

The shape of the column was studied using the grouting method which represented the shape after the loading. The PSD before the installation and after the loading were compared to study the effect of loading. Also, they were compared to the PSD after the installation to distinguish the proportions of crushing attributed to either of the installation or loading.

Tests 6, 7, 8, 9 and 10

These tests were the same as tests 2, 3, 4 and 5, except that the CC/CB was used as a recycled material for the installation of VSCs.

Tests 11, 12, 13 and 14

Similar to the tests on the granite and the CC/CB, the IBAA (1) was used in both the installation and the installation/loading.

Tests 15 and 16

In these tests no column was constructed, the clay was prepared and then loaded under the same conditions as tests 1 to 14. The purpose was to compare the load-deformation of the host ground when it was not reinforced with any columns as opposed to the reinforcement with various stone columns.

Tests 17, 18, 19, 20, 21, 22 and 23

As opposed to series 1 where the various materials were used in the modelling of VSC, in these tests (series 2), only the granite was used. Also, the columns were directly loaded after the installation where the installation time was variable in these tests (refer to Table 6.3).

In all the LUC tests and series 1 and 3 of the SUC tests, the installation time used by the concrete poker was 20 seconds per layer. In these tests; 10, 30 and 90 seconds of compaction per layer were compared to the 20 second compaction.

The densities of the columns achieved were recorded and compared to the usual installation method. Also, the column shape was studied after the loading.

Tests 24, 25, 26 and 27

In series 3 of the SUC tests (refer to Table 6.4), the effect of the column material contaminated by fines was studied on the load carrying capacity of the stone column. Material used was the granite which was contaminated by crushed fragments of granite below 2mm. The columns were loaded after the installation and results of the load carrying capacity were compared to a column with no fines.

The clay was prepared in the same way as other the SUC tests. The column material was prepared differently. The usual aggregate sizes of 2 to 9.5mm were washed, dried and sieved. In order to add the fines, the granite was crushed in the LA machine for 100 minutes, and 1500 rpm. Using trial and error, 8 balls in the LA machine created fines of below 2mm in a well graded range.

Sieve sizes of 2, 1.18mm and 600, 425, 300, 212, 150, 75 and 63 μm were used to perform the PSD on the crushed granite. When a well-graded range was obtained, the crushed material was added to the usual granite.

Based on the standards more than 10% fines is not acceptable in the aggregates used for the VSCs (ICE, 1987; BRE, 2000), also, other researchers studied the effects of 10 and 20% fines in the aggregates on the behaviour of VSC (McKelvey *et al.*, 2002). Based on these guidelines, 10% and 20% fines were added to the material used for the column installation.

6.12 Evaluation of the small unit cell tests

6.12.1 Errors in the laboratory tests

Similar sources of errors described in section 6.10 for the LUC tests exist for the SUC tests as well. The soil for the host ground was reused and some loss of moisture content was unavoidable. However, it has been discussed in the results (refer to sections 8.2.1,

8.3.1 and 8.4.1) that the undrained strength was still within the range required for these tests.

The installation method contributed to the various column densities and shapes, especially when different compaction times of 10, 30 and 90 seconds per layer were used.

6.12.2 Comparison and repeats

Clear instructions were provided to repeat and reproduce the SUC tests. In these tests due to the limited sources of the material available the tests were repeated only once.

In case of different installation times and contamination of the column material with fines, the granite was the only aggregate tested. The trends observed may not be generalized for all the primary and alternative aggregates. However, they provided understanding of the behaviour of the columns under similar installation and loading conditions.

6.13 Summary of unit cell testing

In this chapter, the unit cell concept was used to study the behavior of the single stone column in soft clay. Aggregates were used in two types of the large and the small unit cells to study the various factors affecting the performance of VSCs in the short-term.

In the chapter, the simplifying assumptions, the measurements, the factors studied and the instrumentation in both the large and the small cells were presented and the differences in aim and procedure of each were described.

CHAPTER SEVEN

RESULTS AND DISCUSSIONS- PART 2: THE LARGE UNIT CELL TESTS

7 RESULTS AND DISCUSSIONS-PART 2: THE LARGE UNIT CELL TESTS

In this chapter the large unit cell tests results were presented followed by the interpretation and discussions. Firstly, the quality control measures are presented which included the moisture content and the undrained strength of the host ground, followed by the particle size distribution and the density of the columns constructed.

Secondly, the load-deformation behaviour of various columns constructed in the unit cell are compared. The aim of this research was to compare the columns of the primary and the recycled aggregates and the load-deformation results were a main part of the discussions. Various factors such as wet and dry columns and the short-term and long-term behaviour are included in these results.

The settlement of the various columns was estimated using Priebe's method (Priebe, 2005) and compared to the actual settlements of the columns tested in the LUC.

The water level measurements were performed in the LUC tests at various stages of during the installation, during loading and after the tests and the results are compared and discussed.

7.1 Introduction to results and discussions of the large unit cell tests

The method of preparation, measurements, instrumentation and all the LUC tests details were explained in chapter 6 (refer to Table 6.1 for the tests).

The various aspects of the LUC testing enabled the comparison of the recycled aggregates with the granite (PA) in the context of VSC installation and loading. These aspects included the load-deformation behaviour under the same loading conditions, the water level changes during the loading and the settlement improvement of the various columns constructed.

The water level changes measured at various distances from the centre of the column and at various depths indicated the behaviour of the surrounding soil at different stages of the construction and loading.

Table 6.1 in chapter 6 (refer to sections 6.3.1 and 6.7), summarizes all the 15 tests performed in the large container. The same test numbers and test names were used in this chapter.

7.2 Quality control of the host ground

The host ground was China clay with 41% moisture content to provide the undrained strength of 10 to 25kPa under the controlled compaction condition (refer to sections 6.5.1, 6.6.3, 6.6.4 and 6.7 for the details).

In the pilot test, only three layers of clay were used as opposed to all the other 14 tests where 9 layers were compacted for the construction of an end-bearing column.

In these tests, for the quality assurance; samples of moisture content were taken at various stages before and after the tests.

Two moisture content samples were taken from each layer of the host ground after the mixing process (refer to section 6.6.3); followed by the 3 samples from the core extruded during the installation and column formation and finally the samples taken one week after each test from all the layers of the host ground.

After the tests, 4 samples of the moisture content and the undrained strength from each of the layer of Kaolin provided the information on the host ground for the quality control (refer to section 6.5.1).

The average values of the moisture contents and the undrained strengths were calculated for each layer at the boundary condition (refer to section 6.5.1), and the detailed results were presented in Appendix 5 (refer to CD). It was recommended to check these values at various locations closer or further from the column in future research.

Table 7.1 summarizes the range of the moisture content and the undrained strength values obtained with accuracies of 0.01(%) and (± 2) kPa, respectively.

Table 7.1: Quality control of the host ground properties in the various LUC tests

Test name	Test number	Moisture content range before the test (after mixing) (%)	Moisture content range after the test (%)	Moisture content of the core extruded for the column installation (%)	Undrained strength of the host ground after the test (kPa)
Pilot test	1	Not measured	Not measured	Not measured	17-23
No column-axial loading	2	40-42	39-42	No core extruded	18-22
No column	6	40-42	39-42	No core extruded	16-22
Dry primary aggregate	3	Not measured	Not measured	Not measured	Not measured
Dry primary aggregate-repeat	8	39-42	38-41	40-42	18-23
Long-term primary aggregate	15	40-42	39-42	41-43	13-20
Wet primary aggregate	13	39-42	39-41	39-42	Not measured
Wet primary aggregate-repeat	14	39-45	39-42	39-42	16-22
Wet recycled aggregate	11	41-43	40-42	41-43	13-18
Wet recycled aggregate-repeat	12	41-43	39-41	40-43	16-21
Crushed concrete and brick	4	38-42	39-41	39-41	19-23
Crushed concrete and brick-repeat	9	40-44	39-42	40-42	15-22
IBAA (1)	5	38-42	39-41	39-43	14-23
IBAA (1)-repeat	10	40-42	39-42	40-42	15-19
IBAA (2)	7	39-41	38-41	39-41	16-25

As observed in Table 7.1, the clay mixed provided the range of the moisture contents required for the tests. In cases where slightly higher values were recorded, the samples were probably taken from the parts close to the base of the mixer where slightly higher moisture content existed due to the mixing procedure used. The slight difference did not affect the condition of the clay as due to the transfer and compaction of the clay in the cell, slight loss of moisture content was expected.

The moisture content range from the core and the layers after the tests, show a very consistent range of 38 to 43% which provided the undrained strength required.

There was slight loss of the moisture content throughout the whole process which was unavoidable, but the results of the undrained strength confirmed the suitability of the surrounding clay condition for all the tests. All the undrained strength values were above 10 and below 25kPa.

Table 7.2 shows that in the long-term test (test 15), on the column of granite, slightly lower values of undrained strength were observed compared to the other PA tests, especially in the top layers. This could be due to the transfer of the water from the sand layer on the top which was soaked and kept wet throughout the entire test procedure. The values presented were recorded one week after unloading. The values at each of the layers are the average of the 4 samples taken from that specific layer.

Table 7.2: Quality control of the host ground properties in test 15

Layer	Depth form surface (mm)	Moisture content before the test (%)	Moisture content after the test (%)	Undrained strength after the test (kPa)
9 (Top)	120	41.54	40.93	13.25
8	200	41.73	41.18	15
7	280	41.09	41.10	18
6	360	41.04	39.89	19.5
5	440	41.37	40.98	19
4	520	41.56	40.84	19.5
3	600	41.35	39.87	19
2	680	41.00	39.73	18.5
1 (Base)	760	41.72	39.71	19

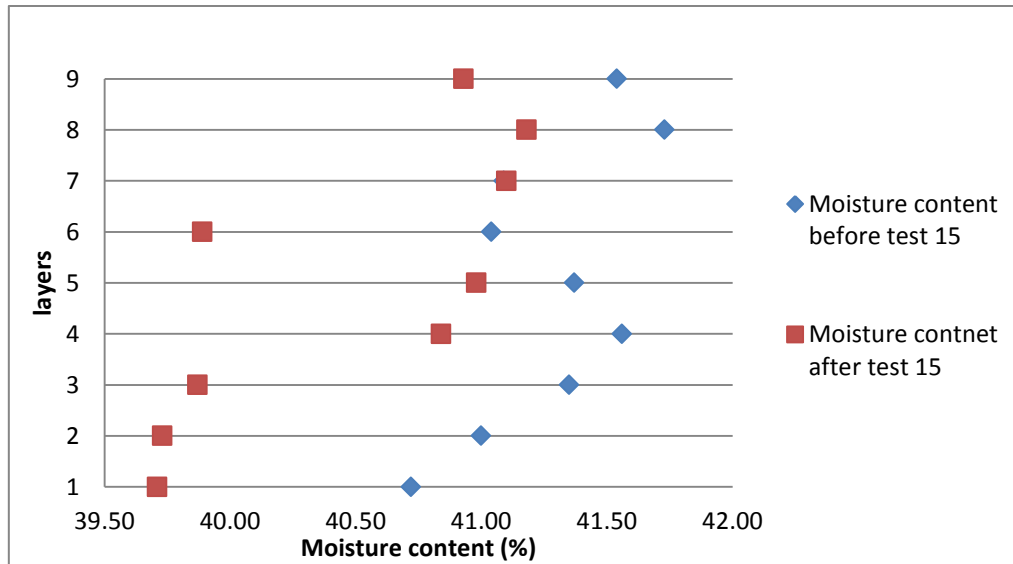


Figure 7.1: Moisture content before and after test 15

As observed in Table 7.2, the moisture content values after the mixing were very consistent and were between 41 and 42%. After the test, the moisture content values decreased slightly for a maximum of 2% which was expected due to loss of the moisture during the transfer and compaction of the soil in the cell. The values of the undrained strength validate the suitability of the condition of the soil based on the requirements.

According to Figure 7.1, the values of the moisture content reduced more in the last three layers near the base of the tank compared to the other 6 layers. This drop in the moisture content was less than 1.5% and was negligible. Some loss of moisture was expected near the base where the water could travel into the layer of the sand underneath.

Figure 7.2 provided results of the moisture content before and after test 14, in which it could be observed that the difference in the moisture content was minimal in the middle layers before and after the test, whereas bigger gaps were observed in the top and bottom layers. This could be due to the loss of moisture content from the top of the container throughout the whole process of testing. Water could be transferred to the sand near the base and reduce the values of the moisture content slightly.

In 8 layers the loss of moisture content was observed after the test, except for layer 8 in which the moisture content increased. This is the layer in which the model piezometers were installed and the water from the saturated sand around the piezometers could travel into the surrounding clay (refer to sections 6.6.2 and 6.7).

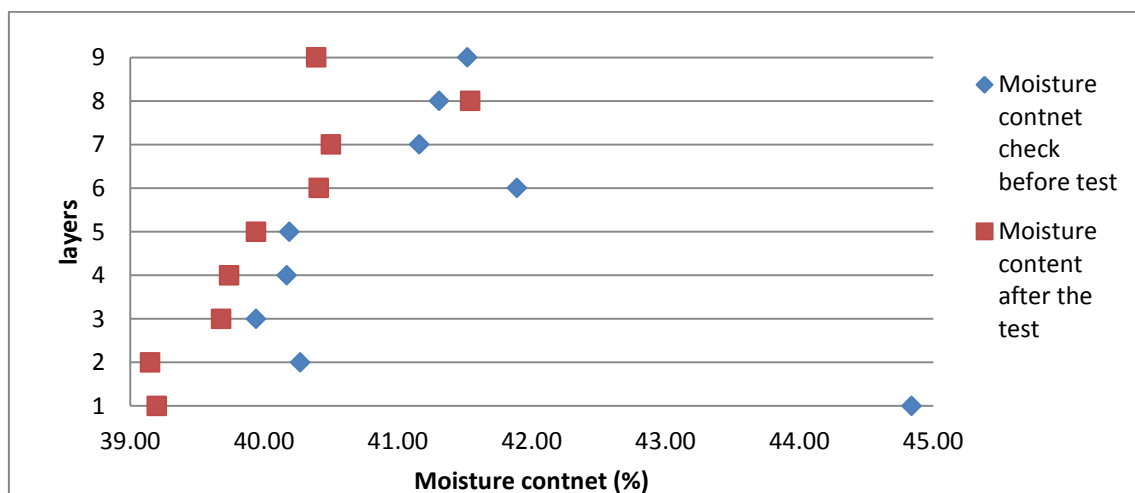


Figure 7.2: Moisture content changes before and after test 14 in the large unit cell

It can be concluded that the moisture content decreased slightly after the tests, but the range provided the strength required for the host ground. Details of all of the LUC tests were provided in Appendix 5 (refer to CD).

Figure 7.3 is an example of the undrained strength changes with the depth in test 7, performed on the column of the IBAA (2). There was no specific trend observed in the changes of the undrained strength values with depth; however it seemed that after the first 4 top layers, the undrained strength values decreased with an increase in the depth. In the top layers the loss of moisture content could contribute to the increase in the undrained strength.

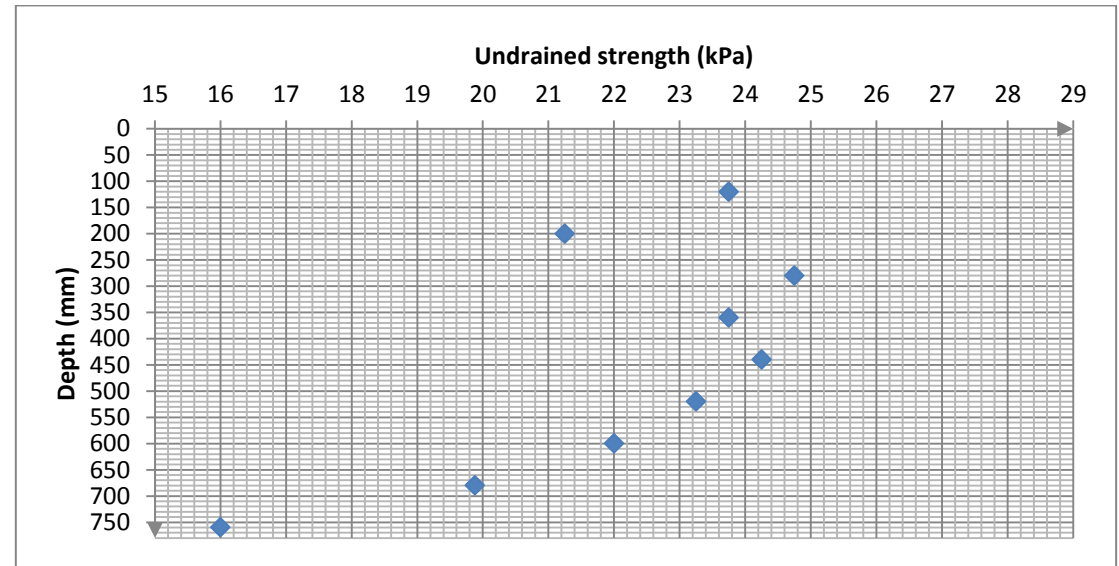


Figure 7.3: The Undrained strength changes with the depth after test 7 in the large unit cell

Figure 7.4 compares the undrained strength values of all the LUC tests, except for tests 1, 2, 3 and 13 (refer to Table 6.1), where complete data were not available.

This figure confirmed the values of the undrained strength to be between 10 and 25kPa. There was no particular trend regarding the undrained strength variations with the depth.

In tests 4, 5, 7 and 8 the values increased slightly in the middle and reduced again near the base. The reason might be related to the higher level of compaction in the middle layers. The lower strength values near the base could be related to the slight increase in the moisture content values in some of the tests.

As the saturated sand existed at the base of the container, some of the water could transfer into the bottom layers of the clay and higher moisture content can result in slightly lower undrained strength values.

There was also no particular difference in the trends between the wet and the dry tests and the long term test did not show any particular difference in terms of the moisture content values after the test.

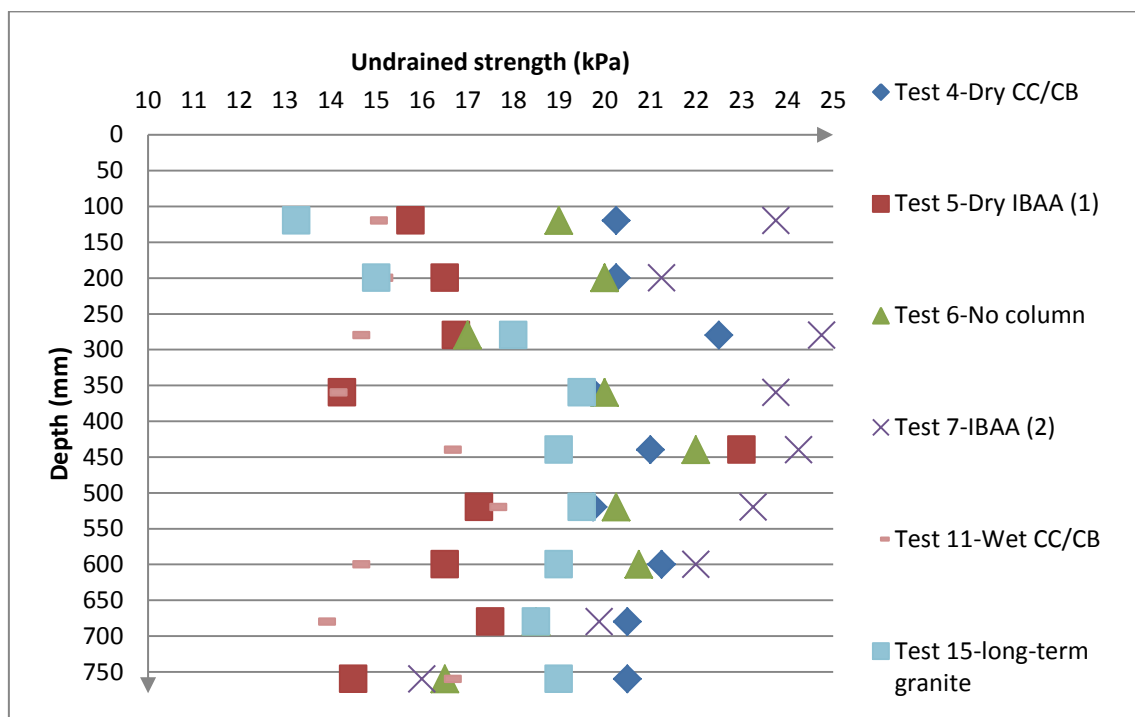


Figure 7.4: The undrained strength values of the clay after the tests in the LUC container

7.3 Quality control of the column material

7.3.1 Particle size distribution

After the host ground was prepared for the unit cell testing, the aggregates were washed, dried and subject to the PSD to be used in the stone columns. The reason for performing the PSD was to make sure that the sizes between 2 to 9.5mm were used. No particles below 2mm in size were used in the material for the column construction in the LUC tests (refer to section 6.7).

Figure 7.5 shows the PSD of all the materials used in the LUC tests before installation. The effect of the installation and loading on the crushing of these materials was not the subject of the LUC tests and was discussed in the SUC results (refer to section 8.2.2).

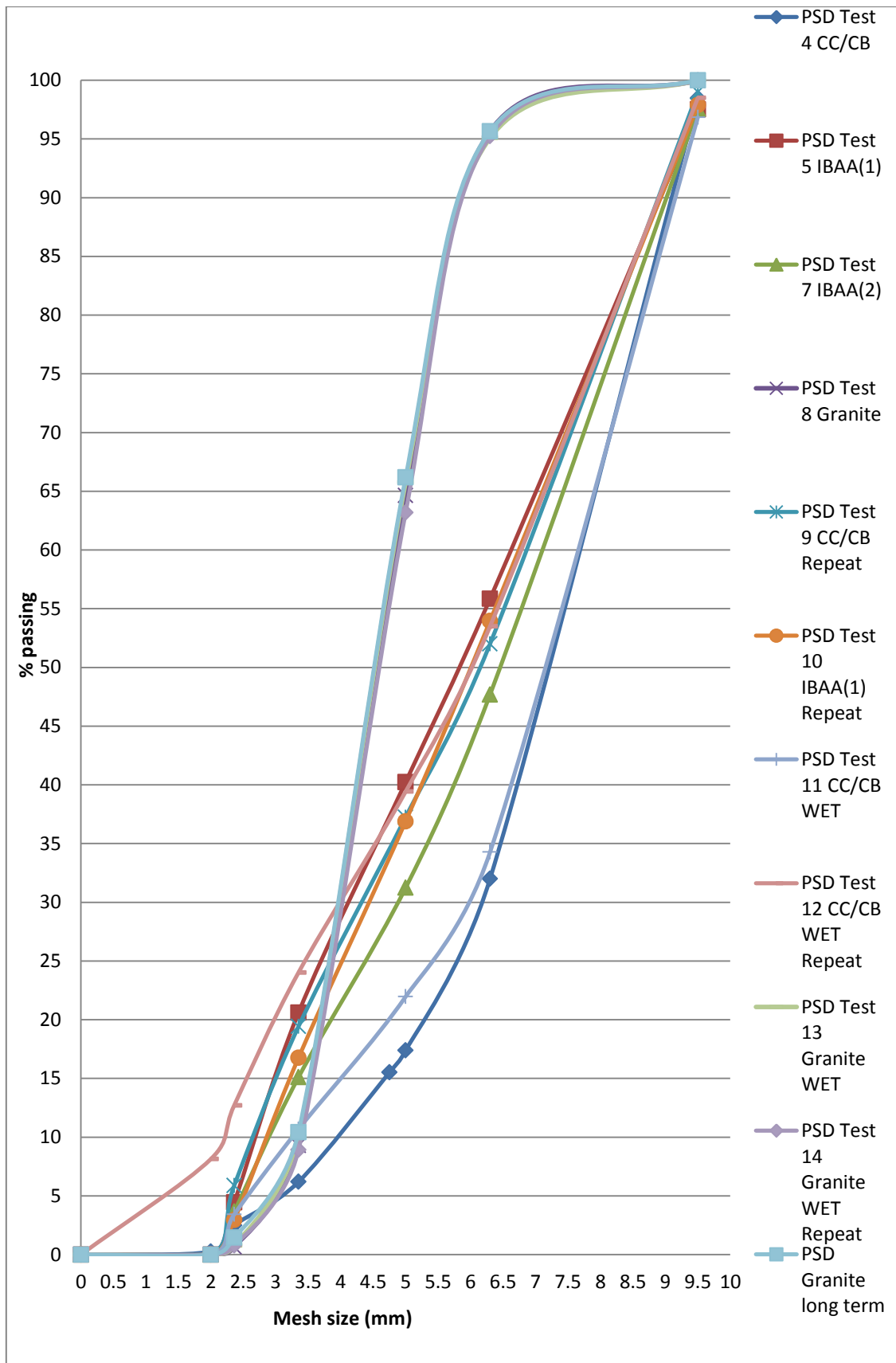


Figure 7.5: The particle size distribution of the aggregates used in this study before the installation in the single columns in the large unit cell

As observed in Figure 7.5, only 11 graphs were presented out of the 15 tests. Tests 2 and 6 were performed on the clay only and as no column was tested, the aggregates were not used.

In test 1, which was the pilot test, sand was used to try the installation method of the columns and the results of this test were not comparable with the other tests. Therefore, the PSD was not performed on the sand. The PSD was not performed on the granite used in test 3, as this was one of the earlier tests and the quality control tests were not developed yet.

One of the main factors studied in the LUC tests was to compare the performance of the various types of the primary and the recycled aggregates in the construction of VSCs and as an important part of this investigation, the various PSDs were compared for the materials used. The materials could be more uniformly graded or alternatively well-graded.

Figure 7.5 demonstrated both of these types of the PSD for the various materials. There was no right pattern and distribution for the materials for use in the VSCs, however, this study addressed the effect of the PSD on the performance of the columns constructed (refer to sections 7.4.3 to 7.4.7).

It was observed from the PSD trends that granite which was used in tests, 8, 13, 14 and 15 had a consistent trend, where approximately 60% of the material was below 5mm in size. The PSD curve was very uniform compared to the other recycled materials.

For the IBAA (1), both curves were very similar in test 5 and its repeat in test 10.

Compared to the granite more fines exist in the IBAA (1) and 60% of the particles were smaller than 6mm. The curves were showing well-graded pattern compared to the granite.

The IBAA (2) had slightly more fines compared to the IBAA (1) which represented more distribution in the grading.

On the other hand, the results of the CC/CB were varied and two of the samples in the tests 4 and 11 had approximately between 10 to 30% more fines than the samples in tests 9 and 12. This variation could be due to the crushing of this material for the tests, which created various sizes and although the sampling was accurately done to represent all particle sizes, in some tests, the smaller range and in the other two the bigger range of the sizes were collected. As each of the trends was repeated once for the CC/CB; the aggregate range between the two trends was considered as the typical aggregate sizes of this source.

Figure 7.6 shows the average PSD of the four materials used in the LUC. It was observed that the IBAA's had more fines compared to the CC/CB. Also, the RAs used had wider range of the various aggregate sizes compared to the granite, which could result in better packing of the aggregates when vibrated during the installation of VSC.

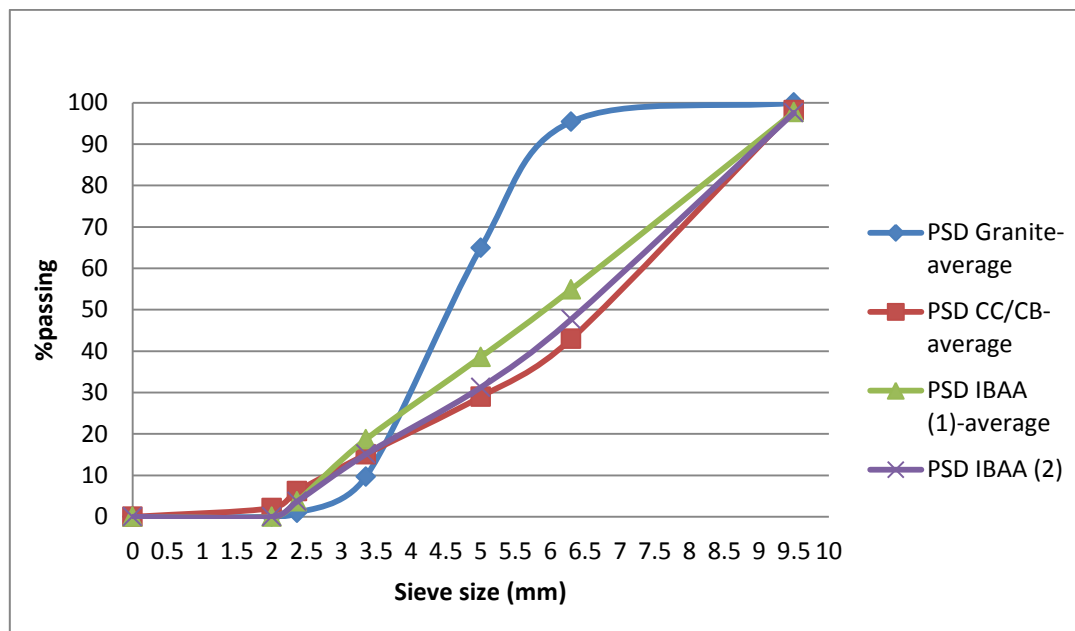


Figure 7.6: Average PSD of the 4 aggregates used in the large unit cell tests

7.3.2 Density of the stone columns

The density of the columns installed was estimated using the approximate volume of the column to be constructed and the actual values of the aggregates used for each column.

The volume was variable for the different tests, despite using the same method of installation. The model vibro-float could create various diameters during the installation due to the pressures exerted (refer to section 6.4.2).

Table 7.3 shows the results of the column densities estimation for all the columns constructed.

Table 7.3: Density of the columns constructed in the large unit cell and the angle of shearing resistance of the aggregates

Test name	Test number	Column density (kg/m^3)	Angle of shearing resistance measured in this research (degrees)
Dry primary aggregate	3	Not measured	47
Dry primary aggregate-repeat	8	1686.43	47
Long-term primary aggregate	15	1786.80	47
Wet primary aggregate	13	1776.95	-
Wet primary aggregate-repeat	14	1895.21	-
Wet recycled aggregate	11	1262.12	-
Wet recycled aggregate-repeat	12	1756.92	-
Crushed concrete and crick	4	1228.22	40.2
Crushed concrete and brick-repeat	9	1521.04	40.2
IBAA (1)	5	1215.50	41.5
IBAA (1)-repeat	10	1577.51	41.5
IBAA (2)	7	1449.94	40.2

As observed in Table 7.3, the dry granite which was used in both the short and the long-term tests provided columns with similar densities with less than 10% difference which was negligible.

On the other hand, for the dry CC/CB, less than 25% difference existed which could not be ignored. This could be due to the exertion of pressure during the installation in some of the tests. Also, if the small cavities exist near the column position in the surrounding

clay, some of the material used in the installation could fill the cavities and the density achieved could be higher than the other columns of the same material.

It was mentioned in the PSD results (see section 7.3.1) that the tests on the CC/CB showed variable PSDs in the material and the density difference could be contributed to the various sizes of the material used in the column construction.

The same difference in the column densities existed for the columns of the IBAA (1). In case of the IBAA (2) not enough material was available to repeat the test.

For the wet aggregate tests, in both cases of the granite and the CC/CB, the densities were similar except for test 11 on the wet CC/CB. The same reason of error in the installation and PSD resulted in the lower density achieved. The difference in the three other density values in the wet aggregate tests was less than 10%.

In case of tests 4 and 11, where the PSD curve showed lower percentage of fines in the CC/CB, the density of the column achieved was lower. On the other hand, where a higher percentage of fines were observed across the PSD curve, a higher density was achieved.

Therefore, the densities calculated could be a combination of the various factors such as the proper column formation, the percentage of fines and the smaller particles in the PSD range which could positively affect the degree of packing, the cavities existence in the installation and the quality of workmanship.

On the other hand, the density of the columns seemed to be irrelevant to the condition of the aggregates (wet or dry) for the materials tested in this research.

The effects of the installation and the quality of workmanship could be observed in the IBAA (1) and its repeat where despite having a similar PSD, the densities were 30% different.

The angle of shearing resistance was another factor which was different for the primary and the various RAs. Based on the study by McKelvey *et al.* (2002), the condition of wet and dry does not affect the angle of shearing resistance significantly in the shear box tests.

In terms of basalt (PA), wet condition caused a reduction of 3 degrees in the angel of shearing resistance. In case of the crushed concrete, there was no difference between the two conditions in the angle of shearing resistance (McKelvey *et al.*, 2002).

The shear box test was not performed on the wet aggregates in this research due to insufficient quantities of the materials available (refer to section 4.5.1). Based on the research by McKelvey *et al.*, (2002) where the values of the angle of shearing resistance for the wet and dry aggregates was very similar, it was concluded that the difference in the densities of the wet and dry materials used in this research was mainly a factor of the quality of workmanship and the PSD. Higher magnitudes of the angle of shearing resistance led to a higher stress concentration and resulted in slightly better packing of the material and ultimately higher densities of columns.

Based on the PSD, the angle of shearing resistance and the densities of columns constructed, the results of the stress-strain of the columns under static loading were further analyzed.

7.4 Loading of columns

After the column installation, the single stone column was quickly loaded under the strain-controlled condition. The load-deformation behaviour was observed and compared for the various columns.

The factors such as the density of the column, the PSD of the column material, the material condition (wet/dry), the angle of shearing resistance, the material shape and

crushability were used in the interpretation of the behaviour of the various aggregates used in the LUC tests.

Firstly, tests 2 and 6, where the no column was constructed were compared with each other. Followed by the comparison of the various primary and RA columns and the wet and dry conditions, and lastly the long-term test was compared with the other columns of the primary aggregates.

7.4.1 The No column test

In these tests, clay was prepared and the loading plates were located in the assumed location of the stone column and then loaded. Two plates were used; the small plate to model the axial and the big plate to model the foundation loads in tests 2 and 6, respectively.

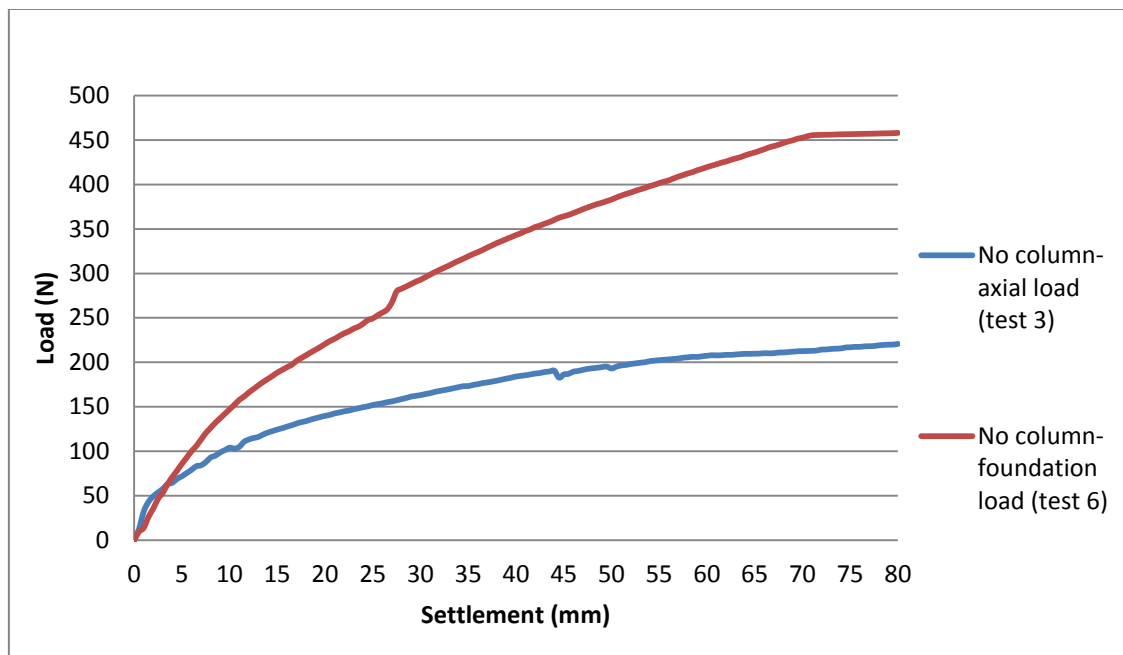


Figure 7.7: Load-settlement behaviour of the soil with no stone columns under the two axial and the foundation loads

Both tests were loaded until the maximum deformation of 80mm which was well beyond the failure condition was achieved. In the beginning of the loading, below 5mm of settlement, the axial plate seemed to produce higher loads compared to the foundation plate; this could be due to the initial punching of the plate into the soil. After a certain point, the big plate (test 6) showed much higher values of load and the difference gradually increased up to two times the maximum value of the axial plate at 80mm settlement. As the load could not be compared unless applied on the same unit of area, the stress-strain curves of the same tests were drawn in Figure 7.8.

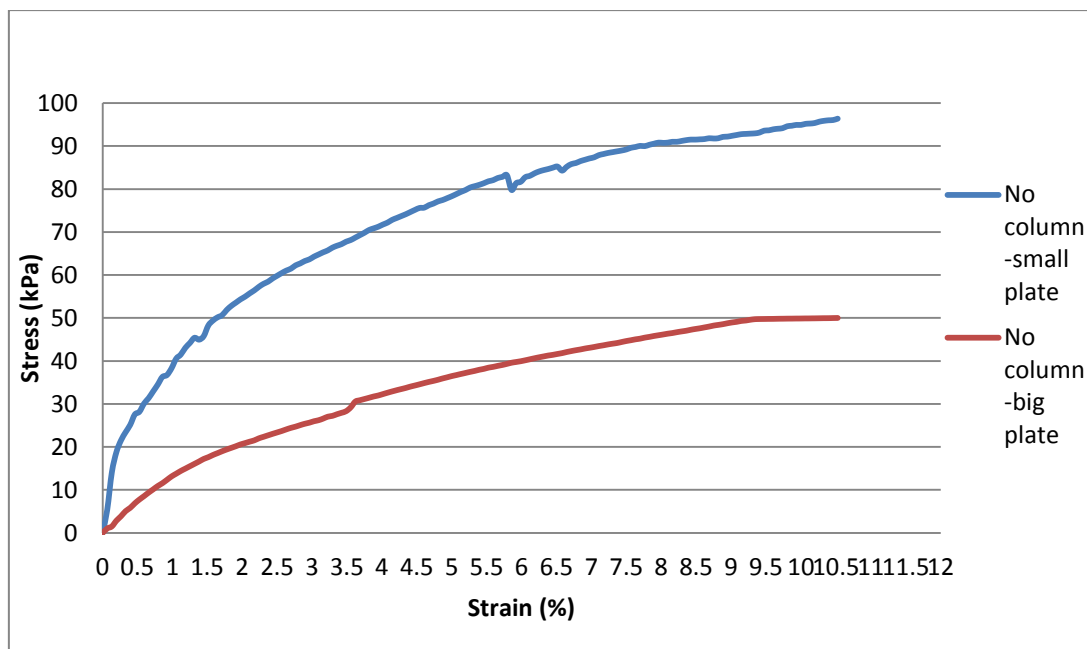


Figure 7.8: Stress-strain curves of the no stone columns under the axial and foundation loads

The exact opposite trend was observed here, where the big plate used in the loading resulted in lower stress values compared to the axial plate at each specific strain.

The curves were obtained by dividing the loads applied to the plan area of each of the plates to achieve the stress.

The strain was calculated as the ratio of the deformations to the assumed column length (760mm). The trend observed showed that higher values of stress were expected from the axial loading at each specific strain. The results of these two tests was used to compare the axial and foundation loading and showed that the foundation load caused lower stress in the ground.

In the rest of the models in the LUC, the foundation plate was used to load the columns and the results were compared to test 6.

7.4.2 Columns of the dry primary aggregates

Tests 3 and 8 were performed on the columns of dry granite. In test 3 as it was one of the initial tests, the measurements were not performed completely. The density of the column constructed in test 8, as well as the angle of shearing resistance of the granite was presented in Table 7.4.

Table 7.4: Properties of the columns of granite in the large unit cell tests

Test name	Test number	Column density (kg/m^3)	Internal angle of shearing resistance measured in this research (degrees)
Dry primary aggregate	3	Not measured	47
Dry primary aggregate-repeat	8	1686.43	47

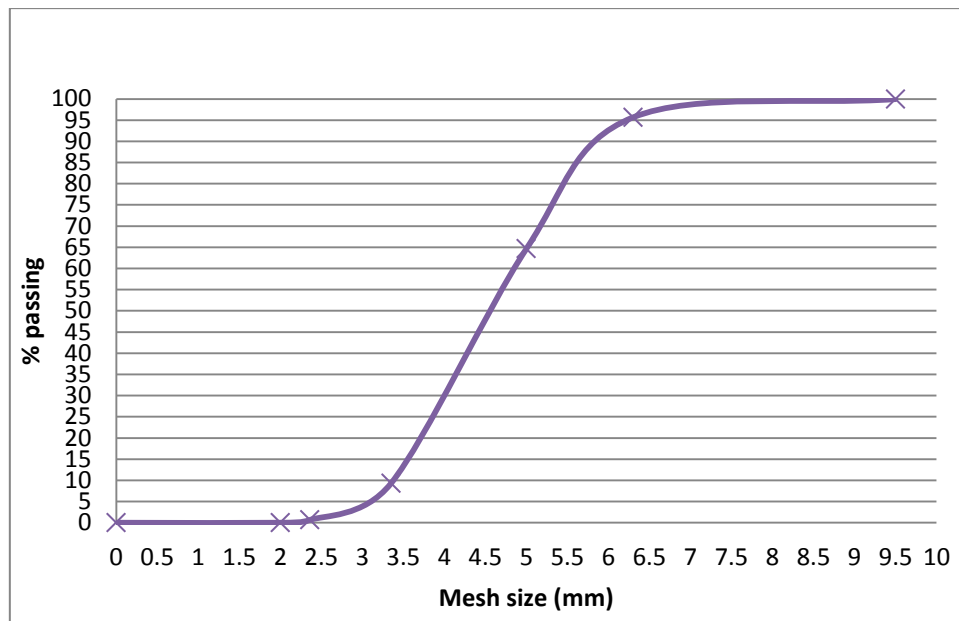


Figure 7.9: Particle size distribution of the granite before the installation-test 8

As the information for the PSD and the densities of the columns were not available for both tests, the results of the stress-strain during the loading of these columns could not be compared using this information.

Figure 7.10 showed the comparison of tests 3 and 8; the average stress-strain curve of the columns of the granite and the stress-strain curve with the no column.

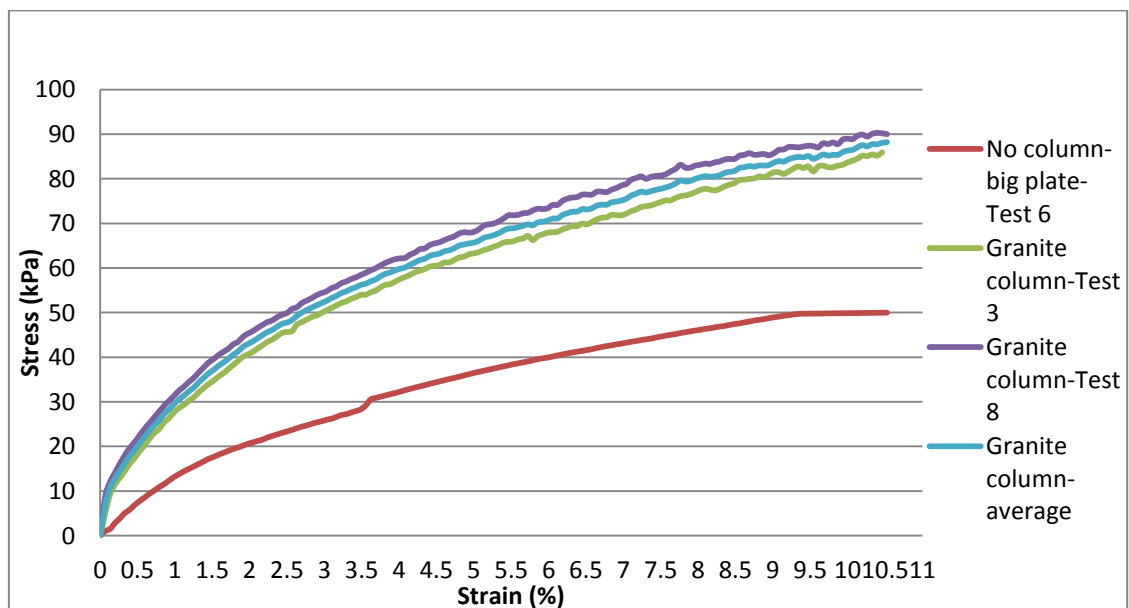


Figure 7.10: Stress-strain of the columns of granite in the large unit cell tests

The graphs represented stress values at the maximum strains of 10.5%. This strain was calculated using the ratio of the deformation to the column length. The maximum deformation was 80mm which was well beyond the failure and the usual settlement values of the stone columns constructed.

At this point the column had effectively failed in settlement. The failure point could be defined for the stone column using the various methods, such as Hughes and Withers (1974) and Zakariya (2001).

The most common failure definitions were related to the peak value of the load, the foundation width (Zakariya, 2001) and the column diameter (Hughes and Withers, 1974; Al-Mosawe *et al.*, 1985).

In this research all the possible analysis was used in defining the failures of the columns constructed.

As observed in Figure 7.10, there was no specific point which could be considered as the peak stress; however, the 80mm deformation (equivalent to 10.5% strain) was well beyond the settlement failure of the columns.

- The diameter of the column and the loading plate were 54 and 108mm.

According to Zakariya (2001), the failure is the load at 10% of the foundation width in deformation. Based on this definition, the stress or load at 1.42% strain was considered as the failure point.

- Hughes and Withers (1974), defined the same criteria at 58% of the stone column diameter, whereas Al-Mosawe *et al.* (1985), argued this ratio to be 60% of the column diameter. Based on these calculations, at strains of 4.12 and 4.26% the stress or the load obtained was the failure point.

As observed in Figure 7.10, where the single columns of granite were installed in the soil, the failure could be defined at the approximate points of 1.5 or 4.5% strain.

At these strains the values of the stress capacity improved approximately 122 and 83% compared to the no column loading. This meant that using the columns of PA could increase the load carrying capacity significantly regardless of the point of failure definition. This trend seemed to start from the lower strain values and continued to the higher strains beyond the failure of the stone columns.

It was observed in Figure 7.10 that the tests on the granite were both showing very similar results in terms of the load carrying capacity.

Black *et al.*, (2007a), modelled the columns of basalt in peat in the laboratory tests on end-bearing and partial length columns. In the full-length columns, the load-deformation characteristics were improved in the ground up to 1.5 times compared to the no column condition.

The comparison with this study was not possible as the the scaling used; the material source and the host ground properties were different from this research. However, the results of the improvement in the stress-strain behaviour of the ground when the column of the granite was constructed, agree with the other research where a single full-length column improved the bearing capacity and the settlement of the host ground (Black *et al.*, 2007a; Black *et al.*, 2007b; Black *et al.*, 2011; Sivakumar *et al.*, 2007 and Sivakumar *et al.*, 2004).

7.4.3 Columns of primary and recycled aggregates

In Figure 7.11, the various materials tested in this research were compared for their stress-strain behaviour. The trend shown for each material was the average stress values from the two tests performed, except for the IBAA (2) where material was available for one test only.

It was observed that the construction of the stone column using the dry aggregates improves the load carrying capacity significantly regardless of the column material and

its type (PA or RA). All the curves showed higher values of the stress at the same values of the strains compared to the no column test.

The IBAA (2) showed higher load carrying capacity compared to all the other RAs and even higher than the granite (PA). This could be contributed to its structure and the ash matrix which held the material together and also, the effect of its well-graded PSD which resulted in better packing of the column materials during the installation and loading.

The CC/CB and the IBAA (1) had very similar trends and the granite had the lowest load carrying capacity throughout the loading compared to all the other columns of aggregates.

The IBAA (1) and the CC/CB showed a slight difference in the beginning of the loading and towards the end. Initially the IBAA (1) had higher stress values probably due to its structure and nature which caused better packing of the material under the lower stress values. The stress values decreased slightly compared to the CC/CB after 2.5% strain due to the possible crushing of the glass pieces. At around 7.5% strain and well beyond the settlement failure, the two materials showed very similar values of the stress at each value of the strain.

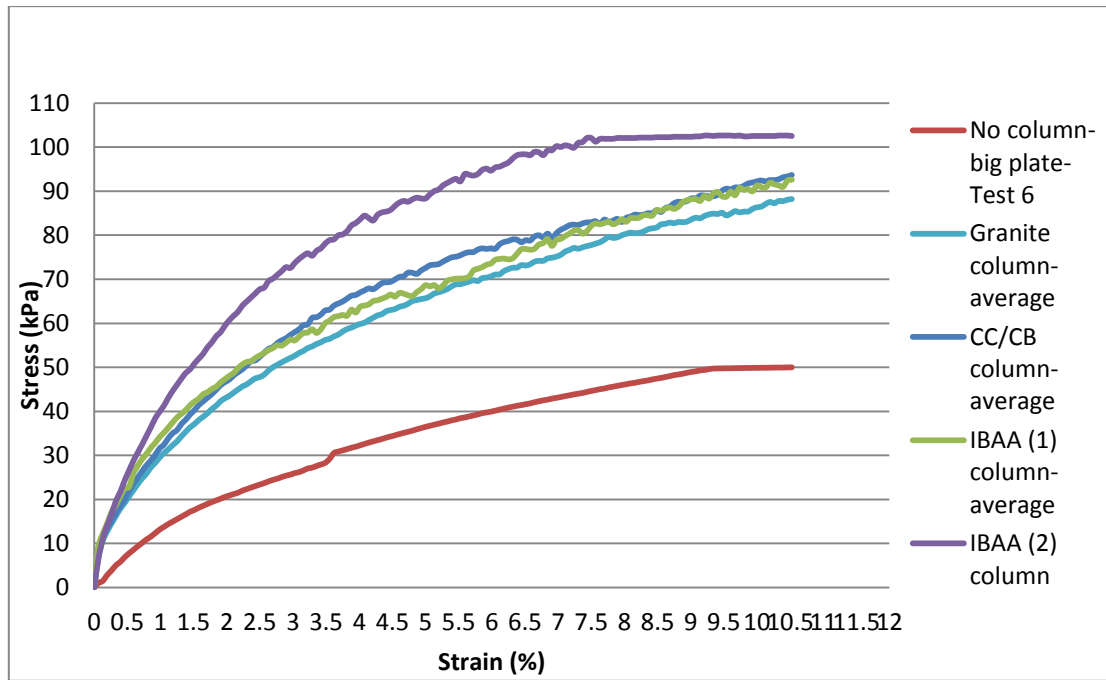


Figure 7.11: The stress-strain curves of the primary and the recycled aggregates in the large unit cell tests

At the failure points of approximately 1.5 and 4.5% (Zakariya, 2001; Hughes and Withers, 1974; Al-Mosawe *et al.*, 1985) the stress values improved compared to the no column condition and the estimated improvement results were presented in Table 7.5.

Table 7.5: Improvement of stress carrying capacity of stone columns of various materials compared to no column

Failure point at the strain value of (%)	Stress improvement of the column of granite compared to no column	Stress improvement of the column of CC/CB compared to no column	Stress improvement of the column of IBAA (1) compared to no column	Stress improvement of the column of IBAA (2) compared to no column
1.5	100%	128%	128%	189%
4.5	83%	106%	95%	156%

As observed in Table 7.5, the stress carrying capacity increased over 100% more in the columns of PA and the RA compared to the no column condition at a strain of 1.5%.

If the failure is defined at 4.5% strain, still a significant improvement was observed with a minimum of 83% in case of the column of granite. In both the failure points, the IBAA (2) outperformed the other columns by a large margin.

The analysis of the various columns could also be related to the angle of shearing resistance, the PSD and the densities of the columns achieved.

Table 7.6: Densities and the internal angle of shearing resistance of the various stone columns

Column material	Average column density estimated for each test and its repeat (kg/m^3)	Internal angle of shearing resistance (degrees)	Average density of the materials in the shear box (kg/m^3)
Granite	1686.43	47	1718.06
CC/CB	1374.63	40.2	1364.23
IBAA (1)	1396.51	41.5	1479.01
IBAA (2)	1449.94	40.2	1427.79

As observed in Table 7.6 the values of the angle of shearing resistance were obtained using the small shear box test with various the materials. The difference in the density of the materials in the box was due to the nature and the PSD of these aggregates.

The significant difference was in the granite where more material was compacted and sheared in the box. Therefore, the values of the angle of shearing resistance could be related to the densities obtained. When a higher density in the box was achieved; due to more contact between the particles; a higher angle of shearing resistance was obtained.

The same difference was observed in the stone columns, where the column of granite had a higher density compared to the recycled aggregates. This difference could cause different behaviour of the materials under the stone column loading.

However, according to the PSD curves in Figure 7.6, the IBAA (2) had more spread concentration of the various sizes compared to the other materials. This difference seemed to be comparable with the stress-strain curves observed for the four aggregates tested.

When a well-graded PSD existed, the load carrying capacity increased. This trend could be compared specially in the granite with a uniform PSD and the IBAA (2) which was well-graded and the results of the stress-strain curves showed a much higher load carrying capacity for the latter.

The nature and the structure of the materials could also affect the load carrying capacity under the static loading of the stone columns after the installation. In case of the IBAA (2), the structure and the existence of the ash held the particles together and provided a stronger column under the static loads. This aspect agreed with the results of the aggregate index tests, where the IBAA (2) outperformed the other recycled aggregates in some of the tests.

The shape and the angularity of the aggregates could also affect the density and ultimately the load carrying capacity of the columns constructed. It was observed in Figure 7.11 that the IBAA (2) curve became steady after the strains of approximately 7.5%. Although this strain point was beyond the failure, it was possible that if the tests were continued for more than 80mm settlement, the granite and the other RAs would catch up with the stresses obtained in the IBAA (2). However, this was not a practical study as the failure occurs before these strain values.

7.4.4 The wet primary and recycled aggregates

Four tests were performed on the wet aggregates in the large unit cell. The wet granite was tested as the wet primary aggregate versus the wet CC/CB as the only recycled aggregate to be tested at the wet condition. Not enough quantities of the IBAA were available to perform the wet tests on. On both the granite and the CC/CB two tests with similar conditions were performed.

In Figure 7.12 the average values of the stress-strain curves were compared for the dry and the wet materials. All these trends were also compared to the no column condition.

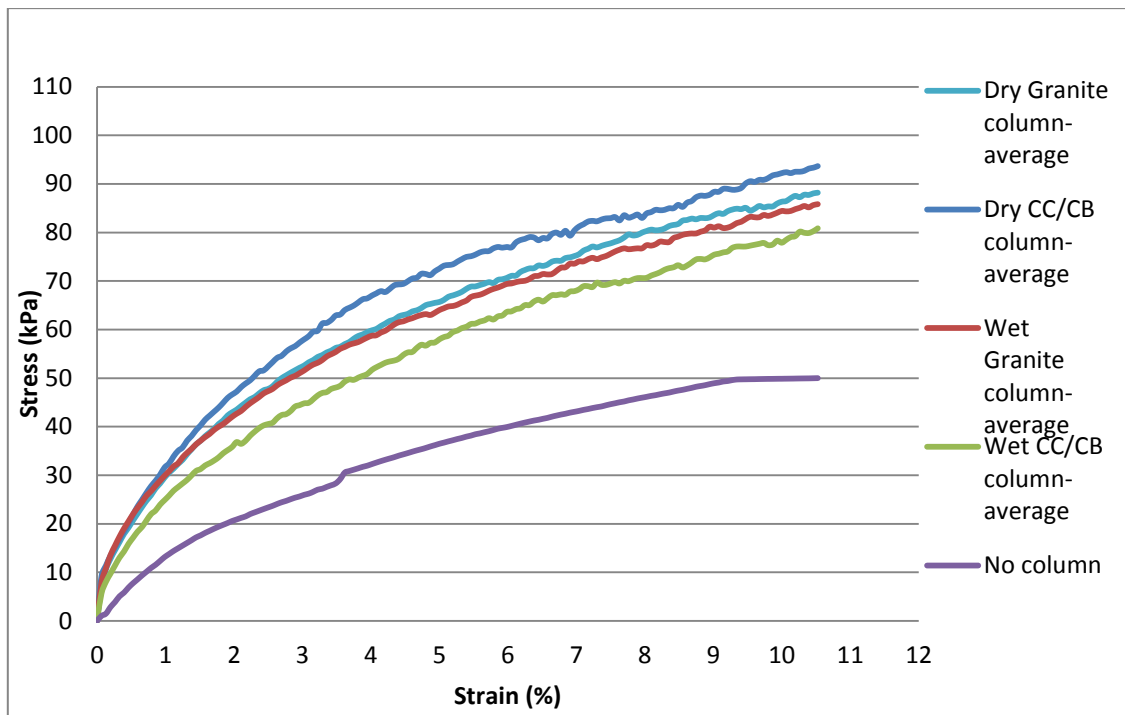


Figure 7.12: The wet and dry primary and recycled aggregate tests in the large unit cell

It was observed from the stress-strain curves that both the wet and the dry conditions of the primary and the recycled aggregates provided columns with a load carrying capacity significantly higher than the condition of the no column.

It was observed that in the dry tests the CC/CB outperformed the granite in terms of the load carrying capacity; however, the wet granite had a higher stress capacity compared to the wet CC/CB.

This test was only performed on one type of the recycled aggregates, but it was possible that the wet and dry conditions could affect certain aggregates more than the others.

In case of the CC/CB the moisture might have been absorbed by the brick and the cement in the concrete particles and affected the performance. This difference in the load carrying capacity of the wet PA and the RA was less than 10kPa across the strain values of up to 10%.

It was also observed that the wet and the dry granite showed very close values of the stress throughout the curve, whereas, the CC/CB showed a bigger gap in the wet and the dry tests up to the maximum difference of 15kPa. As mentioned before all the four tests showed significant improvement in the load carrying capacity of the ground compared to the no column condition, but the wet recycled aggregate had the lowest stress at all points.

The internal angle of shearing resistance obtained in the study by McKelvey *et al.* (2002) on the wet and dry aggregates suggested that the wet primary aggregate had a lower angle of shearing resistance compared to the dry PA.

On the other hand, the crushed concrete showed the same values in both the wet and the dry conditions (McKelvey *et al.*, 2002).

The small shear box test on the wet aggregates was not performed in this research (due to insufficient materials sources), and the results of the study by McKelvey *et al.*, (2002) could not be elaborated for the findings of this research for the wet and dry conditions. Even if the wet shear box tests were done, the nature and the structure and the source of the aggregates were different and could create unpredictable results in terms of the load carrying capacity.

Table 7.7 compares the densities of the columns constructed using the wet aggregates.

Table 7.7: Average densities of the wet and dry columns constructed

Test name	Test number	Average column density (kg/m^3)
Wet primary aggregate-average	13, 14	1836.08
Wet recycled aggregate (CC/CB)-average	11, 12	1509.52

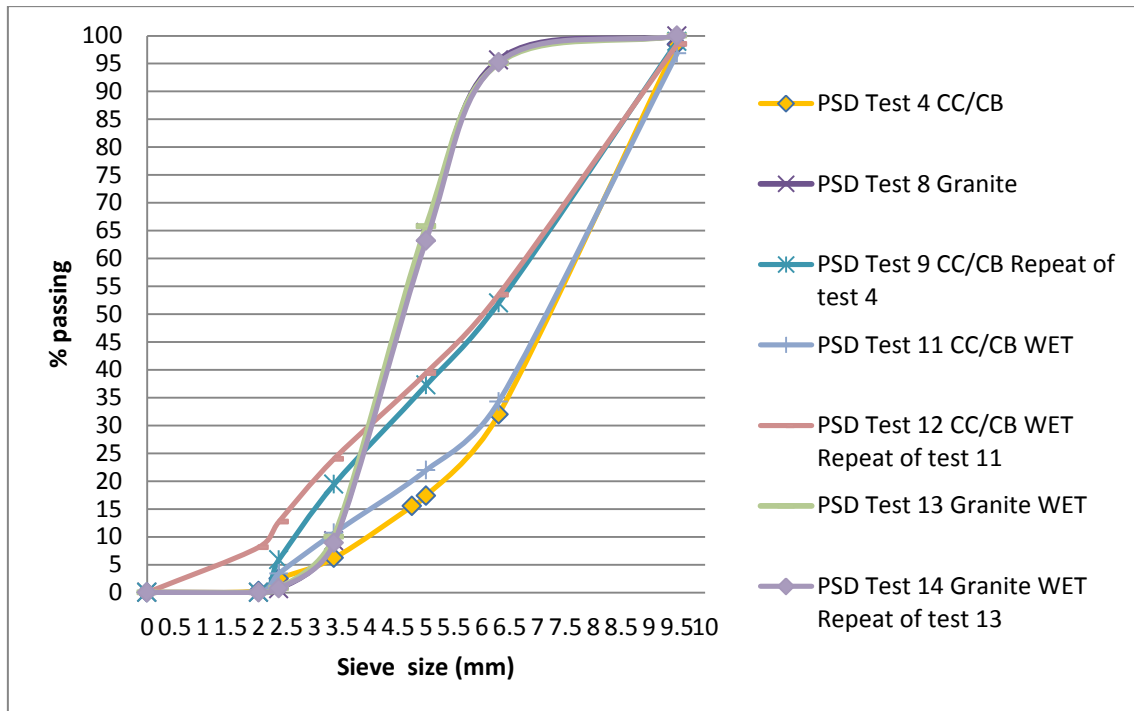


Figure 7.13: Dry PSD of the granite and the CC/CB before being used in the dry and wet tests

It was observed that the average densities of the columns of granite and the CC/CB were very different in the wet tests. The column of the wet granite showed higher load carrying capacity compared to the wet CC/CB, and it could be related to its higher column density achieved during the installation process.

In the CC/CB, despite having various trends in the PSD, the general range was well-graded compared to the granite. The results of the load carrying capacity of the wet primary and recycled aggregates seemed different from the dry tests in terms of the PSD factor. In the dry tests, the well-graded material resulted in higher load carrying capacity while in the wet tests; the PSD seemed a secondary factor compared to the possible loss of strength in the wet condition.

The addition of moisture to the aggregates can happen during the storage, the transportation or the installation of the stone columns and this factor should be considered in the short and long-term behaviour of the VSCs.

In this chapter for the LUC tests, only the PSD before the installation was presented as a controlling measure to make sure the proper size aggregates were being used in the construction of the columns. The comparison of the PSD before and after the installation and the loading of the columns was not the objective of these tests and this factor was fully studied in the SUC tests and was presented in section 8.2.

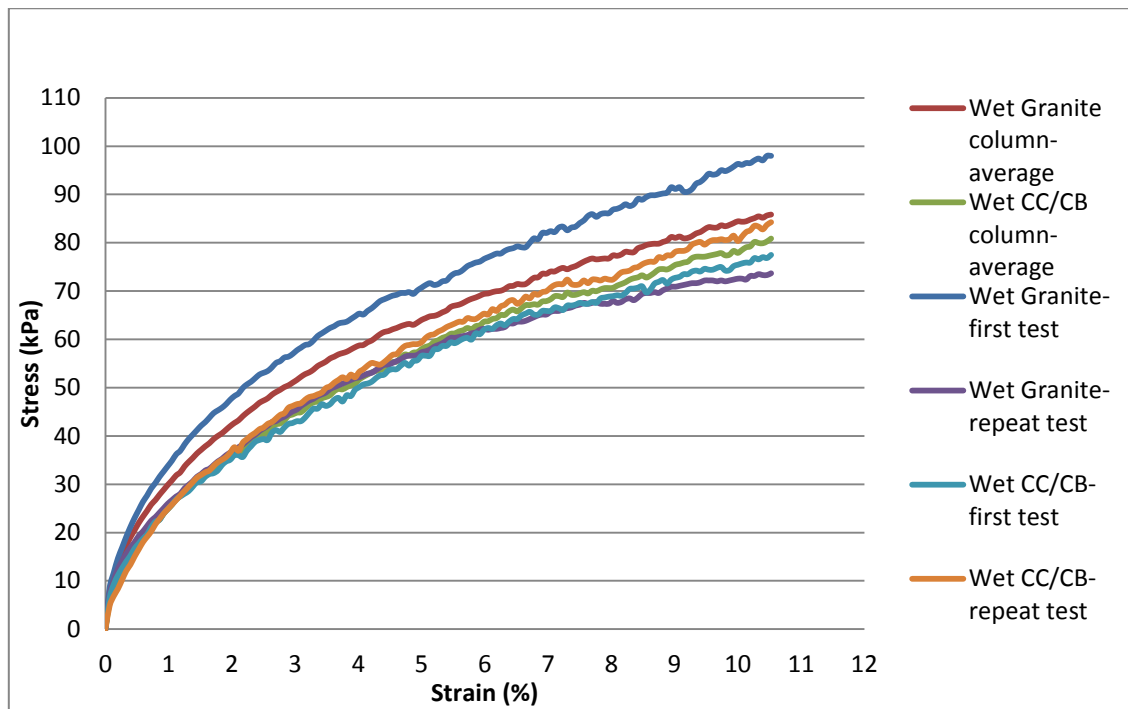


Figure 7.14: All the wet tests and the averages in the large unit cell

Figure 7.14 showed all the tests and the averages on both wet materials. It was observed that the wet granite had variable load carrying capacity in the two tests performed, as opposed to the CC/CB where both the results were very close.

The process of soaking of the material could contribute to the variable wet granite results, where the temperature and the loss of moisture could affect the soaking procedure. Also, during the installation when the wet source was used, water could transfer to the system during the charges of the aggregates which increased the water

level in the unit cell and could have led to different performance in the load carrying capacity.

7.4.5 All the materials tests including the wet and dry aggregates

Figure 7.15 is a combination of the wet and the dry results. The curves presented were the average values of the two tests performed on each material except for the IBAA (2).

It was observed that the wet materials regardless of the type of the aggregates showed lower stress capacity compared to all the dry primary and recycled materials.

The wet CC/CB provided the weakest column as opposed to the dry IBAA (2) which had the highest load carrying capacity. The difference was significant up to 35kPa less stress capacity in the wet CC/CB. It was possible that the stress in the IBAA (2) became steady while the stress was still increasing in the wet CC/CB and it would outperform the IBAA (2) at higher values of the strains.

But as the maximum strain in these tests was beyond the settlement failures, the trends obtained were more representative of the behaviour of these aggregates in the context of VSC.

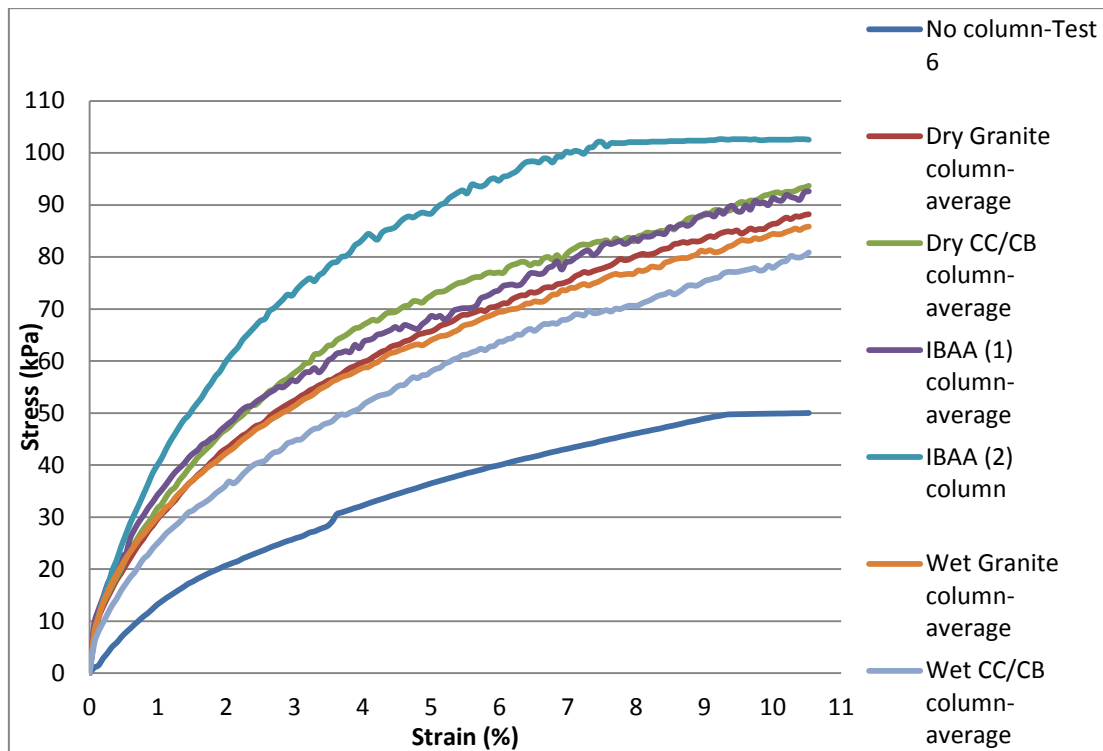


Figure 7.15: The wet and dry aggregates, the average values in the large unit cell tests

Following the IBAA (2), the IBAA (1) and the dry CC/CB showed better results compared to both the wet and dry PA. Apart from the condition (wet/dry), the PSD and the level of packing seemed to be the most important factors for the materials tested. The angle of shearing resistance and the column density were two other factors affecting the load carrying capacity of the columns. It was apparent that regardless of the type of material, the wet condition was a critical factor in the performance of the VSC in the short term.

7.4.6 Short-term versus long-term tests

Figure 7.16 shows the short and the long-term tests on the dry PA. The same granite was used for all the three tests. The PSD, the angle of shearing resistance and the shape of aggregates used in all the three tests was similar. The only variation was less than 6% difference in the density of columns formed.

As it was observed in Figure 7.16, the long-term test on the granite showed poor results compared to the short-term tests on the same material. Although the density of the column formed was slightly higher in the long-term test, the stress-strain behaviour showed a lower bearing capacity.

It is also observed that even the long-term test in which the column was loaded three months after the installation provided improvement for the host ground in terms of the load carrying capacity compared to the no column test. This improvement was up to values of 46% across the strains and the long-term results were close to the short-term columns of granite with up to 23% lower values of the stress throughout the curve.

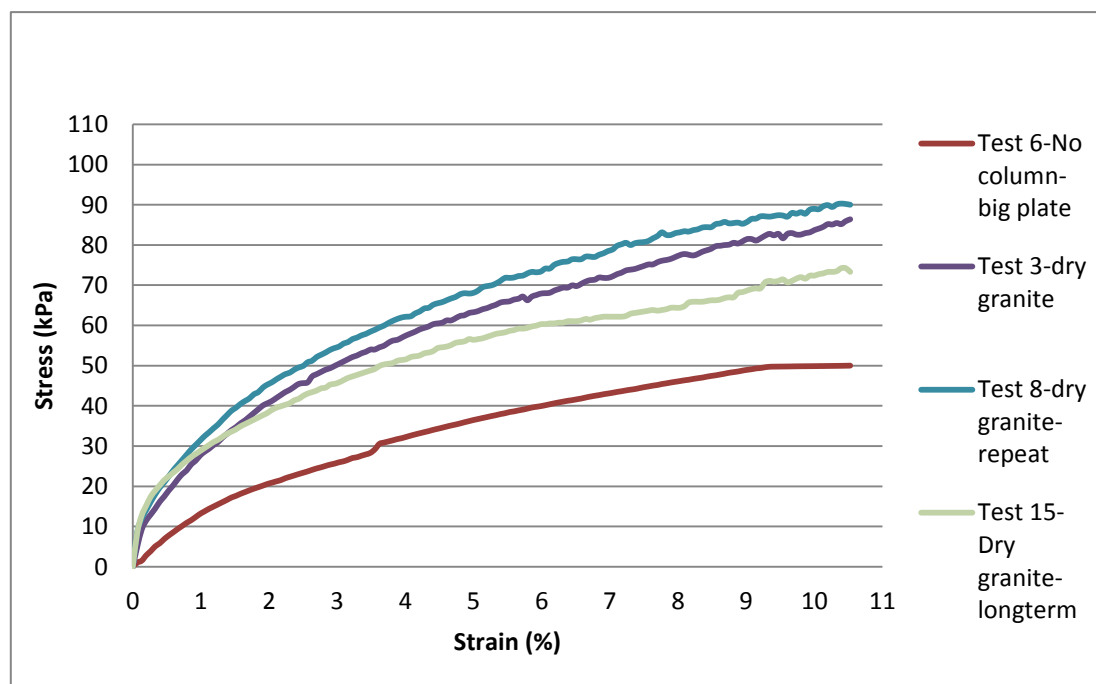


Figure 7.16: The short and the long-term tests on the dry granite

In order to understand the reason behind the variation in the short and the long-term tests, the results of the long-term test were compared to both the wet and dry short-term tests on the granite.

Figure 7.17 showed that the results of the stress-strain on the long-term test on the dry aggregates were close to the values obtained for the wet tests. Both the CC/CB and the granite under the wet condition had similar values across the curve to the dry long-term test on the PA.

It was concluded that regardless of the densities of columns, leaving the material in the host ground before the loading could change their condition from the dry to wet where water could be absorbed by the aggregates from the surrounding ground.

As a result, the performance of the long-term test was more similar to the short-term tests on the wet aggregates, and confirmed that the condition could affect the performance of the materials in the column far more than the density and the PSD.

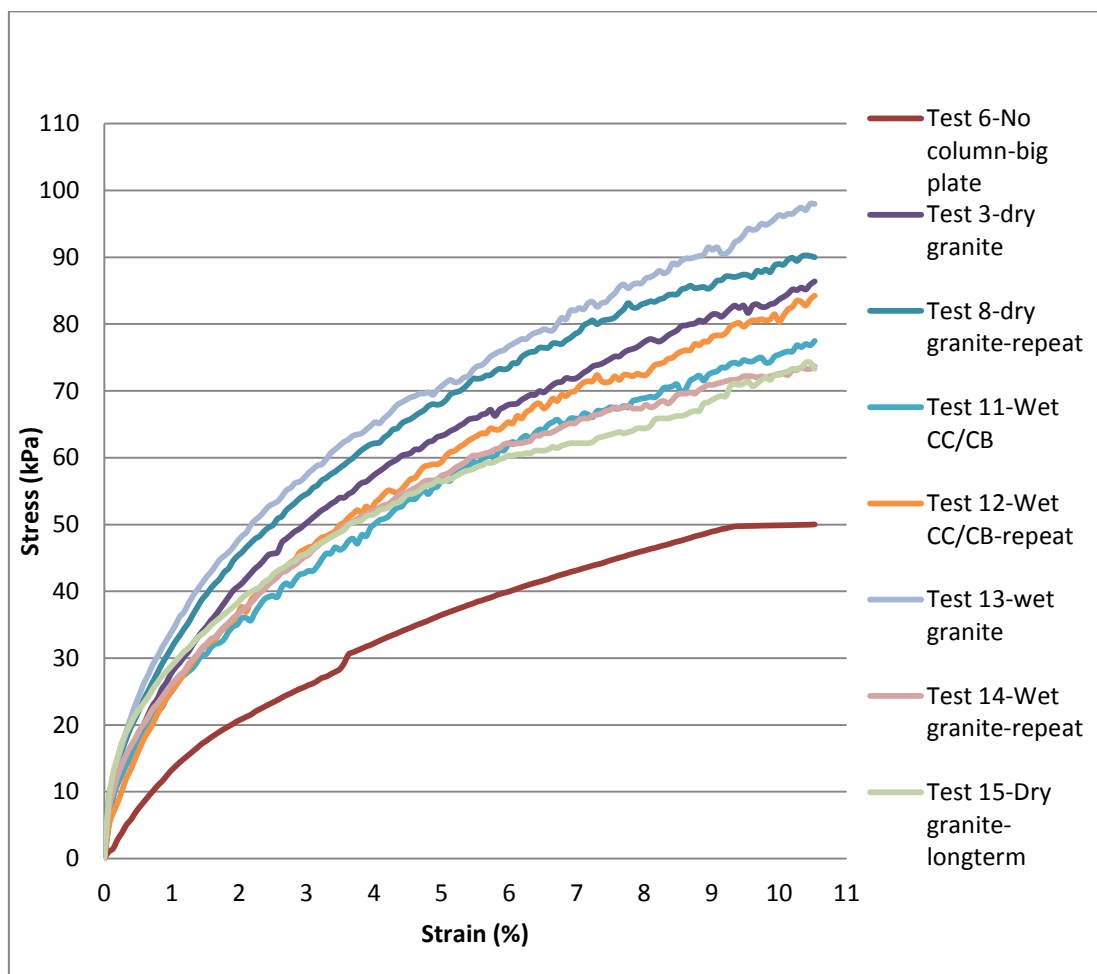


Figure 7.17: Comparison of the wet short-term with the dry long-term tests

7.4.7 Sand column

The pilot test (test 1) in the LUC was on a column of Leighton Buzzard sand constructed in a host ground consisting of both layers of sand and clay. This test was only performed to check the process of installation and loading.

As the host ground, the material used and the axial plate were different from all the other LUC tests, the results could not be compared.

The load-deformation results of this test were only presented in Appendix 5 (refer to CD).

7.5 Errors in the LUC tests

The errors were estimated for the LUC tests based on the repeat results. The results were available for two tests performed on the dry granite in the short-term, two tests on the wet granite, two tests on the dry CC/CB, two tests on the wet CC/CB and two tests on the IBAA (1).

The errors were not presented for the IBAA (2) and the long-term tests as the tests could not be repeated. All these tests mentioned were performed under the foundation type plate.

The error were estimated for the mean values of the results based on the standard deviation. The detailed calculations were presented in Appendix 5 (refer to CD).

Figures 7.18 to 7.22 showed the errors for the tests and the repeats.

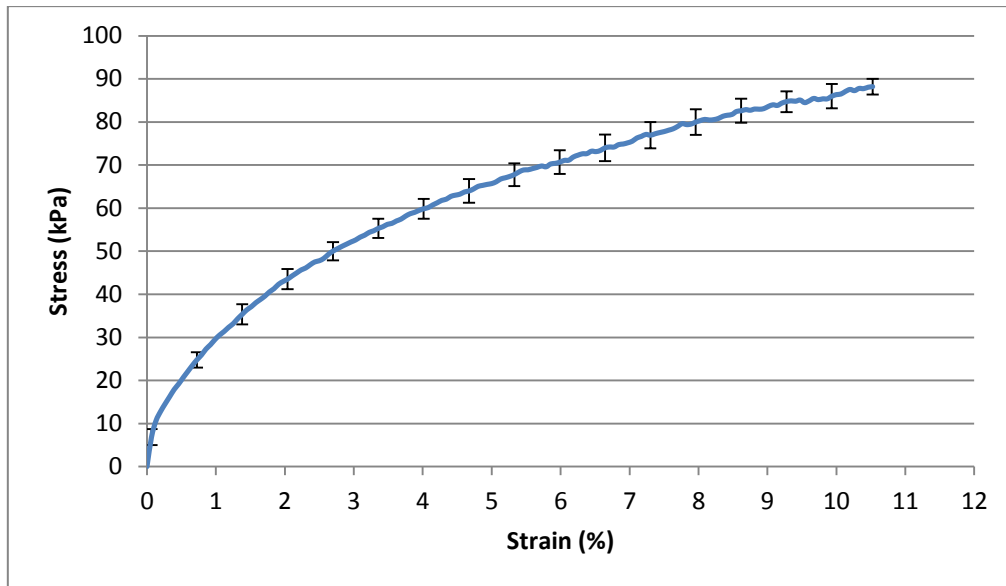


Figure 7.18: The errors for the dry granite tests (tests 3 and 8)

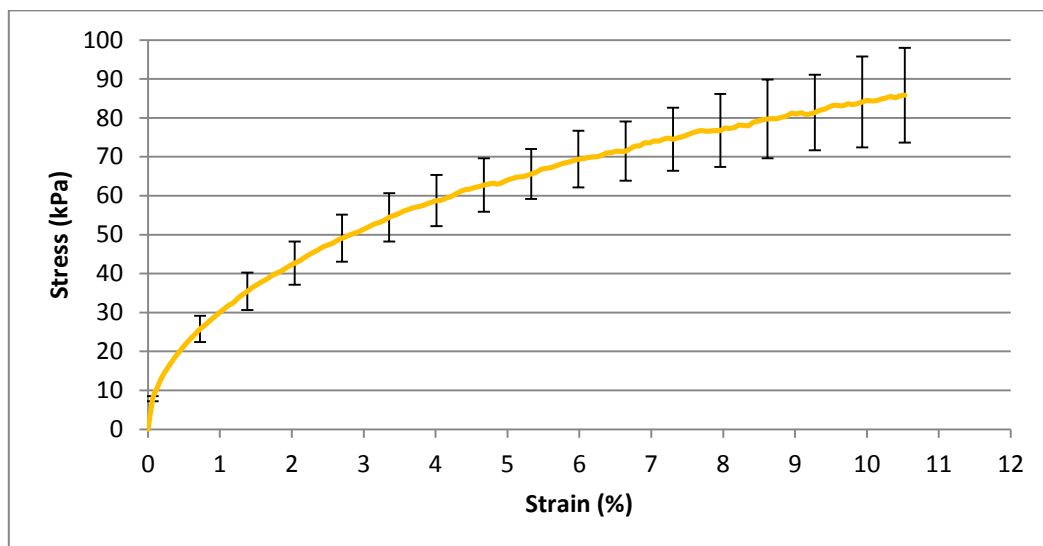


Figure 7.19: The errors for the wet granite tests (tests 13 and 14)

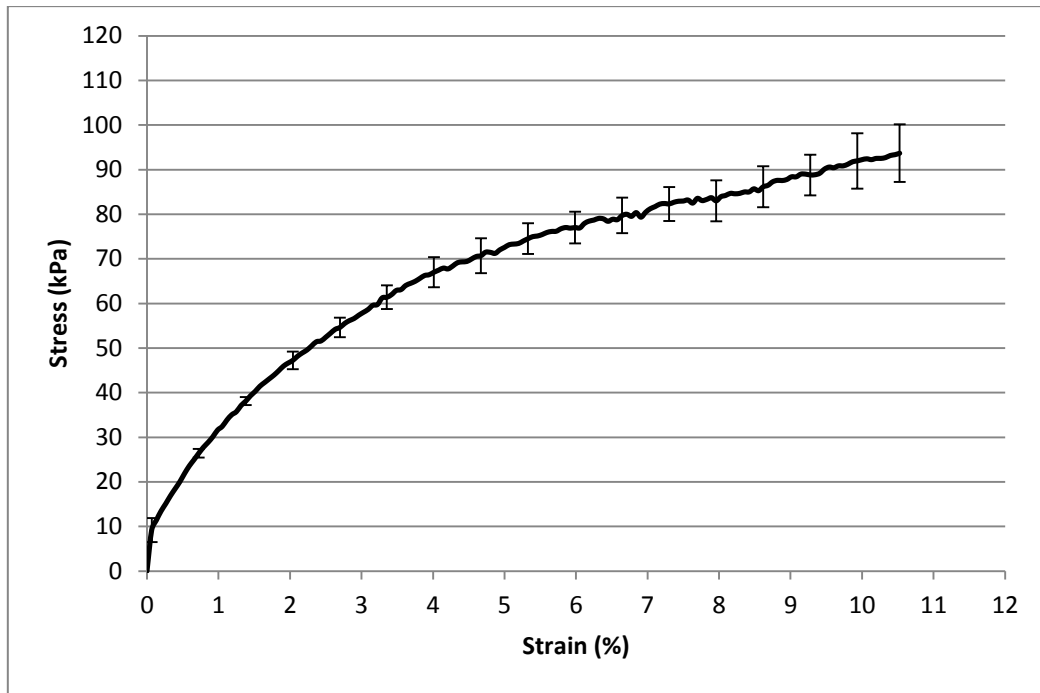


Figure 7.20: The errors for the dry CC/CB tests (tests 4 and 9)

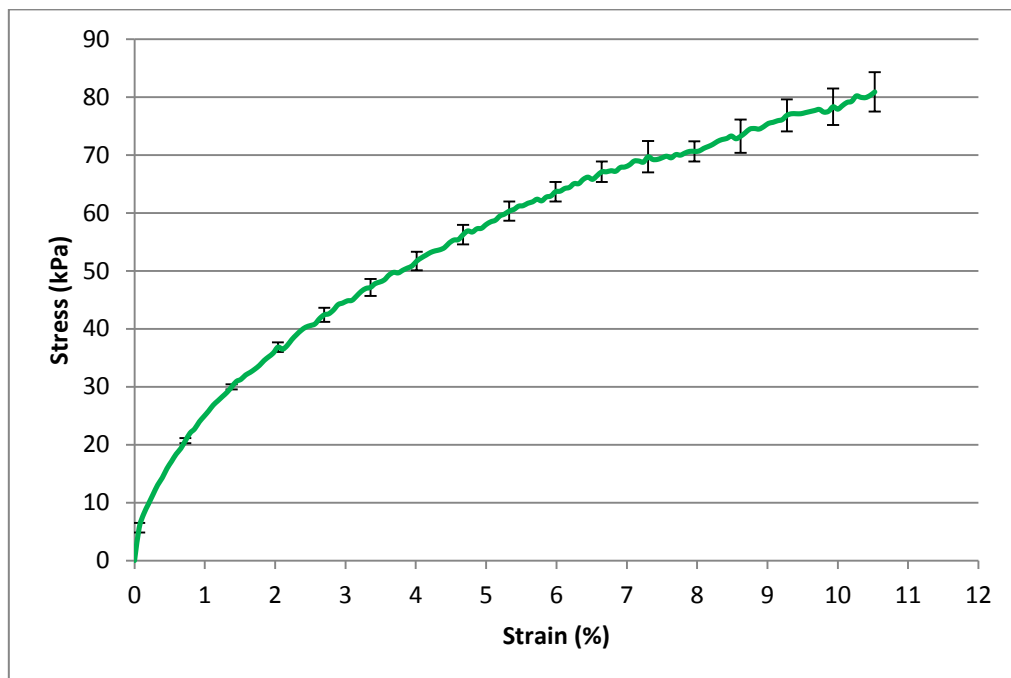


Figure 7.21: The errors for the wet CC/CB tests (tests 11 and 12)

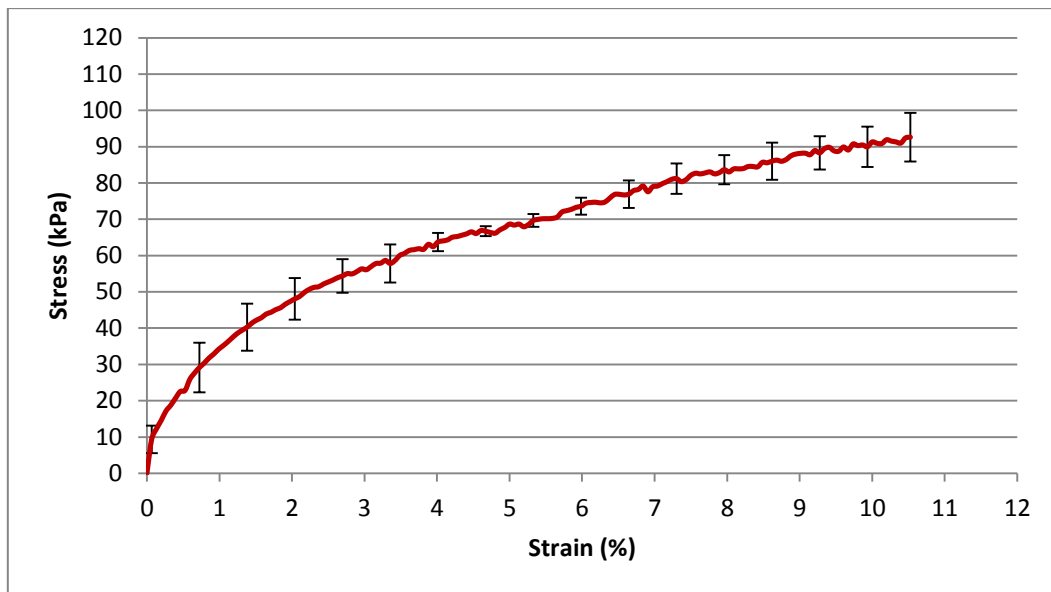


Figure 7.22: The errors for the IBAA (1) tests (tests 5 and 10)

It was observed in Figures 7.18 to 7.22 that the errors had various patterns for the different column aggregates.

Tests on the wet granite showed higher values of the errors compared to the dry granite tests. The errors were due to the soaking procedure and its effects on the properties and the strength of the aggregates. This effect was addressed in the research by Steele (2004), where the soaked tests were recommended on the various aggregates to assess their properties.

The errors in the dry and the wet CC/CB were discussed in section 7.4.3 and the variations in the PSD of the materials used in tests created variations in the stress-strain behaviour of the tests under the same loading condition.

It was observed that the errors increased gradually with the increase in the strains in all the tests except for the IBAA (1). Figure 7.22 showed that the errors in the IBAA (1) tests were minimal after the strains of around 4% and increased again after 6.5% strain.

If the failure was considered at the strains of 1.5 or 4.5%, the errors in the IBAA (1) were more than the other materials tested before the failure. This could be related to the unexpected behaviour of this material under the loads due to its structure and nature. The glass pieces in the IBAA (1) broke after a certain load in one test, or started breaking since the beginning of the loading in another test.

Table 7.8 compares the maximum errors of the various materials at the failure strains of 1.5 and 4.5%.

Table 7.8: The errors in the dry and wet tests and repeats

Failure strain (%)	Maximum standard errors in columns of dry granite (\pm kPa)	Maximum standard errors in columns of dry CC/CB (\pm kPa)	Maximum standard errors in columns of IBAA (1) (\pm kPa)	Maximum standard errors in columns of wet CC/CB (\pm kPa)	Maximum standard errors in columns of wet granite (\pm kPa)
1.5	2.5	1	7	0.6	5
4.5	2.8	3.5	2	1.5	7

The errors obtained were below 10% and were negligible. The exception was the IBAA (1) where the nature of the material created unexpected trends in the results. Also, the condition of aggregates caused unpredictable results in the tests and repeats due to the effects on the properties of the aggregates which affected the performance of VSC in the short-term.

7.6 Settlement estimations

7.6.1 Priebe's method

The settlement estimations were performed on the LUC tests for both the PA and the RAs. Priebe's method of settlement estimation is commonly used in practice as it is easy and straight forward (Douglas and Schaefer, 2012). The simplifying assumptions were considered in the initial method which was modified in later years (Priebe, 2005). One of the initial assumptions was related to the compressibility of the column material

which was not considered. The curves used for the settlement improvement factor were modified and the following procedure was used to estimate the settlement of the primary and the recycled aggregates in this research (refer to Appendix 5 on the CD for the details):

- As in the LUC, the area of the loading (foundation type plate) was small compared to the depth of the treated area (height of 760mm); the three dimensional settlement estimation was used.
- Firstly, the one dimensional settlement improvement was calculated and the settlement ratio factor was used to modify the results to the three dimensional estimations.
- As the tests were rapidly constructed and loaded, the consolidation and the long-term settlements were not considered in this research. Only the immediate settlement values were used.
- The area replacement ratio and the angles of shearing resistance of the columns constructed were used to estimate the improvement factor from Priebe's method.
- The angle of shearing resistance of the granite was 47 degrees. The angle of shearing resistance of 45 degrees was considered for the aggregate which is a typical value used in the design (Serridge, 2006) and also to consider the possible errors regarding the use of the small shear box test to obtain the angle of shearing resistance instead of the large shear box apparatus in this research. The angle of shearing resistance was used to estimate the improvement factor for the granite. On the other hand, all the RAs had the angles of shearing resistance close to 40 degrees and one value of improvement factor was considered for all the RAs.

- Using the area replacement ratio, the angles of shearing resistance and the compressibility of the columns, the improvement factor of the settlement was applied to test 6 in the LUC where the no column was tested.
- Using the two factors for the PA and the RAs, the predicted settlement values based on Priebe's method were obtained for the PA and RAs.

On the other hand, all the PA and RAs were tested in the LUC container under the foundation type loading plate. The actual settlements or strains obtained in the LUC tests were compared to the predicted values based on Priebe's method. The details of the calculations were presented in Appendix 5 (refer to CD).

7.6.2 The settlement comparisons

Figure 7.23 shows the comparison of the strains estimated according to Priebe's method and the actual measurements for the dry column of the granite. The LUC test on the dry granite was repeated and the actual values presented were the average of the two tests performed.

Priebe's improvement factor was applied to the settlement values of the untreated soil (test 6). Figure 7.23 shows that the actual values of the stress-strain were very different from the strain values predicted.

At any specific stress value, the strains could be compared for the actual and the estimated curves. It was apparent that at any stress point in the graph, the values of the strain for the estimations were much higher than the actual strain values. The high strain values meant higher settlement prediction based on Priebe's method; which made the results of the estimation too conservative for the LUC tests. The actual values of the settlement in the LUC tests on the dry granite were much lower than the prediction by Priebe's method.

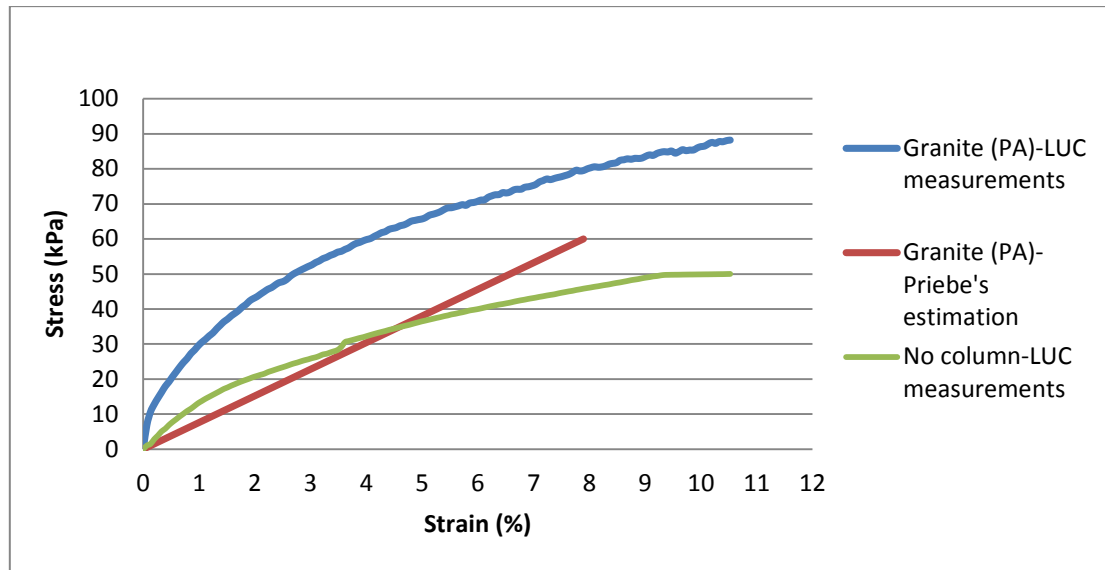


Figure 7.23: Stress-strain estimation and measured for the LUC tests on the dry granite

Figure 7.24 showed the same comparison for all of the primary and the recycled aggregates. The same angle of shearing resistance was used for all the RAs resulting in one trend of settlement prediction based on Priebe's method.

The values of the predictions for the PA and the RAs were very similar for the materials tested in this study. The maximum settlement values according to Priebe's method were 6.4 and 7.2mm for the PA and the RAs, respectively. It was expected to have higher values of the settlements for the RAs compared to the PA.

Similar to the PA trend, Priebe's settlement prediction method was too conservative for the RAs. The biggest difference existed for the IBAA (2) and the predicted values, where for each specific stress, the strains were much lower in the actual measurements for the IBAA (2) compared to the predictions.

Findings of this comparison for the large scale tests versus Priebe's predictions agreed with the findings by Douglass and Schaefer (2012) on 250 cases of settlement

validations. In the study the actual measurements were compared to the predictions by Priebe's method which was frequently used in practice and the predictions were 89% conservative.

Similar results were obtained in this research using both the primary and the recycled aggregates. However, direct comparison was not possible due to various factors such as single versus group of columns, the host ground properties, the various aggregates and the assumptions used in Priebe's method of estimation. Other important factors affecting the results included the area replacement ratio and the compressibility of the ground and the column materials.

The analysis was not performed on the wet materials, as the values of the angle of shearing resistance in the wet condition were unknown and this value is the most important factor in Priebe's settlement prediction method (Priebe, 1995).

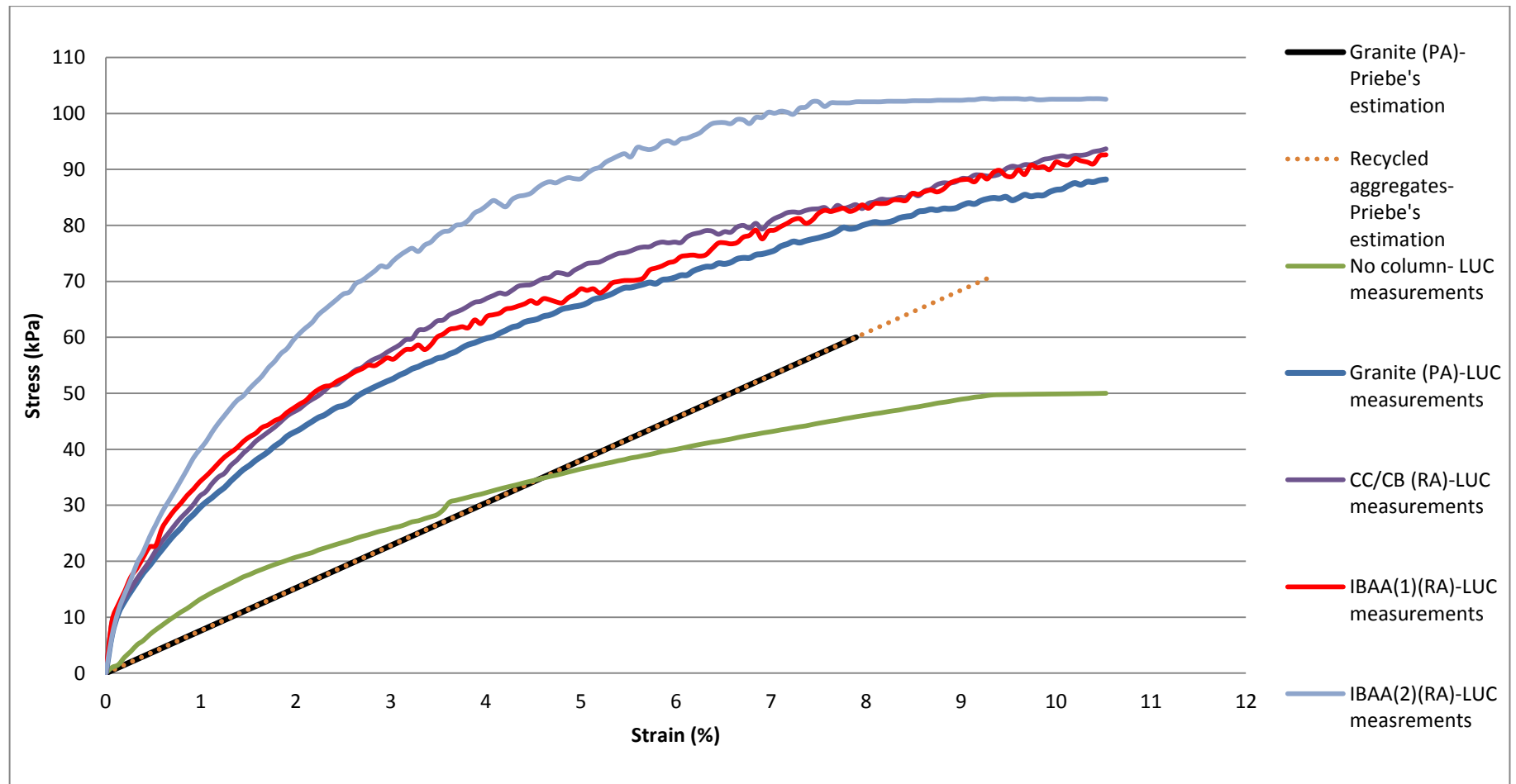


Figure 7.24: Stress-strain estimation and measured for the LUC tests on the dry primary and recycled aggregates

7.7 Water level changes

In the LUC tests, the model piezometers at depths and porous stone at the base of the cell were used to monitor the water level changes during the various stages of the tests. Monitoring started after the clay was mixed and compacted into the cell and continued during the installation and the loading of the column and also 48 hours after the column was unloaded. These values were measured at 7 points: at the base of the cell (porous stone) and at the various depths and radii from the column centre (Piezometers). For the location of the piezometers and the numbers refer to Figure 6.1 and Table 6.5.

The values recorded were water level and not the excess pore water pressures as the clay used was only compacted in layers and not consolidated. The whole process of the preparation, the installation and the loading was a fast process and did not provide the time for the layers of the clay to consolidate in the large cell.

As the values of the water level measured in these tests could not be compared to previous research on the excess pore water pressure changes during the installation and loading (Castro and Sagasetta, 2012); only the behaviour of the host ground was interpreted at various stages of the installation and loading using the data obtained in this research.

7.7.1 Stages of the water level measurements

The measurements of the water level changes were not taken for all the LUC tests.

Table 7.9 shows the stages at which the water level changes were monitored for the LUC tests. The details of the measurements were attached in Appendix 5 (refer to CD).

Table 7.9: Stages of the measurements of the water levels for the LUC tests using the 6 piezometers and the porous stone

Test	Stage of water level measurement			
	Before installation	During installation	During loading	After unloading
Pilot test	-	-	-	-
No column-small plate	-	-	*	-
Granite	-	-	*	-
CC/CB	-	-	*	*
IBAA (1)	-	-	*	*
No column-big plate	-	-	*	*
IBAA (2)	-	-	*	*
Granite-repeat	-	-	*	*
CC/CB-repeat	*	*	*	*
IBAA (1)-repeat	-	-	*	*
Wet CC/CB	*	*	*	*
Wet CC/CB-repeat	*	*	*	*
Wet granite	*	*	*	*
Wet granite-repeat	*	*	*	*
Long-term granite	*	*	*	*

NB: *: measurements taken; -: measurements not taken

As observed in Table 7.9, the water level measurements were performed at four stages:

- (1) Before the installation: After the clay was mixed and compacted in the LUC, water levels were measured before the installation started from the porous stone and the piezometers.
- (2) During the installation: As soon as the hole was formed and charges of the aggregates were poured into the hole (refer to section 6.7.2), the water levels were monitored at each stage of the aggregate pouring and compaction. This stage took around 15 minutes until the column formed reached the surface of the host ground. The values were taken from the porous stone at base and the six piezometers.
- (3) During the loading: This stage was the most important part of data collection for the water levels. Loading of the column took around 67 minutes and at every 0.5mm of deformation, the load and seven values of the water levels were recorded until the maximum travel of 80mm was achieved. In total 160 values

were recorded during the loading from each piezometer and these values were used to interpret the water dissipating through the column (from the porous stone) and the changes in the surrounding soil during the loading (from the piezometers).

- (4) After the unloading: the values of the water levels from the piezometers and the porous stone were recorded for 48 hours after unloading to monitor the water dissipation through the column and the possible changes in the surrounding soil.

7.7.2 Comparisons of the water levels

Table 7.10 summarizes the general trends observed. As enormous amount of data was available for the comparison, the examples representative of the findings were discussed instead. All the water level measurements and their changes were attached in Appendix 5 (refer to CD).

Table 7.10: Summary of the monitoring of the water levels in the large unit cell tests

Stage	Porous stone (base)	Bottom piezometers	Middle piezometers	Top piezometers	Examples
Before installation	At this stage increase in the water level was observed from the porous stone. After 2 days, the values started to reduce and increased again with the start of installation	Both the piezometers at this level regardless of their distance to the centre of the column showed slight increase and then decrease in the water levels	Both piezometers, regardless of their distance from the centre of the column, showed very slight increase in the water levels and then dropped very quickly within a few hours	Both the piezometers, regardless of their distance from the centre of the column, showed very slight increase in the water levels and then dropped very quickly within a few hours	The water level changes from the porous stone from tests 9 and 13 were presented and compared before the installation. The base of the column showed more variation compared to the other piezometers at this stage
During installation	Increase in the water level was significant since installation started, the values increased as the column installation proceeded and reached the surface	Fluctuations were observed in both the piezometers at this level, regardless of their distance from the column, as the installation proceeded to higher levels, fluctuations at the base reduced	Fluctuations were observed in both of the piezometers at this level, regardless of their distance from the column	Fluctuations were observed in both of the piezometers at this level, regardless of their distance from the column	Tests 9 and 11 were compared during the installation to represent the wet and dry RA (CC/CB) being used, all the values of the piezometers and the porous stone were presented to compare the fluctuations at various stages of the aggregate compaction
During loading	The most significant changes were observed at the base during this stage, where the values of the water levels increased since the loading started; the values represented the water transfer through the column during the loading	Water levels increased in both of the piezometers, there was no particular trend comparing the relation between the increase in water level for the two distances from the centre of the column, in many cases the piezometers far from the centre showed similar increase in the water levels to the closer piezometer	Water levels increased in both of the piezometers, there was no particular trend comparing the relation between the increase in the water level for the two distances from the centre of the column; in many cases the piezometers far from the centre showed similar increase in the water levels to the closer	Water levels increased in both of the piezometers, but reduced quickly or became steady even during the loading. Although these piezometers were located close to the surface, it seemed most of the water level changes were near the base of the column. At this level, the piezometer closer to the centre of the column	Three points of the base and the middle were considered the most sensitive areas due to the drainage and bulging of the column, respectively. Tests on the 4 types of aggregates in this research were compared at the base, also, at the middle piezometers, close and far from the centre of the column

			piezometer	showed more variations and increase in the water level quantities	
After unloading	During the 48 hours of monitoring after the unloading, the values at the base kept increasing and after 24 hours started to decrease gradually; more significant changes were observed at the base compared to the other piezometers	Slight increase in the water levels was observed during the 48 hours of monitoring	Water levels decreased quickly since the loading stopped in both of the piezometers regardless of their distance from the centre of column	Water levels decreased quickly since the loading stopped in both of the piezometers regardless of their distance from the centre of column	-

The location of the piezometers and the porous stone and their distances from the centre of the column was shown schematically in Figure 6.1 and Table 6.5. The names used in this figure can assist in the results interpretations based on the graphs in Figures 7.25 to 7.31.

7.7.3 Comparison of the water level changes before the installation

In order to compare the changes in the host ground after the soil was compacted and before the installation started; the water levels were monitored at various depths.

The most significant changes were observed at the base where the water could be transferred into the porous stone for all the tests.

The results of the water levels from two of the LUC tests at the base (number 1) were presented in Figure 7.25. The results were available for several tests, but analysis was not related to the type of column constructed as at this stage all the tests were similar. These graphs were representative of the trends obtained in most of the LUC tests.

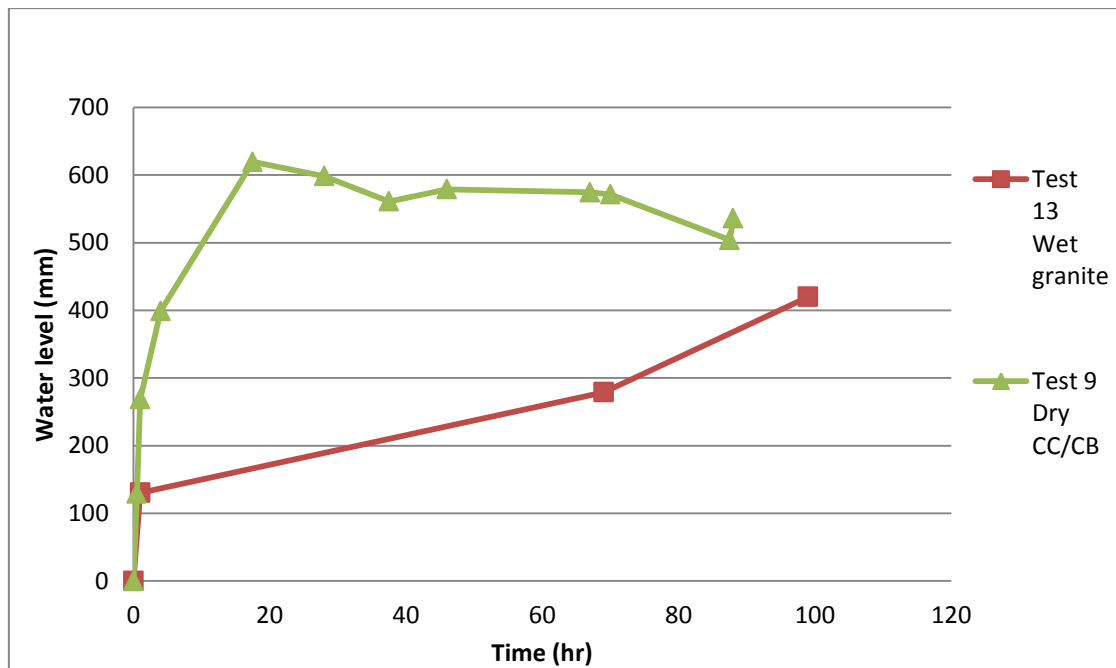


Figure 7.25: The water levels of the clay at base for test 9 (Dry CC/CB) and test 13 (Wet Granite) before the columns were installed

It was observed that in both of the tests, the water level at the base (number 1) increased rapidly with the first few hours. These water level measurements were taken from the host ground over the porous stone. At this stage for both of the tests (9 and 13) the columns were not constructed yet, therefore, the graphs in Figure 7.25 were not related to the type of the aggregates or their condition (wet or dry) and the aggregate names and conditions were only used to distinguish the tests' names.

In the tests, the increase in the water level was continued until the next stage which was the installation of the column; however, in other tests such as test 9, the water levels increased rapidly within 24 hours and slight drops were observed in the trend.

At the next stage (the installation of the columns) sudden increase in the water levels started after approximately 3 to 4 days. The monitoring did not provide the information on the behaviour of the columns at this stage as these measurements were from the

compacted and prepared Kaolin over the porous stone before the column installation phase.

The results showed that in the host ground the water level was changing near the base (number 1) where the porous stone was provided and the process of the drainage and consolidation started since the host material was prepared through the porous stone at the base where the water in the clay could dissipate. However, the next stage of the installation commenced very quickly and before the consolidation took place, the columns were installed and loaded.

7.7.4 Comparison of the water level changes during the installation

Figure 7.26 shows the water level changes during the installation for the porous stone (number 1) and the piezometers used in test 11. This test was performed on the wet CC/CB and the installation was quickly done at stages of pouring the aggregate and vibrations using the concrete poker. The installation started from the base until the column reached the surface of the host ground.

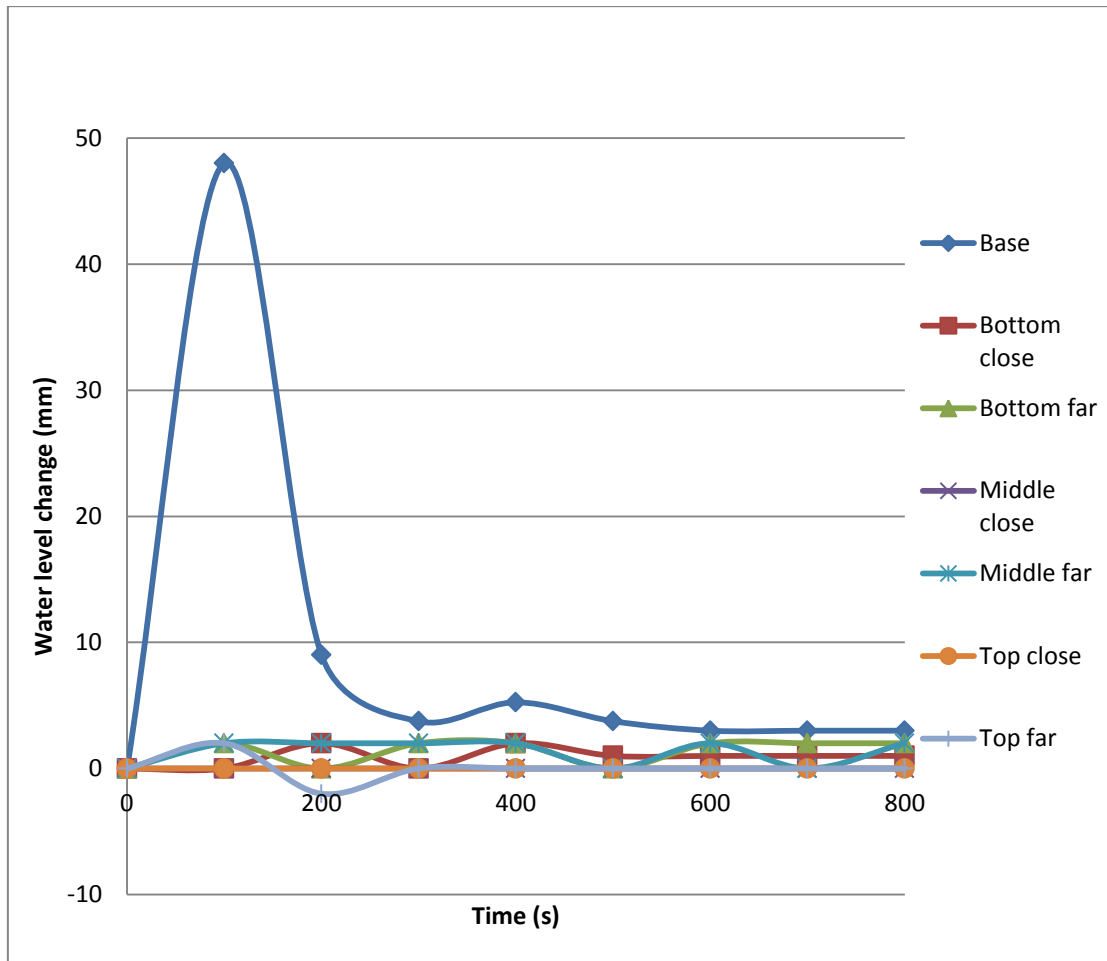


Figure 7.26: The water level changes during the installation of the wet CC/CB

Based on Figure 7.26 it was observed that most changes were recorded near the base (number 1) and from the porous stone compared to the piezometers which were placed in the clay. The porous stone could absorb more water at the base of the columns as both the column material and the porous stone had high permeability as opposed to the clay.

All the piezometers showed water level changes of ± 5 mm, whereas, the base showed changes of up to 50mm in the first two minutes of installation. The reason for the significant change at the base could be related to the early stage of installation where the

column material was poured and vibrated to form the column near the base. Existing water in the column material could be easily transferred to the porous stone at the base.

Fluctuations were observed in most of the readings throughout the entire process of installation. In the beginning due to the aggregate compaction near the base, the base (number 1) and the two bottom piezometers (numbers 2 and 3) showed higher levels of fluctuations. As the column construction proceeded, changes in higher levels of the host ground caused the middle piezometers (numbers 4 and 5) to show more variations in the water levels. The fluctuations disappeared and the trends became steady as the column reached the surface.

It was also observed that in most of the tests, the top piezometers showed lowest values of fluctuations in the water levels throughout the installation process. Even the initial fluctuations in the top far piezometer (number 7) were reduced and zeroed very quickly. The initial vibrations could be due to the general vibrations induced in the system due to the compaction of the aggregates.

Figure 7.27 showed the same analysis on the dry column of CC/CB. It was observed that changes in the water levels in the piezometers were between $\pm 5\text{mm}$. The difference in test 9 and 11 was in the water level changes measured at the base (number 1).

In test 9 on the dry aggregates, the water level changes at the base were a lot smaller compared to the wet aggregate test installation. The reason could be contributed to the condition of the aggregates used. When the wet aggregates were used in installation, the water used for soaking of the aggregates could be transferred into the base throughout the entire process of installation. In the dry aggregate installation, the changes in the

first two minutes of installation were one fifth of the changes during the wet aggregate installation process.

The fluctuations in the various levels of the piezometers were more apparent in Figure 7.27. As the time increased and the installation proceeded, initially the water level changes were observed near the base (number 1) and at the bottom piezometers (numbers 2 and 3); the fluctuations gradually moved to the middle piezometers (numbers 4 and 5) and finally reached the level of the top piezometers (numbers 6 and 7) near the end of the installation process. The changes in the top piezometers (numbers 6 and 7) were much smaller than the bottom and middle piezometers (numbers 2, 3, 4 and 5).

After the base (number 1) with the highest values of the water level changes, the bottom far piezometer (number 3) showed more changes compared to the other piezometers at various levels. It also showed that the vertical changes at the various levels were more significant compared to the radial changes in the water levels during the installation. The piezometers located closer to the centre of the column (numbers 2,4 and 6) did not show more change in the water levels compared to the ones installed further away (numbers 3,5 and 7).

Balaam and Booker (1981) studied radial and vertical changes in the excess pore water pressures in the stone column and stated that the radial dissipation is more than the vertical one in the stone columns. The comparison could not be used with this research as the water levels measured were different from the excess pore water pressure measurements in the saturated soil and also, the location of the piezometers was in the surrounding clay and not in the column.

As opposed to the study by Balaam and Booker (1981), Weber (2004) discussed loss of radial pressure in the unit cell due to the smearing of the surrounding soil and the aggregates. It seemed that this research confirmed that the water level changes were more significant in the vertical direction compared to the radial changes. The trends observed also showed the level of the ground in which more stress changes were observed at each level of column installation.

Castro and Sagaseta (2012) measured the values of the excess pore water pressure during the installation of VSCs in the field. Column groups were constructed and the peak values of the excess pore water pressures were obtained when the vibro-float reached the level of each piezometer. It was also concluded that the vibrational forces were transferred to the system during the installation of the columns. The results of the peak excess pore water pressures were analyzed based on the analytical methods and it was observed that the installation of the neighboring columns affects the results of the excess pore water pressure during the installation and measurements were different from the analytical results after the installation of column finished and when the neighboring column construction started.

Also, similar to this research the peak of the excess pore water pressure was obtained at larger depths. On the other hand, the excess pore water pressure dissipation was very fast in the radial direction (Castro and Sagaseta, 2012).

The results of the water level changes in this research showed that the vertical direction through the column showed more water dissipation compared to the surrounding soil. However, in this research the soil was only partially saturated and the results could not be directly compared to the excess pore water pressure measurements in other published

work. The results could only be used as guidelines on how the water level was expected to behave and also to interpret how the installation process affected the unit cell.

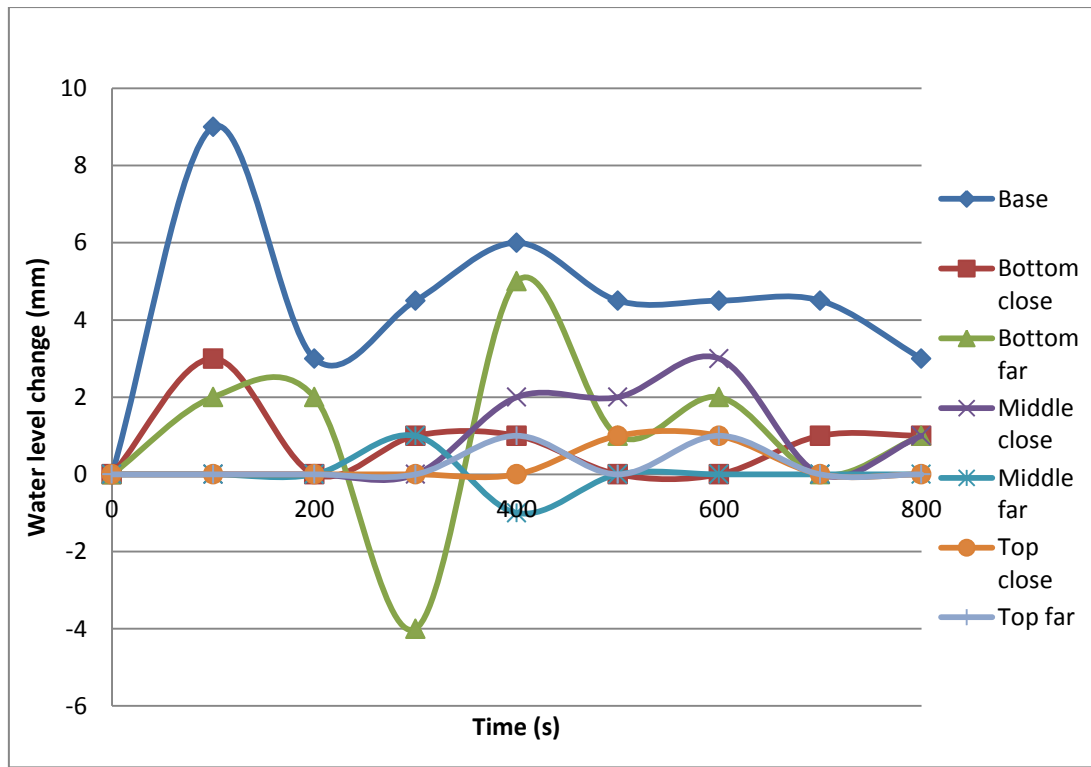


Figure 7.27: The water level changes during installation of the column of dry CC/CB

7.7.5 Comparison of the water level changes during the loading

Figures 7.28 to 7.30 show the water level changes during loading. Test 7 (IBAA (2)), test 8 (dry granite), test 9 (dry CC/CB) and test 10 (IBAA (1)) were used to demonstrate the water level changes at two levels of the base and the middle where the column bulging happens during the loading (refer to Table 6.1 and Figure 6.1).

A total of 160 values of the water levels were recorded during loading for each of the piezometers at each test. The data was analyzed and the four materials used in this research were compared.

Figure 7.28 shows the water level changes from the base (number 1) at the porous stone for these four materials; the stress-time graph for the CC/CB was also shown in this Figure. It was observed that the CC/CB had more fluctuations during the loading compared to the other materials tested and that is the reason the stress changes with time were shown to highlight the possibility of the stress and its fluctuations affecting the fluctuations of the water level changes at the base for the CC/CB. However, the main reason for these fluctuations was associated with the nature and the porosity of the CC/CB which affected its water absorption from the surrounding soil and ultimately more fluctuations as there was more water transferred through the column to the base.

The IBAA's and the granite showed similar range of variations between 4 and -2 mm. It seemed at higher stresses towards the end of the test, the fluctuations and the water level changes were more intense compared to the beginning under lower stress values.

The CC/CB showed water level changes up to more than 10 times the other materials. The results recorded at the base showed the water being transferred into the column during the loading and it seemed that the column of CC/CB provided better drainage during the loading compared to the other materials. The CC/CB could absorb more water from the surrounding clay during the loading of the column due to this material's nature. The water absorbed could show more fluctuations during the loading in the water level changes recorded.

For the CC/CB there were certain points where significant water level changes were observed at approximately 2, 25, 44 and 62 minutes after the loading started.

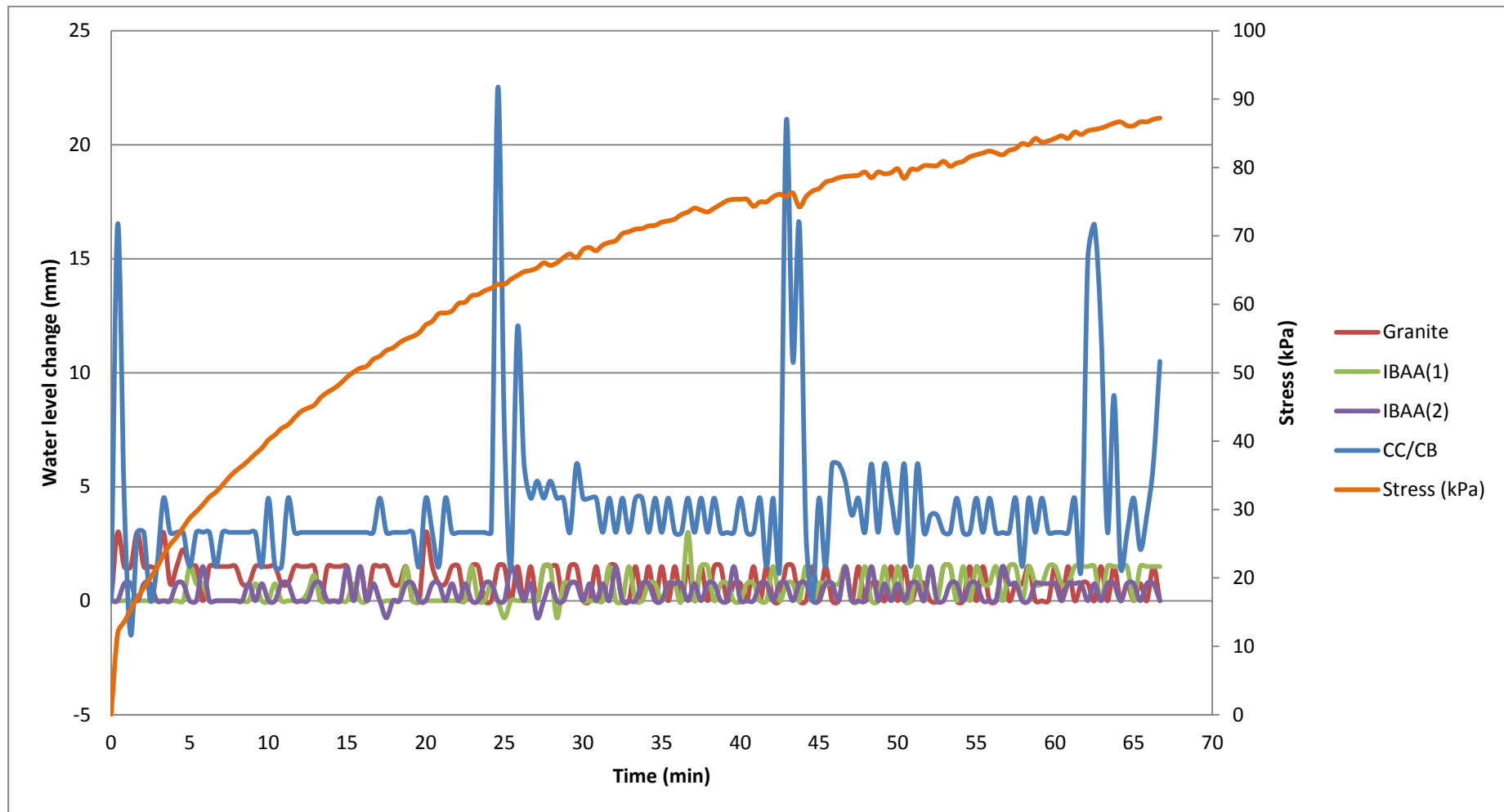


Figure 7.28: The water level changes during loading at the base of the primary and recycled aggregate columns compared at various stress changes of test 9 (the dry CC/CB)

The stress changes were presented against the time of loading for the CC/CB in Figure 7.28. It was observed that in the beginning of the loading, there was a sharp increase in the stress values with the time. The sudden increase in the stress values possibly resulted in higher values of the water level changes at the same time on the CC/CB especially at the base. There were other stress points where the sudden increase or decrease (or failures) in the column during the loading caused unexpected changes in the column and the surrounding soil. However, the most important factor was not the loading and was the nature and porosity of the CC/CB which caused more water absorption from the surrounding soil into the column during the loading phase and Figure 7.29 was presented to consider the possibility of the loading effects.

Figure 7.29 shows the same materials when the water level changes were analyzed during the loading at the level of bulging for the middle piezometer close to the centre of the column. The piezometer was located at a distance equivalent to the column diameter from the centre of the column. This distance was 54mm.

Similar to the results obtained from the base, at this level, the CC/CB showed highest variations in the water level changes due to its nature and level of water absorption from the surrounding soil. Compared to the water level changes at the base, the IBAAAs showed more changes throughout the whole loading process.

Based on the nature of the materials used in the stone columns constructed and the level of packing and the PSD, the drainage through the column during the quick loading might be different for the various materials. This trend was observed in Figure 7.28 where the readings of the base of the column were analyzed.

On the other hand, in the middle level (numbers 4 and 5) where the bulging happens, all the columns and the surrounding soils go through the stress changes. The changes were observed in Figure 7.29 where water level fluctuations during the loading were intense for all the materials during this stage. The fluctuations were frequent compared to the base (number 1) where only sudden changes happened at specific stress points.

The magnitudes of the water level changes were smaller in the middle (numbers 4 and 5) compared to the base, as the clay was not as permeable as the column material and lower water levels were obtained during the loading at the middle level.

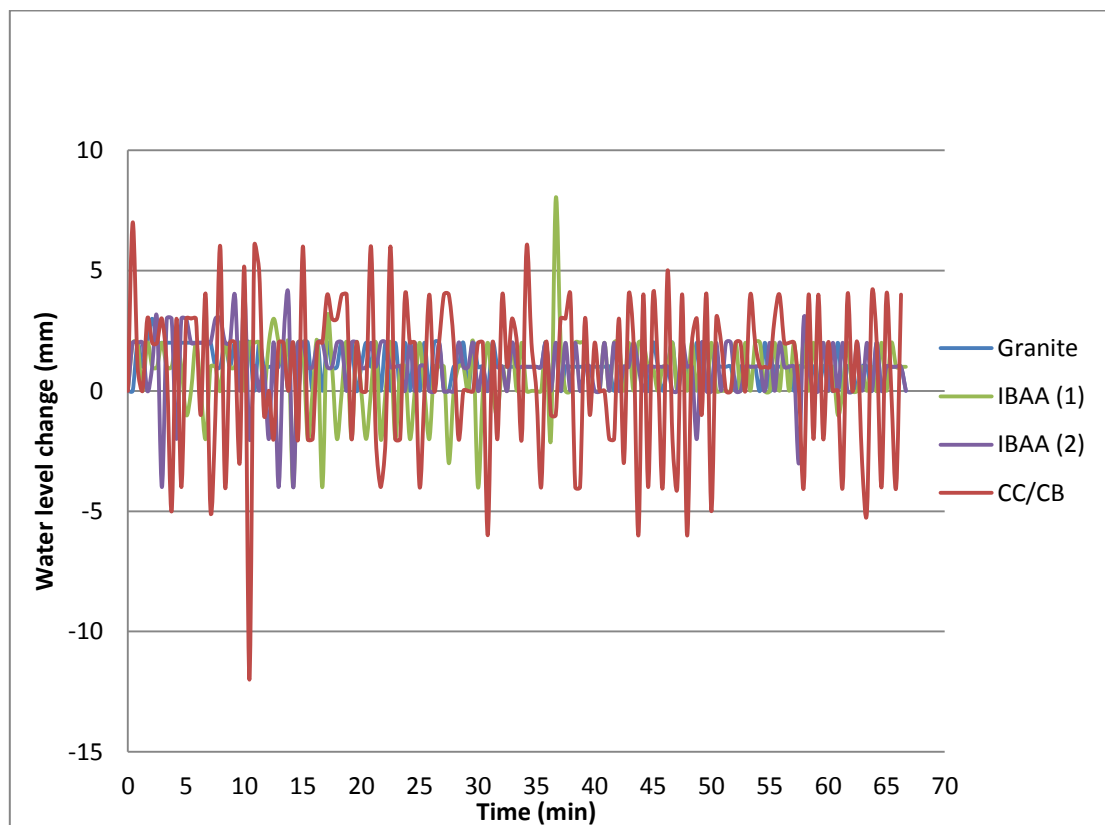


Figure 7.29: The water level changes during loading at the middle close piezometer for the primary and recycled aggregates

Figure 7.30 shows the water level changes at the middle piezometer which was 108mm far from the centre of the column. Similar to the closer piezometer, frequent fluctuations

were observed during the loading for all the four aggregates. The difference with the piezometer closer to the column centre was that the magnitudes of the water level changes were generally smaller than the close piezometer. It showed that more stress changes occurred closer to the column centre during the loading at the level of bulging.

Similar to Figure 7.29, the water level changes were more sudden and sharp in the CC/CB, followed by the IBAA's and then the granite. This could be related to the nature of the RAs used and the porosity and the level of water absorption of these aggregates.

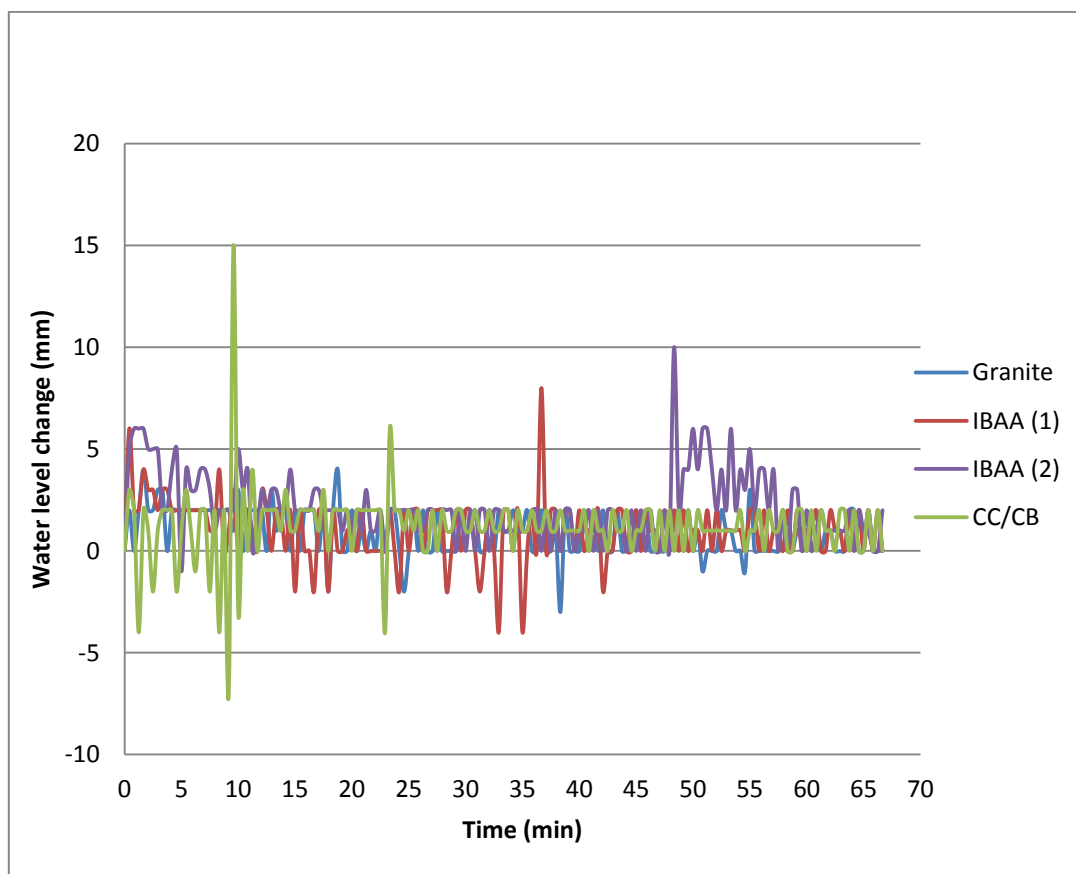


Figure 7.30: The water level changes during the loading at the middle far piezometer for the primary and recycled aggregates

7.7.6 Comparison of the water levels during the loading for the short and the long-term tests

Tests 8 and 15 were compared during the loading to show the water level changes at the base when the dry granite was modelled in the columns.

Test 8 was a short-term test, in which after the column installation, the column was quickly loaded.

Test 15 was prepared similar to test 8, however, after the column installation it was left for 3 months before loading. The consolidation process in the clay started during the time that the installed column was left in the clay. Also, the water dissipated through the column.

Figure 7.31 compares the water level changes at the base during the loading for the two columns in tests 8 and 15 (the short-term and the long-term).

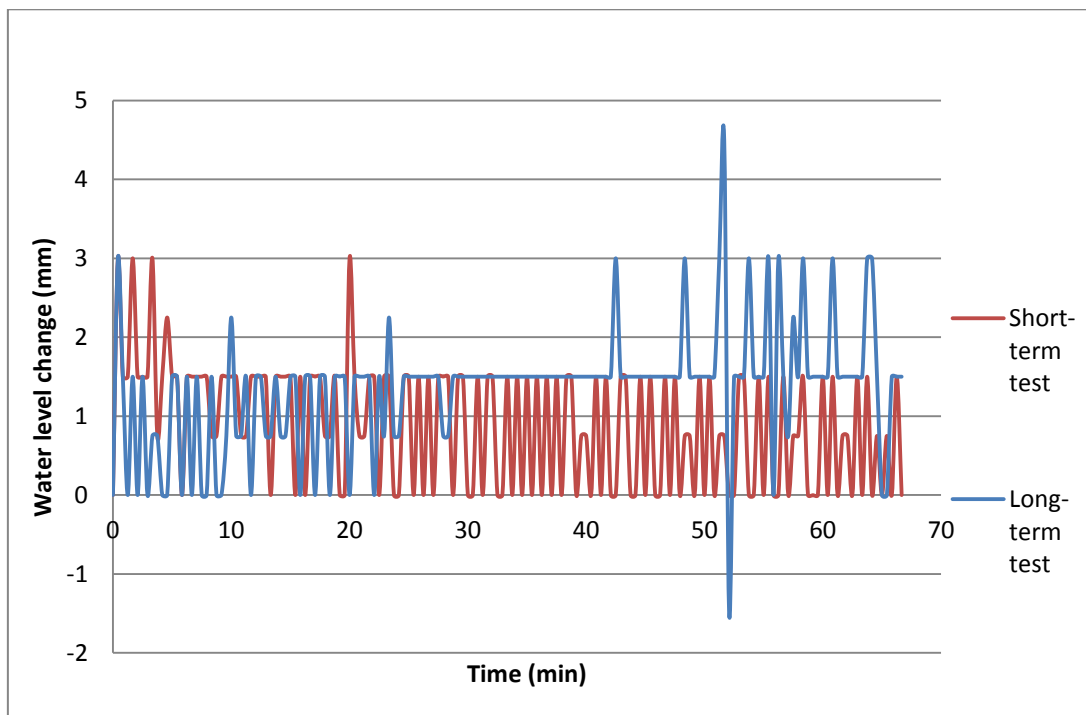


Figure 7.31: The comparison of the water level changes at the base of the short and the long-term tests on columns of PA

It was observed that both of the columns showed water level changes in the beginning of the loading process. As a higher stress was applied to the columns, the long-term column showed steady trend in the water level changes compared to the short-term test, where the fluctuations continued throughout the loading.

Towards the end, at higher stress points, more significant fluctuations were observed in the long-term test where changes of up to 5 times the short-term test were recorded in the long-term column.

In the long-term test as the column was left after the installation, it was expected that due to the water dissipations from the base (number 1), it would show less water level change during the loading. However, it seemed that in the beginning when additional stresses were applied to the system and also, towards the end when higher stress values were applied; the water level increased sharply at the base of the long-term column. On the other hand, the short-term test showed frequent changes throughout the loading from the beginning until the loading stopped.

7.8 Evaluation of the LUC tests results

7.8.1 Errors in the large unit cell tests

Errors of the measurements and analysis could be related to the various stages of preparation, column installation, loading and the methods of measurements.

During the preparation of the tests and after unloading, several quality control measures were introduced such as the moisture content and the undrained shear strength tests.

During the installation process, as the procedure was explained in chapter 6; the forces exerted by the concrete poker could cause various columns to be constructed with variable densities.

The materials used were controlled for their PSD and the angle of shearing resistance before the tests. The unexpected results and column behaviours were analyzed due to the errors and variations in the PSD and the properties of the materials used.

During the loading in a short period of less than 70 minutes, values of stress and water levels were read at every 0.5mm of deformation until the 80mm travel was achieved. The measurements had errors and the tests were repeated to ensure the results obtained were consistent. Errors of the stress-strain tests were estimated and the reasons were contributed to the variations in the column density, the PSD and the nature of the materials used for the testing. The water level readings created errors of (± 0.2) mL for the porous stone and (± 0.1) mL for the piezometers.

After unloading monitoring of the water levels showed that the water level changes became steady and the values decreased at all the levels of the measurements.

7.8.2 Comparison and repeats

The tests performed were repeated on the granite, the CC/CB and the IBAA (1). In case of the IBAA (2) and the long-term test, repeats were not possible. The wet and the dry materials were compared. In case of the RAs only the CC/CB could be tested and repeated. Repeating the results on the RAs assisted in better understanding of their behaviour in the LUC tests.

The materials used in this research were unique and could only be compared against each other. The other published studies used other sources of column materials such as

sand, gravel and other primary and recycled aggregates. The nature of the material used in the modelling, the PSD and the angle of shearing resistance as well as the condition of aggregates were critical factors that made a direct comparison of the various columns challenging.

The LUC tests could be reproduced; however the material source could be different and create variations in the results. However, the aim was to observe and compare the actual primary and recycled aggregates in this context where the aggregate index tests might have suggested that many of the RAs were unsuitable for the use in the VSC construction and loading.

7.9 Summary of the LUC tests results

The main findings of the 15 LUC tests results were summarized below:

- 1) The quality control tests on the host ground proved that the moisture content and the undrained shear strength required for the VSC modelling in the LUC container was achieved for all of the 15 tests.
- 2) The quality control tests on the aggregates showed that the materials used for the column formation (the granite, CC/CB, IBAA (1) and IBAA (2)) had various PSDs. The RAs used in this research were well-graded compared to the more uniformly graded granite.
- 3) The densities estimated from the columns formed in the unit cell showed that the various PSDs and the nature and the shape of the aggregates created columns of various densities.
- 4) The installation process and the vibrations exerted on the same type of aggregates caused columns of various densities to be formed.

- 5) When the columns were loaded, the foundation loading caused lower stress distributions on the column and the surrounding clay compared to the axial plate at each specific strain. Therefore, the foundation loading was used in the rest of the LUC tests.
- 6) All the constructed columns (regardless of the type of the aggregates used) improved the load carrying capacity of the host ground significantly by at least 80%.
- 7) The column of the IBAA (2) improved the load carrying capacity of the composite (the column and the clay) more than the other columns of the PA and the RAs by at least 180% improvement.
- 8) The significant improvement in the load carrying capacity for the column of the IBAA (2) was contributed to its well-graded PSD which caused better packing of the column in the ground. Also, the nature and the ash matrix of this material held the column together at the lower strains.
- 9) The most important factor affecting the load carrying capacity was the condition of the aggregates (wet/dry). The wet aggregate columns had lower load carrying capacity compared to the dry columns.
- 10) The only long-term test on the granite (test 15) showed that the long-term column left in the ground absorbed water from the surrounding soil and reduced the load carrying capacity of the column similar to the weaker wet aggregate columns tested.
- 11) The settlement of the columns was both estimated using the Priebe's method and also measured in the actual tests performed in the LUC. The results showed that the Priebe's method was highly conservative for both the columns of the PA and the RAs.

- 12) The water level changes measured in the partially saturated clay of the LUC tests showed that the surrounding soil changed since the installation of the stone columns started.
- 13) More water was transferred through the column (as a granular material) compared to the surrounding soil. In other words, the vertical water dissipation was more than the radial dissipation rate.
- 14) During the loading of the columns, the CC/CB absorbed the water from the surrounding clay due to its nature and showed more fluctuations in the water level changes at this stage compared to the other columns of the PA and the RAs.

The findings showed that despite the various results of the aggregate index tests, the aggregates behave differently in the context of VSCs and the aggregate index tests alone are not enough to predict the suitability of the various aggregates for the use in the installation and loading of the VSCs. The study of the materials in the context of installation and loading of the VSC is required for comprehensive understanding of the primary and the recycled aggregates used in the VSC construction.

CHAPTER EIGHT

RESULTS AND DISCUSSIONS- PART 3- THE SMALL UNIT CELL TESTS

8 RESULTS AND DISCUSSIONS- PART 3- THE SMALL UNIT CELL TESTS

In this chapter the results and discussions of the three series of tests performed on the small unit cell were provided. A total of 27 tests were performed on the primary (granite) and recycled aggregates (CC/CB and IBAA (1)).

Series 1 discussed the effects of installation and loading on the crushing of various recycled aggregates that were compared to the crushing of the granite (PA).

Series 2 compared the effects of the energy of installation on the crushability and ultimately the load carrying capacity of the granite.

The last series (series 3) studied the effects of contamination of the column material with fines on the load carrying capacity of the columns of granite and compared the performance with the columns of aggregates that were not contaminated.

Comparing the densities of the columns constructed, the installation impacts, the crushability of the aggregates during installation and loading and the shape of the columns constructed were among the most important discussions and findings of this chapter.

8.1 Introduction to the results and discussions of the small unit cell tests

The method of preparation, measurements, instrumentation and the factors studied in the small unit cell (SUC) tests were explained in chapter 6 (refer to sections 6.4, 6.5, 6.6 and 6.8). Tables of the three series of tests performed in the SUC were presented in section 6.3.2.

Various aspects of the performance and comparison of the primary and recycled aggregates were modelled in the LUC container.

Other factors such as the crushability of aggregates under the installation forces compared to loading; the effects of installation energy on the aggregate crushability and the contamination of aggregates with fines were performed under the axial loading of a single column in a smaller scale. The small container provided the opportunity for the researcher to study more factors separately using fewer quantities of the host and the column materials. The tests were repeated in all the three series.

The factors studied could be compared in various tests and the results can be related to the findings of the LUC tests discussed in chapter 7. However, the limitations of the SUC tests (scaling and axial loading) compared to the LUC tests should be considered.

In the SUC tests, only the axial loading was performed due to the smaller size of the container used and the boundary condition limitations.

The clay used as the host ground was reused from the LUC tests; however, the quality control measures (the moisture content and the undrained strength tests) were taken to ensure the requirements for the column installation were met.

Columns constructed had the diameter of 54 mm but smaller lengths of 420mm compared to the 760mm length columns constructed in the LUC tests.

Similar to the LUC tests the end-bearing columns were constructed in the soft clay on the firm base of the cell. Similar to the LUC tests, the Static loading was applied to the columns through the axial plate.

8.2 Results and discussions of Series 1- The crushability of the materials

In series 1, the granite, CC/CB and IBAA (1) were modelled in single columns. The procedure of the preparations and findings of each of the test were explained in chapter 6 (see section 6.8).

16 tests were performed in this series to compare the crushing of the aggregates under installation forces and installation and loading. Enough quantities of the IBAA (2) were not available for these tests and only the CC/CB and IBAA (1) were compared to the granite.

Test 1 was a pilot test in which the aggregates were compacted in layer using a compaction hammer. The quantity of the granite used to form the column resulted in a higher density of the column compared to all the other 15 tests. As the compaction was not the standard method of installation in this research, it was abandoned after the pilot test, and the other 15 columns were constructed using the concrete poker.

Each material was installed in the column and after installation; aggregates were vacuumed out and subject to the PSD. The test was repeated on the same material when after the installation; the material in the column was loaded and after the unloading; the material was vacuumed out and subject to the PSD. This comparison assisted in understanding the behaviour of the material under the installation forces and the loading separately.

Various aspects of the results were compared in sections 8.2.1 to 8.2.7:

8.2.1 Quality control of the host ground

The host ground used in all the SUC tests was reused from the LUC tests after cleaning.

The quality control tests included the moisture content and the undrained shear tests using the hand vane performed after each test.

When the soil was cleaned out of the LUC tests, its moisture content and the undrained strength were measured at each layer (of the 9 layers of the clay compacted in the LUC container). Therefore, in the beginning of the SUC tests, the water content test was not repeated. After the clay was placed in the SUC; each layer was compacted for 4 minutes to form a total of 5 layers. The clay compacted was then left in the cell overnight for homogenization.

The first moisture content test in the SUC was performed during the installation of the columns. When the core was extruded to form a hole for the aggregate compaction, three samples were taken from the top, the middle and the bottom of the core.

After installation and loading, the columns were unloaded and the shape of the column was studied using the vacuum and grouting method described in chapter 6 (see section 6.5.5). After 24 hours once the grout was set, the surrounding clay was cleaned in layers where the moisture content and the vane shear tests were performed at each layer at the boundary condition (at a radius of 2.5 times the column diameter).

The average values of the moisture contents and the undrained strengths were calculated for each layer, and the detailed results were presented in Appendix 6 (refer to CD).

Table 8.1 summarizes the range of the moisture content and the undrained strength values obtained with accuracies of 0.01(%) and (± 2) kPa, respectively.

Table 8.1: Quality control of the host ground in the SUC tests-series 1

Test name	Test number	Moisture content range after the test (%)	Moisture content of the core extruded for column installation (%)	Undrained strength of the host ground after the test (kPa)
Pilot	1	39-41	39-41	17-21
Granite-loaded	2	39-42	39-41	18-21
Granite-installation	3	38-40	38-40	18-25
Granite-installation-repeat	4	37-39	37-39	22-27
Granite-loaded-repeat	5	37-40	37-39	17-28
CC/CB-installation	6	38-40	37-39	17-22
CC/CB-loaded	7	38-40	39-41	17-23
CC/CB-installation-repeat	8	38-40	38-39	20-23
CC/CB-loaded-repeat	9	36-37	37-38	35-39
CC/CB-loaded-repeat 2	10	39-41	40-42	22-26
IBAA(1)-installation	11	38-41	38-40	18-23
IBAA(1)-loaded	12	38-40	38-41	18-22
IBAA(1)-installation-repeat	13	39-41	39-41	15-19
IBAA(1)-loaded-repeat	14	38-40	39-40	16-21
No column-loaded	15	39-41	-	14-22
No column-loaded-repeat	16	39-41	-	18-20

As observed in Table 8.1, the clay reused provided the range of the moisture contents required for the unit cell tests except for test 9 in which the reduction in the moisture content caused extreme increase in the values of the undrained strength beyond the maximum requirement of 25kPa. Test 9 on the CC/CB was a repeat test (for test 7) but had to be repeated a second time to make sure the undrained strength required existed in the host ground (test 10 was a repeat test for test 9).

It was also observed that due slight loss of the moisture content during the procedure of reusing the clay, the undrained strength values increased to over 20kPa in most cases which were slightly higher than the values measured for the LUC tests (see section 7.2).

8.2.2 Quality control of the column material

The particle size distribution (PSD), the angle of shearing resistance and the density of columns constructed were the quality control factors in the interpretation of the behaviour of the columns in the SUC tests.

Material used for each column was subject to the PSD before each test. As the aim of the first series of the tests was to compare the crushing of the materials before and after installation or before and after loading; the PSD was performed after each of these stages.

The angle of shearing resistance was obtained for various materials and the details of the results were presented in chapter 5 (see section 5.5.6.4). The same aggregates were used in these tests in the dry condition.

The density of the columns constructed was estimated for each of the unit cell tests. The quantity of the aggregates consumed in the column construction was measured to be used to estimate the column density based on the estimated volume of the column constructed. The variations of the densities was due to the different PSD ranges available for each of the materials which resulted in different levels of packing and interlocking of the aggregates in the columns which was fully explained in section 8.2.3.

Table 8.2 shows the results of the column density estimation for all the columns constructed in series 1 of the SUC tests.

The column density, the angle of shearing resistance and the PSD were used in the analysis of results in the following sections.

Table 8.2: Density of the columns constructed in the small unit cell and the angle of shearing resistance of the aggregates-series 1

Test name	Test number	Column density (kg/m^3)	Angle of shearing resistance measured in this research (degrees)
Pilot	1	2294.27	47
Granite-loaded	2	1900.67	47
Granite-installation	3	1578.6	47
Granite-installation-repeat	4	1913.64	47
Granite-loaded-repeat	5	1574.75	47
CC/CB-installation	6	1685.98	40.2
CC/CB-loaded	7	1593.28	40.2
CC/CB-installation-repeat	8	1590.8	40.2
CC/CB-loaded-repeat	9	1407.55	40.2
CC/CB-loaded-repeat 2	10	1436.82	40.2
IBAA(1)-installation	11	1565.996	41.5
IBAA(1)-loaded	12	1724.18	41.5
IBAA(1)-installation-repeat	13	1593.23	41.5
IBAA(1)-loaded-repeat	14	1508.67	41.5
No column-loaded	15	-	-
No column-loaded-repeat	16	-	-

8.2.3 The particle size distribution before and after column installation

In these tests and their repeats on the granite, the CC/CB and the IBAA (1), the columns were constructed using the usual method of compacting for 20 seconds per layer via the vibrating hammer similar to the LUC tests.

After the installation material was vacuumed out and subject to the PSD. The changes during the installation in terms of the crushing of the aggregates were presented in Figures 8.1 to 8.4.

Figure 8.1 shows the PSD of the granite before and after installation in test 2 and its repeat. It was observed that the level of crushing of the granite at the stage of installation was minimal. The Vibrational forces of the concrete poker used affected the PSD of the granite only slightly in the first test. Slightly more fines were produced in

the range of 4.5 to 6mm. The results could not be generalized for the granite used in practice, as the method of installation and its energy and scaling effects of the particles used in the modelling affected the results obtained.

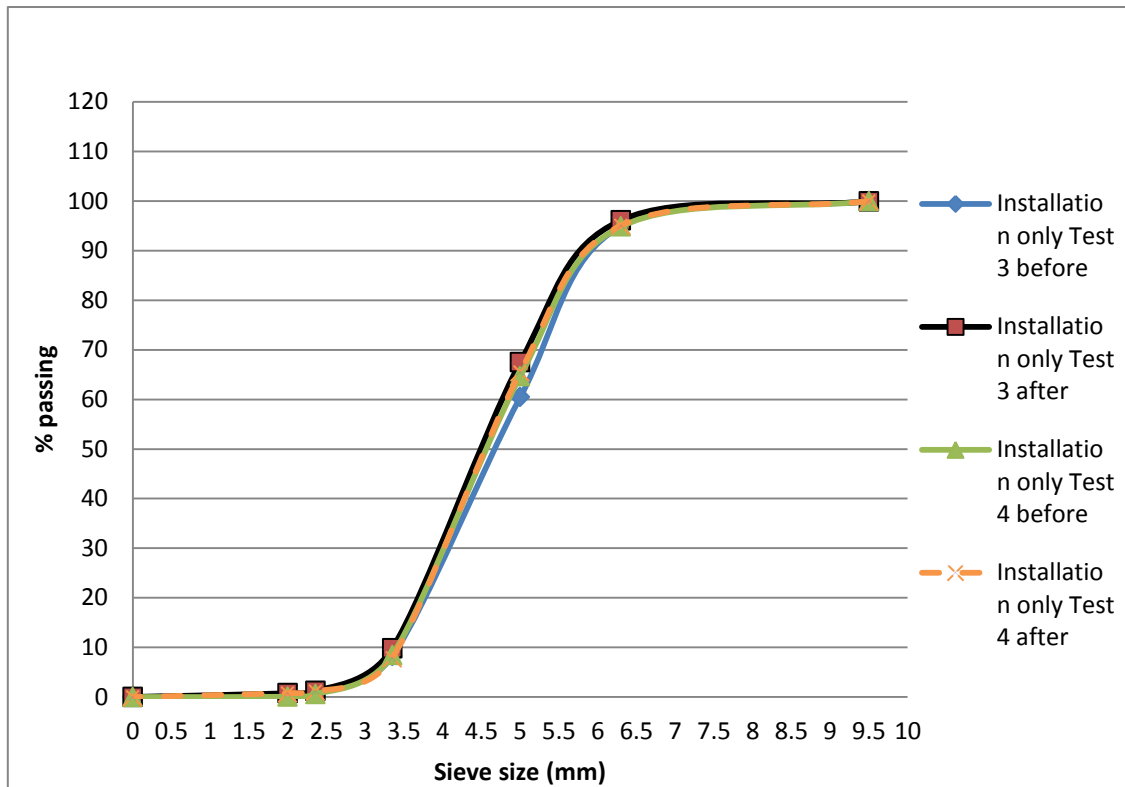


Figure 8.1: PSD of the granite before and after installation

Figure 8.2 shows the PSD of the CC/CB as a recycled aggregate before and after installation. More crushing was observed in the repeat test compared to the first one. Also, compared to the granite more aggregate crushing was observed for this material. However, the crushing was less than 10% and was only observed in the particle ranges between 4 to 6mm.

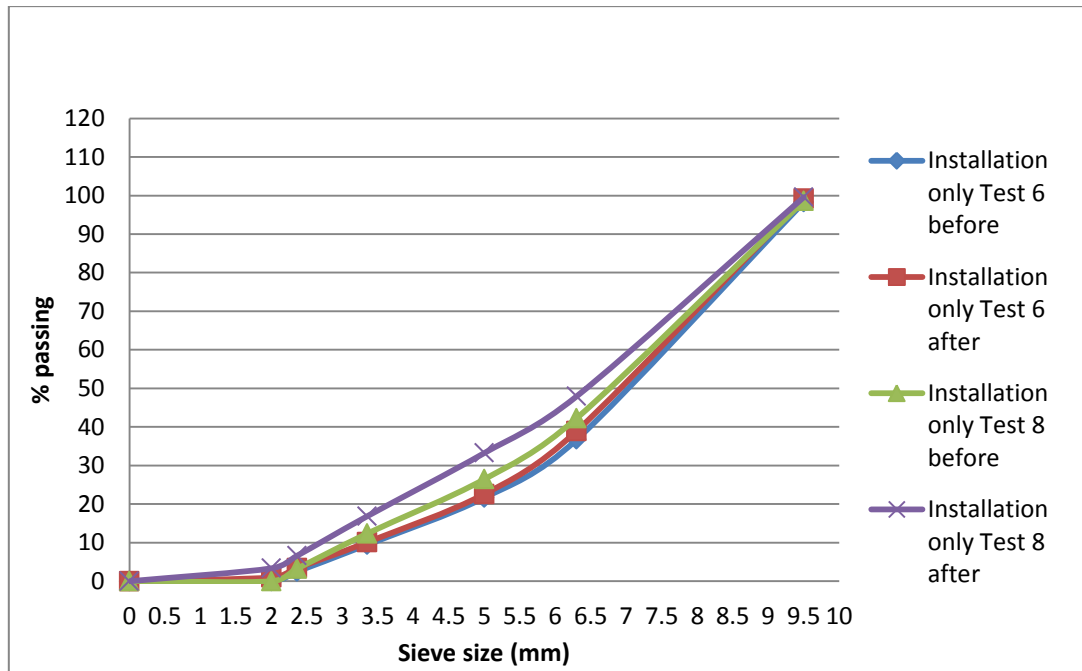


Figure 8.2: PSD of the CC/CB before and after installation

Figure 8.3 shows the same comparison for the IBAA (1). Similar to the CC/CB, the crushing was more than the granite during installation. Also, the repeat test showed higher level of crushing compared to the first test performed on this material. As opposed to the previous two materials, crushing was spread over the entire PSD curve of the IBAA (1) and all the aggregate sizes seemed to crush during installation of this material. Smaller percentage of crushing was observed compared to the CC/CB, to values of up to 5%.

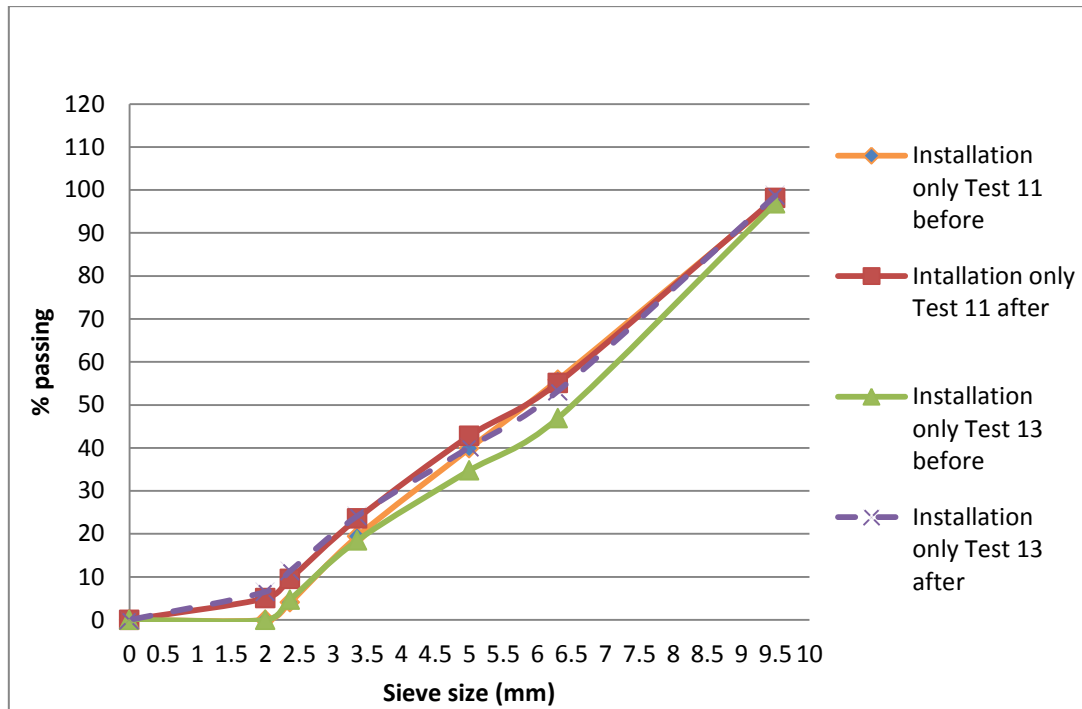


Figure 8.3: PSD of the IBAA (1) before and after installation

Figure 8.4 compares all the three materials tested for the PSD before and after installation. The average of the two tests performed on each aggregate was used to represent the crushing behaviour of the materials at this stage.

It was observed that the recycled aggregates used in this research showed higher level of crushing during installation compared to the granite (PA). The trends of the crushing observed for both the RAs were very similar. It could be concluded from this graph that the crushing during installation was negligible for all the PA and RAs used in the SUC in this research. However, the small scale used in this research could be the reason as opposed to the powerful equipment used in practice that may cause more crushing on all aggregate types during the installation process.

Based on the densities of the columns constructed, it can be observed in Table 8.2 that all these tests on the granite and the RAs showed very close range of column densities

between 1500 to 1600 kg/m^3 . Only the CC/CB in the first test had slightly higher a column density. Also, the angle of shearing resistance was higher in case of the granite compared to the RAs. This parameter as well as the original PSD of the granite could contribute to the lower levels of crushing achieved. The higher angle of shearing resistance of the PA prevented it from crushing during installation.

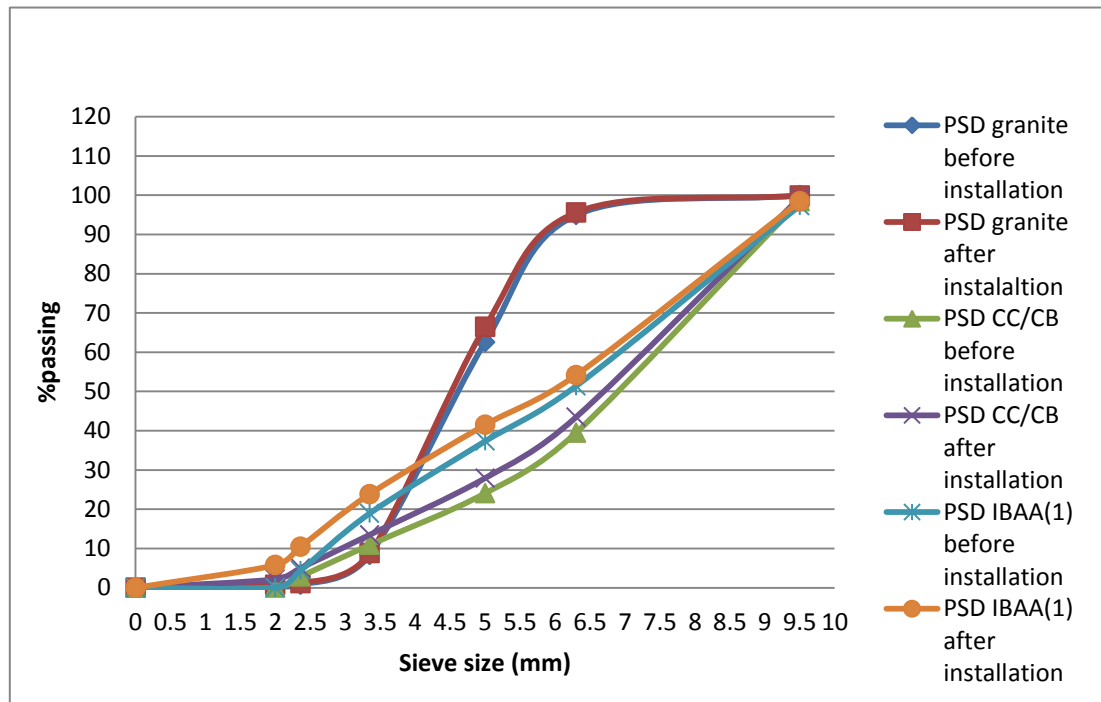


Figure 8.4: PSD of the three aggregates before and after installation

The variation between the natural and the alternative aggregates could be contributed to their original PSD range available where for the natural aggregate the material was more uniformly graded as opposed to the more well-graded RAs produced after the original materials were crushed to be scaled for the LUC and the SUC modelling (refer to section 5.4.1). The original crushing and sieving of the aggregates in order to prepare them for the SUC tests could have also affected their strength and crushability under similar installation impacts compared to the granite which was supplied with the required aggregate size.

8.2.4 Particle size distribution before and after column loading

In these tests, the columns were constructed using the same method explained in section 8.2.3. However, after the installation the material was not vacuumed out of the column. The column was loaded quickly after installation via an axial plate. The maximum travel of 30mm was achieved in all the tests which was beyond the failure point of the columns tested. The load-deformation measurements were taken at every 0.5mm of settlements. The results of the stress-strain behaviour of the three materials were presented in section 8.2.6.

Before the material was used in each test; the PSD was performed and compared to the results after unloading. When the column was unloaded, the aggregates were vacuumed out and the shape of the column was studied.

The results of the PSD before and after loading were compared in Figures 8.5 to 8.8.

Figure 8.5 shows the PSD before and after loading for the granite. Almost no crushing could be seen in the trend. The results of the main test and the repeat were very close with less than 10% error.

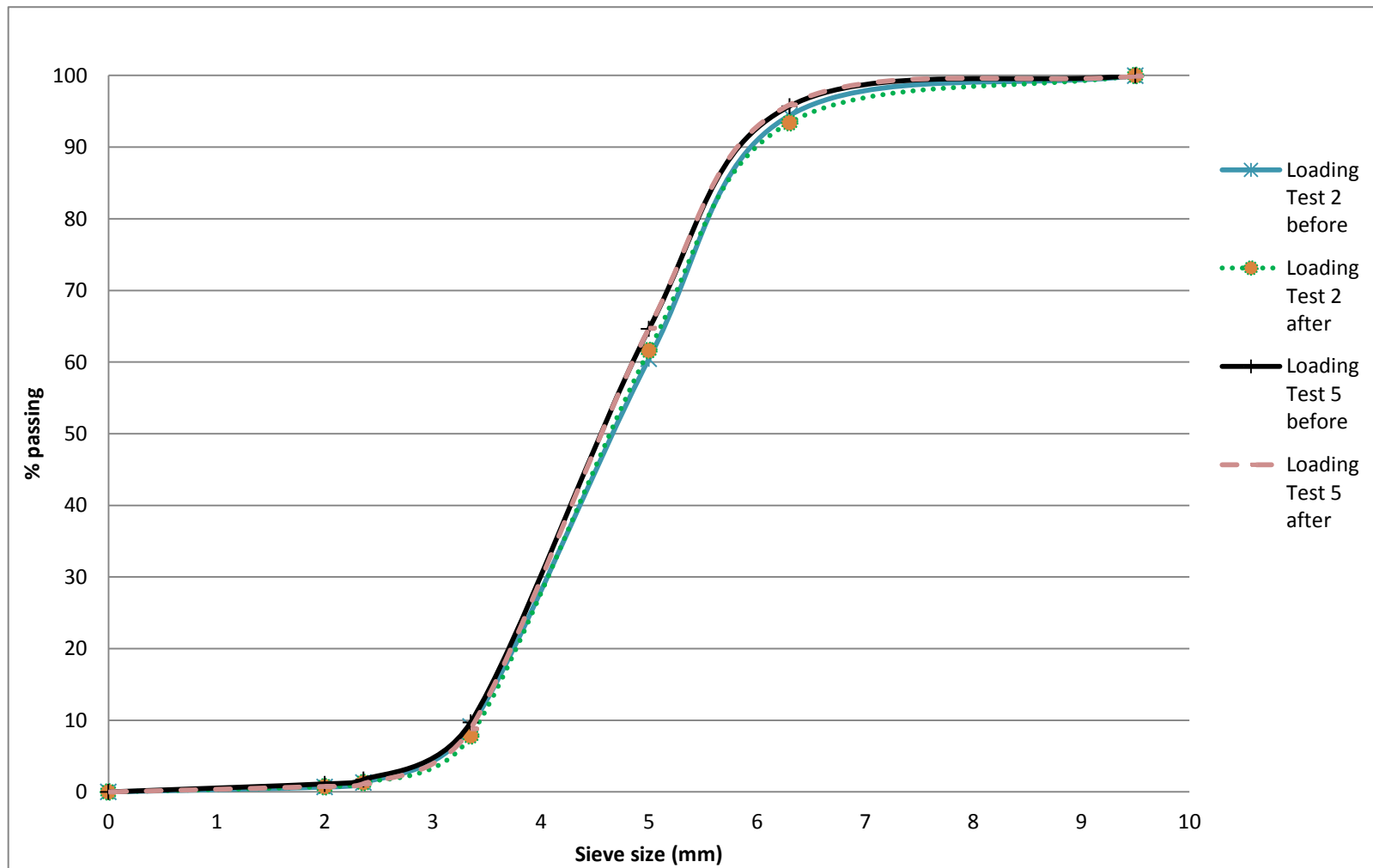


Figure 8.5: PSD of the granite before and after loading

Figure 8.6 shows the same results for the CC/CB. Three columns of the CC/CB were loaded. It seemed that the second repeat test (test 10 was a repeat test for tests 7 and 9) showed more crushing during the loading compared to the first two tests. This test showed crushing of up to 20% and twice the crushing in the first two tests (tests 7 and 9). As the same material was tested, the error observed could be due to the additional pressures exerted during the installation of the last column of the CC/CB by the concrete poker. Similar ranges of the column densities were observed for the three loading tests performed on the CC/CB.

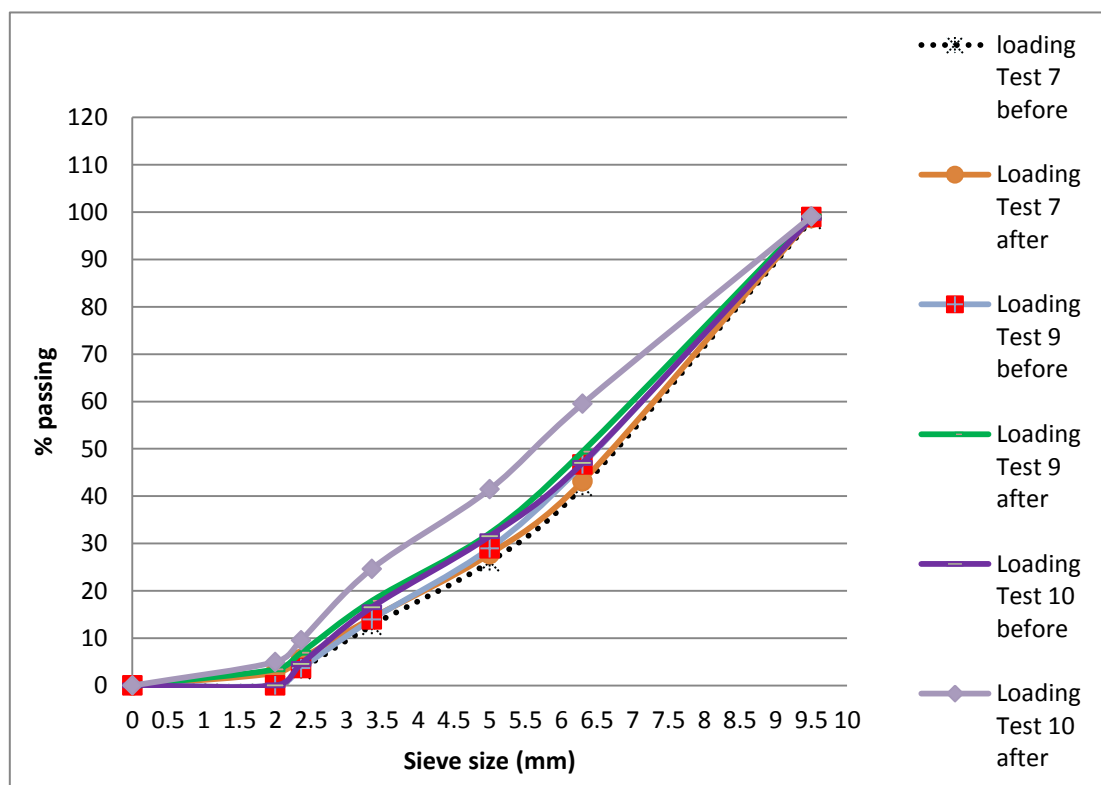


Figure 8.6: PSD of the CC/CB before and after loading

Figure 8.7 shows the PSD before and after loading for the IBAA (1). The trends observed showed that almost no crushing occurred before and after the loading in the IBAA (1) tested. The crushing of the IBAA (1) was insignificant throughout the whole process of the installation and loading and followed the granite in this aspect. The

CC/CB showed more crushing during the entire process compared the other two aggregates.

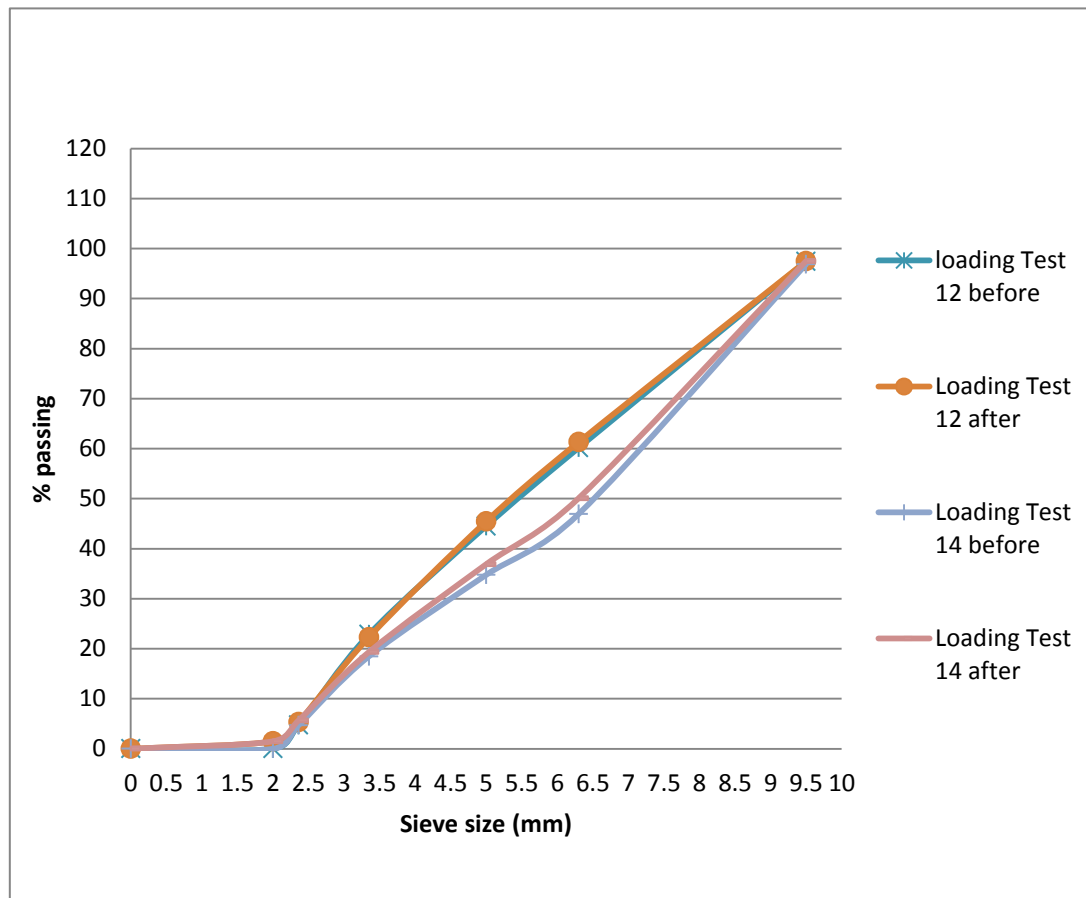


Figure 8.7: PSD of the IBAA (1) before and after loading

All of the three materials tested were compared in Figure 8.8 for the PSD before and after loading. Average values of the crushing were used in this graph to compare the various materials. It was observed that the crushing was minimal in the granite compared to the other two aggregates closely followed by the IBAA (1). The CC/CB went through more crushing during the installation and loading.

It seemed that apart from the granite, where a higher angle of shearing resistance was obtained in the shear box test, the other two recycled aggregates that had similar angle of shearing resistance were different in crushing because of their structure. The IBAA

(1) had a different structure that could hold the material together under the sustained loads.

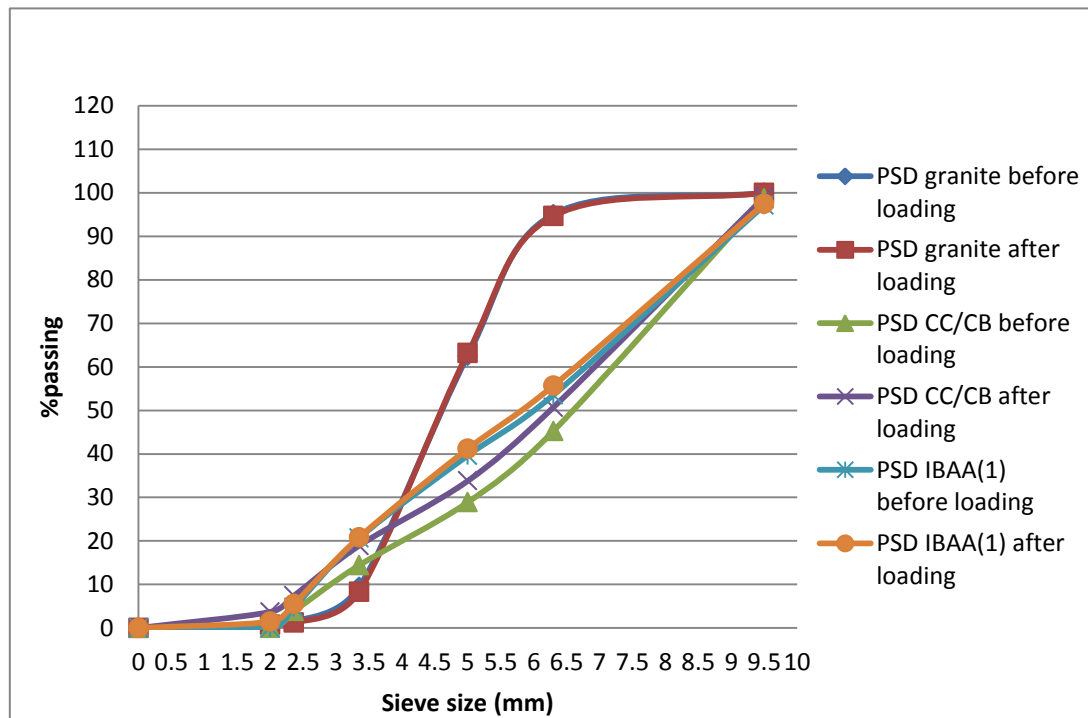


Figure 8.8: PSD of all the three aggregates before and after loading

8.2.5 Crushing of the aggregates during installation and loading

In order to compare the crushing for the installation and loading, the average values of the crushing at each stage were presented in Figures 8.9 to 8.11 for the PA and the RAs.

Figure 8.9 shows that the granite was not crushed during the SUC tests under the installation or loading. The trends of the PSD were similar and crushing in the granite was negligible compared to the other two RAs both during the installation and loading.

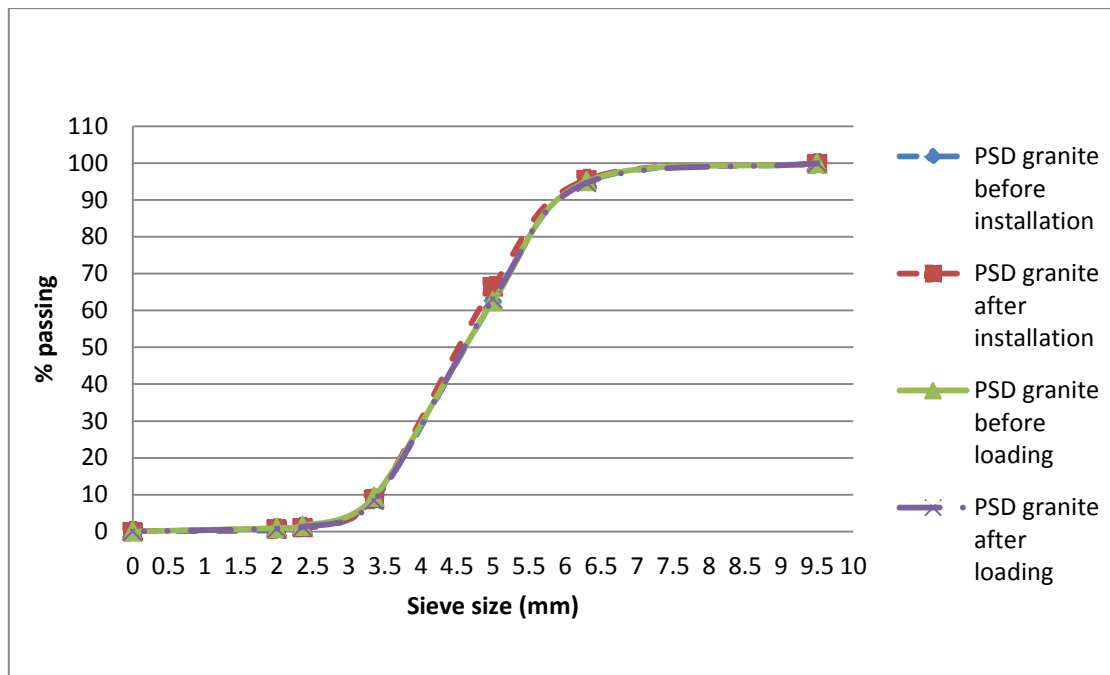


Figure 8.9: PSD of the granite during installation versus during loading

Figure 8.10 shows the change in the PSD of the CC/CB both during the installation and the loading processes from which the level of crushing of the material can be interpreted. It was observed that the majority of crushing could be contributed to the installation process for this material and the loading procedure slightly increased the crushing. For the maximum values of the crushing; more than half of the particle crushing occurred during the installation process.

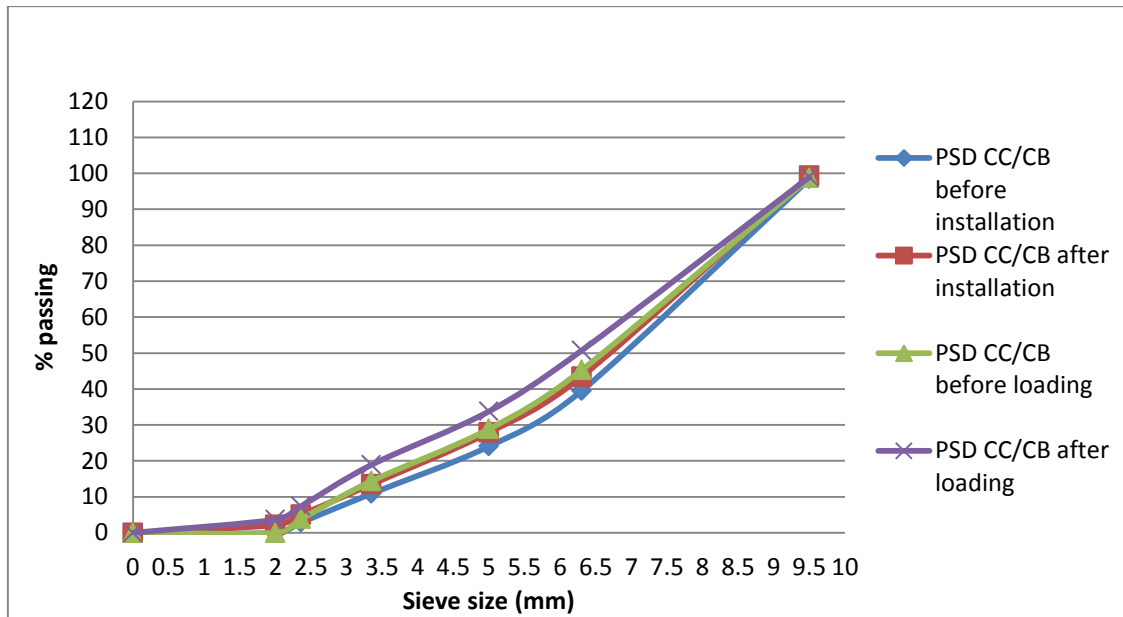


Figure 8.10: PSD of the CC/CB during installation versus during loading

Similar to the CC/CB, it was observed in Figure 8.11 that the majority of the crushing of the IBAA (1) could be contributed to the installation process. The crushing during the loading was insignificant compared to the installation process.

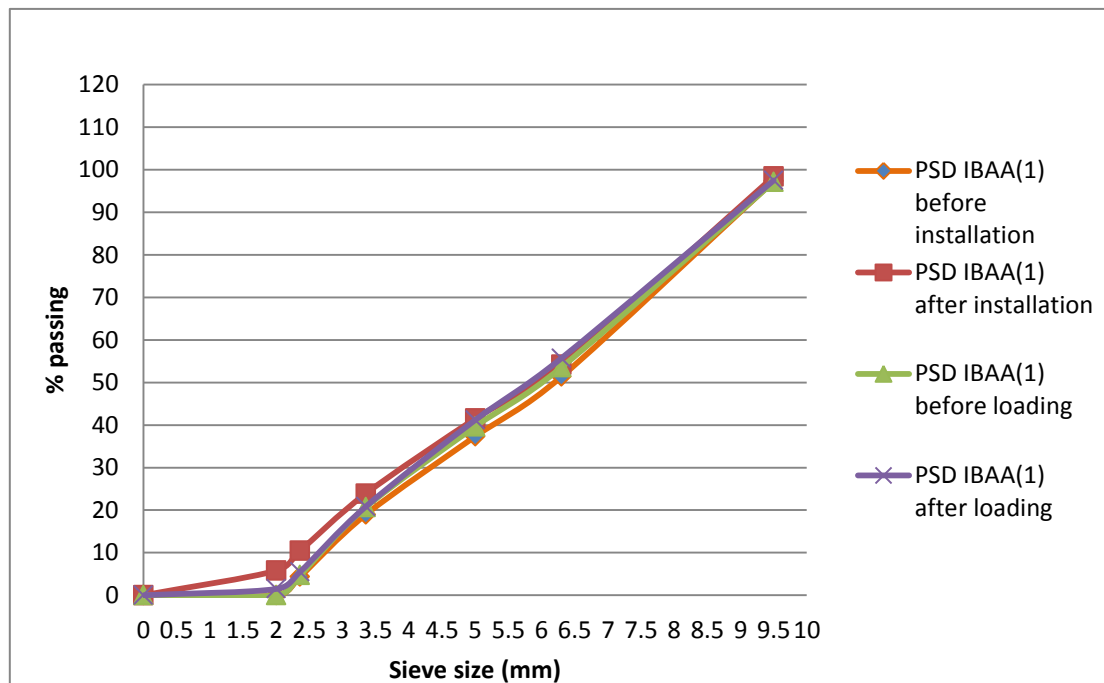


Figure 8.11: PSD of the IBAA (1) during installation versus during loading

It was observed in the SUC tests that the recycled aggregates crushed more both during installation and loading compared to the granite. Also, both the RAs tested crushed more during the installation compared to the loading. The vibrational forces of the vibro-float can have the same effect on materials whereas, the loading of the columns could increase the packing of the aggregates and the dense column formed during the loading might prevent further crushing of the particles. During the installation, a lot of aggregate crushing could reduce the angle of shearing resistance and the overall behaviour of the column could be affected.

It seemed that the structure and the nature of the material source were important in terms of the crushability during the installation and loading. In this research the RAs with a lower angle of shearing resistance values compared to the granite performed poorly during the installation in terms of the crushing. However, the values of the crushing obtained in this research were all below 10% and were negligible. The scaling effect should be considered as in the real scale VSC practice more crushing during the installation could happen.

8.2.6 Loading of the columns in series 1

In series 1 of the SUC tests, 10 tests out of the 16 were loaded after the installation. Although the loading was only performed to compare its effects on the aggregate crushing compared to the installation process; the results of the stress-strain curves obtained for each material were presented in this section.

Firstly, tests 15 and 16 on the no columns were compared and the average of the stress-strain properties of these two tests was used to compare the other columns of the primary or the RAs with.

8.2.6.1 The No column test

In these tests, the clay was prepared, and the axial loading plate was located in the assumed location of the stone column and then the host ground was loaded.

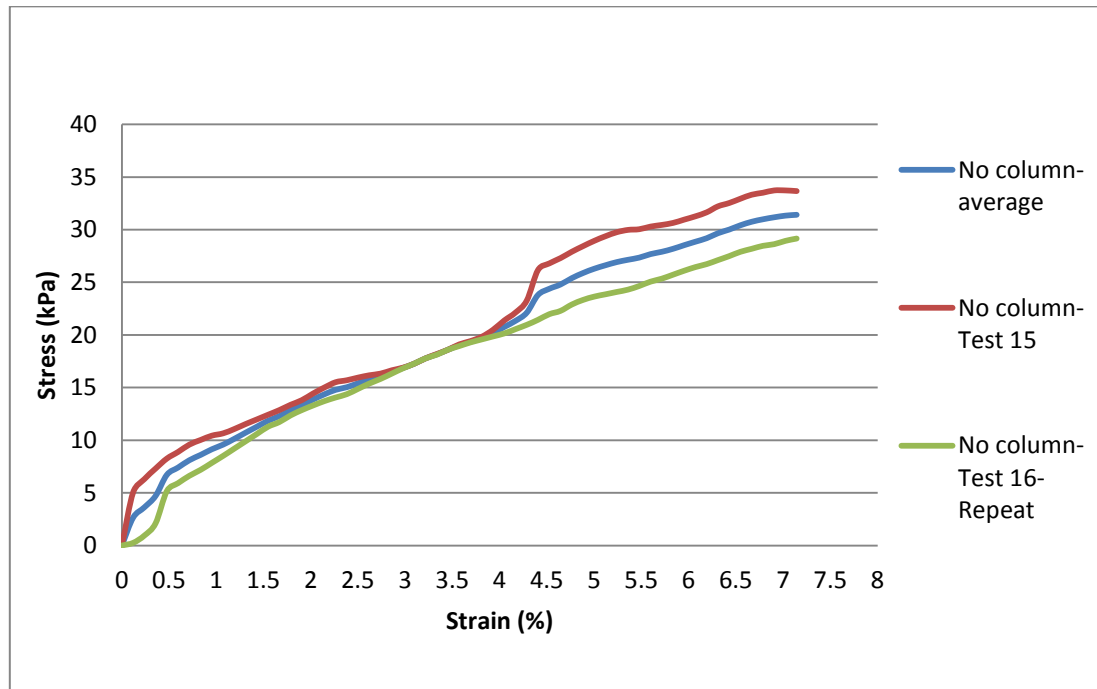


Figure 8.12: The stress-strain of the no column test loaded in the small unit cell container under the axial plate

It was observed in Figure 8.12 that both of the tests (15 and 16) had very similar trends in loading. The maximum travel of 30mm was used and divided by the depth of the treated area (420mm) at each point of the loading to provide the strain changes recorded.

Reduction in the stress values was observed at an approximately 2.5% strain, followed by a steady increase in both of the tests. At strains of 4.5% the stress increased suddenly and more deviation was observed between the two tests at higher stress values. A 17% deviation was observed towards the end of the loading.

Based on the failure definition by Zakariya (2001); a strain of 1.3% was where the failure should be compared for the two tests. At this point, the results were very close in both of the tests and the stresses of around 12 kPa were observed in the clay. This stress could be compared to the other tests where the columns were constructed and improvement in the load carrying capacity could be observed.

Hughes and Withers (1974) defined the failure at 58% of the column diameter, which was 7.5% strain. This was beyond the loading of this column and as the axial plate was used in the small cell, this definition was not used to compare the failures of the various tests. The overall trends observed and the stresses at the strain of 1.3% were compared for various tests.

8.2.6.2 The Pilot test

This test was performed to check the overall process of the loading and study of the shape of the columns. The results could not be compared to the other tests in the SUC as the method of installation was different from the compaction via the concrete poker. Due to excessive energy of the compaction by the standard compaction hammer, the column constructed had a higher density of over 20% compared to the other columns of the granite constructed.

Figure 8.13 compared the pilot test column with the no column in terms of the stress-strain behaviour. It was observed that the pilot test showed much higher stress values at each strain. There was a peak in the stress at a strain of approximately 2.5% at which the stress was 10 times higher than the no column loading condition. Even at a lower failure strain of 1.3% an improvement of 800% was achieved. After a certain point, the stress became steady and started to reduce. This change was well beyond the points of failures of the column constructed.

Figure 8.14 compared the pilot test with the other columns of the granite (PA) that were constructed using the concrete poker. At the peak stress of the pilot test, the stress was at least 122% higher than the columns of the granite in tests 2 and 5. As this column was not representative of the load carrying capacity of the columns constructed in the SUC tests, the results were not used in the analysis and further comparisons.

Tests 2 and 5 were compared in Figure 8.14 where a significant improvement was observed in the stress-strain patterns compared to the clay loaded without a stone column. A 25% difference was observed between test 2 and its repeat which was not negligible. This was due to the significant difference in the density of the columns constructed and the error of installation. The column with a higher density in test 2 showed a higher load carrying capacity compared to test 5.

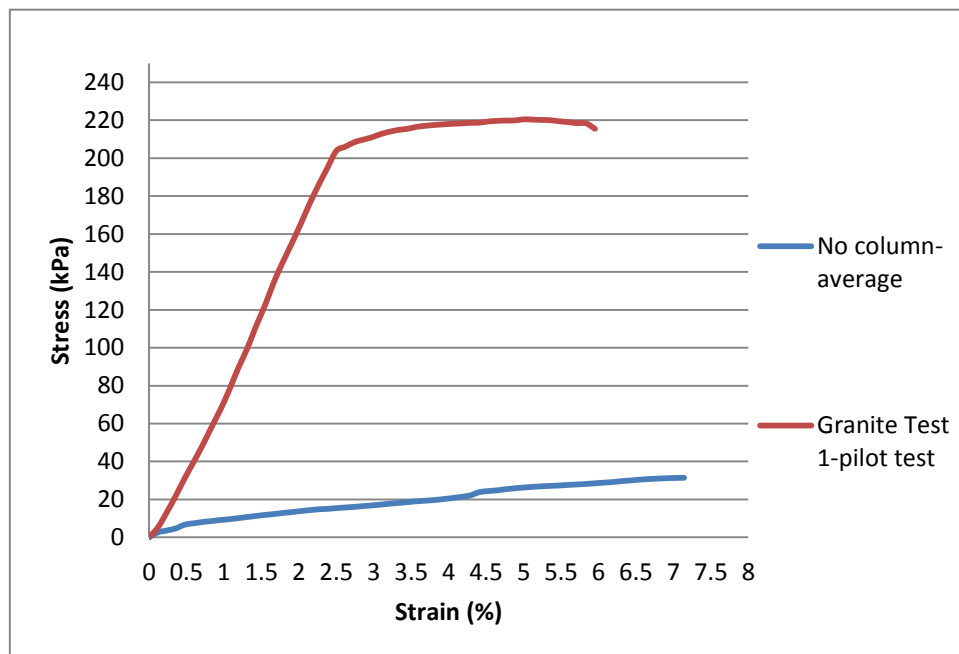


Figure 8.13: The stress-strain comparison of the pilot test and the no column in the small unit cell container under the axial plate

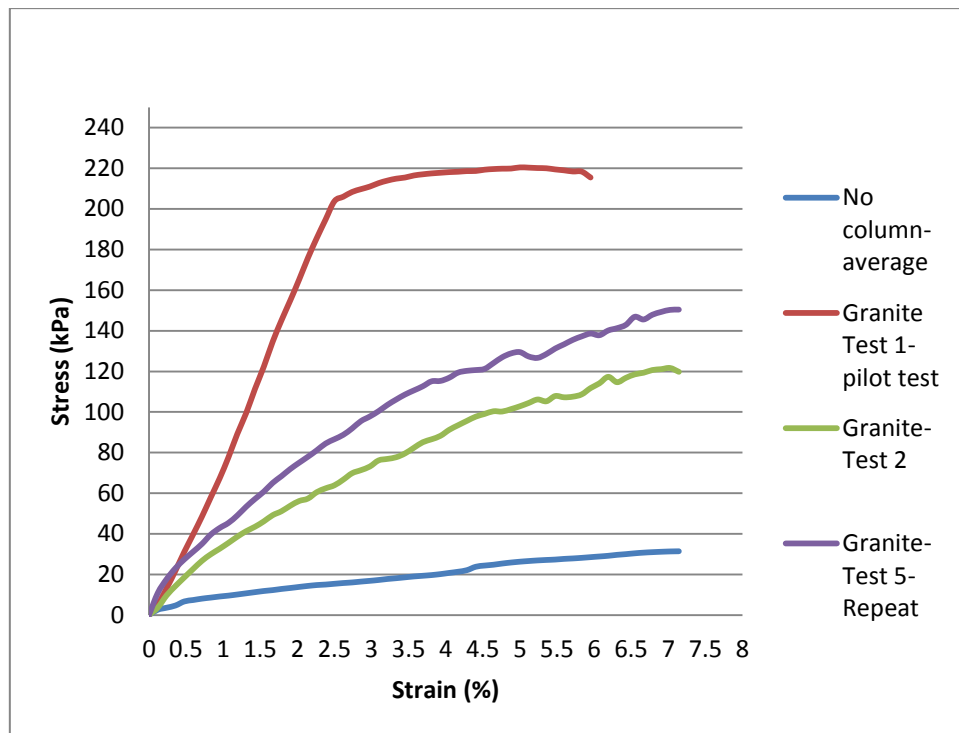


Figure 8.14: The stress-strain relationships for the pilot test compared to the other columns of the granite in the small unit cell

8.2.6.3 Columns of the recycled aggregates

Figure 8.15 shows the results of the load carrying capacity of the columns of the CC/CB. Due to the host ground error of the loss of the moisture content, test 9 was repeated in test 10. Despite having similar column densities, test 9 showed the variation in stress behaviour compared to the first two tests. The results were affected by the properties of the host ground that provided a higher undrained strength and higher stress values at each strain.

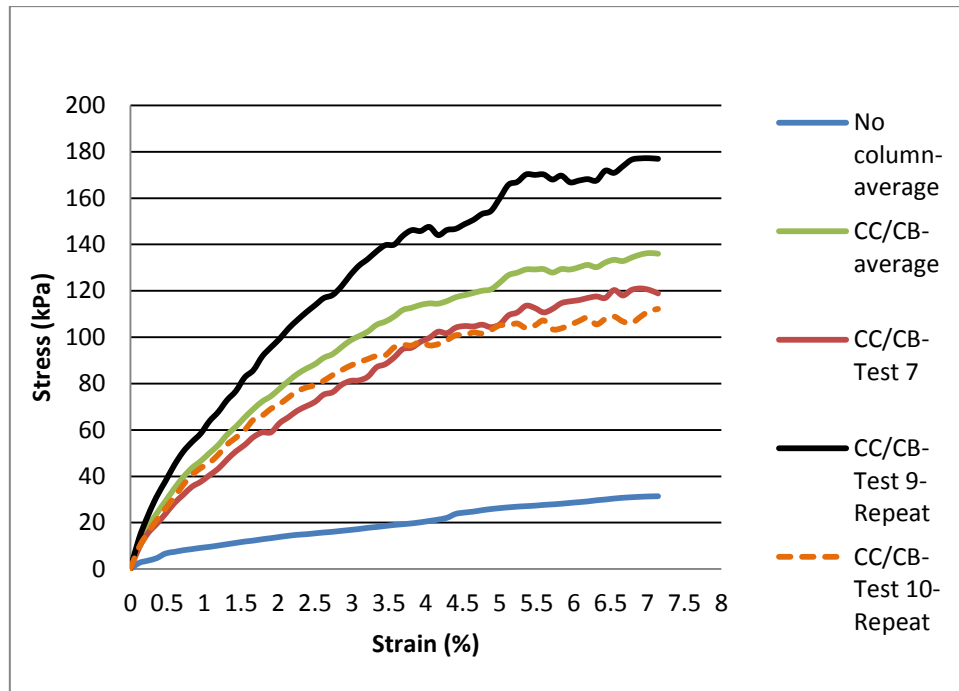


Figure 8.15: The stress-strain relationships of the columns of the CC/CB under the axial plate loading in the small unit cell

Figure 8.16 shows the stress-strain behaviour of the IBAA (1). The results were more consistent in the initial test and its repeat. Fluctuations were observed in the trends which confirmed the failure of the material above the strains of 1.3%. The higher levels of stress in test 12 compared to the repeat test could be contributed to the density of column achieved which was around 15% higher. The stress behaviour improved 5 times compared to the no column condition for columns of the IBAA (1) at strains of 1.5% which showed significant improvement when the column of the RA was constructed.

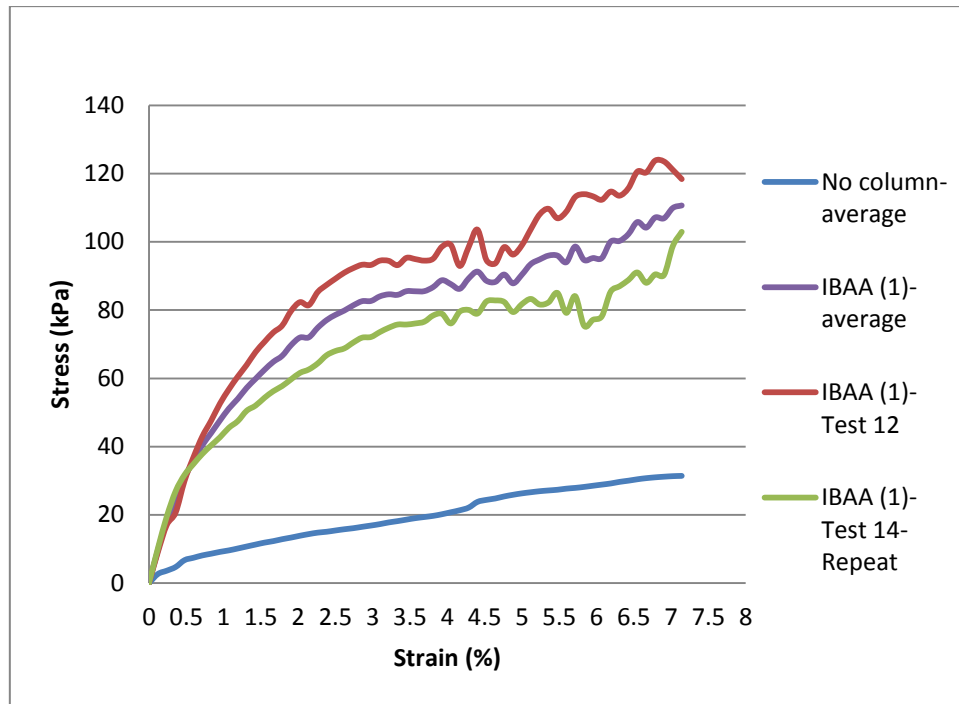


Figure 8.16: The stress-strain relationships of the columns of the IBAA (1) loaded under the axial plate in the small unit cell

8.2.6.4 Columns of the primary and the recycled aggregates

Figure 8.17 compares the load carrying capacity of the various materials tested in series 1 in the SUC. The average values of the stress-strain curves were used in the modelling. Test 9 on the CC/CB was not considered in the average of the values of the stress obtained due to the error of host ground properties.

Based on this figure, in the initial part of the loading and the lower strain values, the IBAA (1) outperformed the other two materials. The CC/CB followed the IBAA (1) and the granite performed poorer than the two RAs modelled. So far the results agreed with the results of the LUC tests when the columns of the granite and the RAs were loaded under the foundation type plate. However, after the 2.5% strain, the pattern changed where the granite showed higher stress followed by the CC/CB and the IBAA (1) at the same strains.

As the axial loading was applied it could be concluded that at lower strains, the RAs performed better than the granite, but after a certain increase in the loading, the granite caught up with the RAs and ultimately outperformed both of the RAs used. On the other hand, the axial loading was not a good representation of the actual loading condition of the VSCs in practice where the foundation type loading is usually applied. The data obtained was used to compare the columns of the PA and the RAs constructed in this research under similar construction and loading conditions.

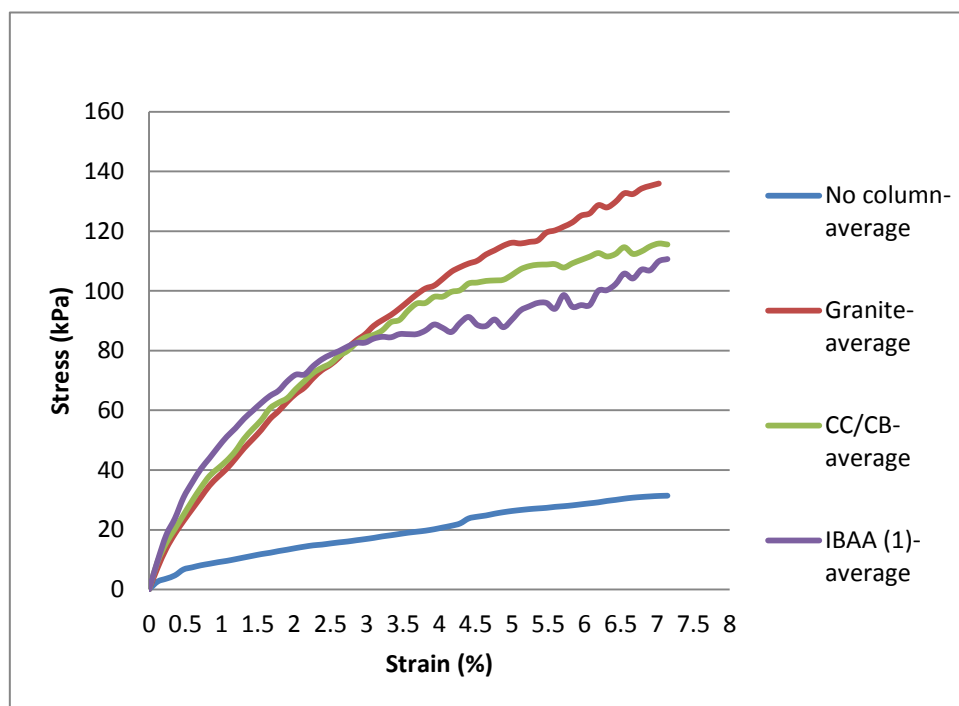


Figure 8.17: The stress-strain comparison of the granite and the recycled aggregates under the axial loading in the small unit cell

8.2.7 Shape of the columns

The shapes of the columns constructed were investigated after each test in series 1. The column shape after the installation was compared with the column shape after the loading for each material (the granite, the CC/CB and the IBAA (1)).

Figure 8.18 shows the columns of granite, the CC/CB and the IBAA (1) where the installation only was compared to the installation and loading.

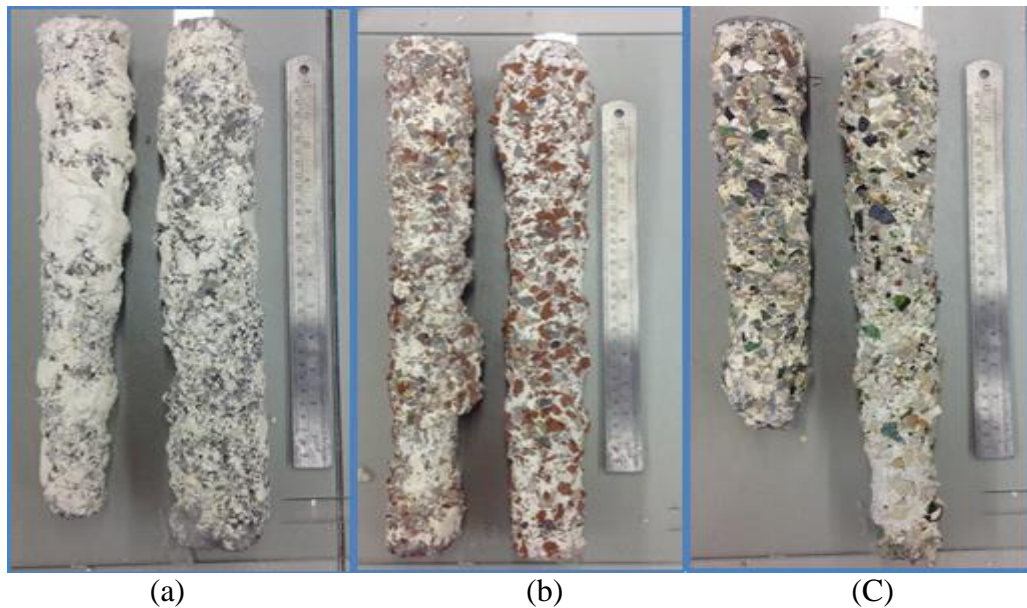


Figure 8.18: Shapes of the columns after installation versus after loading (a) the column of granite, left: installation only, right: loaded; (b) the column of CC/CB, left: installation only, right: loaded; (c) the column of IBAA (1), left: installation only, right: loaded

As observed in Figure 8.18, the shapes of the columns after installation only were different from installation and loading. The stages of installation where the aggregates were poured and compacted could be observed in the installation only columns. On the other hand, the bulging was observed in the columns that were loaded.

For the columns of granite (Figure 8.18 (a)), the stages of the installation were observed. The diameter of the column achieved was variable at each stage of installation. At greater depths the column diameter was smaller than designed. This could be due to the partial collapse of the clay into the soil due to the concrete poker vibrations.

The different diameters and lengths in the columns achieved were related to the quality of workmanship. The installation process and the level of vibration and compaction of the aggregates could create under or over-treatment in the ground. Therefore, smaller or bigger diameters than designed could be achieved at various depths. During the

installation the quantities of the aggregates used can help in evaluation of proper column formation.

For the column of granite which was loaded, the overall diameter was bigger than the column which was only installed. Apart from the stages of the installation, the bulging and the deformations near top of the columns were significant. As opposed to the column of granite which was not loaded, the diameter seemed more consistent throughout the length.

The quality of workmanship was the key in forming the columns with the proper diameter in practice as the design parameters affect the performance of the columns both in the short and the long-term.

In practice the proper diameter can be achieved by controlling the amounts of aggregates used in installation and the level of compacting the aggregates which can be controlled on site, however, each material is different in terms of the PSD and might be compacted differently as the particles pack differently under the same installation forces. Previous experience on similar materials can help in better quality control of the installation process (Bell, 2004).

For the columns of the CC/CB (Figure 8.18 (b)), the stages of installation were observed, where even in the column that was only installed and not loaded, it seemed that a small cavity surrounding the column was filled with extra material.

Similar to the column of granite, the diameter was variable at various depths and the diameter reduced near the base of the column. In the column of CC/CB that was loaded after the installation, the bulging was apparent near the top and the diameter achieved was more consistent along the length.

The column of the IBAA (1) showed a similar shape to the CC/CB both after the installation and after loading. In the column that was only installed, the grout was not properly set to form the whole length. This could be related to the nature of the material and the wide range of the aggregate sizes that prevented the vacuuming to be performed properly.

The IBAA (1) might have penetrated into the clay or were contaminated by the surrounding clay and the grout could not fully penetrate into the column near the base. From the parts of the IBAA (1) columns extruded, it was observed that the column diameter was reducing with depth. The column of IBAA (1) that was loaded also showed a reduced diameter with the length. It was concluded that the IBAA (1) caused improper column formation during the installation due to its nature that could easily mix with the wet surrounding clay and prevented the proper compaction by the concrete poker. In the column of the IBAA (1) that was loaded, the bulging was observed but was less symmetrical all around the column.

The various lengths of the columns observed were results of improper grout penetration and lack of complete column shape formation after the grout was set. This happened near the base where the grout could not always penetrate easily. Also, the material itself can penetrate into the clay and cause various columns diameters to be formed.

It was observed that for some of the columns, the material type (IBAA (1)) prevented the proper grout penetration near the base of the cell and the fine nature of the material prevented the grout setting procedure. The researcher could not extract the full column length from the container in the columns of the IBAA (1) as the grout did not penetrate the base and the column was loose and could not be extracted.

8.3 Results and discussions of Series 2- The effect of installation energy

In series 2, only the granite was modelled in the SUC as a single column due to lack of sufficient RA sources available. Procedures of the preparations for each of the test were explained in chapter 6 (see sections 6.8 and 6.11).

7 tests were performed in this series to compare the effects of installation time on the crushing and load carrying capacity of the columns.

In all the other LUC and SUC tests, the usual installation time of 20 seconds compaction per layer of the aggregates was used. In the second series of the tests this time was changed to 10, 30 and 90 seconds per layer of aggregates.

The density of columns constructed was recorded. Also, the columns were loaded quickly after the installation to compare the load carrying capacity of various columns. The Shapes of the columns were observed via the grouting method to understand the effects of installation time on the performance of the columns of granite.

Not enough quantities of the CC/CB and IBAAAs were available for the modelling of various installations in the SUC.

The loading procedure was similar to the other SUC tests, where an axial plate was used over the column.

Various aspects of the results were compared in the following sections:

8.3.1 Quality control of the host ground

Similar to series 1, the quality control tests of the host ground were performed during the installation (the three moisture content samples from the core) and after the unloading (during the cleaning of layers).

The host ground was reused from the LUC tests. The quality control tests included the moisture content and the undrained shear tests using the hand vane performed after each of the SUC test.

Table 8.3 summarizes the range of the moisture content and the undrained strength values obtained with accuracies of 0.01(%) and (± 2) kPa, respectively.

Table 8.3: Quality control of the host ground-series 2

Test name	Test number	Moisture content range after the test (%)	Moisture content of the core extruded for column installation (%)	Undrained strength of the host ground after the test (kPa)
20 second installation	17	38-39	39-40	23-29
30 second installation	18	38-40	39-40	17-24
10 second installation	19	37-40	39-40	18-22
90 second installation	20	36-39	36-39	21-24
90 second installation-repeat	21	37-39	36-38	24-27
10 second installation-repeat	22	38-40	38-40	19-24
30 second installation-repeat	23	37-39	37-40	20-23

As observed in Table 8.3, as the soil was reused from the LUC tests, similar to series 1, slightly lower moisture content values resulted in the increase in the undrained strength of the soil. The range was still acceptable for the construction of the columns in the SUC.

8.3.2 Quality control of the column material

The particle size distribution (PSD) and density of the columns constructed were used in interpretation of the behavior of various columns constructed in series 2.

The PSD was performed before the installation and after the loading. The crushing of the materials due to the installation and loading could be compared to the PSD before each test to study effects of the various installation times on the same material.

The angle of shearing resistance of the material was measured via the shear box test and 47 degrees was obtained for the granite.

The dry granite was used in all tests in series 2. The density of the columns constructed were estimated and recorded based on the quantity of the material used and the approximate volume of the column.

Table 8.4 shows the results of the column density estimation for all the columns constructed in series 2 of the SUC tests.

Table 8.4: Densities of the columns constructed in the small unit cell-series 2

Test name	Test number	Column density (kg/m^3)
20 second installation	17	1781.56
30 second installation	18	1731.25
10 second installation	19	1515.38
90 second installation	20	1908.84
90 second installation-repeat	21	1760.39
10 second installation-repeat	22	1693.19
30 second installation-repeat	23	1686.74

It was observed in Table 8.4 that a slight variation existed for the 90 second installation between the test and the repeat. It was recorded by the researcher that during the installation of the repeat test, less effort was utilized to compact the layers of the aggregates and the results could be considered as an error in the installation process.

The Other times of installation used in the tests and the repeats show very similar densities achieved in the columns. It was also observed that the increase in time of installation increased the column density.

For the 30 and 10 second installations, the results of the column densities were very similar to the 20 second installation; however, the 90 second compaction per layer of the aggregates had more impact on the column density achieved.

8.3.3 Particle size distribution

The PSD was compared before the granite was used for each test. After unloading, the aggregates were vacuumed out and subject to further PSD. In the 90 second installation tests (tests 20 and 21), the columns were constructed under higher level of energy; therefore, during vacuuming, the aggregates were taken out in 4 sections separately from the top, the middle top, the middle base and the base. The PSD was performed separately on each section to study if the aggregate crushing was more concentrated in a specific part of the column. However, the results were very similar and this method of the PSD was not carried out for the other tests.

The average PSD curves of the granite before and after each test were presented in Figure 8.19. Figure 8.19 showed that the crushing of granite after these tests was minimal. This might be related to the nature of aggregate and as a primary source, the granite had high strength and high angle of shearing resistance that prevented the crushing of the material via various methods of installations used in this research.

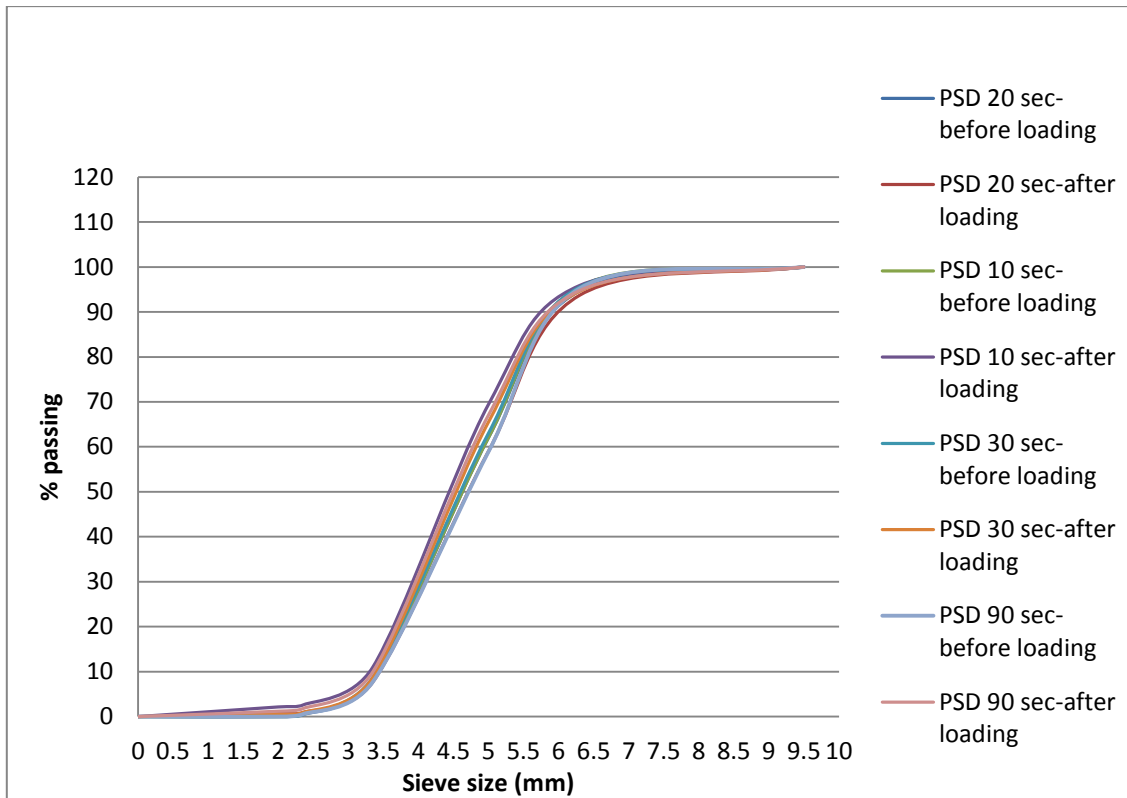


Figure 8.19: PSD of the granite before and after the tests, for the 10, 20, 30 and 90 seconds of compaction during installations

Details of the comparisons of tests are presented in Appendix 7(refer to CD).

8.3.4 Loading of the columns in series 2

Columns of the granite were loaded quickly after the installation, and the results of the stress-strain behavior were shown in Figure 8.20. It was observed that increasing the time of installation increased the column density and ultimately the load-carrying capacity of the column and host ground.

It was also observed that changing the time of installation from 20 to 30 seconds per layer did not have a significant impact on the stress strain behavior of the column.

On the other hand, decrease of the time to 10 or increase to 90 seconds affected the column behavior dramatically. Increasing the time from 30 seconds to three times its value increased the stress values by up to 30% at specific strains. Also, only 10 seconds reduction in the time of installation changed the level of improvement in the stresses from 60% to 40%.

If a 1.3% strain was considered as the failure point, even the 10 second installation of the column of granite improved the stress-strain behavior significantly; however, the higher installation time caused the column to outperform the others in terms of the stress-strain behaviour.

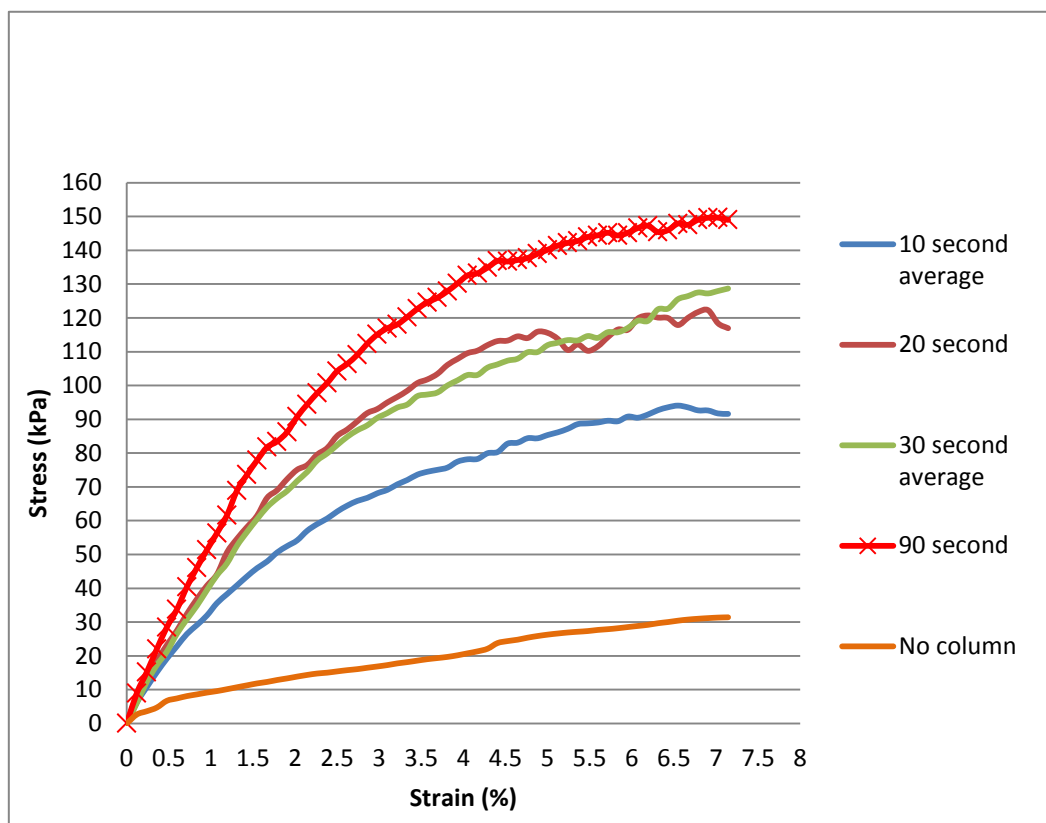


Figure 8.20: The stress-strain behaviour of the columns of the granite constructed under various installation times

The effects of the change of time of installation on the performance of VSC cannot be easily interpreted. Increase in the time of vibration could cause more aggregate crushing during the installation, especially in case of the weaker sources.

Also, the increase in the time of installation can result in higher density of the columns achieved and the need for more material to be used in the column construction and can increase the costs of projects.

Finally, other effects of over-treatment such as ground heave should be considered in estimation of the density of column and the stress-strain behavior under various loads. Heave can cause severe damage to the neighboring structures (McCabe *et al.*, 2013).

8.3.5 Shape of the columns

The shapes of columns constructed were investigated after each test in series 2. The column shape was compared for the columns constructed via the concrete poker using the times of compaction of 10, 20, 30 and 90 seconds per layer of aggregate.

Figure 8.21 shows the columns of granite, compacted by the concrete poker at various levels of energy:

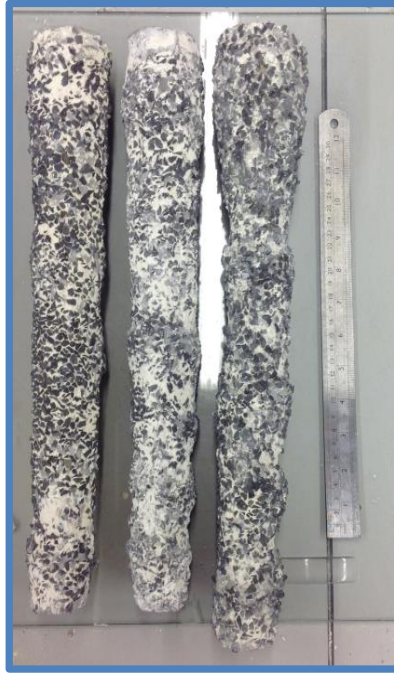


Figure 8.21: Column shapes in series 2, from left to right: 10, 30 and 90 seconds of compaction per layers

It was observed that the column diameter increased as the time of installation per layer increased. The 90 second installation time created a column with the significant difference in the diameter and length compared to the other two columns. The steps of installation and bulging were more apparent in this column. Sharp edges showed higher level of penetration of the material into the host ground during the installation.

On the other hand, the columns constructed using the 10 and 30 seconds of compaction were very similar in the diameter and length. Due to the loose column formation in the 10 second of compaction, more deformations were observed under the area of bulging. The shape of this column confirmed its low bearing capacity.

8.4 Results and discussions of Series 3- The contamination with fines

In the last four tests in the SUC, the effects of the contamination of aggregates with fines on the performance of VSC were modelled. Due to the limited sources of the RAs,

only the granite was used in three tests. List of the tests was presented in chapter 6 (refer to Table 6.4), followed by the descriptions of each of the test (refer to section 6.11).

Similar to the other two SUC tests, the axial loading was applied over the single stone column. The columns were installed and quickly loaded. The column material was granite which was replaced by 10 or 20% crushed granite.

In order to provide the fines, granite was crushed in the LA machine and a range of fines was provided to be added to the original PSD of 2 to 9.5mm.

Various aspects of the results were compared in sections 8.4.1 to 8.4.4:

8.4.1 Quality control of the host ground

Similar to series 1 and 2, the host ground was controlled by the moisture content and the undrained strength values. The three samples of the moisture content were taken during installation and also, the clay which was reused from the LUC tests was subject to the moisture content and the hand vane shear tests after the test finished.

The range of the values obtained was presented in Table 8.5. The errors of 0.01(%) and (± 2) kPa existed for the moisture content and the undrained shear strength values, respectively.

Table 8.5: Quality control of the host ground-series 3

Test name	Test number	Moisture content range after the test (%)	Moisture content of the core extruded for column installation (%)	Undrained strength of the host ground after the test (kPa)
10% fines contamination	24	36-41	39-43	19-22
20% fines contamination	25	39-41	40-42	17-21
10% fines contamination-repeat	26	37-41	39-41	18-23
20% fiens contamination-repeat	27	37-40	38-40	19-22

According to Table 8.5, the range of the moisture contents and the undrained strength values were suitable for the column installation despite that the host ground was reused from the LUC tests.

8.4.2 Quality control of the column material

Densities of the columns constructed were used in interpretation of the behaviour of the various columns loaded in series 3 of tests. The angle of shearing resistance of the material was 47 degrees based on the shear box tests (refer to section 5.5.6.4) and the same material (dry granite) was used in all of the four tests performed.

The PSD was performed before installation on the granite ranging between 2 to 9.5mm. Separate PSD was performed on the crushed granite before it was added to the original material used in the tests.

The PSD results of the crushed granite were presented in Figure 8.22 where it was observed that the crushed material covered a range of sizes between 1.18 mm and 63 μm . The crushed material was used to replace 10 and 20% of the granite prepared for the installation in the four tests of series 3.

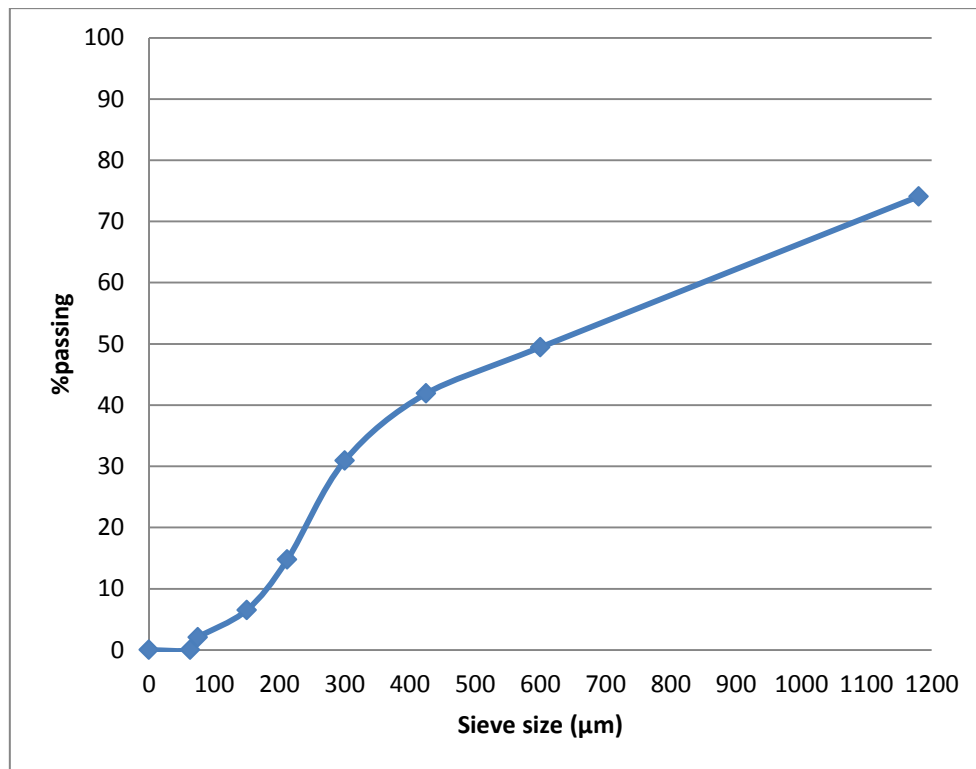


Figure 8.22: PSD of the crushed granite used for series 3 of the columns in the SUC tests

Density of the columns constructed were estimated and recorded based on the quantity of material used and the approximate volume of the columns.

Table 8.6 shows the results of the column density estimation for all the columns constructed in series 3 of the SUC tests.

Table 8.6: Densities of the columns constructed in the small unit cell-series 3

Test name	Test number	Column density (kg/m^3)
10% fines contamination	24	1817.60
20% fines contamination	25	1733.60
10% fines contamination-repeat	26	1666.31
20% fines contamination-repeat	27	1806.98

Table 8.6 showed the densities of the columns in two tests of 10 and 20% contamination with fines and the repeat tests. Slight error in the densities was observed which was mainly due to the errors of the installation process. Apart from the installation method, existence of fines affected the behaviour of the column since the installation started. Fines could easily penetrate into the column and stick to the surrounding clay and therefore, affect the ultimate density achieved. The results of the load carrying capacity of the columns were compared and the percentage of fines and the densities achieved were the critical factors in understanding the column behaviour.

8.4.3 Loading of the columns in series 3

After the clay preparation, the aggregate was prepared where the granular granite was mixed with the crushed granite. The installation commenced and the columns were quickly loaded after the installation under the axial plate. The stress- strain behavior of the granite with 0, 10 and 20% fines was compared in Figure 8.23.

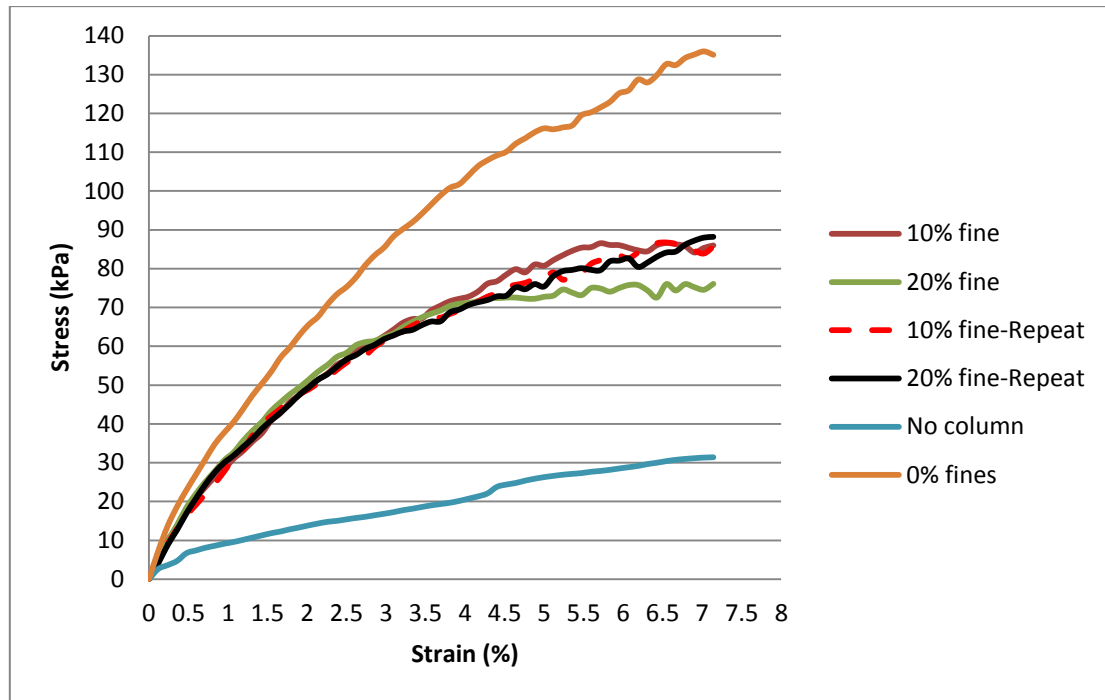


Figure 8.23: Comparison of the columns of granite contaminated by 0, 10 and 20% fines

Firstly, it was observed that the construction of the stone column regardless of its contamination with fines improved the load-settlement behavior significantly compared to the no column test by approximately 40% at the failure strain of 1.3%.

However, the columns in which the aggregates were contaminated by even 10% fines performed poorly compared to the 0% contaminated column due to the change in the angle of shearing resistance of the material used to form the column.

It was also concluded that the 10 and 20% contamination had similar effects on the stress-strain behavior of the columns at the lower strains, although, the column contaminated with 20% fines had slightly lower stress values at each strain.

In the initial part of the curves at the lower strains, the 10 and 20% fines were performing similarly, but under higher stresses the difference becomes more apparent.

It can be concluded that the addition of fines (even 10%) can reduce the load carrying capacity of the composite (the column and the host ground). The best improvement was achieved when only granular granite was used in the column formation

A well-graded material can form a better packed column and carry higher loads, however, addition of dust or powdered fines can reduce the angle of shearing resistance of the column and reduce the load carrying capacity.

In this research only the 10 and 20% fines were compare; whereas addition of more than 20% fines might affect the load carrying capacity up to the point that the existence of the fines would be redundant. Also, the addition of the fines can block the drainage provided by the stone columns and cause long –term settlements in the ground.

The study by McKelvey *et al.*, (2002) investigated the effects of adding 10 and 20% clay slurry to primary and recycled aggregates. The material source in this research was different from the aggregate sources tested in that study and also, the clay slurry could have various effects on the column material.

In this research the crushed granite was added to the granite to avoid the complications of interpreting the results of the effects of another component on the granite. In the tests performed on the PA and the RA by McKelvey *et al.*, (2002); the angle of shearing resistance of all the materials were reduced by over 10% due to the addition of clay slurry. In this research the angle of shearing resistance was not studied under the condition of the contamination of aggregates with fines, however, the stress-strain behaviours showed poor results of the load-settlement behaviour.

Based on the results of this research and previous published work, the storage and transportation of the aggregates should be carefully considered before the use of

material in the VSC construction, as the addition of fines due to storage, transportation, wind and flood can result in poor performance of the column when loaded (McKelvey *et al.*, 2002).

8.4.4 Shape of the columns

The shape of the columns was investigated after the loading in series 3 of the SUC tests.

Figure 8.24 shows the shapes of the columns constructed with 10 and 20% fines.



Figure 8.24: Columns contaminated with fines, left to right: the granite contaminated by 10% fines, the granite contaminated by 20% fines

It was observed that in both of the tests the column diameter was variable along the length of the column due to the existence of the fines that penetrated into the surrounding clay and also prevented the grout to stick the aggregates together.

Also, the bulging area and the deformations were different. In case of the higher level of contamination with fines (20%), the column was deformed more significantly under the

similar loading conditions compared to the column with the lower percentage of fines (10%).

In both of the columns contaminated by 10 and 20% fines, the stages of installation (the stages where the aggregates were poured and compacted at each layer) were observed. The deformations due to the loading of the columns can be observed near the top parts which were the bulging areas under similar static loading. The addition of fines caused bigger area of bulging with a bigger diameter which meant more deformations and lower load carrying capacity.

8.5 Evaluation of the SUC tests results

8.5.1 Errors in the small unit cell tests

The errors of the measurements and analysis were related to the various stages of the preparation, the column installation method, the loading and the methods of measurements.

For the host ground preparations, slight loss of the moisture content happened as the soil was reused in all the SUC tests, from the LUC container. In order to make sure the soil had the undrained strength of 10 to 25kPa, the moisture content and the vane shear tests were performed.

During the installation phase, the forces exerted by the concrete poker caused various columns to be constructed with variable shapes and densities.

The Loading of the columns in all of the SUC tests was performed via an axial plate which could not be compared to the LUC results due to the variations in the stress-strain behaviour under these two types of loading. However, the same method was used in all

the SUC tests to enable the researcher to compare the results of all of the tests performed in the SUC container.

8.5.2 Comparison and repeats

Tests performed were repeated once on the granite, the CC/CB and the IBAA (1) in series 1. The procedure was exactly explained for each test (refer to section 6.11), so the tests can be reproduced, however, source of material is unique for each project and the properties of aggregates may vary and cause different results under the same conditions.

In series 2 and 3, only granite was used to model the effects of installation and contamination of the aggregates with fines. Not enough sources of the RAs were available for these tests, but the tests on the granite were repeated once and compared. The results of the repeats in all the SUC tests were very close with small error margins. The errors encountered were related to the quality of workmanship during the installation of the columns.

Other published work on aggregates contaminated with fines could not be directly compared to the material tests in series 3 as in this research the material was contaminated by the crushed granite and also the loading condition to estimate the aggregates behavior was different (McKevey et al., 2002).

8.6 Summary of the SUC tests results

The main findings of the 27 SUC tests results were summarized below:

- 1) In series 1, the quality control tests on the host ground proved that the moisture content and the undrained shear strength required for the VSC modelling in the SUC container was slightly higher than the LUC tests; however, the ranges obtained were still acceptable for the construction of the columns.

- 2) In series 1, the quality control tests on the aggregates showed that the materials used for the column formation (the granite, the CC/CB, and the IBAA (1)) had various PSDs which resulted in various degrees of packing of the materials in the columns and the various densities obtained. The RAs used in this research were well-graded compared to the more uniformly graded granite.
- 3) In series 1, the PSD before and after the installation was compared for the three aggregates tested and it was concluded that during the installation stage, the CC/CB crushed more than the IBAA (1) and the level of the crushing of the granite during the loading was minimal which was related to its strength, the angle of shearing resistance and the fact that the RAs were already crushed and sieved to provide the right range for the SUC tests which could have affected their hardness.
- 4) In series 1, the PSD was compared before the installation and after the loading; it was observed that the RAs crushed more during the loading compared to the granite; with the most crushing observed for the CC/CB.
- 5) In series 1, the level of crushing was compared for all the three aggregates tested at stages of the installation and the loading. Apart from the granite which had negligible crushing at both stages, the recycled aggregates crushed more during the installation compared to the loading. It was possible that the material was better packed under the loading and the dense column prevented further crushing of the particles throughout the loading.
- 6) In series 1, when the single columns were loaded under the axial plate, similar to the LUC tests results, the columns of the RAs outperformed the column of the granite in the load carrying capacity due to their well-graded PSD and better packing of the columns in the host ground.

- 7) In series 1, all of the columns constructed improved the load carrying capacity of the ground significantly by at least 120%. However, at the lower strains the RAs outperformed the granite, whereas, at the higher strains; the granite had higher load carrying capacity compared to the RAs. But the higher strains were beyond the failure of the columns.
- 8) In series 1, the shapes of the columns showed that the diameter of the column reduced with the depth and the bulging was observed in the columns that were loaded. Also, the IBAA (1) column formation was incomplete as the smaller particles penetrated into the surrounding clay and prevented the grout to set and form the column.
- 9) In series 2, the increase in the time of installation on the charges of the granite caused higher column densities to be obtained. The increase in the densities increased the load carrying capacity of the columns. However, higher density meant more quantities of aggregates to be used which can lead to uneconomical construction and over-treatment that can cause ground heave. Increasing the time of the vibrations can cause more crushing of the aggregates and change in the angle of shearing resistance.
- 10) In series 2, the increase in the time of installation per layer of aggregates from 20 to 30 seconds did not create a significant change in the load carrying capacity. However, the increase in the time from 30 to 90 seconds increased the load carrying capacity at least 3 times. On the other hand, the reduction of the time from 20 to 10 seconds caused the level of improvement in the ground to be reduced from 60 to 40%.

- 11) In series 2, the shape of the columns showed that the 10 second compacted column was loosely formed and went through higher levels of deformations during the loading.
- 12) In series 3, fines were added to the granite and addition of fines affected the installation procedure and variable densities in the columns achieved.
- 13) In series 3, the addition of 10 and 20% fines affected the load carrying capacity of the column significantly. Even 10% addition of fines caused up to 75% reduction in the stresses at specific strains. On the other hand, 10 and 20% fines created similar columns in terms of the load carrying capacity.
- 14) In series 3, the addition of fines caused more bulging and deformations in the columns loaded and the column contaminated with 20% fines showed more deformations compared to the 10% contaminated column.

The findings showed that despite the various results of the aggregate index tests, the RAs can be used in the context of the VSCs. The crushing of the aggregates during the installation can affect the behavior of the column more than during the loading. Also, the contamination of the column material with fines can significantly reduce the performance of the stone columns under static loading. The time of the installation for each layer of aggregates should be sufficient to compact them enough; at the same time should not damage the aggregates by crushing or affecting the treated by over-treatment.

CHAPTER NINE

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

9 CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

In this chapter the main findings of the aggregate index tests results (chapter 5), the LUC tests results (chapter 7) and the results of the three series of tests performed in the SUC were summarized. The findings were related to the main aim of this research presented in chapter 1.

In order to improve the tests performed in this research, recommendations were provided for future research.

9.1 Research aim and the main findings

Traditionally natural sources of aggregates were used as the column material in the construction of VSCs. In recent years, the use of alternative aggregates (AA) has been encouraged in geotechnical engineering due to sustainability reasons and the PA sources becoming scarce (Jefferson *et al.*, 2010).

On the other hand, there are certain barriers against the use of AAs in the practice of VSCs:

- 1) Lack of reliable sources or lack of records regarding the quality and strength of the materials can prevent the engineers from use of AAs in the design and construction of VSCs.
- 2) The tests introduced by the standards are mainly index tests that do not represent the installation and loading conditions of the aggregates used for the construction of the VSCs (ICE, 1987; BRE, 2000).
- 3) The recommendations are not clear regarding distinguishable criteria for primary and AAs and specific index tests for each category.
- 4) The effects of the use of AAs in the long-term, under various loads applied to the VSCs are still unknown.

In previous research, the aggregate index tests were used on various primary and alternative aggregates to understand the aggregate properties such as the hardness, the angle of shearing resistance and the porosity (Chidioglou *et al.*, 2009; McKelvey *et al.*, 2004; Steele, 2004; Schouenborg, 2005).

The index tests did not consider the unique conditions of the installation process and loading of the aggregates in the context of the VSCs.

Other previous research tested a single or column groups under various installation and loading conditions. However, in most of these tests the actual aggregates were not used. Sand or gravel or in fewer cases only primary aggregates were modelled in the installation and loading of the VSCs (Hughes and Withers, 1974; Barksdale and Bachus, 1983; Black *et al.*, 2007).

In this research, three recycled (CC/CB, IBAA (1) and IBAA (2)) and one primary (granite) aggregates were selected for laboratory testing. The laboratory testing of the stone columns provided controllable and repeatable conditions of column installation and loading under which various aspects of the performance of the VSCs was studied.

Instead of sand or gravel or only PAs, for the first time the actual recycled sources were used in the installation and loading of a single stone column and the behavior of these aggregates was compared in the actual context of the VSC.

The aggregate index tests recommended by the standards were performed on all the PA and RAs. The results showed that in most of the aggregate index tests (ACV, TFV and LA tests) the RAs performed poorly or marginal and based on the aggregate index tests criteria they could not be used for the construction of VSCs.

However, in this research the validity and relevance of these tests regarding the performance of the VSC was studied via two sets of the LUC and the SUC tests.

In these tests the short-term behaviour (with the exception of test 15 in the LUC) of the single stone column was compared for the primary and the three recycled aggregates.

It was concluded that despite unacceptable results in the index tests, the RAs perform satisfactorily in the context of the stone column and also, outperformed the PA (granite)

in the stress-strain comparison under similar installation and loading at the lower strains before the settlement failure of the single column happened.

The PSD and its range were found to be one of the most important factors affecting the column density and formation and ultimately the load carrying capacity of the column.

The condition of the aggregates (wet/dry) was another important factor that affected the load carrying capacity and the short-term performance of the single columns modelled in this research.

The findings of the aggregate index tests, the SUC and the LUC were presented in sections 9.2 to 9.4.

9.2 Conclusions-The aggregate index tests

The PSD, the shear box test, ACV, AIV, The LA and TFV were the tests performed on the granite and the three recycled aggregates (CC/CB, IBAA (1) and IBAA (2)).

- 1) The shear box test showed the angle of shearing resistance of 47 degrees for the granite and angles of shearing resistance between 40 to 41 degrees for the three RAs. All these results were in the range acceptable for the material used in the practice of the VSCs in the UK (Serridge, 2005).
- 2) The RAs used in this research were crushed and sieved to provide the range between 2 to 9.5 mm. The granite was supplied within this range. The range was selected based on the scaling of the stone column size and the boundary conditions in the LUC and the SUC.
- 3) The PSD performed on the aggregates showed that the granite was supplied with a uniformly graded range. Whereas, the RAs had a well-graded PSD.

- 4) The AIV showed that all the primary and recycled aggregates sustained impact forces of this test. However, the IBAA (1) performed worse than the rest followed by IBAA (2), the CC/CB and the granite. The nature of the IBAA (1) which consisted of glass and ceramic pieces affected its performance.
- 5) The ACV showed that all the aggregates used in this research including the granite used in the large and small unit cell modelling were unsuitable under prolonged loads.
- 6) The TFV test showed that all the three RAs were unsuitable for the use in the VSC construction and only the granite performed satisfactorily.
- 7) The LA test is only recommended by ICE (1987) and its results were not used in the analysis of the unit cell modelling. All the three recycled aggregates failed the criteria of these tests; however, the condition of this test cannot be compared to the condition of aggregates under the installation and loading of the VSCs.
- 8) In the AIV, ACV, TFV and the LA tests, the CC/CB outperformed the IBAA's.

9.3 Conclusions-The LUC tests

The LUC was used to model the installation process and the loading of a single stone column using the primary and the three RAs.

The columns were installed under similar conditions using a dry top-feed method where the aggregates were charged and compacted for 20 seconds per layer via a concrete poker.

The strain-controlled loading was applied using a foundation plate over the single column.

15 tests were performed; 14 of which were short-term tests where the column was installed and quickly loaded. Only the last test was a long-term test, where the column was constructed but left for three months before the loading.

The stress-strain behaviour of the column and the surrounding soil and the water level changes at the base of the column and the various depths and radii from the column centre (measured in the partially saturated clay) were compared for the columns of primary and RAs.

The main findings of the 15 LUC tests results were summarized below:

- 1) The quality control tests before and after each of the LUC tests on the host ground proved that the required moisture content range (38 to 42%) and the undrained shear strength (10-25 kPa) for the VSC modelling in soft Kaolin was achieved for all the 15 tests.
- 2) The quality control tests on the aggregates showed that the materials used for the column formation (the granite, CC/CB, IBAA (1) and IBAA (2)) had various PSDs. The RAs used in this research were well-graded compared to the more uniformly graded granite. The range of the densities estimated from the columns formed in the unit cell (1200 to 1900 kg/m^3) showed that the various PSDs and the nature and the shape of the aggregates created columns of various densities under the same installation methods.
- 3) The installation process and the vibrations exerted on the same type of aggregates caused columns of various densities to be formed. Therefore, it was concluded that the quality of workmanship in quantities of the material charged and the level of vibrations can affect the densities achieved and the ultimate load carrying capacities of the columns constructed.

- 4) When the columns were loaded, the foundation loading caused lower stress distributions on the column and the surrounding clay compared to the axial plate at each specific strain. At the failure point of 1.5% of strain, the axial plate applied three times the stress applied by the foundation load on the ground. As the foundation loading was more comparable to the practice of the VSCs, it was used in the rest of the LUC tests.
- 5) All the constructed columns (regardless of the type of the aggregates used) improved the load carrying capacity of the host ground significantly by at least 80% proving that even the RAs can be used in the practice of the VSCs and improve the bearing capacity and the settlement of the host ground.
- 6) The column of the IBAA (2) improved the load carrying capacity of the composite (the column and the clay) more than the other columns of the PA and the RAs by at least 180%. Despite showing poor results compared to the granite in the aggregate index test, the well-graded PSD and the nature and ash matrix of the IBAA (2) resulted in better packing of the column material and prevented its breakage and column deformation under prolonged static loading of the short-term test in the LUC.
- 7) The columns of the RAs outperformed the granite in the stress-strain behaviour tested. The IBAA (2) outperformed all the other materials, followed by the IBAA (1) (at the lower failure strain of 1.5%) and the CC/CB (at the higher failure strain of 4.5%). The granite used showed lower stresses at each strain compared to the RAs however; at the higher strains (beyond a strain of 10% and the failure of the column) the granite seemed to outperform the other materials.
- 8) Apart from the PSD and its range, the most important factor affecting the load carrying capacity was the condition of the aggregates (wet/dry). The wet

aggregate columns had lower load carrying capacity compared to the dry columns by a maximum of approximately 10% at strains of 10%.

- 9) The only long-term test on the granite (test 15) showed that the long-term column left in the ground before the loading, absorbed the water from the surrounding soil and due to its changed condition reduced the load carrying capacity of the column by approximately 20%, similar to the weaker wet aggregate columns tested.
- 10) As opposed to the dry aggregate tests, where the RAs outperformed the granite; in the wet tests performed on the CC/CB and compared to the wet granite; the wet RA performed poorly compared to the wet PA by approximately 5%. This concluded that the RAs might be more sensitive towards the condition (wet/dry) and when used under the ground water level, the type of the material and its behaviour under the influence of the water should be considered in the material selection and the design of the VSCs.
- 11) The settlement of the columns was both estimated using the Priebe's method and also measured in the actual tests performed in the LUC tests. The results showed that the Priebe's method was highly conservative for both the columns of the PA and the RAs. For the granite, at the failure strains of 1.5 and 4.5%, the actual improvement in the settlement behaviour was 200 and 90%, respectively compared to the Priebe's prediction.
- 12) In case of the settlement estimation of the RAs, as the materials tested in this research outperformed the granite in terms of deformations at each specific strain, the Priebe's prediction was even more conservative by approximately 140%.

- 13) It was concluded that the Priebe's method is conservative even for the RAs with a lower angle of shearing resistance compared to the granite and the assumptions of the Priebe's method such as the compressibility ratio of the column to the surrounding soil could be improved to provide more realistic settlement estimations for the columns of the RAs. Also, the RAs can be confidently used for the construction of the VSCs to improve the settlement of the ground if the angle of shearing resistance is known. The shear box test is recommended to obtain this parameter.
- 14) The water level changes measured in the partially saturated clay of the LUC tests showed that the surrounding soil changed since the installation of the stone columns started especially in the area of the bulging which confirms that the column installation causes pressure changes in the surrounding soil both during the installation of the columns and the loading.
- 15) More water was transferred through the column (as a granular material) compared to the surrounding soil at both stages of the installation of the column and the loading. In other words, the vertical water dissipation was more than the radial dissipation rate.
- 16) During the installation, as the column was formed from the bottom towards the surface, the water level changes and fluctuation were more significant at the level of aggregate compaction via the concrete poker. In the beginning more water level change was observed at the base and as the installation progressed the water level at the base became steady. This confirmed the previous field measurements by Castro and Sagasetta (2012). The other piezometers in the surrounding soil showed similar behaviour; however, the water level changes at the base were up to 9 times the quantities of the water level changes at the

piezometers in the surrounding clay. This was because the column was granular and had higher permeability than the clay.

- 17) Throughout the entire loading, the water levels fluctuated at the base of the column and also most apparently at the level of bulging of the column which showed the stress changes in the surrounding soil and the column being compressed during the loading.
- 18) During the loading of the columns, the CC/CB column absorbed the water from the surrounding clay due to its nature and showed up to 5 times more changes in the water level at this stage compared to the other columns of the PA and the RAs.
- 19) The findings showed that despite the various results of the aggregate index tests, the aggregates behave differently in the context of VSCs and the aggregate index tests alone are not enough to predict the suitability of the various aggregates for the use in the installation and loading of the VSCs. Therefore, the study of the materials in the context of installation and loading of the VSC is required for comprehensive understanding of the primary and the recycled aggregates when used in the VSC construction.
- 20) The most important tests based on this research are the PSD and its range, the shear box test (for the angle of shearing resistance) and field testing of the RAs in the stone column installation and loading before a RA is selected for the design and construction of the VSC. The condition of the aggregates (wet/dry) affects the performance of the VSCs in the short-term and should be considered in the design and construction.

9.4 Conclusions-The SUC tests

The SUC was used to study the following factors:

- The effects of the installation forces on the crushing of the aggregates versus the effects of the loading on the primary (granite) and RAs (CC/CB and IBAA (1)) (series 1)
- The effects of the installation time (energy) on the columns of granite formed and their density, the level of crushing and their load carrying capacity (series 2)
- The effects of contamination of the column of granite with powdered fines on their load carrying capacity (series 3)

Similar to the LUC, a single column was installed under the similar dry top-feed method (except for series 2 tests which had installation times of 10, 20, 30 and 90 seconds per layer as opposed to all the other LUC and SUC tests with the installation time of 20 seconds per layer) and loaded under the static loads. However, as opposed to the LUC tests the plate used for all the SUC tests was the axial plate.

27 tests were performed and the densities of the columns, the stress-strain behaviour of the column and the surrounding soil and the change in the PSDs of the materials were among the most important measurements in the SUC tests.

The main findings of the 27 SUC tests results were summarized below:

- 1) In series 1, the quality control tests on the host ground proved that the acceptable moisture content range (38-42%) and the undrained shear strength (10-25 kPa) required for the VSC modelling in the SUC container were achieved; however, the values were slightly higher than the LUC tests as the Kaolin was reused from

the LUC tests and slight loss of the moisture content and therefore increase in the undrained shear strength values were inevitable.

- 2) In series 1, the quality control tests on the aggregates showed that the materials used for the column formation (the granite, the CC/CB, and the IBAA (1)) had various PSDs which resulted in various degrees of packing of the materials in the columns during the installation and the various densities obtained. The RAs used in this research were well-graded compared to the more uniformly graded granite.
- 3) In series 1, the PSD before and after the installation was compared for the three aggregates tested and it was concluded that during the installation stage, the CC/CB crushed more than the IBAA (1) (by a maximum of approximately 5%) and the level of the crushing of the granite during the loading was minimal which was related to its strength, the angle of shearing resistance and the fact that the RAs were already crushed and sieved to provide the right range for the SUC tests which might have affected their hardness.
- 4) In series 1, the PSD was compared before the installation and after the loading for the granite, the CC/CB and the IBAA (1); it was observed that the RAs crushed slightly more during the loading compared to the granite (by approximately 2%); with the most crushing observed for the CC/CB. The nature of the IBAA (1) held the material together under the vibrational forces of the installation and the sustained loads of the axial plate. The brick in the CC/CB was not as hard as the other materials tested and therefore, cause more change in the PSD changes of the CC/CB compared to the other aggregates tested.
- 5) In series 1, the level of crushing was compared for all the three aggregates tested at the two stages of installation and loading. Apart from the granite which had

negligible crushing at both of the stages, the recycled aggregates crushed more (by a maximum of approximately 5%) during the installation compared to the loading. It was possible that the material was better packed under the loading and the dense column prevented further crushing of the particles throughout the loading.

- 6) It was concluded that the material source can go through crushing even before the column is loaded and therefore, the effects of the installation forces on the crushing of the RAs should be considered in the design and construction of the VSCs especially when the RA sources are considered.
- 7) In series 1, when the single columns were loaded under the axial plate, similar to the LUC tests results, all the columns regardless of the primary or recycled aggregates being used in their construction improved the load carrying capacity of the host ground by at least 120%.
- 8) In series 1, under the axial loading, similar to the LUC tests, the columns of the RAs outperformed the column of the granite in the load carrying capacity by more than 30% due to their well-graded PSD and better packing of the columns in the host ground.
- 9) In series 1, all the columns constructed improved the load carrying capacity of the ground significantly by at least 120%. However, at the lower strains the RAs outperformed the granite, whereas, at the higher strains (above the failure of the columns); the granite had higher load carrying capacity compared to the RAs.
- 10) In series 1, the shapes of the columns showed that the diameter of the column reduced with the depth as the columns were not properly formed due to the existence of the finer particles in the RAs that penetrated into the surrounding clay. Also, the nature of the IBAA (1) prevented the grot penetrating into the

base of the column and the shape was not observed. It was concluded that the quality of workmanship is a critical factor to make sure that enough material is charged into the ground at depths and compacted properly via the vibro-float to make sure the designed column diameter and length are achieved.

- 11) In series 2, the increase in the time of installation from 20 to 90 seconds on the charges of the granite caused up to 10% higher column densities to be obtained. This increase in the densities increased the load carrying capacity of the columns by over 30%. However, the higher density means more quantities of the aggregates to be used which can lead to uneconomical construction and over-treatment that can cause ground heave.
- 12) In series 2, increasing the time of the vibrations caused more crushing of the aggregates by a maximum of approximately 5% in the granite. In practice of the VSCs, the same level of the crushing can change the angle of shearing resistance and affect the load carrying capacity and settlement of the VSCs in both the short and the long-term.
- 13) In series 2, the increase in the time of installation per layer of aggregates from 20 to 30 seconds did not create a significant change in the load carrying capacity. However, the increase in the time from 30 to 90 seconds increased the load carrying capacity by at least 3 times. On the other hand, the reduction of the time from 20 to 10 seconds caused the level of improvement in the ground to be reduced from 60 to 40%.
- 14) In series 2, the shape of the columns showed that the 10 second compacted column was loosely formed and went through higher levels of deformations during the loading. Therefore, similar to over-treatment that can negatively affect the performance of the VSCs, under-treatment can cause improper column

formation (the diameter and the length) and reduce the load carrying capacity and increase the settlements of the ground significantly. The quality of workmanship is the key in controlling the quantities of the aggregates charged and the level of compaction achieved at each layer of installation.

- 15) In series 3, powdered granite was added to the 2 to 9.5 mm granite and the addition of fines affected the installation procedure by the penetration of the fines into the surrounding clay and requiring more aggregates and as a result variable densities in the columns were achieved.
- 16) In series 3, the addition of 10 and 20% fines affected the load carrying capacity of the column significantly. Even 10% addition of fines caused up to 25% reduction in the stresses at the failure strain. On the other hand, 10 and 20% fines created similar columns in terms of the load carrying capacity at the strains below the failure of the columns. In practice addition of fines during the storage, the transportation and the installation should be avoided in order to achieve the designed load carrying capacity.
- 17) In series 3, the addition of fines caused more bulging and deformations in the columns loaded and the column contaminated with 20% fines showed more deformations compared to the 10% contaminated column.
- 18) The findings showed that despite the various results of the aggregate index tests, the RAs can be used in the context of the VSCs. The crushing of the aggregates during the installation can affect the behavior of the column more than during the loading depending on the properties of the aggregates. Also, the contamination of the column material with fines can significantly reduce the performance of the stone columns under static loading (by up to 30%). The time of the installation for each layer of aggregates should be sufficient to compact

them properly at the same time should not damage the aggregates by crushing or affect the ground by over-treatment.

9.5 The most important factors affecting the performance of the VSCs

In this research despite the poor aggregate index tests results of the RAs, the materials were modelled in the context of the installation and loading in a single column.

Various factors affected the performance of the single columns which were tested in the short-term.

The most important factors affecting the performance of the VSCs in the short-term which were found in this research were listed below:

- The PSD and its range: well-graded aggregates can form a dense column with a higher load-carrying capacity regardless of the type of the column material (primary or recycled)
- The condition of the aggregates: the wet condition weakens the materials in the column and reduces the load carrying capacity even in the short-term
- The crushing of the aggregates during the installation process: the energy of the installation can affect the particles and ultimately the load carrying capacity, at the same time over or under-treatment affect the performance of the columns in both the short and the long-term
- The addition of fines: even 10% fines added to the column material can reduce the load carrying capacity by 25% and therefore should be avoided for better performance of the VSCs.
- Other parameters such as the angle of shearing resistance and the aggregate index tests can assist the prediction and interpretation of the behaviour of

various primary and alternative aggregates in the context of the VSCs; *e.g.*, the AIV and the ACV can predict the behaviour of the aggregates under the loading, whereas, the TFV and The LA tests can assist in the prediction of the behaviour of the aggregates under the installation forces.

9.6 Recommendations for future research

The following variations from this research are recommend for the laboratory testing that can improve the aggregate index tests and the unit cell tests results obtained in this research:

- 1) Instead of the host ground used in the LUC tests (soft Kaolin), other problematic soils such as peat or collapsible soils in which the VSCs are constructed can be used in the modelling.
- 2) Due to the time constrains, the host ground was only compacted. But it can be consolidated for better quality of the host ground conditions (the moisture content, the degree of saturation and the undrained strength).
- 3) Various AA sources can be tested under the same installation and loading conditions of the LUC and the SUC tests. In this research only one primary and three recycled aggregates were used.
- 4) In this research the aggregates were formed into a column via the dry top-feed method of installation. The other methods of installation such as wet and bottom-feed installations can be used and compared for their effects on the various aggregates
- 5) Wet aggregate index tests are recommended to be included for the study of the durability and deterioration of the AAs; especially the wet shear box test

for the comparison of the angle of shearing resistance of the wet materials to be compared to the dry ones

- 6) In this research the TFV test was performed under the static loads; however, cyclic loading can provide better understanding of the aggregates behaviors under the installation vibrations and can be added to the current results of TFV obtained
- 7) In the SUC (series 2 and 3) due to lack of sufficient availability of the RAs, the tests were only performed on the granite. The same tests can be repeated for the RAs to be compared to the PA.
- 8) In the LUC due to the lack of time and materials, the wet tests were only compared for one type of the RAs with the granite. Other RAs should also be tested in the wet condition. The long-term test was only performed on the granite, and other RAs can also be tested long after the installation is completed.
- 9) In this research only the end-bearing columns were tested. End-bearing versus floating columns can be compared using the LUC or the SUC tests for comparison of the performance of both the PA and the RAs under the short-term static loading.
- 10) The long-term loading of the unit cell tests can provide the knowledge on the behaviour of the various aggregates in the long-term where the aggregates deterioration can affect the performance of the VSCs.

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Appendix 1: Results of host ground tests

1. China clay composition

Table 1: Technical data of English China clay of type Puroflo 50 provided by WBB

Devon Clays Ltd

Analysis	Results																				
Particle size distribution	Equivalent spherical diameter Microns: 1___2___5___10___20 % passing: 37 49 76 94 99																				
PH value	5.1																				
Mineralogical composition (derived from X-ray diffraction measurements and calculations based on chemical analysis)	<table> <tr> <th>Composition</th><th>Rational analysis</th></tr> <tr> <td>Kaolinite</td><td>64</td></tr> <tr> <td>Potash Mica</td><td>24</td></tr> <tr> <td>Soda Mica</td><td>2</td></tr> <tr> <td>Quartz</td><td>6</td></tr> </table>	Composition	Rational analysis	Kaolinite	64	Potash Mica	24	Soda Mica	2	Quartz	6										
Composition	Rational analysis																				
Kaolinite	64																				
Potash Mica	24																				
Soda Mica	2																				
Quartz	6																				
Chemical analysis	<table> <tr> <th colspan="2">Ultimate analysis (%)</th></tr> <tr> <td>SiO_2</td><td>48.8</td></tr> <tr> <td>TiO_2</td><td><0.1</td></tr> <tr> <td>Al_2O_3</td><td>35.4</td></tr> <tr> <td>Fe_2O_3</td><td>0.8</td></tr> <tr> <td>CaO</td><td>0.1</td></tr> <tr> <td>MgO</td><td>0.2</td></tr> <tr> <td>K_2O</td><td>2.8</td></tr> <tr> <td>Na_2O</td><td>0.2</td></tr> <tr> <td>Loss on ignition</td><td>11.4</td></tr> </table>	Ultimate analysis (%)		SiO_2	48.8	TiO_2	<0.1	Al_2O_3	35.4	Fe_2O_3	0.8	CaO	0.1	MgO	0.2	K_2O	2.8	Na_2O	0.2	Loss on ignition	11.4
Ultimate analysis (%)																					
SiO_2	48.8																				
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CaO	0.1																				
MgO	0.2																				
K_2O	2.8																				
Na_2O	0.2																				
Loss on ignition	11.4																				
Residue (measured by wet screening on a 35 μ mesh, equivalent to 300 BSS)	Average <0.1%																				
Surface area	8-10 m^2/g																				

2. Natural moisture content of clay

Table 2: Natural moisture content, sample 1

Container	Weight of container (m_1)	Weight of container and wet soil (m_2)	Weight of container and dry soil (m_3)	$m_2 - m_3$	$m_3 - m_1$	$w = \frac{m_2 - m_3}{m_3 - m_1} (\%)$
A	5.84	35.96	35.80	0.16	29.96	0.53
B	5.72	35.64	35.48	0.16	29.76	0.54
C	5.78	35.70	35.54	0.16	29.76	0.54

Table 3: Natural moisture content, sample 2

Container	Weight of container (m_1)	Weight of container and wet soil (m_2)	Weight of container and dry soil (m_3)	$m_2 - m_3$	$m_3 - m_1$	$w = \frac{m_2 - m_3}{m_3 - m_1} (\%)$
A	5.60	29.64	29.44	0.20	23.84	0.83
B	5.24	30.16	29.94	0.22	24.7	0.89
C	5.83	27.67	27.48	0.19	21.65	0.88

Table 4: Natural moisture content, sample 3

Container	Weight of container (m_1)	Weight of container and wet soil (m_2)	Weight of container and dry soil (m_3)	$m_2 - m_3$	$m_3 - m_1$	$w = \frac{m_2 - m_3}{m_3 - m_1}$ (%)
A	6.02	29.32	29.11	0.22	23.09	0.95
B	6.01	19.22	19.12	0.10	13.11	0.76
C	5.90	29.76	29.57	0.19	23.67	0.80

3. Plasticity index of China clay

Liquid limit with distilled water- Sample 1

Table 5: LL with distilled water- Sample 1

Container	Cone penetration (mm)		Average cone penetration (mm)	Weight of container (m_1)	Weight of container and wet soil (m_2)	Weight of container and dry soil (m_3)	$m_2 - m_3$	$m_3 - m_1$	$w = \frac{m_2 - m_3}{m_3 - m_1}$ (%)
A	132	127	129.5	4.84	37.89	27.41	10.48	22.57	46.43
B	129	134	131.5	5.54	36.59	26.16	10.43	20.62	50.58
C	141	140	140.5	5.55	47.48	33.02	14.46	27.47	52.64
D	234	229	231.5	5.36	44.18	29.87	14.31	24.51	58.38

Liquid limit with distilled water- Sample 2

Table 6: LL with distilled water- Sample 2

Container	Cone penetration (mm)		Average cone penetration (mm)	Weight of container (m_1)	Weight of container and wet soil (m_2)	Weight of container and dry soil (m_3)	$m_2 - m_3$	$m_3 - m_1$	$w = \frac{m_2 - m_3}{m_3 - m_1} (\%)$
A	101	104	102.5	4.85	22.59	16.83	5.76	11.98	48.08
B	132	132	132	5.55	29.87	21.65	8.22	16.1	51.06
C	178	180	179	5.55	31.14	21.98	9.16	16.43	55.75
D	229	230	229.5	5.36	35.67	24.33	11.34	18.97	59.78
E	291	291	291	5.43	40.34	27.05	13.29	21.62	61.47

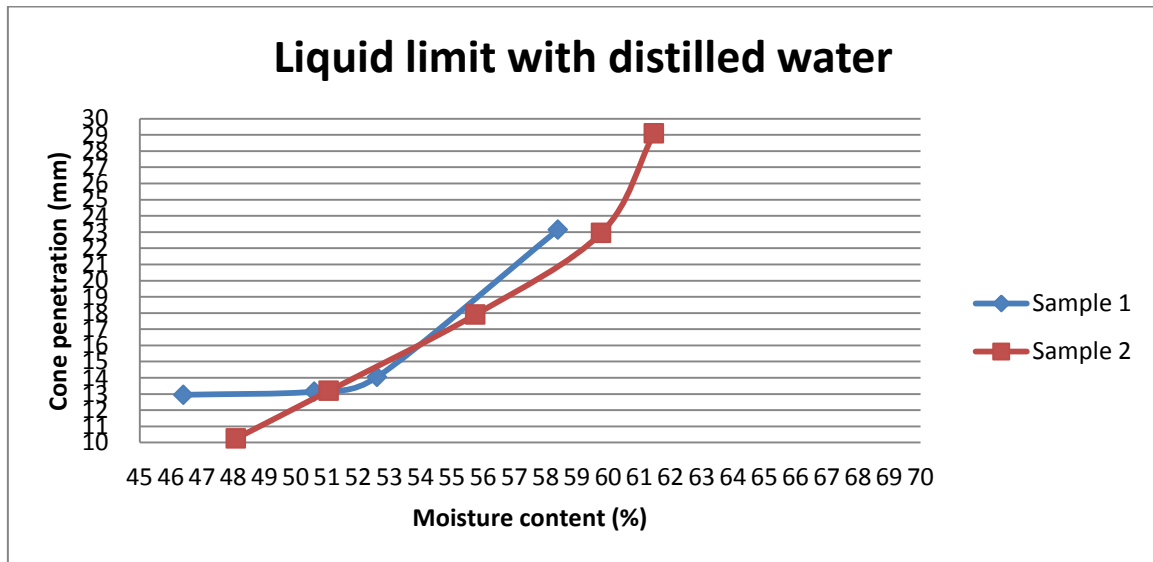


Figure 1: LL with distilled water

Plastic limit with distilled water- Sample 1

Table 7: PL with distilled water- Sample 1

Container	Weight of container (m_1)	Weight of container and wet soil (m_2)	Weight of container and dry soil (m_3)	$m_2 - m_3$	$m_3 - m_1$	$w = \frac{m_2 - m_3}{m_3 - m_1}$ (%)	Average
A	9.74	12.51	11.87	0.64	2.13	30.05	29.98% (30%)
B	9.83	12.50	11.89	0.61	2.06	29.61	
C	24.37	27.54	26.81	0.73	2.44	29.92	
D	23.34	26.39	25.68	0.71	2.34	30.34	

Plastic limit with distilled water- Sample 2

Table 8: PL with distilled water- Sample 2

Container	Weight of container (m_1)	Weight of container and wet soil (m_2)	Weight of container and dry soil (m_3)	$m_2 - m_3$	$m_3 - m_1$	$w = \frac{m_2 - m_3}{m_3 - m_1}$ (%)	Average
A	9.74	13.51	12.65	0.86	2.91	29.55	30.73% (31%)
B	9.83	13.17	12.37	0.80	2.54	31.50	
C	24.37	27.45	26.74	0.71	2.37	29.96	
D	23.34	26.15	25.47	0.68	2.13	31.92	

Liquid limit with tap water- Sample 3

Table 9: LL with tap water- Sample 3

Container	Average cone penetration (mm)	Moisture content (%)
A	43.57	11.36
B	46.1	13.75
C	58.37	23.2
D	62.3	24.2

Liquid limit with tap water- Sample 4

Table 10: LL with tap water- Sample 4

Container	Average cone penetration (mm)	Moisture content (%)
A	44.48	12.65
B	47.72	14.75
C	48.33	16
D	65.65	29.25

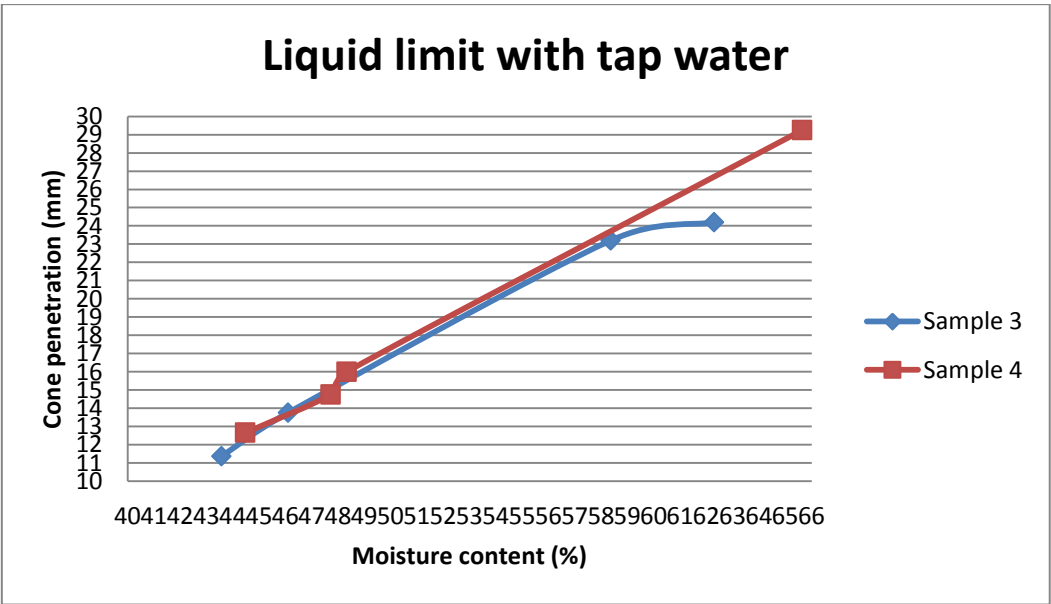


Figure 2: LL with tap water

Plastic limit with tap water- Sample 3

Table 11: PL with tap water- Sample 2

Container	$w = \frac{m_2 - m_3}{m_3 - m_1}$ (%)	Average
A	33.20	33.66
B	34.20	
C	33.34	
D	33.90	

Table 11: PL with tap water- Sample 2

Plastic limit with tap water- Sample 4

Table 12: PL with tap water- Sample 4

Container	$w = \frac{m_2 - m_3}{m_3 - m_1}$ (%)	Average
A	34.43	33.90
B	34.53	
C	33.94	
D	32.68	

4. Specific gravity of China clay

Table 13: Specific gravity of China clay

Container	A	B	C	D
Weight of bottle	26.3002	26.7460	24.9053	25.8327
Weight of stopper	4.6117	4.6622	4.6410	4.6558
Weight of bottle and stopper (m_1)	30.9119	31.4082	29.5463	30.4885
Weight of bottle and soil (m_2)	28.0975	28.7744	26.7072	27.8677
Weight of bottle, stopper, soil and water (m_3)	85.5137	86.6317	82.0469	83.4864
Weight of bottle, stopper and water (m_4)	84.4152	85.3762	80.9292	82.2020
$m_2 - m_1$	1.7973	2.0284	1.8019	2.035
$m_4 - m_1$	53.5033	53.9680	51.3829	51.7135
$m_3 - m_2$	57.4162	57.8573	55.3397	55.6187
$(m_4 - m_1) - (m_3 - m_2)$	0.6988	0.7729	0.6842	0.7506
$\frac{G_L(m_2 - m_1)}{(m_4 - m_1) - (m_3 - m_2)}$	2.5720	2.6244	2.6336	2.7112
Average SG	2.6353			

5. Standard compaction test on clay

Sample 1:

Table 14: Standard compaction test on China clay

Test	1	2	3	4	5
Moisture (%)	19.8	26.42	27.93	31.04	33.24
Dry density (Mg/m ³)	1.48	1.48	1.49	1.47	1.41
Undrained shear strength (kPa)	No data	94.25	104.25	73.5	61.25
Dry density (Mg/m ³) at zero air (sat)	1.72	1.54	1.51	1.44	1.39
5% void line	1.63	1.46	1.43	1.37	1.32
10% void line	1.54	1.39	1.36	1.29	1.26

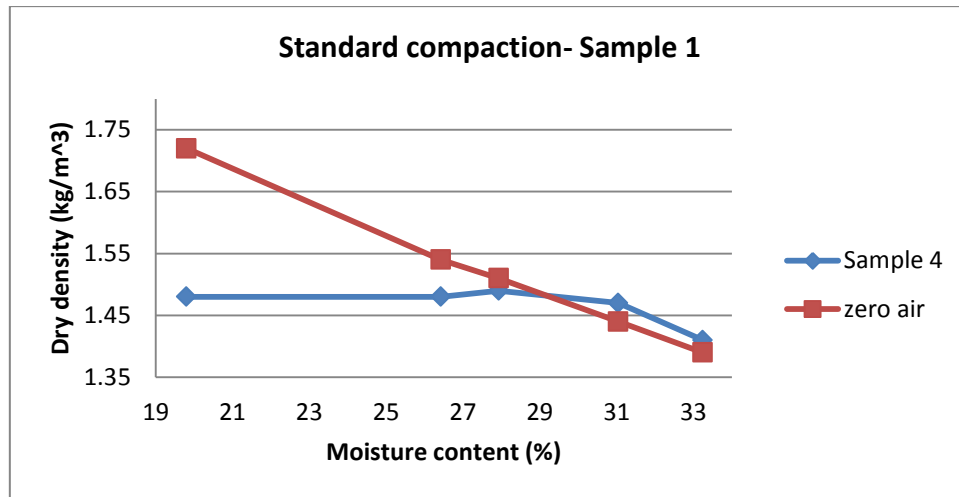


Figure 3: Standard compaction test-sample 1

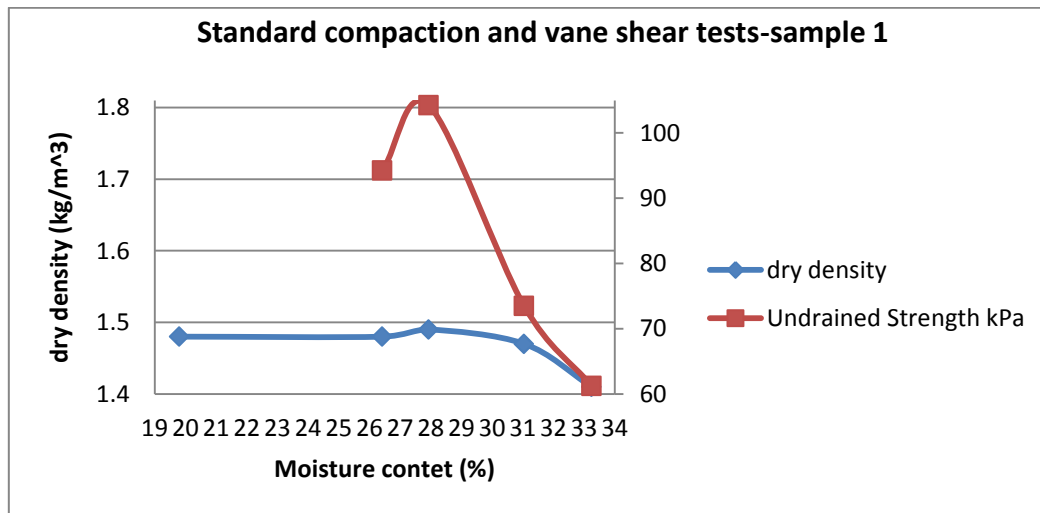


Figure 4: Dry density and undrained strength of sample 1 (China clay)

Sample 2:

Test	1	2	3	4	5
Moisture (%)	23.67	27.37	28.57	32.36	33.27
Dry density (Mg/m ³)	1.41	1.49	1.5	1.42	1.4
Undrained shear strength (kPa)	105.5	105.25	105.25	67.75	57.5
Dry density (Mg/m ³) at zero air (sat)	1.61	1.52	1.49	1.41	1.39
5% void line	1.53	1.44	1.42	1.34	1.32
10% void line	1.45	1.37	1.34	1.27	1.25

Table 15: Standard compaction test on China clay- Repeat

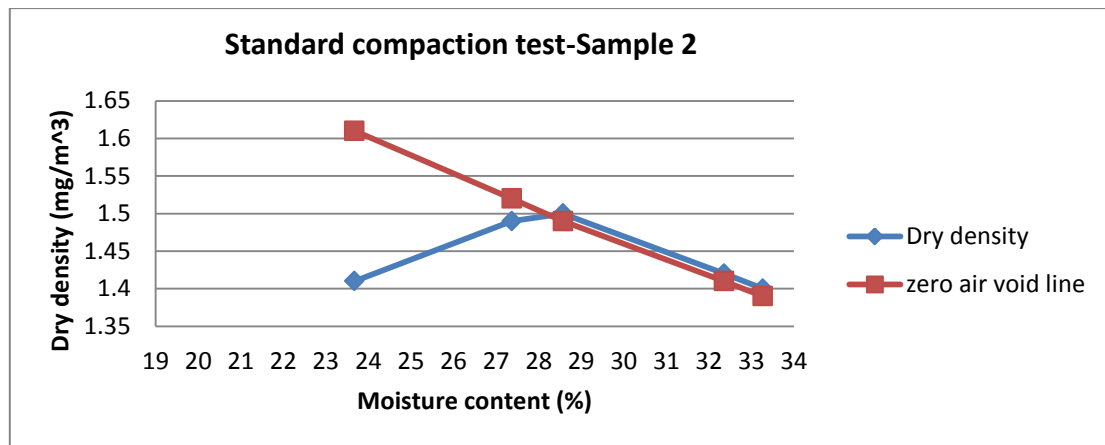


Figure 5: Standard compaction test-sample 2

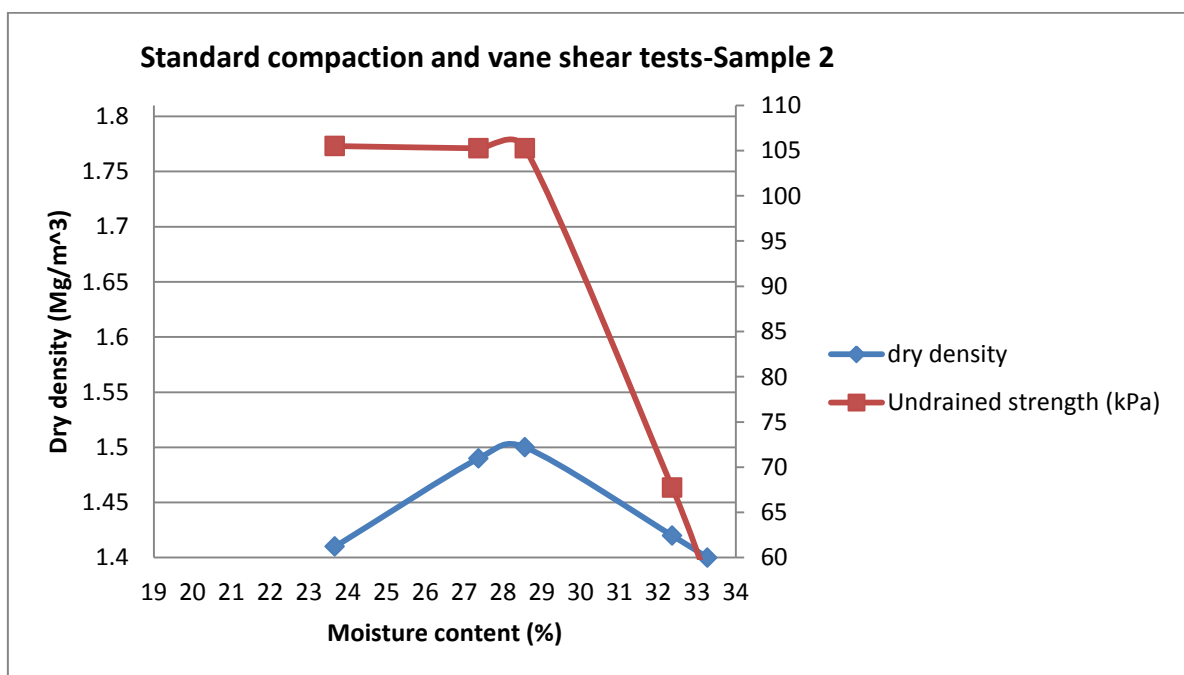


Figure 6: Dry density and undrained strength of sample 2 (China clay)

6. Vibrating hammer compaction test on clay- 10 seconds per layer

The results of this trial have not been used due to significant error in the short time of vibrations.

Table 16: Vibrating hammer compaction- 10 seconds

W (%)	Dry density (Mg/m ³)	Undrained strength (kPa)
30	1.16	88
33.47	1.34	72.67
35.18	1.37	49
36.56	1.32	35.5
38.41	1.29	24.5
41.63	1.23	15
42.86	1.21	12.75
44.12	1.19	9.25

7. Vibrating hammer compaction test on clay- 15 seconds per layer

Sample 1:

Table 17: Vibrating hammer compaction- 5 layers- 15 seconds per layer

Test	W (%)	Dry density (Mg/m ³)	Undrained strength (kPa)	Zero air void line (Mg/m ³)
test 1	24.12	1.31	122.5	1.64
Test 2	27.1	1.44	104.25	1.56
test 3	29.95	1.43	110.75	1.49
test 4	33.33	1.37	64	1.42
test 5	37.01	1.28	32.25	1.35
test 6	39.93	1.21	19	1.3
test 7	43.1	1.14	12.25	1.25

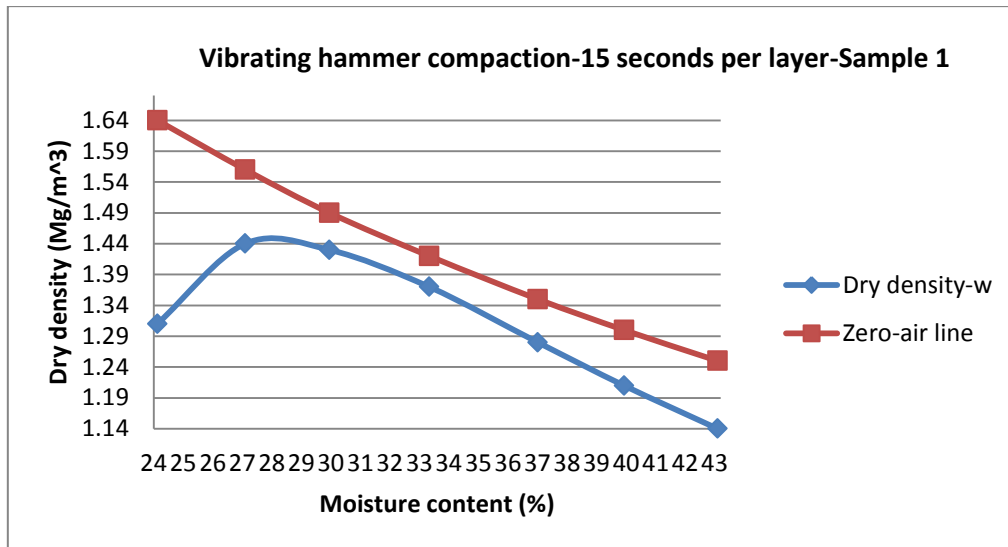


Figure 7: Vibrating hammer compaction-15 seconds per layer

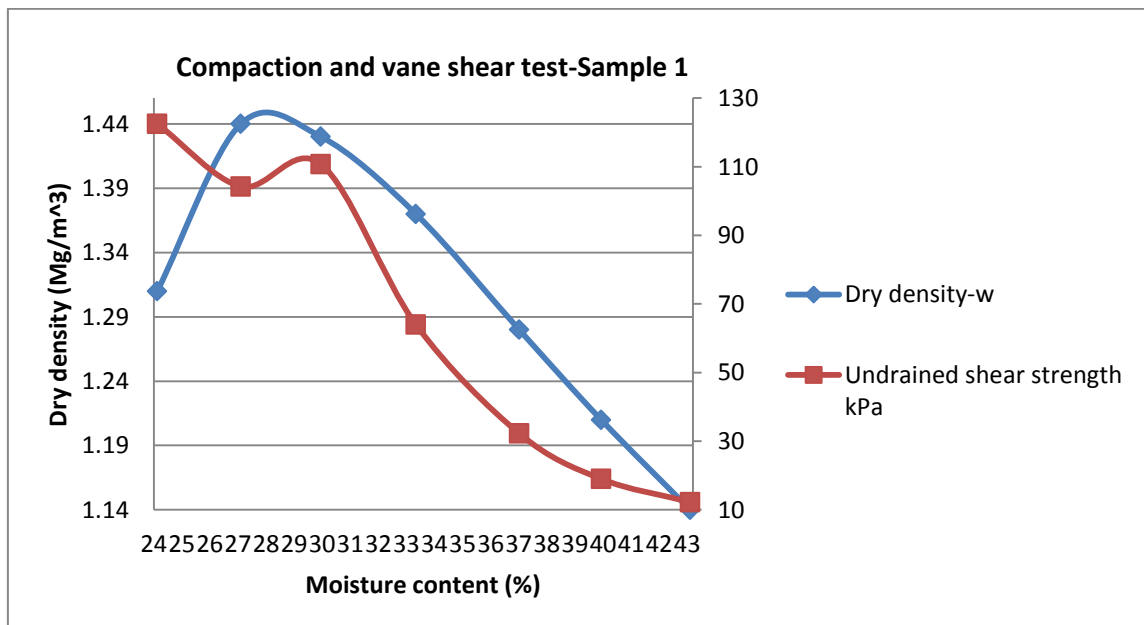


Figure 8: Dry density and undrained strength of vibrating hammer compaction sample 1
(China clay)

Sample 2:

Table 18: 15 seconds compaction per layer- Sample 2

W (%)	Dry density (Mg/m ³)	Undrained strength (kPa)
30.22	1.23	99.5
33.6	1.4	66.5
36.72	1.33	39
40.54	1.24	18.5
44.3	1.19	10.5

Sample 3:

Table 19: 15 seconds compaction per layer- Sample 3

W (%)	Dry density (Mg/m ³)	Undrained strength (kPa)
30.16	1.26	113.5
34.04	1.37	60
37.38	1.29	33.5
40.54	1.24	19.75
44.41	1.17	9.5

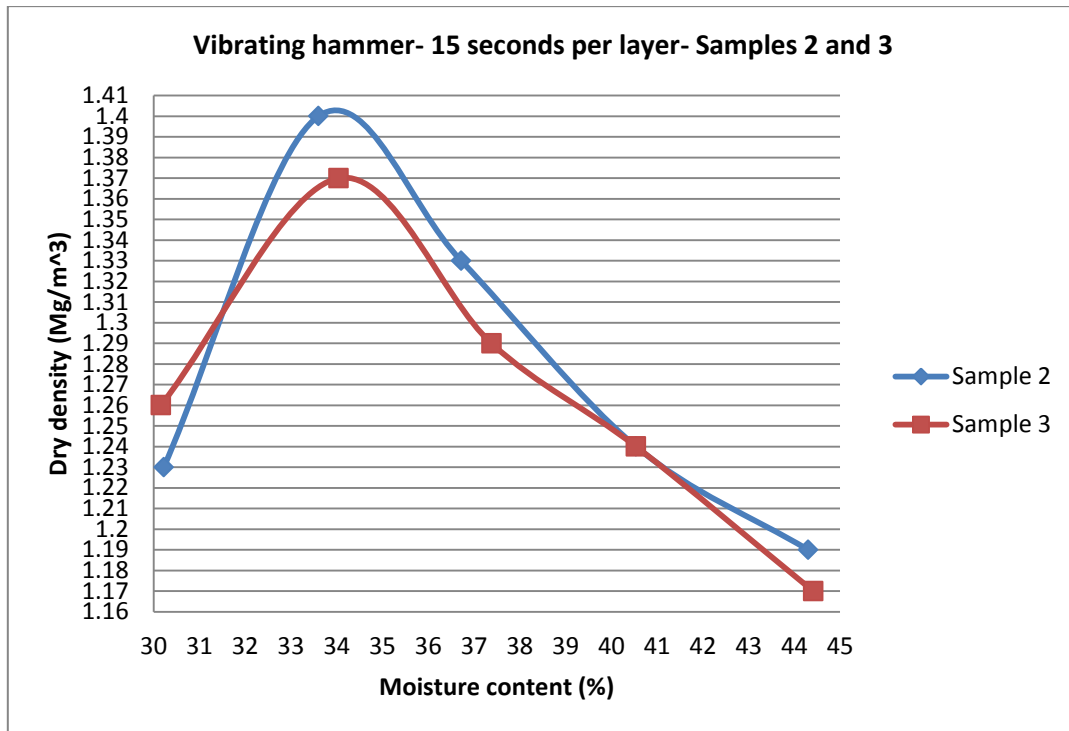


Figure 9: Dry density-Samples 2 and 3- 15 seconds per layer

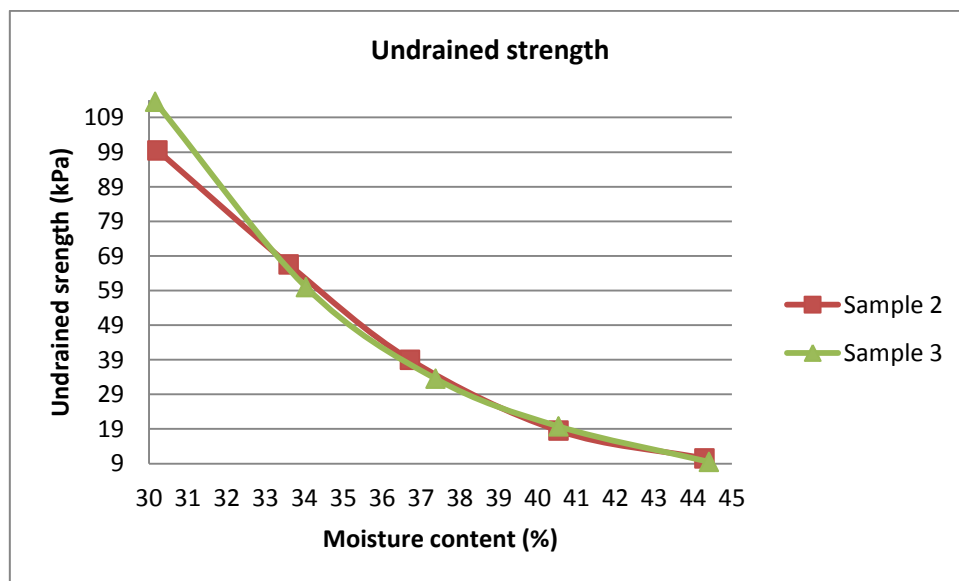


Figure 10: Undrained shear strength-Samples 2 and 3- 15 seconds per layer

8. Host ground requirements for unit cell testing

Test 1: Small unit cell container- three layers

Table 20: Untrained strength of three layers

Layer	Reading 1 (kPa)	Reading 2 (kPa)	Reading 3 (kPa)	Reading 4 (kPa)	Average undrained strength (kPa)
1	13	17	11	15	14
2	18	16	16	16	16.5
3	18	17	18	19	18

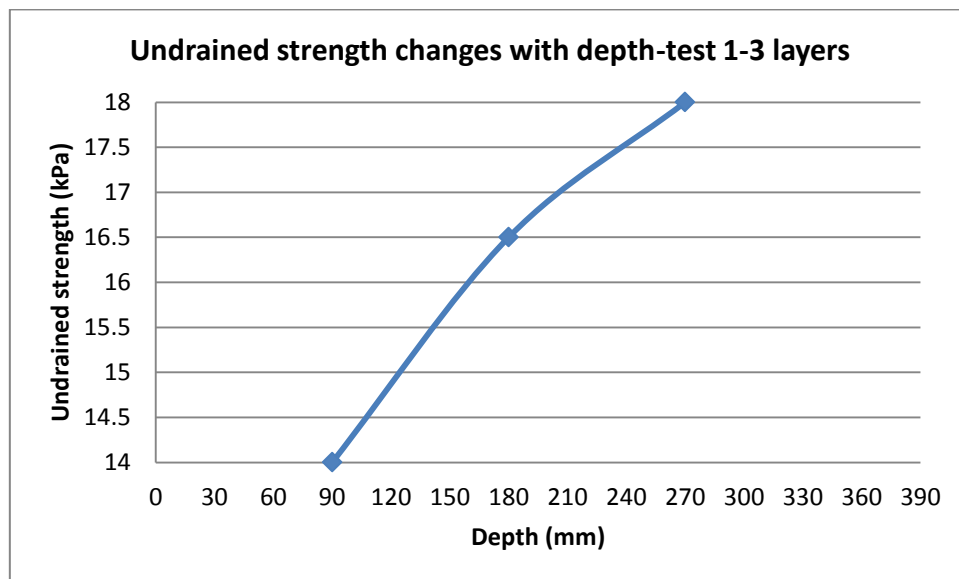


Figure 11: Variation of undrained strength with depth from top of small unit cell container

Moisture content samples from 5 cores; core one is located at centre of unit cell container where stone column would be constructed in the unit cell tests.

Table 21: Moisture content of 5 cores extruded from test 1

Depth at which samples are taken (mm) from the top of the container	Moisture content of core 1(%)	Moisture content of core 2(%)	Moisture content of core 3(%)	Moisture content of core 4(%)	Moisture content of core 5(%)
0	40.04	40.15	38.85	39.85	40.39
30	41.07	40.45	39.96	40.13	40.03
60	41.16	41.53	40.81	41.3	40.6
90	41.17	41.48	40.98	41.2	41.38
120	40.94	41.09	40.4	40.48	41.46
150	41.18	40.83	40.73	40.46	41.41
180	41.52	41.91	40.48	40.52	40.51
210	41.49	41.52	40.96	40.71	41.27
240	41.64	41.22	41.87	41.71	41.35
270	42.07	42.27	41.62	41.38	41.43

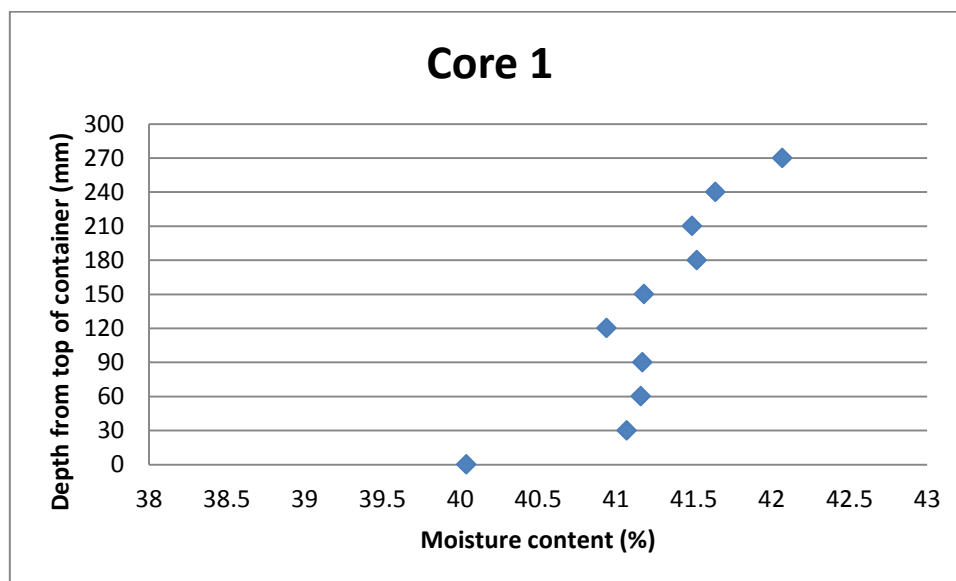


Figure 12: Moisture content variations with depth-core 1

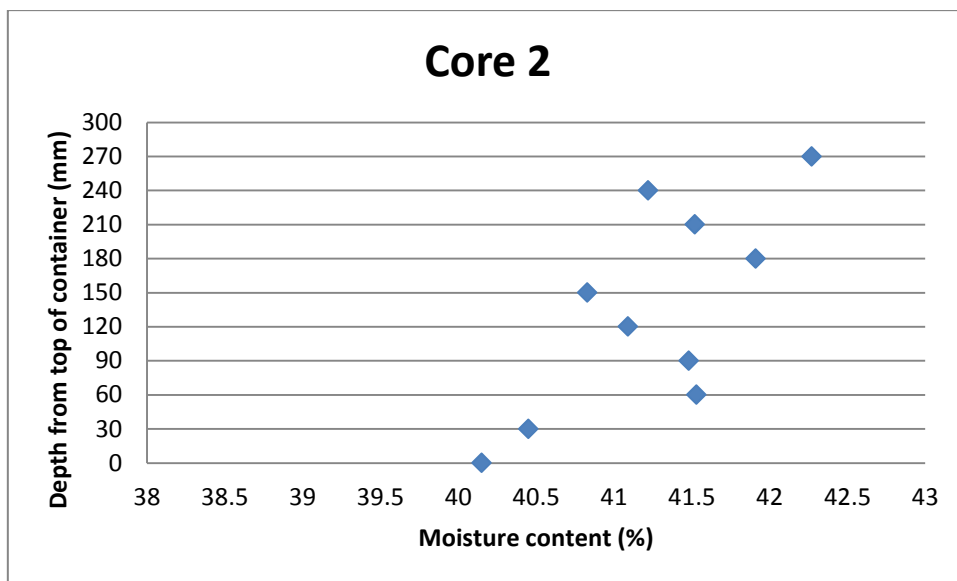


Figure 13: Moisture content variations with depth-core 2

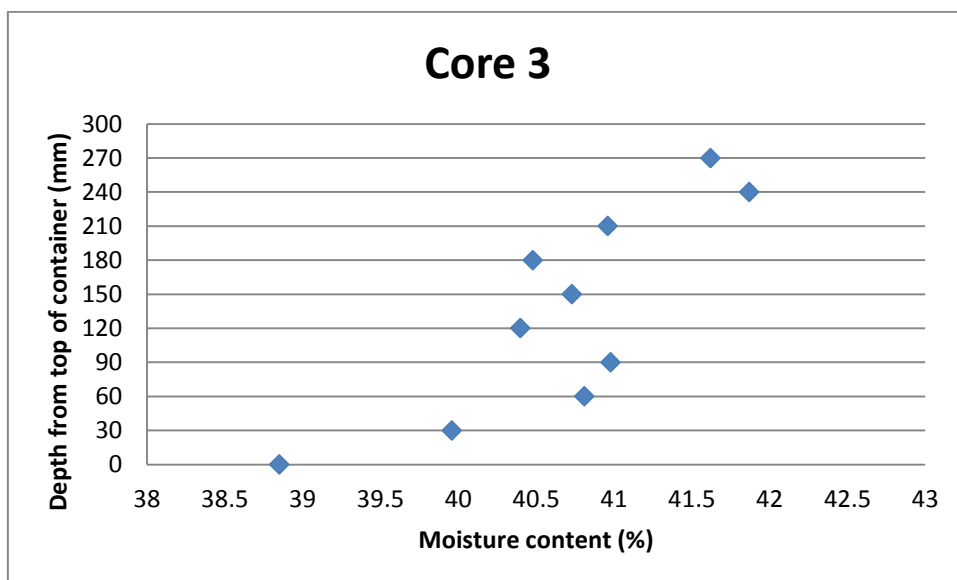


Figure 14: Moisture content variations with depth-core 3

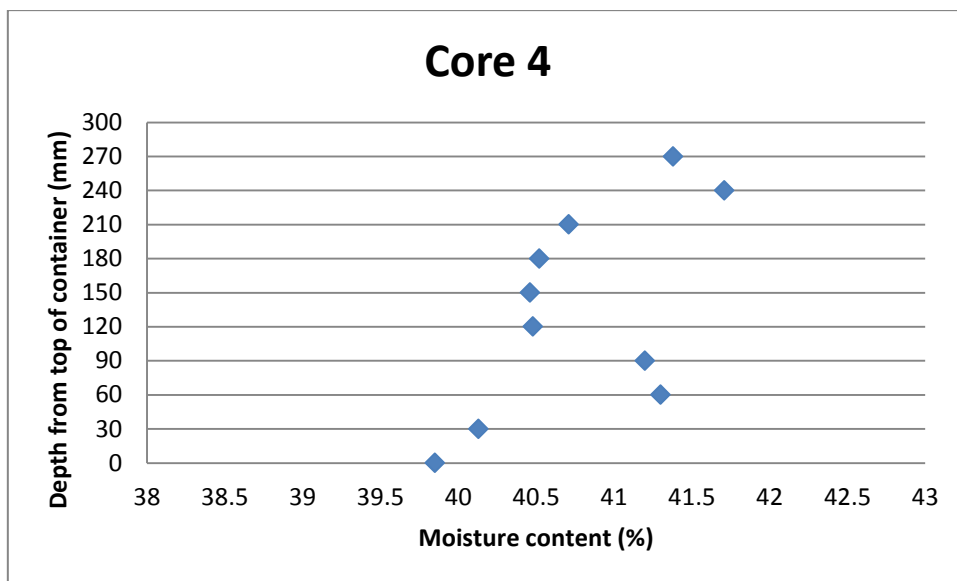


Figure 15: Moisture content variations with depth-core 4

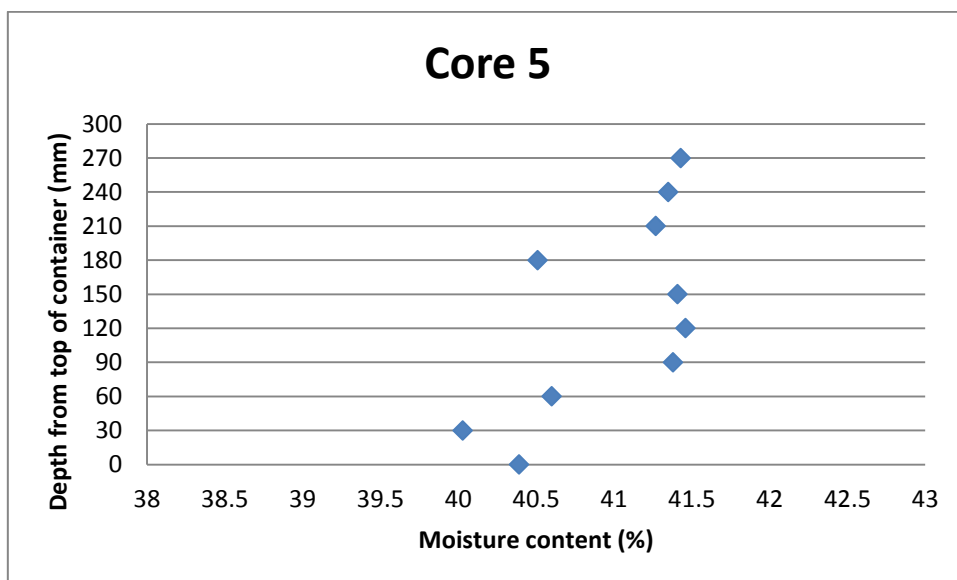


Figure 16: Moisture content variations with depth-core 5

Test 2: Small unit cell container- five layers

Table 22: Undrained strength of 5 layers

Layer	Reading 1 (kPa)	Reading 2 (kPa)	Reading 3 (kPa)	Reading 4 (kPa)	Average undrained strength (kPa)
1	16	15	16	20	16.75
2	16	16	13	17	15.5
3	15	11	18	15	14.75
4	14	16.5	13	13	14.125
5	13	17	12	16	14.5

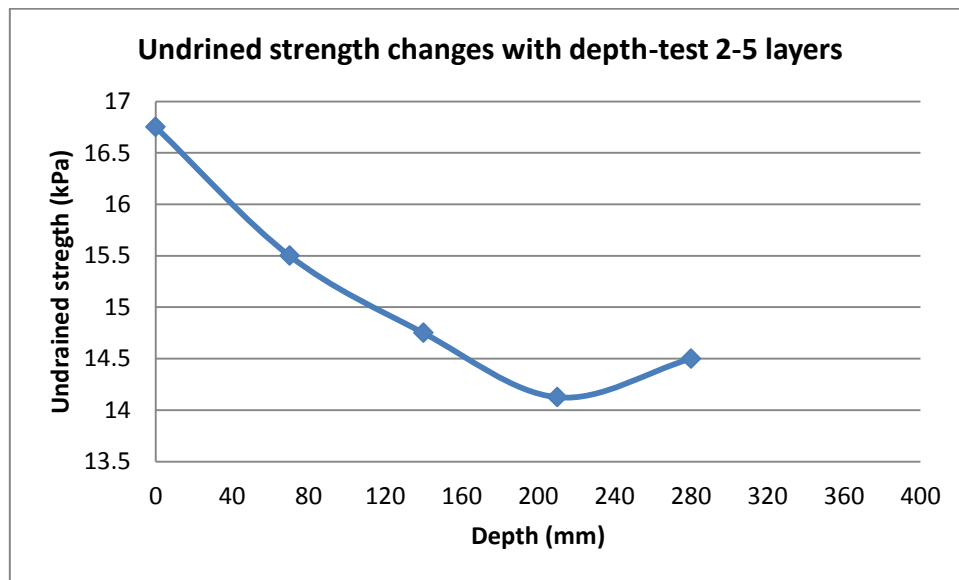


Figure 17: Variation of undrained strength with depth from top of small unit cell container

Table 23: Moisture content of 5 cores extruded from test 2

Depth at which samples are taken (mm) from the top of the container	Moisture content of core 1(%)	Moisture content of core 2(%)	Moisture content of core 3(%)	Moisture content of core 4(%)	Moisture content of core 5(%)
0	42.16	41.07	41	39.74	40.09
30	41.61	40.6	40.41	41.06	40.32
60	41.95	40.81	40.72	40.5	41.27
90	42.43	40.75	43.2	41.2	41.42
120	40.98	40.76	41.76	40.84	42.39
150	40.77	42.06	42.89	41.38	42.13
180	41.35	42.13	42.62	42.19	42.11
210	40.74	41.96	42.28	41.14	42.11
240	40.99	42.09	41.57	42.2	42.64
270	41	42.03	41.57	42.04	41.4

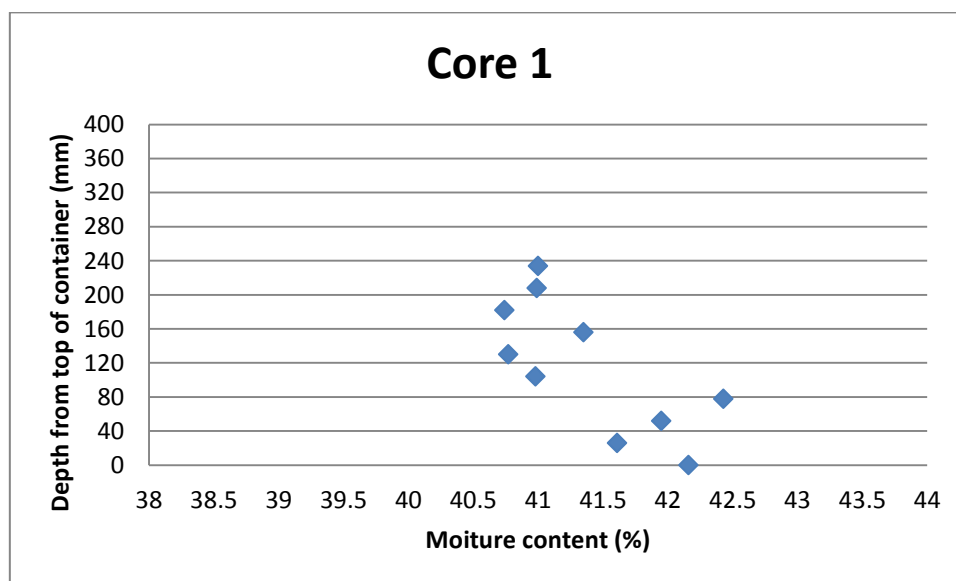


Figure 18: Moisture content variations with depth-core 1

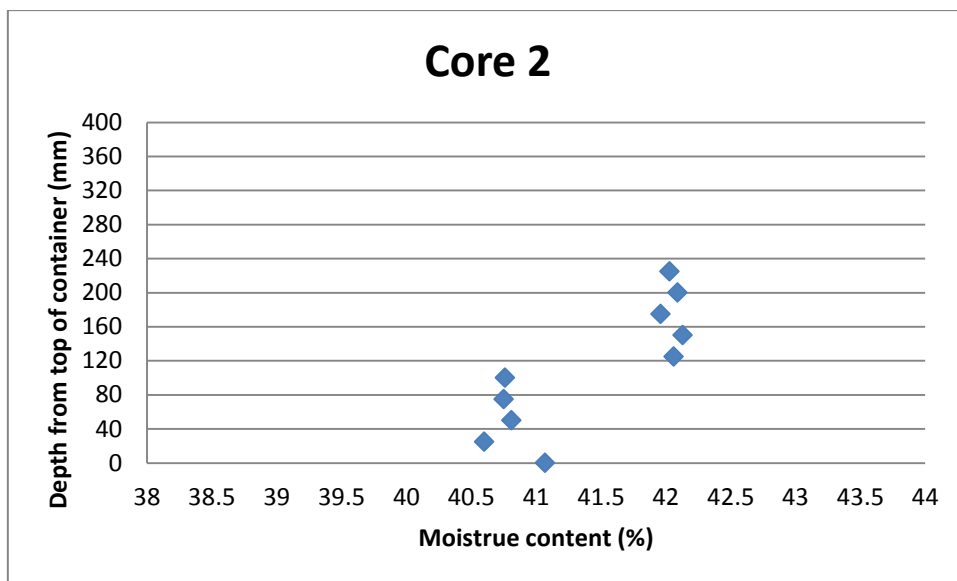


Figure 19: Moisture content variations with depth-core 2

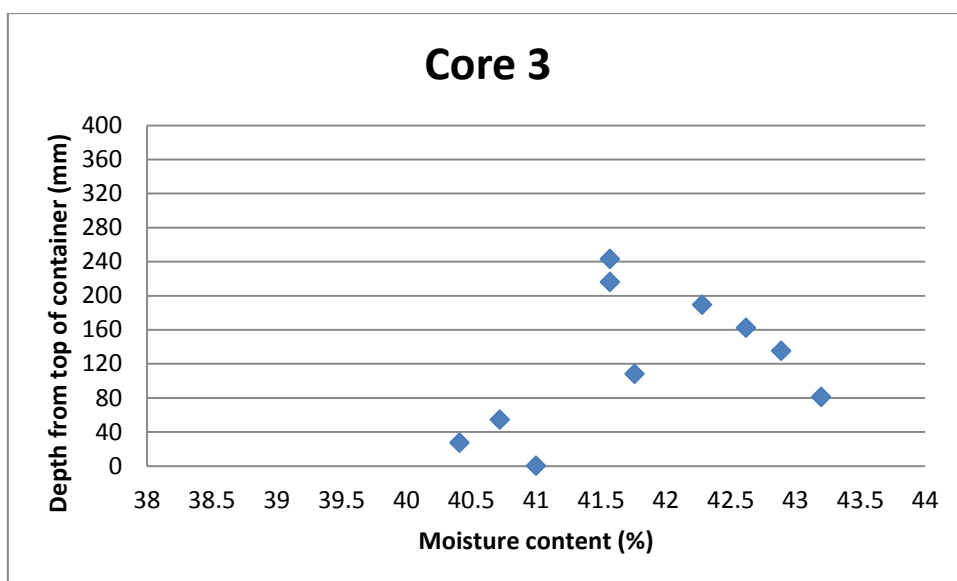


Figure 20: Moisture content variations with depth-core 3

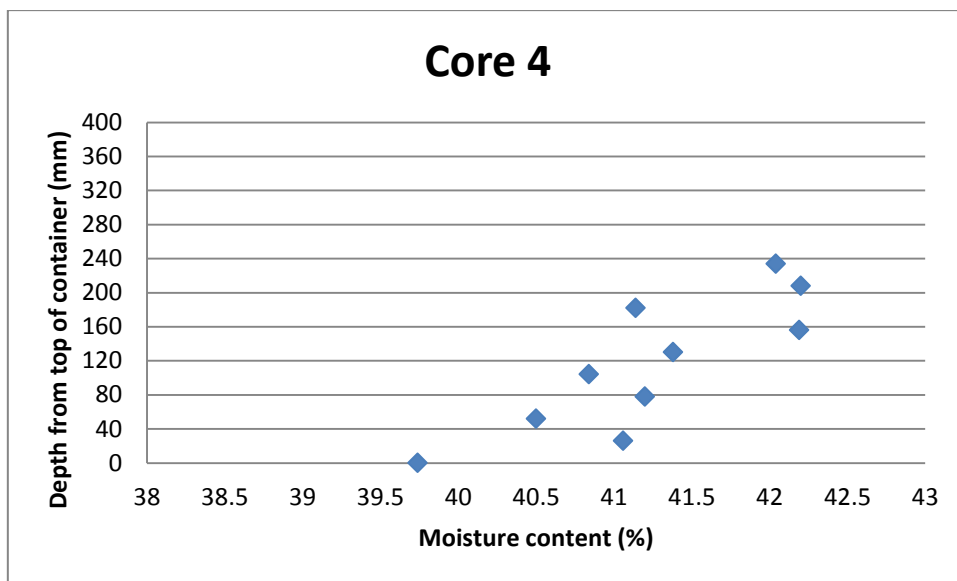


Figure 21 Moisture content variations with depth-core 4

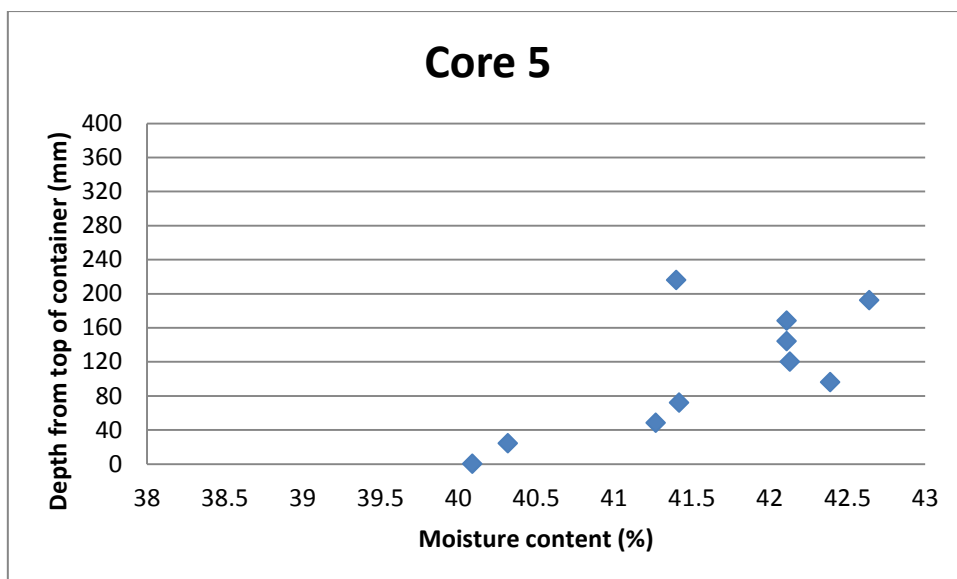


Figure 22 Moisture content variations with depth-core 5

Table 24: Density of layer 1

Moisture content %	Dry density Mg/m ³	Zero air density	5% air density	10% air density
40.95	1.22	1.282	1.218	1.154
40.63	1.25	1.288	1.223	1.159
40.77	1.24	1.285	1.221	1.157
41.21	1.28	1.278	1.214	1.15

Table 25: Density of layer 2

Moisture content %	Dry density Mg/m ³	Zero air density	5% air density	10% air density
40.76	1.34	1.285	1.221	1.157
40.54	1.31	1.289	1.225	1.16
40.85	1.3	1.284	1.22	1.156
41.2	1.29	1.278	1.214	1.15

Table 26: Density of layer 3

Moisture content %	Dry density Mg/m ³	Zero air density	5% air density	10% air density
40.72	1.29	1.286	1.222	1.157
40.76	1.26	1.285	1.221	1.157
40.78	1.24	1.285	1.221	1.157
40.44	1.28	1.291	1.226	1.162

Table 27: Density of layer 4

Moisture content %	Dry density Mg/m ³	Zero air density	5% air density	10% air density
41.74	1.28	1.269	1.206	1.142
42.35	1.26	1.26	1.197	1.134
42.74	1.22	1.253	1.191	1.128
41.8	1.26	1.268	1.205	1.142

Table 28: Density of layer 5

Moisture content %	Dry density Mg/m ³	Zero air density	5% air density	10% air density
42.03	1.27	1.265	1.202	1.138
41.33	1.3	1.277	1.213	1.149
41.59	1.25	1.272	1.208	1.145
41.81	1.31	1.268	1.205	1.141

Appendix 2: Compaction energy for large and small unit cells

Compaction energy for the tank:

Compaction energy for the standard (Proctor) test:

$$\text{Compactive effort} = 2.5 \text{ kg} \times \frac{300 \text{ mm}}{1000 \text{ mm}} \times 27 \times 3 = 60.75 \text{ kg.m}$$

$$60.75 \times 9.81 \frac{\text{m}}{\text{s}^2} = 596 \text{ N.m} = 596 \text{ J}$$

$$\text{volume of the mould} = 1000 \text{ cm}^3 = 0.001 \text{ m}^3$$

$$\text{Work} = \frac{596 \text{ J}}{0.001 \text{ m}^3} = 596 \text{ kJ/m}^3$$

Tank size:

$$\text{Internal diameter} = 319 \text{ mm}$$

$$\text{height} = 425 \text{ mm}$$

$$\text{Volume} = \frac{\Pi \times \left(\frac{319}{1000}\right)^2}{4} \times \frac{425}{1000} = 0.034 \text{ m}^3$$

Energy for the tank:

$$\text{Volume} = 0.034 \text{ m}^3$$

$$\text{Work} = 596 \text{ kJ/m}^3$$

$$\text{Total energy for the tank} = 0.034 \times 596 = 20.264 \text{ kJ}$$

$$\text{No. of layers} = 3$$

$$\text{Energy per layer} = \frac{20.264 \text{ kJ}}{3} = 6.75 \text{ kJ}$$

The vibrating hammer specifications:

$$240 \text{ v}$$

$$3.9 \text{ A}$$

$$900 \text{ w}$$

$$25 - 60 \text{ Hz}$$

$$\text{weight of the hammer} = 2533.7 \text{ g}$$

Time of compaction for each layer:

$$power = \frac{work(energy)}{time}$$

$$t = \frac{w}{p} \text{ sec}$$

$$time \text{ per layer} = \frac{energy \text{ per layer}}{power} = \frac{6.75kJ}{900watt} = \frac{6.75 \times 1000J}{900watt} = 7.5Sec$$

Appendix 3: Results of tests on column’s materials

1. Particle size distribution of aggregates

Big granite:

Table 29: PSD of big Granite

Sieve size (mm)	Percentage passing
pan	0%
20	0.01%
31.5	7.7%
37.5	16.3%
50	77.1%
75	100%

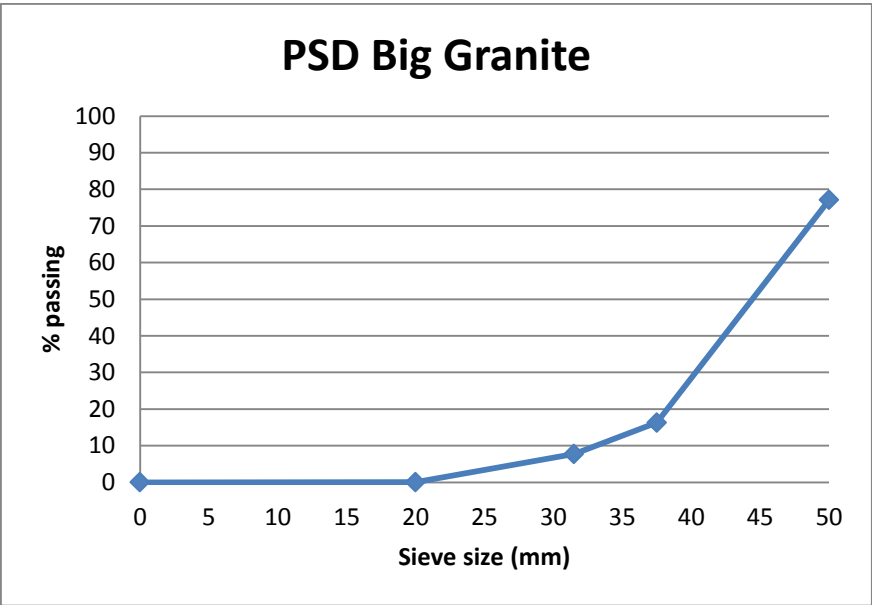


Figure 23: PSD of big granite before crushing via the brick crusher

Small granite used in the unit cell tests:

Table 30: PSD of small Granite

Sieve size (mm)	w remaining (g)	%remained	%passing
9.5	0	0	100
6.3	67	4.457158	95.54284
5	548.8	36.50878	59.03406
3.35	780.8	51.94252	7.091538
2.36	98.1	6.526078	0.56546
2	8.5	0.56546	0
0	0	0	0
Sum	1503.2		

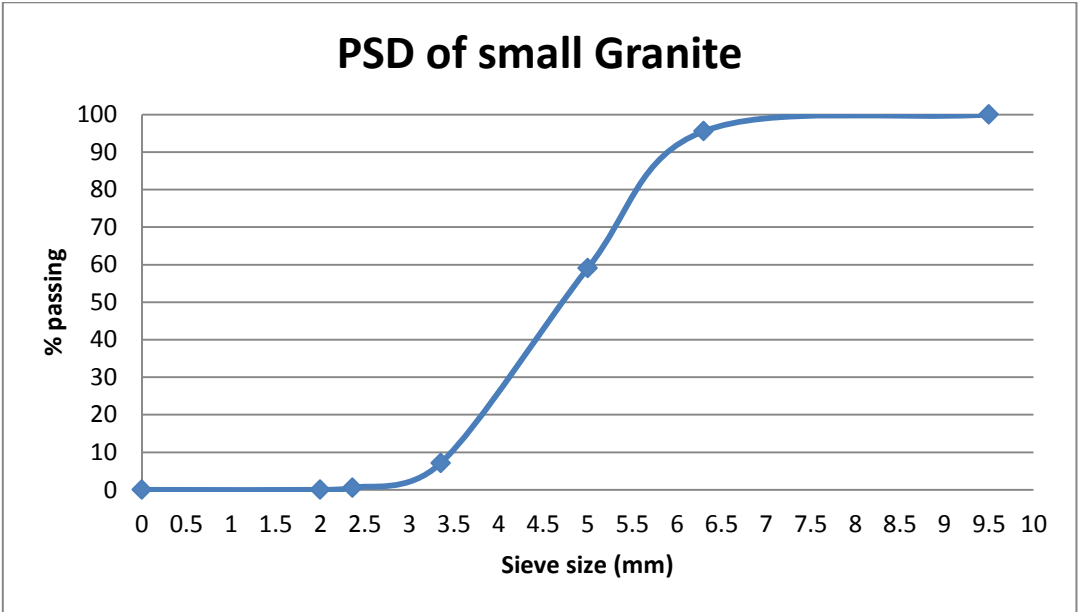


Figure 24: PSD of small Granite used in unit cell tests

Crushed concrete and brick:

Table 31: PSD of crushed concrete and brick

Sieve size (mm)	Percentage passing
pan	0%
20	0.5%
31.5	11.5%
37.5	32.4%
50	67.5%
75	100%

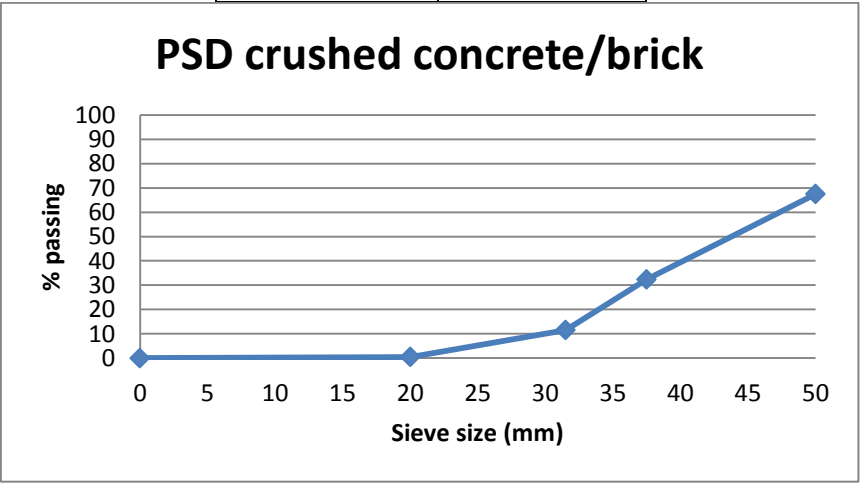


Figure 13: PSD of crushed concrete and brick before crushing via the brick crusher

IBAA (1):

Table 32: PSD of IBAA (1)

Sieve size	Percentage passing
pan	0%
5 mm	0.2%
10 mm	13.4%
14 mm	57.4%
20 mm	93.3%
50 mm	100%

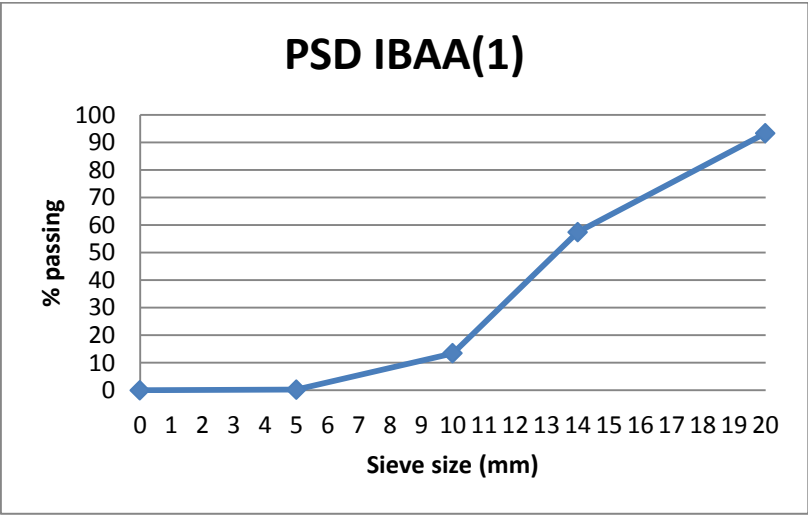


Figure 14: PSD of IBAA (1)

IBAA (2)

Table 33: PSD of IBAA (2)

Sieve size (mm)	Percentage passing
Pan	0%
5	7.5%
10	31.9%
14	51.9%
20	66.7%
50 mm	100%

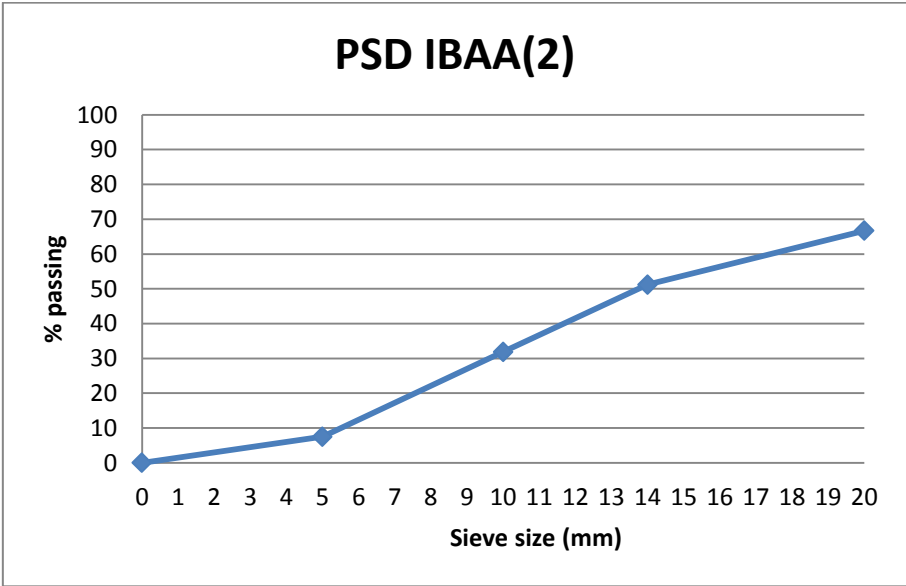


Figure 15: PSD of IBAA (2)

2. Aggregate impact value

$$AIV = \frac{M_2}{M_1}$$

Where M_1 is total mass of the sample in grams;

And M_2 is mass of material passing 2.36mm sieve in grams

Big Granite

Sample 1:

$$AIV = \frac{27.6}{626.3} \times 100 = 4.4\%$$

Sample 2:

$$AIV = \frac{25.1}{579.9} \times 100 = 4.3\%$$

Sample 3:

$$AIV = \frac{23.4}{622.8} \times 100 = 3.7\%$$

$$\text{Mean } AIV = \frac{(4.4 + 4.3 + 3.7)}{3} = 4.1\%$$

Small Granite

This test requires aggregate range of 10 to 14mm; however, the range of 2 to 9.5mm available from small granite has been used in this test.

Sample 1:

$$AIV = \frac{66.8}{605.3} \times 100 = 11\%$$

Sample 2:

$$AIV = \frac{77.2}{583.7} \times 100 = 13.2\%$$

Sample 3:

$$AIV = \frac{80.7}{578.7} \times 100 = 13.9\%$$

$$\text{Mean } AIV = \frac{(11 + 13.2 + 13.9)}{3} = 12.7\%$$

Crushed concrete and brick

Sample 1:

$$AIV = \frac{88.4}{492.3} \times 100 = 17.9\%$$

Sample 2:

$$AIV = \frac{86.7}{479.4} \times 100 = 18\%$$

Sample 3:

$$AIV = \frac{75.6}{468.4} \times 100 = 16.1\%$$

$$\text{Mean } AIV = \frac{(17.9 + 18 + 16.1)}{3} = 17.3\%$$

IBAA (1)

Sample 1:

$$AIV = \frac{137.9}{466} \times 100 = 29.6\%$$

Sample 2:

$$AIV = \frac{131.4}{485.7} \times 100 = 27\%$$

Sample 3:

$$AIV = \frac{124.9}{467.3} \times 100 = 26.7\%$$

$$\text{Mean } AIV = \frac{(29.6 + 27 + 26.7)}{3} = 27.8\%$$

IBAA (2)

Sample 1:

$$AIV = \frac{112}{537.4} \times 100 = 20.8\%$$

Sample 2:

$$AIV = \frac{123.4}{531.9} \times 100 = 23.2\%$$

$$\text{Mean } AIV = \frac{(20.8 + 23.2)}{2} = 22\%$$

3. Aggregate crushing value

$$ACV = \frac{M_2}{M_1}$$

Where M_1 is total mass of the sample in grams;

And M_2 is mass of material passing 2.36 mm sieve in grams

Big Granite

Sample 1:

$$ACV = \frac{470.5}{1966.4} \times 100 = 23.9\%$$

Sample 2:

$$ACV = \frac{472.9}{1912.4} \times 100 = 24.7\%$$

Sample 3:

$$ACV = \frac{476.1}{1845.5} \times 100 = 25.8\%$$

$$Mean \ ACV = \frac{(23.9 + 24.7 + 25.8)}{3} = 24.8\%$$

Small Granite

Sample 1:

$$ACV = \frac{790.3}{1879.5} \times 100 = 42\%$$

Sample 2:

$$ACV = \frac{777.5}{1930.5} \times 100 = 40.3\%$$

Sample 3:

$$ACV = \frac{738.8}{1930.5} \times 100 = 38.3\%$$

$$\text{Mean } ACV = \frac{(42 + 40.3 + 38.3)}{3} = 40.2\%$$

Crushed concrete and brick

Sample 1:

$$ACV = \frac{560.9}{1638.2} \times 100 = 34.2\%$$

Sample 2:

$$ACV = \frac{550.5}{1637.4} \times 100 = 33.6\%$$

Sample 3:

$$ACV = \frac{538.9}{1595.3} \times 100 = 33.8\%$$

$$\text{Mean } ACV = \frac{(34.2 + 33.6 + 33.8)}{3} = 33.9\%$$

IBAA (1)

Sample 1:

$$ACV = \frac{712.6}{1532.4} \times 100 = 46.5\%$$

Sample 2:

$$ACV = \frac{736.7}{1542.3} \times 100 = 47.8\%$$

Sample 3:

$$ACV = \frac{766.7}{1576.9} \times 100 = 48.6\%$$

$$Mean \ ACV = \frac{(46.5 + 47.8 + 48.6)}{3} = 47.6\%$$

IBAA (2)

Sample 1:

$$ACV = \frac{697.1}{1697.7} \times 100 = 41.1\%$$

4. Ten percent fines value

$$F = \frac{14f}{m + 4}$$

$$m = \frac{M_2}{M_1} \times 100$$

Where F is the force in kN, required for 10% fines to be produced for each specimen

f, is the maximum force applied in kN

m, is the percentage of material passing the 2.36mm sieve at the maximum force

M_1 is total mass of the sample (grams)

M_2 is mass of material passing 2.36 mm sieve (grams)

Big Granite

$$f = 125kN$$

$$M_1 = 1759.4g$$

$$M_2 = 178.4g$$

$$m = \frac{178.4}{1759.4} \times 100 = 10.1\%$$

$$F = \frac{14 \times 125}{10.1 + 4} = 124.1kN$$

Small Granite

$$f = 75kN$$

$$M_1 = 1907.5g$$

$$M_2 = 166g$$

$$m = \frac{166}{1907.5} \times 100 = 8.7\%$$

$$F = \frac{14 \times 75}{8.7 + 4} = 82.7kN$$

Crushed concrete and brick

$$f = 50kN$$

$$M_1 = 1518.9g$$

$$M_2 = 158.1g$$

$$m = \frac{158.1}{1518.9} \times 100 = 10.4\%$$

$$F = \frac{14 \times 50}{10.4 + 4} = 48.6kN$$

IBAA (1)

$$f = 37.5kN$$

$$M_1 = 1555.9g$$

$$M_2 = 135.3g$$

$$m = \frac{135.3}{1555.9} \times 100 = 8.7\%$$

$$F = \frac{14 \times 37.5}{8.7 + 4} = 41.3kN$$

IBAA (2)

$$f = 43.75kN$$

$$M_1 = 1662.6g$$

$$M_2 = 199.8g$$

$$m = \frac{199.8}{1662.6} \times 100 = 12.01\%$$

$$F = \frac{14 \times 43.75}{12.01 + 4} = 38.25kN$$

5. Los Angeles test

$$LA = \frac{5000 - m}{50}$$

Where m is the mass of material retained on the 1.6mm sieve (grams).

Big Granite

Total mass=5098g

m=4291.2g

$$LA = \frac{5000 - 4291.2}{50} = 14.176$$

Crushed concrete and brick

Total mass=5057.5g

m=3444.7g

$$LA = \frac{5000 - 3444.7}{50} = 31.106$$

IBAA (1)

Total mass=5015.7g

m=2866.6g

$$LA = \frac{5000 - 2866.6}{50} = 42.668$$

IBAA (2)

Total mass=4990.2g

m=2780.4g

$$LA = \frac{5000 - 2780.4}{50} = 44.392$$

Appendix 4: Shear box tests results (Attached CD)

Appendix 5: Large unit cell tests results (Attached CD)

Appendix 6: Small unit cell results-series 1 (Attached CD)

Appendix 7: Small unit cell results-series 2 (Attached CD)

Appendix 8: Small unit cell results-series 3 (Attached CD)