The Use of Recycled and Secondary Aggregates Within Granular Columns

by

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Abstract

The construction industry utilises 90 % of aggregates. It is recognised that the overuse or mismanagement of a resource can lead to its depletion or exhaustion. The reliance of the construction industry on primary aggregates is unsustainable and the overexploitation of sand and other primary aggregates has depleted reserves in recent decades, causing damage to the environment and creating the potential for shortages. Simultaneously the construction industry is responsible for producing millions of tons of waste, sending approximately 44% to landfill.

The adoption of alternative aggregates, i.e., recycled aggregate (RA) or secondary aggregates (SA), offers a potential solution. However, perceived barriers such as lack of confidence and/or perceived risk with the product (perception that they are inferior to PA, or that there are issues with consistency), a lack of suitable specifications and testing protocols (reliability and quality control issues for certain applications), and certification of the produced product. In addition, there may be a lack of awareness of AA products and issues such as supply–demand and waste management licensing regulations/environmental issues may further hinder their adoption.

There has been a significant research effort to address these barriers through research into the use of alternative aggregates in geotechnics, but a significant amount of this research has proposed lower utility application such as fills, which is a positive step, but is a missed opportunity as these alternative aggregates have potential to be used in higher utility applications such as granular columns.

Granular columns offer a cost-effective, low-carbon ground improvement solution. The technique, suitable for weak cohesive and granular soils, comprises the backfilling of bores with aggregate, forming a composite ground that is stiffer that the original soil.

The use of alternative aggregates within granular columns is not a new concept and has been adopted to some extent in industry but there is scope for increased uptake, and this is hindered by the previously mentioned barriers.

Extensive laboratory research has shown the successful use of alternative aggregates within granular column construction, materials studied include tyre chips, fly ash, bottom ash, steel slag, crushed polypropylene. Waste glass has been suggested as a possible aggregate for granular columns by numerous researchers but has not yet been studied extensively.

In this research the suitability of three alternative aggregates, waste container glass, waste flat glass and APCr (waste ash treated by accelerated carbonation), for use within granular columns is explored. The performance of these alternative aggregates when used to construct granular columns was compared against a primary aggregate, granite.

Two types of tests, static loading and constant rate of strain, were undertaken to evaluate the impact of the type of loading on column behaviour and to assess the suitability of constant rate of strain tests, which are adopted frequently but do not replicate the loading experienced by granular columns in practice.

Laboratory scale granular column tests were completed to assess and compare the performance of each of the materials. It is acknowledged that there are limitations in the use of laboratory scale tests for granular columns, however, there are clear benefits to conducting these tests particularly when determining feasibility such as repeatability, speed and low-cost.

Prior to the model column tests, aggregate index tests that are suggested to indicate the suitability of an aggregate for use within granular columns, including the aggregate crushing value, aggregate impact value, LA value, flakiness, and angle of friction were conducted for each of the materials. The results indicated that the alternative aggregates, APCr and both types of waste glass, would perform poorly if adopted for granular columns.

However, in the model column tests the alternative aggregates performed well and the capacity of APCr was comparable to granite despite the apparent poor quality exhibited in the index tests. The improvement factors achieved in the constant rate of strain tests, based on the average test results, indicate that the APCr and flat glass perform comparably to granite with improvement factors of 2.0 and 1.8 respectively. The container glass performed less well but still achieved an improvement factor of 1.5.

The results for the static load tests, conducted on granite, container glass and APCr columns, corroborated the findings of the constant rate of strain tests as the improvement factors measured for ACPr, granite and container glass were 2.2, 2.0 and 1.5 respectively.

The results of this research are encouraging as the work highlights a potential valuable use for these waste materials. It is hoped that the demonstration of the aggregate index tests being a poor indication of the material behaviour within granular columns inspires investigation into other materials that may ordinarily be dismissed, reducing waste sent to landfill and contributing the achievement of a circular economy.

Dedication

To my parents and sister Rachel, thank you for your unwavering support throughout the highs and lows of this process.

To my husband-to-be, Thomas, thank you for all your love, support, and encouragement.

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Thank you to my lead supervisor Dr Royal, without whom none of this would be possible. I would also like to thank Prof Jefferson for his continued help and guidance throughout the duration of my studies.

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Abbreviations

AA	Alternative Aggregates (such as recycled and secondary)
ACT	Accelerated Carbonation Treatment
APCr	Air Pollution Control Residues
A_r	Area Replacement Ratio
d	Aggregate Particle Diameter
D	Granular Column Diameter
EoW	End of Waste
I_f	Improvement Factor
MSWI	Municipal Solid Waste Incinerator Ash
GC	Granular Column
PA	Primary Aggregate
RA	Recycled Aggregate
SA	Secondary Aggregate
VSC	Vibrostone Column
WGC	Waste Glass Cullet
WtE	Waste to Energy

1 Introduction

The construction industry consumes large quantities of primary aggregates (PA) annually, within Europe this figure equates to 1.8 Gt (British Geological Survey, 2017). This is inherently an unsustainable situation as these aggregates are a limited resource (Cowell and Owens, 1998; Langer, 2002; Baker and Hendy, 2005) and the engineering properties often exceed the requirements for some applications (Royal and Jefferson, 2017). In addition, the disposal of secondary or alternative materials, which have potential to be adopted as alternative aggregates, unnecessarily to landfill is a waste of precious resources and does not contribute to the circular economy that is a political goal for many countries including the UK, China, USA, Japan and the European Union (Lazarevic, Buclet and Brandt, 2012; Ajayi and Oyedele, 2017; HM Government, 2018) . For further detail on the benefits of increasing the adoption of alternative aggregates please see published paper 'The Use of Recycled and Secondary Aggregates to Achieve a Circular Economy within Geotechnical Engineering' contained in Appendix A.

Please note that in this thesis, the term 'aggregates' is taken in a broad sense; it encompasses coarse grained materials that could be used in geotechnical engineering applications. This definition would include materials traditionally supplied from quarries and borrow pits for construction processes; materials excavated, and possibly processed (i.e., crushed, sorted to produce specified particle size distributions, etc.) and marketed, before being consumed by the geotechnical industry. These aggregates are defined in the catchall term Primary Aggregates (PA). They (AA) have not been produced as wastes and then repurposed to form a viable material for another application, although it is possible to produce viable materials from such waste streams. These wastes (potentially with some processing) can have suitable engineering properties suitable for geotechnical engineering purposes and are termed Alternative Aggregates (AA). AA can comprise materials that are simply reused/recycled or have been processed to form a new material. Materials that historically have been reused, without significant processing, might include pulverised fuel ash, colliery spoil, shredded tyres, glass cullet, etc.; such aggregates are termed Recycled Aggregates (RA) herein. Some waste materials require additional intervention before a suitable aggregate can be developed; these interventions might include thermal treatment, chemical stabilisation, etc. and are termed herein as Secondary Aggregates (SA). An example of this would be Lytag, the lightweight aggregates produced via sintering pulverised fuel ash.

Whilst there is an apparent abundance of natural aggregates on Earth, there doesn't appear to be a pressing need to look for alternatives. However, the Mineral Products Association (MPA) (2020) states that, based on the 10-year average, the quantity of aggregates extracted and used each year exceeds the new reserves that have been granted planning permission, thus resulting in a long-term trend of reserve depletion. This is an issue that most urgently affects sand and gravel as there is only an estimated 8 years' supply remaining in England (Mineral Products Association, 2020). However, crushed rock reserves are also of concern as only 75 % of (compared to 63 % of sand and gravel) reserves were replenished between 2009 and 2018 (Mineral Products Association, 2020). It is estimated that in the UK alone, over 5 billion tonnes of aggregates will be required (even when factoring in more efficient construction methods), and clearly, shortfalls are likely to occur in the near future unless alternative sources can be found (Mineral Products Association, 2016; 2020)

Availability is not the only factor when assessing the viability of a resource, as there are clearly societal, environmental, and economic considerations. The development of land has led to sand and gravel resources becoming inaccessible (leaving the accessible resources nearing exhaustion) and increasing the costs, both financial and energy, associated with extraction (Thompson *et al.*, 2008; Bloodworth, Scott and McEvoy, 2009; Mineral Products Association, 2016). Since aggregate sources are not always located where they are required, there are large regions around the world where sources of PA are non-existent. This varying distribution of PA leads to additional pressure being placed upon areas of resource (Langer, 2002; Baker and Hendy, 2005; Ismail, Hoe and Ramli, 2013; Hossain *et al.*, 2016). An example of this is the UK, where there is an imbalance in the areas of aggregate supply and demand as 70 % of hard the rock sources lie within the south-west and East Midlands regions (of England and Wales) (Ellis, 2003) whereas the area of greatest development, i.e., the greatest demand for material, is located in the south-east (Cowell and Owens, 1998; Bloodworth, Scott and McEvoy, 2009; Mankelow, Oyo-Ita and Birkin, 2010; Hossain *et al.*, 2016; Zuo *et al.*, 2018). This region has accounted for one-third of the total construction activity in the UK for at least the past twenty years (Gunn et al., 2008; Mankelow, Oyo-Ita and Birkin, 2010). In 2004, only 40 % of aggregates could be sourced in the south-east; thus, materials were transported over 100 miles.

It would be impossible to import sufficient quantities of aggregates to meet England's requirements so the need for an indigenous supply is of huge importance (Brown et al. 2008), adopting AA provides one solution to maintaining this supply whilst reducing pressure on existing sources of PA.

The UK is one of the highest adopters of recycled and secondary aggregates, approximately 25 % of aggregates utilised are from recycled or secondary sources, which is encouraging but given the pressing need to reduce reliance on PA there is more work to be done to increase the uptake of AA (Mankelow *et al.*, 2008; Ismail, Hoe and Ramli, 2013; Mineral Products Association, 2016, 2020). There has been much research into the use of secondary and primary aggregates within the construction industry although a large proportion of this is focused on the concrete sector and not in geotechnics specifically. In addition, much of the work, that does not focus on the use of PA/SA within concrete mixes, identifies relatively low utility, i.e. value, applications such as fills (Gomes Correia, Winter and Puppala, 2016; Zukri and Nazir, 2018). Whilst this is still a positive step in reducing waste sent to landfill, particularly since fill applications account for approximately 30 % of all aggregates consumed within the construction industry (Brown et al., 2008), it is also a wasted opportunity to utilise these materials in higher utility applications such as granular columns.

Granular columns (GCs) (which are also known as vibrostone Columns or granular piles) are a commonly used ground improvement technique (Egan and Slocombe, 2010). These columns provide both increased strength in soft soils (commonly weak clay soils), by changing the load transfer mechanism and mobilising the passive strength of the ground, and by potentially accelerating consolidation rates due to the increase in drainage pathways (Sivakumar *et al.*, 2004, 2021; Bryan, James and Jonathan, 2007). There have been calls for the adoption of recycled and secondary aggregates within granular columns for many years (Serridge, 2005a; Tranter, Jefferson and Ghataora, 2008) and there has been a significant amount of research that shows the successful use of AA within granular columns (Amini, 2015; Ayothiraman and

Soumya, 2015; Moradi *et al.*, 2018; Zukri and Nazir, 2018) but this does not appear to be reflected in industry as there are more limited published case studies of the use of AA in practice. Materials shown to be of benefit for GC construction include: waste tyre chips, railway ballast, crushed concrete and bottom ash (Serridge, 2005; Tranter, Jefferson and Ghataora, 2008; Amini, 2015; Ayothiraman and Soumya, 2015).

This research focuses on the potential use of three materials, as possible alternatives to a primary aggregate (crushed granite chips) in GC applications. These AA were container glass cullet (RA), flat glass (window) cullet (RA) and an Air Pollution Control residue (APCr); which is created via accelerated carbonisation (at room temperature) of industrial waste streams (incinerator fly ash). Waste container glass has been cited as a potential aggregate for granular columns but has not yet been explored (Zukri and Nazir, 2018), window glass is poorly recycled and often is landfilled, yet could be as suitable as container glass cullet (which is, on occasion, adopted in geotechnical engineering applications) and APCr are defined as lightweight aggregates and are currently utilised within concrete block construction but has not yet been adopted within geotechnical applications. This study was undertaken to contribute to the body of evidence that AA are viable alternatives to PA in geotechnical applications; confidence in these materials is fundamentally required if uptake is to be increased and the circular economy, championed by the UK's Government, is embraced.

1.1 Granular Columns

Granular columns, also known under the collective term 'vibrostone columns' are a commonly used ground improvement technique in soft soils. There two main column construction processes that can be adopted, bottom-feed or top-feed, and the method chosen is dependent on the ground conditions. The bottom-feed method comprises the aggregate being fed into the bore via a tube attached to the vibroflot, enabling aggregate to be placed at the base whilst the bore is supported by the vibroflot. The bottom-feed method is most suited for use when open bores would become unstable, such as in very soft soil or where there is a high-water table. The top-feed method, suitable for more stable ground, involves a bore being formed and the aggregate tipped from the surface, the top-feed approach will be adopted within this research. For both construction methods, each lift of aggregate is vibrated (via the vibroflot) and the material is pushed outwards into the ground, creating a column of aggregate which has a greater diameter than the original bore.

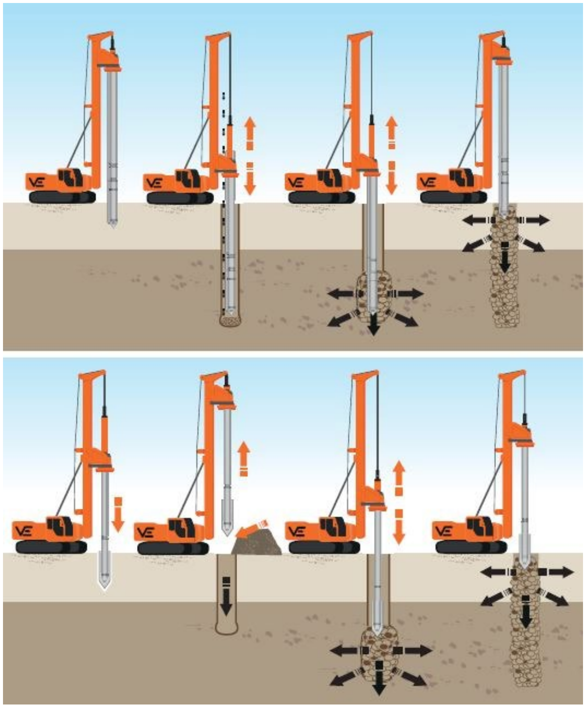


Figure 1-1: Bottom-Feed (top) and Top-Feed(bottom) granular column construction methods (Van Elle, 2020)

The installation of GCs within soft cohesive soils has the following main benefits (Hu, 1995):

- Increase in load bearing capacity of the ground (by introducing a stiffer material)
- Reduction in settlement
- Reduction in time required for consolidation to occur (by introducing additional drainage paths)

Granular columns can either be end-bearing (i.e., the columns toe into a more competent strata) or ‘floating’ where the columns terminate within a soft stratum. When the columns are loaded, they tend to bulge outwards, as shown in Figure 1.2, drawing on the lateral support (passive earth pressure) offered by the surrounding soil. When this mechanism is repeated for multiple columns, i.e., a group, this additional radial confinement generated by the column bulging works to increase the bearing capacity of the ground. The way in which granular columns transfer load to the ground differs to piled foundation, as it illustrated in Fig. 1.2. As piles are rigid structures they do not deform, or bulge, under load instead they achieve their capacity by skin friction and end bearing. The load transfer mechanisms of granular columns will be discussed in further detail in Chapter 2.0.

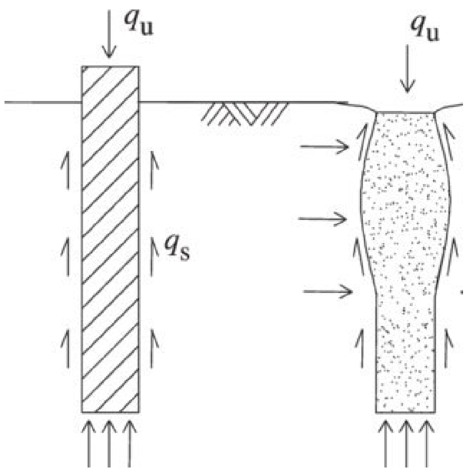


Figure 1-2: (a)) Mechanisms of load transfer for a rigid pile (left) and granular column (right) (Hughes and Withers, 1974)

1.2 Research Aims, Questions and Objectives

1.2.1 Aim:

The aim of this research is to investigate if recycled/secondary aggregates can be utilised successfully within granular columns, when compared to the performance of a primary aggregate (granite chips), via an experimental, laboratory based, investigation. If these materials are suitable then adoption by the industry would contribute to meeting the pressing need to develop the circular economy, reduce dependence on limited resources (i.e. primary aggregates, which can have engineering properties that exceed the requirements of the application they are used), reduce waste sent to landfill and sequester carbon dioxide (Alfieri *et al.*, 2012; Gunning and Hills, 2014), which is a timely and pressing need.

1.2.2 Questions:

To attempt to address the aim a number of interrelated challenges/uncertainties must be addressed, for example:

- How does the material behave from an engineering viewpoint, both in the short and long-term, when installed in a soft soil as a strengthening granular column, and is this behaviour exhibited consistent and reliable?
- Aggregates are often assessed/categorised for use in granular columns by index tests (such as the Aggregate Impact Value, AIV, Aggregate Crushing Value, ACV, and the Los Angeles Abrasion test, LA). Yet these index tests do not tend to reflect conditions a placed material might experience *in situ*, hence are the current specifications that dictate the potential suitability of aggregates, for use in geotechnical applications, fit for purpose?
- Do the stated (index) criteria reflect how the aggregates perform *in situ* or do they discount the use of RA and SA as they focus predominantly on particle response (i.e., crushing, abrasion, etc., where such materials are likely to perform poorly when compared to the performance of crushed rock and other PA)?
- Are these aggregate performance criteria (informed via the index tests) important if the aggregates are used in a confined environment (such as a granular column) rather than focusing on fabric behaviour (i.e., shear strength parameters, particle size distribution and shape of particles, interaction with water; such properties are related to more than just the particle performance)?
- To what extent do the current specifications (focusing on the index tests outcomes) inhibit the use of recycled/secondary aggregates in geotechnical engineering solutions? Hence, if these materials can be shown to be suitable for given geotechnical applications (via alternative methods to the index testing), are changes, or additional considerations, required in the performance standards so that they better represent the required performance of the proposed

aggregates? I.e., are these index tests fit for purpose or are they unfairly rejecting viable materials that could be used as alternatives to primary aggregates?

The project aim necessitates a significant body of research effort if it is to be fully addressed. It is accepted that more than one doctoral study is required to complete this research and it is hoped that this project inspires further research both in academia and in industry. The aim and objectives of this project were developed to provide fundamental knowledge to contribute to the research effort. The proposed objectives are described below.

1.2.3 Research objectives:

- To undertake two phases of testing: Phase 1 characterises the nature of the three types of RA/AA using the index tests; Phase 2 considers the performance of the RA/AA, with respect to the PA, in bench-scale granular column tests (applying the load via static loading and via a constant rate of strain, CRS).

Phase 1 Objectives:

- To undertake characterisation of the selected alternative aggregates by following standard geotechnical tests, and selected standards (index testing) for the acceptance of recycled/secondary/lightweight aggregates in geotechnical engineering applications
- To undertake characterisation of a PA in parallel with the investigation of the AA, and to compare the outcomes to draw conclusions based on the suitability of the aggregate materials (base on Phase 1 testing alone)

Phase 2 Objectives:

- To develop experiment procedures that allows the investigation of bench-scale testing of granular columns in soft clay soils (under CRS and static loads). This will include:
  - Validation of methodology and identification/limitation of boundary conditions
  - Developing a method to install and densify the AA/PA in the host soil to create the granular column
  - Develop a method to apply the load via CRS and statically
- To determine the suitability of the selected RA/SA for use as aggregates within granular columns through undertaking laboratory scale experimentation. Experimentation to follow approaches used in previous GC experimental investigations

Comparing Phase 1 and Phase 2 Outputs:

- To determine the relative performance of the AA against PA and to develop insight into the suitability of the characterisation/acceptance criteria when considering the performance of the AA (and PA) in the application of granular columns.
- To consider the suitability of the index tests, when used in isolation, to assess the suitability of these tests
- To determine if APCr, recycled container glass and/or crushed window glass would be suitable aggregates as a replacement for the primary aggregates (crushed granite)

1.3 Research Approach

Three materials, waste container glass, waste flat/window glass and APCr (a secondary aggregate produced by the carbonisation of incinerator ash) have been selected for investigation as potential granular column aggregate. The materials were initially assessed using the standard aggregate index tests. The aggregates were then utilised within laboratory-scale model granular column tests to assess their performance in this application. Two different types of load tests, static and constant-rate-of-strain, were conducted to enable an assessment of the column performance under different types of loading to enable an evaluation of the suitability of the laboratory test methodology itself. In addition, two loading plate sizes were trialed in attempt to assess the impact of the boundary conditions imposed by the test container.

It is acknowledged that laboratory studies have limitations with regards to granular column research (Serridge, 2016) . However, due to the main purpose of this research being to compare the relative behavior of a primary aggregate to recycled and secondary aggregates within the context of granular columns the implementation of a well-planned laboratory study has clear benefits.

1.4 Thesis Outline

The table below presents a brief overview of the structure of this thesis.

Table 1.1: Summary of thesis

Chapter	Summary
1. Introduction	*Summarises the area of the study and presents the aim and objectives*
2. Literature Review	*Summarises the current situation with regards to the use of AA within geotechnics and provides background information on the materials selected for investigation in this study*
3. Methodology	*The methodology adopted for both the aggregate index tests and the model column tests, static and constant-rate-of-strain, is presented along with validation of the approach taken.*
4. Discussion: Phase 1	*Presents the results and main findings of the aggregate index tests.*
5. Results: Model Column Tests	*Presents the data gathered in the model column tests*
6. Discussion: Phase 2	*Presents the main findings of the model column tests.*
7. Conclusion	*Summarises the key findings of the research and includes recommendations for future work*

1.5 Summary

This chapter summarised the key drivers for the adoption of AA within the geotechnical industry and presented the case for the use of AA within granular columns, a common ground improvement technique. An overview of the aims and objectives of the research, along with the approach taken to meet them, was also included. The overall aim of this research is to investigate the potential for three waste materials to act as aggregates in granular columns in place of primary aggregates. It is hoped that it will contribute to the body of knowledge in this area and help to encourage industry to increase the adoption of alternative aggregates within geotechnics, in place of primary aggregates, to ensure that finite resources of primary aggregates are used more sustainably and to have less material dumped in landfill; a win-win situation and a contributing step towards the UK developing a truly circular economy.

2 Literature Review

This chapter provides an overview of the motivation for the adoption of AA within granular columns (part 1) and provides the background information on granular columns that underpinned the experimental program (Part 2).

2.1 Part 1: Sustainability and the Construction Industry

The construction industry has significant impacts on the economy, society and the environment (Chang *et al.*, 2018). Sustainability within the industry and how this can be achieved have been a much discussed topic over the past few decades (Abreu *et al.*, 2008; Edil, 2009; Ortiz, Castells and Sonnemann, 2009; Yip Robin and Poon, 2009; Jefferson *et al.*, 2010; Chang *et al.*, 2018).

There are numerous definitions of the sustainability including the well-known “meeting the needs and expectations of the present without compromising future generations to meet their own needs and expectations” stated within the Brundtland Report (World Commission on Environment and Development, 1989). Basu, Misra and Puppla (2015) described sustainability as a system with supplies (i.e., capacity) and demands (i.e., loads), if the supply is greater than the demand then the system is sustainable.

The concept of the circular economy is a developing model related to sustainability designed to improve the efficiency of resource use and reduce losses (Geissdoerfer *et al.*, 2017). As the concept of the CE draws on the principles of multiple schools of thought to propose a new way of thinking of the economy it is difficult to reduce it to a single definition (BS 8001, 2017). However, the concept relies on three fundamental principles (Ellen Macarthur Foundation, 2016):

1. Preserve and enhance natural capital by controlling finite stocks and balancing renewable resource flows
2. Optimise resource yields by circulating products, components and materials in use at the highest utility at all times in both technical and biological cycles
3. Foster system effectiveness by revealing and designing out negative externalities.

It is well recognised that geotechnical engineering can significantly influence the sustainability of the built environment due to its early position within the construction process (Jefferis, 2005, 2008; Jefferson, Hunt and Rogers, 2009; Jefferson, Puppala and Al-Tabbaa, 2010; Basu, Misra and Puppala, 2015; Gajjar, Shah and Shah, 2019). Since geotechnical engineering utilises significant quantities of aggregates in various applications, that include the creation of embankments and cuttings, formation of road base and subbase layers, placement of fills for infrastructure and construction purposes, establishment of foundations (including granular columns) as well as being incorporated into concretes (Royal and Jefferson, 2017), the use of alternative aggregates obtained from more sustainable sources has become an important area of research.

Both the concepts of sustainability and the circular economy are relevant to the global aggregate industry, which is currently under stress as primary aggregates are a limited and valuable resource (Royal and Jefferson, 2017; Chen *et al.*, 2019). In addition, the engineering properties of many primary aggregates often exceed the requirements for a specific purpose and other materials would suffice. Another issue is the significant quantities of waste materials being created by the construction industry which are transported to landfill, or stockpiled in heaps, when, with some treatment or processing these wastes could prove to be an acceptable alternative to primary aggregates (Royal and Jefferson, 2017).

2.1.1 Motivation for the use of Recycled and Secondary Aggregates

The global annual production of primary aggregates, defined as aggregates sourced from natural materials such as rock and sand, is estimated at 40 to 50 billion tonnes, a figure that is increasing year in year (Tam, Soomro and Evangelista ACJ, 2018; Dhir *et al.*, 2017). Given that the construction industry accounts for 8 % of the UK GDP and consumes 25 % of the country's resources there is a great need for the industry to prioritize sustainability and the reduction of waste (Egan and Slocombe, 2010).

The introduction of the landfill tax in 1999 (HM Treasury, 2004) and the aggregate levy in 2002 (Taylor et al., 2006) caused a simultaneous increase in the cost of both sending waste to landfill and utilising natural aggregates. Alternative aggregates are exempt from the levy and the adoption of these materials also avoids the landfill tax, a double win (McKelvey, Sivakumar, Bell and McLaverty, 2004). Another factor is the continually increasing cost of transportation of materials (Chidiroglou *et al.*, 2008); utilising secondary and recycled aggregates that are located closer to construction sites would significantly reduce the impact of these costs (as well as reduce the environmental impacts associated with haulage). Therefore, the adoption of AA appears to make both environmental and economic sense.

Further information on the barrier to the widespread use of AA within geotechnics can be found in the published paper, '*The Use of Recycled and Secondary Aggregates to Achieve a Circular Economy within Geotechnical Engineering*, included in Appendix A. The key points of the paper will be briefly summarised in this chapter.

2.1.2 Barriers to the Widespread Adoption of Alternative Aggregates

The main barriers to the widespread utilisation of recycled and secondary aggregates have been identified as follows (Serridge, 2005; Silva, de Brito and Dhir, 2017):

1. Perception that they are inferior
2. Lack of suitable specifications/testing protocols
3. Lack of awareness
4. Waste management licensing restrictions and environmental issues
5. Supply – demand issues and economics.

2.1.3 The use of recycled and secondary aggregates within granular columns

As part of the drive to move towards a circular economy and increase the sustainability of the construction industry there is a desire to increase sustainability within geotechnical engineering, and specifically ground improvement, by adopting AA within granular columns (Serridge, 2005; R. Tranter, Jefferson and Ghataora, 2008; Serridge and Sarsby, 2010).

To add further motivation to the adoption of granular columns as a ground improvement method, the GC technique can yield reductions in the order of 90 % in terms of embodied carbon (Egan and Slocombe, 2010) when compared to a traditional piled solution. In addition, this 90 % reduction is based on the use of primary aggregate to form the columns and so could further be reduced through the adoption of AA.

To date, there has been a large research effort into the use of AA within concrete mixes (Bhatty and Reid, 1989; Frigione, 2010; Wagih *et al.*, 2013; Shafigh *et al.*, 2014; Lynn, Dhir and Ghataora, 2016; Thomas, Gupta and Panicker, 2016) highlighting the potential value of waste materials. There has also been a significant amount of research into the use of AA within the geotechnical industry but many of potential the applications identified have tended to be of relatively low utility (i.e., low value applications) such as fills. This is still a valid use of the material as fills can occupy large volumes, however, numerous researchers have identified, through laboratory based studies, that these AA could be used in higher utility applications such as granular columns (Serridge, 2005; Amini, 2015; Ayothiraman and Soumya, 2015; Royal and Jefferson, 2017; Moradi *et al.*, 2018; Zukri and Nazir, 2018; Shahverdi, 2020; Kazmi *et al.*, 2022).

The two main benefits of utilising AA for granular columns, which align with the principles of the circular economy, identified by (Serridge, 2005) are:

1. Reducing demand on primary aggregates
2. Reducing disposal of material to landfill

However, Serridge (2005) notes that the best practical option is a key consideration in aggregate use. For example, it is typically less impactful to use materials that are sourced locally (Serridge, 2005) as the cost, both financial and to the environment, of transporting aggregates tend to outweigh the benefits (Thomas *et al.*, 2009). That is, if PA are available much closer to the construction site, then the best option, in terms of the environmental impact, is likely to be the utilisation of these materials rather than the transportation of AA large distances; this is echoed by Jefferis (2008).

Over the past two decades there have been calls to increase the use of AA within granular columns from academia (Serridge, 2005; R. Tranter, Jefferson and Ghataora, 2008; Jefferson *et al.*, 2010; Zukri and Nazir, 2018) but little is published on their use in industry. To date there has been a number of laboratory based research projects investigating the potential use of recycled and secondary materials within granular columns; materials successfully identified as potential GA aggregates include tyre chips, fly ash, bottom ash, steel slag, crushed polypropylene (Amini, 2015; Reddy and Krishna, 2015; Zukri and Nazir, 2018; Moradi *et al.*, 2019).

There are many materials that may have the potential to be used within GCs that have not yet been researched .Two such materials, cited as a potential source for aggregate for GCs, are LECA (a lightweight aggregate formed of expanded clay) and waste glass (Serridge and Sarsby, 2010; Zukri and Nazir, 2018).

2.1.4 Materials Successfully Utilised Within Granular Columns

As mentioned previously, there are few published case studies on the use of alternative aggregates within granular columns which in contrast to the numerous laboratory studies. To date, materials successfully adopted include spent railway ballast, recycled crushed concrete and China clay waste (Serridge and Sarsby, 2010). Contractor Balfour Beatty has stated that it currently utilises approximately 40 % recycled aggregates for granular column construction, but the materials used are not specified (Balfour Beatty, 2022). Another contractor that specialises in granular columns, Keller, was contacted for comment and stated that alternative aggregates are used wherever possible and popular materials include spent ballast and reclaimed gravels. On occasion there is surplus 6F2 (perhaps from a used piling mat) and this material has been successfully used within GCs. The main limitations in the further adoption of AA, as identified by Keller, as detailed below, appear to align with those suggested by Serridge (2005):

- The material isn't produced to a good enough specification to meet Keller's requirements
- The benefit (cost + carbon) of recycled aggregate is completely eroded because it has to travel a great distance
- Insufficient testing has been carried out to demonstrate strict compliance factors and production in line with WRAP protocol

Keller also stated that usage of alternative aggregates is regional, the Midlands and the Southeast tend to utilise more AA than the North where typically quarried stone is used.

It seems the lack of publication of case studies and examples of the successful adoption of RA and SA within the granular column industry is hindering the further widespread usage of these materials. It would be of great benefit for those in industry, or those in academia working closely with industry, to publish more information relating to the use of AA as this would help promote the usage of these materials and increase awareness.

2.1.5 Possible Alternative Aggregates

Through the process of this literature review two types of waste, glass (both container and window/flat glass) and APCr (a waste treated via the accelerated carbonation technique) have been identified as potential AA for use within granular columns.

2.1.5.1 Waste Glass

There has been a considerable amount of research conducted into the use of waste glass within geotechnics (Butler and Hooper, 2011; Arulrajah *et al.*, 2013; Arul. Arulrajah *et al.*, 2014; Fauzi, Djauhari and Juniansyah Fauzi, 2016; Jamshidi *et al.*, 2016; Majdinasab and Yuan, 2019). Current applications of waste glass cullet (WGC) include aggregate within concrete mixes, drainage material, filter media, drainage blankets and load-bearing material in road pavements and asphalt aggregate projects (Atkins, 2009; M. M. Disfani *et al.*, 2011; Arul. Arulrajah *et al.*, 2014; Jamshidi *et al.*, 2016). In addition, case studies of WCG being successfully used as pipe bedding material in Australia have been published, and many states in America and New Zealand have adopted the WGC-blends for use in road construction (Majdinasab and Yuan, 2019).

The use of crushed waste glass within granular columns has been suggested by a number of researchers (Serridge and Sarsby, 2009; Zukri and Nazir, 2018) but to date research into the use of this material within GCs is limited to one recently published study (Kazmi *et al.*, 2022).

2.1.5.1.1 Glass Production and Use

The basic ingredients of glass are sand (60 %), sodium carbonate (21 %) and limestone (19 %) and certain additives are included to achieve different colours or properties (e.g., Iron is added to create brown or green glass and Alumina is added to improve durability) (Atkins, 2009). The production of glass is energy-intensive, requiring the consumption of large amounts of non-renewable resources, see Table 2.1 (M. M. Disfani *et al.*, 2011; A. Arulrajah *et al.*, 2014) hence, there is a ready market for (certain) recycled glass.

Table 2.1: Embodied Energy of Float and Container Glass

Glass Type	Embodied Energy (MJ/kg)	Reference
Float *(i.e., flat/window)*	15.9	(Atkins, 2009)
Container	12.7	(Camaratta, Volkmer and Osorio, 2020)

Glass is used in many different applications (e.g. windows, car windscreens, table wear) but the most common is in containers (Atkins, 2009). In the USA, the largest element of municipal solid waste is food and beverage bottles. Waste container glass cullet (WGC) (crushed glass created from waste glass bottles) is recycled within the glass manufacturing industry (i.e., to create new containers) where possible. Whilst this type of closed-loop recycling of WGA is an energy-efficient use of the material, potentially enabling reductions of 215–250 kg CO_2/tonnes and energy savings of greater than 1.5 GJ/tonnes to be achieved (Testa *et al.*, 2017), the direct re-use of containers would further reduce energy consumption. However, due to the phasing out of collection schemes for used glass containers (particularly in the UK), partly due to the location of treatment plants becoming more centralised, there has been an increase in the prevalence of comingled glass collection schemes (Atkins, 2009). This presents an obvious challenge, the difficult task of separating all WGC back into colour streams (which is required for use in the container glass manufacture due to the differing chemical compositions of each glass colour (Atkins, 2009; Oyedele, Ajayi and Kadiri, 2014). The maximum percentage of cullet that can be used in the manufacture of new glass containers is 70 % for amber, 60 % for flint and 90% for green whilst the composition of container glass (in terms of colour) recovered annually in the UK is 18 % amber, 30 % flint and 52 % green glass (Atkins, 2009). This produces a significant imbalance in the amount that is available, compared with the amount that can be recycled in glass production.

Whilst the varying chemical composition of waste glass, debris content and the ability to sort it by colour are limiting factors in the recycling rates of waste container glass (M. Disfani, Arulrajah, Bo, *et al.*, 2011; A. Arulrajah *et al.*, 2014), the recycling of flat glass (e.g., windows from buildings or vehicles) is substantially lower than cullet, which is largely due to the stricter quality requirements (Cuelho *et al.*, 2008; WRAP, 2008; Heriyanto, Pahlevani and Sahajwalla, 2018; Majdinasab and Yuan, 2019). This is disappointing as it is possible to directly reuse windows (e.g., from buildings), but due to constraints that include: additional time and labour costs for glass removal, ease of damage during transportation and difficulties in matching supply with demand, much of this is landfilled (Guthrie *et al.*, 1999; Butler and Hooper, 2011; Vieitez *et al.*, 2011; Testa *et al.*, 2017).

The constraints previously mentioned lead to significant quantities of waste glass being sent to landfill. Within the EU, this is estimated at 30 %, but this figure is much larger in other parts of the world, see Table 2.2.

Table 2.2: Global Container Glass Consumption and Recycling (Butler and Hooper, 2011)

Country	Re-use (%)	Recycle (%)	Landfill (%)
USA	0	28	72
Australia	0	37	63
Japan	0	14	86
China	50		50
EU	5	64	31

2.1.5.1.2 Properties of Waste Glass

The disposal of WGC in landfills is largely due to the lack of knowledge on the engineering properties of the material and the inclusion of contaminating material (non-recyclable glass) (Chidiroglou, O'Flaherty and Goodwin, 2009; M. Disfani, Arulrajah, Ali, *et al.*, 2011; A. Arulrajah *et al.*, 2014). This is extremely wasteful, especially since studies have shown that recycled glass exhibits geotechnical properties similar to PA, such as sands and gravels (Ali and Arulrajah, 2012; Mueller, Schnell and Ruebner, 2015) and research shows that recycled glass has some favourable characteristics, particularly when in finer fractions (i.e., maximum particle size less than 4.75 mm (M. Disfani, Arulrajah, Ali, *et al.*, 2011)), such as an LA coefficient comparable to natural granite (see Table 2.3). On the Mohr's hardness scale (where talc =1 and diamond = 10) glass has a value of 6 (Atkins, 2009). Furthermore, waste glass is an inert material and nonbiodegradable, thus staying in landfills for up to 1 million years (Amiri, Nazir and

Dehghanbanadaki, 2018) which is an obvious disadvantage in landfills but an advantage for AAs in geotechnical applications.

Table 2.3: Characteristics of Waste Glass Gullet (CWC, 1998; Wartman, Grubb and Nasim, 2004; M. M. Disfani *et al.*, 2011; A. Arulrajah *et al.*, 2014)

Characteristic	Particle size		
	Coarse ***(less than 19 mm)***	**Medium** ***(less than 9.5 mm)***	**Fine** ***(less than 4.75 mm)***
Specific Gravity (Mg/m^3)	2.48	2.48-2.5	2.5
Flakiness (%)	94.7	85.4	-
Standard Proctor Values			
Maximum Density (kN/m^3)	15.6-16.9	16.6-16.8	16.7
Optimum water content (%)	4.7-5.5	12.3-13.6	12.5
Modified Proctor Values			
Maximum Density (kN/m^3)	17.4-18.5	19.5	17.5
Optimum water content (%)	5.2-7.5	8.8	10
CBR	-	31-32	18-21
LA Abrasion Value (%)	29.9-41.7	24-25	-
ϕ' (from Direct Shear Test)(°)	-	47.5	-
ϕ' *(σ_n = 30-120 kPa)*	-	52-53	45-47
ϕ'*(σ_n = 60-240 kPa)*	-	50-51	42-43
ϕ'*(σ_n=120-480kPa)*	-	-	40-41
ϕ' (from Triaxial Test) (°)	42-46	47-48	
ϕ' *(σ'_c = 30-120 kPa)*	-	-	40
ϕ'(σ'_c *= 60-240 kPa)*	-	-	38
ϕ'*(σ'_c=120-480kPa)*	-	-	35
Hydraulic Conductivity (m/sec)	6.24 x 10-4	1.36-6.64 x 10-4	1.75 x 1-5

The effective angles of friction of waste glass cullet, as presented in Table 2.3, are similar to sands and gravels. Evidence suggests the smaller the particle size (in the coarse-grained size range), the less susceptible to crushing the glass becomes (Ooi *et al.*, 2008). However, the potential for particle crushing to occur should be considered when deciding on the application of the material, as this could lead to changes in behaviour when in situ (Ooi *et al.*, 2008).

2.1.5.1.3 Environmental Concerns

Prior to adopting a material, the potential negative effects on the environment, which is a particularly important topic for materials used within ground improvement, must be considered to prevent any harm to the surrounding environment. Disfani, Arulrajah and Bo (2011) found that in the context of highway construction no leaching hazards were presented by the use of waste glass, however, this is in contrast to the findings of other researchers (Tsai, Krogmann and Strom, 2009). In work by Tsai et al (2009) the potential contaminating effects of waste glass are discussed. Rainwater, or in the context of GCs ground water, can leach through waste cullet, dissolving and suspending contaminants such those from residual food/drink and paper labels. Based on this the glass used within this research will be washed twice as a minimum prior to being used to ensure the safety of the researcher. In practice, many waste glass treatment centres have the facility to wash the waste as it is being processed which is likely to be necessary should the material be utilised for granular columns. The use of water to wash materials would reduce the potential for chemicals to cause harm to those handling them whilst preventing the leaching of chemicals into the ground following column installation, although this would clearly increase the environmental impact.

The health and safety of the operatives working with the material also need to be considered prior to utilisation. One key concern over the use of waste glass cullet is the risk of cuts or penetration wounds, however, research by Ali (2012) indicates that particles less than 19 mm do not pose an increased risk of cuts compared to other angular primary aggregates.

2.1.5.1.4 Glass within Granular Columns

To date the use of crushed waste glass within granular columns is limited to one studied conducted by Kazmi (2022). End bearing columns were formed within a 150 x 150 x 150 mm shear box. The column was created using waste glass (maximum particle size of 1.55 mm) and the host soil was kaolin, compacted in three layers. This methodology is not common within granular column research as typically columns are formed within a tank (either rectangular or cylindrical) and are loaded vertically. As granular columns are unbound, they are not suitable to support lateral loading, so the use of the direct shear apparatus appears to be a strange approach. However, is it interesting to note that the inclusion of a waste glass column increased the angle of friction from 14 to 20.7 ° (48 %) when compared to a pure kaolin sample, suggesting some potential for ground improvement.

It is clear that this single study is not sufficient to confirm the suitability of waste glass within granular columns and so further research is much needed.

2.1.5.2 APCr

APCr (Air Pollution Control Residue) is a waste material generated during the process of municipal waste incineration (MSWI) and collected from the chimneys of the incinerator. This thermal residue needs to be disposed of as hazardous waste in landfill (at a cost of over £100 per tonne) (Hills, 2013), however, using the process of carbonisation the waste can be treated to create a viable secondary aggregate (Gunning and Hills, 2014). Currently approximately 15 % of municipal solid waste (MSW) within England is incinerated each year (producing over 1 Mt/pa of ashes) and the successful use of this waste material reduces hazardous waste being sent to landfill and the costs associated.

Carbonation is a naturally occurring reaction, which can be accelerated by exposing reactive materials to a higher concentration of carbon dioxide gas than that available from the atmosphere, a process known as accelerated carbon technology (ACT) (Gunning and Hills, 2014). Carbon dioxide primarily combines with calcium and magnesium to form calcium carbonate ($CaCO_3$). The resulting Calcium carbonate, more voluminous than the reactant minerals, infills pore space which leads to greater retention of contaminants. It through exploiting this mechanism, that fine grained powders, such as APCr, can be cemented together to form aggregates with re-use potential (Gunning and Hills, 2014).

The material formed has a bulk density in the range of 950 – 1100 kg/m^3 (Tota-maharaj, Hills and Monrose, 2017) meaning that it is classified as a lightweight aggregate (light weight aggregates are defined as material with a maximum bulk density of 1200 kg/m^3 (Gunning and Hills, 2014). The low unit weight of the material is an obvious advantage for certain applications such as backfill material. The average induvial pellet strength of the secondary aggregate is 0.28 MPa, exceeding the minimum 0.1 MPa stipulated by the End of Waste specifications (Gunning and Hills, 2014). It is important to note that the properties of the secondary aggregate produced by the accelerated carbonation of wastes fluctuate depending on the source material (Gunning and Hills, 2014).

To date this material has not been explored from a geotechnical perspective, however, some known material properties are presented in Table 2.4.

Table 2.4: Properties of APCr (Tota-maharaj, Hills and Monrose, 2017)

Property	**Value**
Particle size (mm)	4-16
Bulk Density (kg/m^3)	950-1100
Moisture Content *(as delivered)* (%)	8
Water Absorption (18.8 %)	18.8
LA Abrasion (%)	39

Research to date has shown that many wastes (such as Biomass Ash, Cement Kiln Dust, Clinical Ash, PSWI ASH, PFA and Sewage Ash) are suitable for treatment via accelerated carbonation. Wastes from industrial thermal processes, including paper ashes, biomass ashes, pulverised fuel ashes, steel slags and incineration residues have been found to be particularly suitable for the treatment due to their chemical composition (Gunning and Hills, 2014).

Whilst it appears that this waste product requires significant potentially energy intensive treatment when compared to aggregates produced from recycled materials, such as glass, in fact the process is carbon negative (including transportation to treatment plant) and low energy due to the low temperatures required (Gunning and Hills, 2014) for the process. In 2015 5000 t of CO_2 were captured (Hills, 2013).

Another critical aspect of the carbonisation process transforms the hazardous waste into a non-hazardous product. The aggregate has been tested extensively and has been awarded 'end-of-waste' (EOW) status by the Environment Agency (Gunning and Hills, 2014). In order for a waste to achieve EoW status, the following criteria, as set out in Article 28. by the Environmental Protection Agency, must be met (EPA, 2022):

•the substance or object is commonly used for specific purposes

•there is an existing market or demand for the substance or object

•the use is lawful (substance or object fulfils the technical requirements for the specific purposes and meets the existing legislation and standards applicable to products)

•the use will not lead to overall adverse environmental or human health impacts.

To date, secondary aggregate created via the accelerated carbonisation of APCr has been successfully utilised within permeable paving, concrete blocks and as growing media for green roofs but has not yet been used in any geotechnical applications (Hills, 2013; Tota-maharaj, Hills and Monrose, 2017). The strategic locations of the aggregate production plants, competitive pricing, aggregate tax exemption status together with their carbon negative status make these materials a very attractive option for the construction industry (Hills, 2013).

2.2 Part 2: Granular Columns

This section has been written to provide a summary of the granular column ground improvement technique. The key aspects of the technique including column design, material requirements, failure mechanisms and installation processes have been reviewed. It should be noted that the term 'granular columns' is used in this research is used to describe the vibro-replacement technique. This technique is also known as 'vibrostone columns' or 'granular piles' and all three terms are used interchangeably throughout the literature.

2.2.1 An Overview of Ground Improvement and the Benefits of Granular Columns

Nearly all construction projects require some form of ground improvement to be carried out to ensure that a structure has a solid foundation. The three main objectives of any ground improvement work are listed below (Mitchell and Jardine, 2002):

- Reduce and control deformation
- Reduce, control and exclude groundwater
- Reduce susceptibility to erosion

The means of achieving these objectives can fall into three categories (Mitchell and Jardine, 2002):

- Change the state of in situ soil, i.e., stronger, stiffer, denser and more durable
- Change the nature of the in-situ soil i.e., ground becomes different due to inclusion of other materials
- Change the response of in situ soil i.e., via the introduction of other materials the ground becomes a composite material and as a result has increased load carrying capacity and reduced settlements

The installation of granular columns changes the response of the in-situ soil, as detailed in the final bullet point.

2.2.2 The Granular Column Technique

The use of granular columns (GCs) is a ground improvement technique that has been commonly used since the 1960s, although the technique has been dated as far back as the 1830s (Hughes, J.M.; Withers, 1974). The installation of granular columns, a vibro-replacement technique, is currently one of the most common forms of ground improvement (Serridge and Sarsby, 2010;Mitchell and Jardine, 2012).

GC construction involves the partial replacement of in situ soils with compacted vertical columns of aggregates. GCs can be used to improve a variety of weak soils such as soft clay, loose sands and waste fills (Abhishek, Rajyalakshmi and Madhav, 2016), to provide a more suitable foundation for structures. The installation of GCs leads to improved bearing capacity and a reduction in settlements by introducing a material that is stiffer and has a greater angle of friction than the host soil (Muir-Wood, Hu and Nash, 2000) . The columns attract load and increase the process of consolidation in clay soils as the column provides a drainage path for water. GCs can also be used to improve the slope stability of embankments and to increase resistance to liquefaction (Barksdale and Bachus 1983; Alamgir et al. 1996).

When loaded, granular columns deform horizontally or 'bulge' into the soil, distributing stresses across the upper portion of the soil unlike rigid inclusions such as piles which derive load carrying capacity through a combination of skin friction and end bearing (Dheerendra Babu, Nayak and Shivashankar, 2013). Granular columns can also transfer load to greater depths and the extent of this is based on the geometry of the column (Muir-Wood, Hu and Nash, 2000). As GCs are unbound that do not have any capacity for tensile loads (Mohammed et al., 2017). GCs are primarily used to support compressive loads however, in some applications (e.g., at the toe of an embankment) they may be subject to lateral loads (Mohapatra and Rajagopal, 2014).

GCs are cohesionless and so their load bearing capacity is directly related to the internal angle of friction of the column material (Serridge and Sarsby, 2010). Other factors such as the density of the column, the shape/angularity of the column material, the strength of surrounding soil, the drainage conditions and the rate of deformation also have an influence (Hu, 1995). Within the UK the maximum friction angle adopted in stone column design is limited to 45° and is often further downgraded to 40° (Serridge, 2005).

Where shallow foundations are not viable (due to settlement criteria being unable to be met) and deep foundations (i.e. piles) are deemed uneconomical GCs can offer a solution (Mitchell and Jardine, 2002). GCs can offer considerable cost savings, both financial and time, when compared to conventional piling. GCs can also be a more sustainable option when compared to piled foundations although the extent of the reduction in environmental impact is in conjecture. According to Egan and Slocombe (2010) the reduction in embodied carbon achieved through the adoption of GCs can be up to 90 % but contractor Balfour Beatty estimates this reduction to be much lower at 10-25 % (Balfour Beatty, 2022). Either way, the adoption of GCs reduces embodied carbon and both these figures are based on columns formed using PA and so do not take into account the further potential benefits of utilising RA/SA. The embodied carbon reductions associated with the adoption of GCs can largely be attributed to the reduced requirement for materials such as steel and concrete and the reduced amount of spoil generated, translating to less muck away, less trucks on road (Egan and Slocombe, 2010; Serridge, 2005).

Achieving net zero is currently a target for many large geotechnical contractors (e.g. Skanska, Keller), as discussed in the paper titled ‘APPLYING THE CARBON HIERARCHY TO GEOTECHNICAL CONTRACTORS‘(Deamer et al., 2022), the adoption of AA within GCs could support this goal.

2.2.3 Applications and Limitations

Granular Columns can be used to support point loads (i.e. pad footings)(Hughes, Withers, 1974) however, GCs are typically used to support large raft foundations under relatively low loading conditions (Sivakumar, Bell and Black, 2011). To support these spread types of loads GCs are typically installed in groups, in either triangular or square arrangements. When columns are installed in groups, as is generally done in practice, the failure mechanisms differ to single columns due to group interaction (Hanna, Etezad and Ayadat, 2013). When columns are installed at small spacings the confinement provided by the surrounding columns prevents bulging occurring under load and so load is transmitted to greater depths (Muir Wood, Hu and Nash, 2000). In addition, the inclusion of multiple columns translates to additional drainage paths, potentially speeding up the consolidation process and influencing column behaviour.

Granular columns can be used to improve various types of grounds including loose silty sands and soft to firm cohesive soils (Dheerendra Babu, Nayak and Shivashankar, 2013). Research indicates that granular columns in cohesive soils are most effective at improving soils with undrained shear strengths in the range of 15-50 kN/m^2 (Barksdale and Bachus, 1983; Greenwood and Kirsch, 1984). The lower limit is due to very weak soil not offering sufficient lateral support to column. Other research indicates that the lower limit for soil strength able to be improved with GCs is a low as 5 kPa (Hu, 1995; Wehr, 2013). In the current specifying vibro-stone columns document (BRE, 2000) minimum soil strengths of 30 kN/m^2 and 20 kN/m^2 are recommended for dry top feed and dry bottom feed installation methods, respectively. The guidance also states that Soil strengths of less than 20 kN/m^2 may be acceptable in certain circumstances (BRE, 2000).

The sensitivity (st) of cohesive materials must also be taken into account prior to selecting stone columns as an improvement method. Sensitivity is defined by equation 2.1:

St = Cu undisturbed/ Cu remoulded ……………………………. **Equation 2.1**

Where Cu = undrained strength of the soil

Most clays have an *St* value of between 1 to 4 but values as high as 150 have been recorded for soils such as Scandinavian quick clays (McCabe, McNeill and Black, 2007). The use of a vibroflot in soils with high sensitivities can lead to considerable strength losses and ultimately unsuitable conditions for GCs as a ground improvement method (McCabe, McNeill and Black, 2007).

The plasticity Index (Ip) of a soil is also a potential limiting factor for the application of granular columns. The Ip value reflects the potential for shrink-swell behaviour of a soil. The National House Building Council (NHBC) recommends that soils with a greater Ip than 40 should not be treated with GCs (McCabe, McNeill and Black, 2007; NHBC Foundation, 2012).

The application of GCs within organic soils, such as peat, is to be treated with caution due to the high compressibility and lack of lateral support provided by the soil, potentially resulting in large vertical deflections. When the of layer organic material exceeds 1-2 column diameters the technique has historically not been recommended (Barksdale and Bachus, 1983). However, research conducted by Black *et al.* (2007) showed that the capacity of model columns installed in a host soil featuring a layer of peat (thickness equal to 3 time column diameter) was improved by wrapping the columns in a tubular sleeve (a technique known as cased granular columns) (Black *et al.*, 2007), suggesting that with additional measures GCs could be applicable in organic soils. More recent research has indicated that encased columns can be used to significantly improve the bearing capacity of collapsible soils (Al-Obaidy, Jefferson and Ghataora, 2016).

Sites where clay fill has been placed within less than 10 years are not likely to be suitable for treatment with GCs, although granular columns may increase the consolidation process, settlements induced by the load of a structure will not be controlled (McCabe, McNeill and Black, 2007). Equally, GC installation is not recommended for sites which are potentially susceptible to collapse failure (such as former quarry location filled in with loose backfill) due to the potential sudden loss of lateral support (Mitchell and Jardine, 2002; McCabe, McNeill and Black, 2007).

Generally ground improvement in non-cohesive soils is achieved by densifying the existing soil rather than using the vibro replacement technique of granular columns. Serridge and Slocombe provide guidance on when each technique is most applicable as shown by Figure 2.2 below.

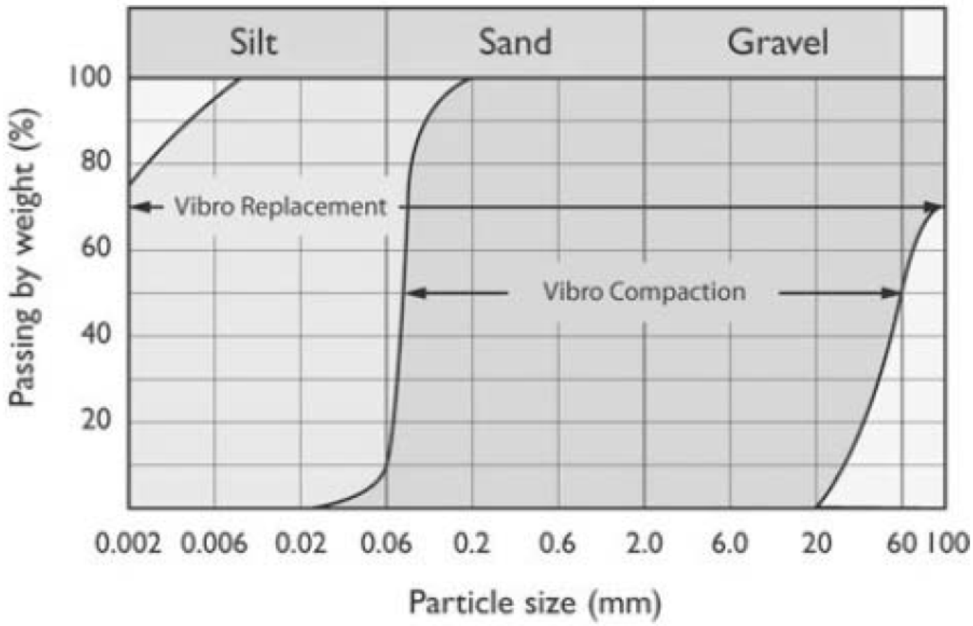


Figure 2-1: Ground conditions and treatment method applicable (Serridge and Slocombe, 2012)

In cohesive soils, such as clays, the key material parameters when designing granular columns is the strength of the host soil (as this determines the extent of lateral restraint provided to the column) and the friction angle of the column aggregate. Other important aspects of column design, that influence behaviour, are related to the geometry of the column, e.g., length/diameter ratio and area replacement ratio.

2.2.4 Specification and Guidance on Granular Columns

There are currently three main specifications and guidance documents that relate to granular columns in the UK:

- BS EN 14731: Ground treatment by deep vibration (2005) (BSI,2005)
- ICE specification for Ground Treatment (1987).
- NHBC Standards, Chapter 4.6 Vibratory Ground Improvement Techniques (2011).

Prior to the publication of EN 14731 the report published by the BRE 'Specifying Stone Columns' (2000) was a key document. This report has been recently archived but is still have available for purchase is still used as a reference by industry (Serridge and Slocombe, 2012). The ICE have provided useful guidance in volume 2 of the Manual of Geotechnical Engineering (Serridge and Slocombe, 2012) and the NHBC reference the use of alternative aggregates within GCs in their 2012 publication ' The use of recycled an secondary materials in residential construction' (NHBC Foundation, 2012). However, overall, the guidance provided for granular column does not seem to be as regularly updated as for other parts of the industry, such as piling where the ICE specification has been updated in 1996, 2007 and 2016. Considering that there has been a continuation of research since the publication of these guidance documents it appears that industry is not keeping up with academia.

2.2.5 Materials

Traditionally PA has been utilised for granular column construction and materials used include quarried rock such as granite and basalt (Serridge and Sarsby, 2010).

According to BS EN 14731 the following material properties are required for GC construction:

- The column material must be sufficiently hard and chemically inert (to maintain stability during installation and working life of the column)
- Column material must be appropriately graded to enable dense columns to form after compaction
- Column material must be compatible with construction plant used during installation i.e., does not cause blockages.

Particle shape is cited as important, in particular the flakiness index (BRE, 2000; Serridge and Slocombe, 2012). Particles which are flaky and elongated are not considered acceptable for stone column construction and it is preferable that properties include rounded, angular and irregular particles (Serridge and Sarsby, 2010). The specification also recommends the following (BRE, 2000):

- Chemically inert
- Able to withstand the impact forces of the poker and retain long-term integrity under applied foundation loads (i.e., it should not break or crush excessively during long-term static loads)
- Will not degrade if saturated with water.

Tests adopted to determine the suitability of an aggregate for use within GCs (according to criteria mentioned above) include (Serridge and Slocombe, 2012; BRE, 2000):

- Particle size distribution (PSD)
- Flakiness
- Ten percent Fines (TFV)
- Aggregate Crushing Value (ACV)
- Aggregate Impact Value (AIV)
- Los Angeles Abrasion Value (most recent test adopted to determine aggregate durability within the European Standards, superseding TFV, ACV and AIV).

Further information on these tests can be found in the Methodology chapter, Table 3.1.

The material used should be well graded, as uniformly aggregates may enable clay to penetrate into the column (Greenwood and Kirsch, 1984) thus reducing the drainage path and resulting rate of consolidation (Dheerendra Babu, Nayak and Shivashankar, 2013).

In work conducted by Brown (1977) a suitability number derived from the PSD curve for a material was suggested, as described by equation 2.2.

Equation 2.2:

$$Brown's\ suitability\ no. = 1.7\sqrt{\frac{3}{D_{50}^2} + \frac{1}{D_{20}^2} + \frac{1}{D_{10}^2}}$$

Where D_{50}, D_{20} and D_{10} (mm) are 50, 20 and 10 % grain size diameters in mm.

Table 2.5: Brown's suitability numbers (Brown, 1977)

Suitability Number	0-10	10-20	20-30	>50
Rating	Excellent	Good	Fair	Unsuitable

Greenwood and Kirch (1984) support Brown's suitability number as a guide to suitable PSDs for use within GCS but state that permeability is also an important factor in determining the effectiveness of compaction. Permeabilities below 10^{-3} cm/s reduce the effectiveness of compaction efforts but rates greater than 1 cm/s slows penetration due to water losses (Greenwood and Kirsch, 1984).

2.2.6 Installation methods

There are three principal methods of stone column installation, dry top-feed, wet process and dry bottom-feed (BS EN 14731, 2005). The method adopted is dependent on the ground conditions encountered. The different methods are summarised in Table 2.6.

It is preferable that material used during the dry-bottom feed process is rounded as this reduces the potential for blockage of the feed tube (Serridge and Sarsby, 2010), it also recommended that the fines content (defined as particles less than 0.063 mm) is kept below 5 % for the same reason. In addition, smaller aggregate sizes tend to be used in the bottom feed systems to further prevent blockages occurring in the feed tube (Serridge and Sarsby, 2010).

The dry-bottom technique is now the most common method of installation due to its lower environmental impact (the wet method requires large quantities of water and creates effluent) (Serridge and Slocombe). Typically, the diameter of column formed using the dry-technique is smaller than when the wet-technique is adopted (Serridge and Slocombe, 2012).

Table 2.6: Summary of column installation methods

Method	Description	Applications	Reference
Dry top-feed	The depth vibrator displaces soil to the column design depth (penetration is assisted with pressurised air). The depth vibrator is held in position for a short time and is then fully retracted. Aggregate is then placed into the bore in lifts, with the depth vibrator being reintroduced into bore to compact each lift. **Aggregates to be graded 40-75mm**	• In soils where an open bore would not be stable • Above the water table	www.keller.com BS EN 14731:, 2005)
Wet top-feed	Method is similar to the dry top-feed but water is used to assist penetration of the depth vibrator. The use of water enables relatively fast penetration through weak soils. At design depth the water flow is reduced and aggregate, which has been heaped around the bore at ground surface, is pushed into the bore. The aggregate passes down the annulus between the depth vibrator and sides of the excavation enabling a column to be formed in short lifts, with each lift being compacted with the vibrator. This method has drawbacks including the requirement of a substantial water supply and controlled disposal of the resulting slurry. **Aggregates to be graded 25-75mm**	• Utilised in weak soils where dry-top feed is not feasible due to unstable ground • Suitable for use in coastal or estuarine environments which feature soft soils	(BS EN 14731:, 2005) (Serridge and Slocombe, 2012)

Dry bottom-feed	The depth vibrator displaces soil to the column design depth (penetration is assisted with pressurised air). As the depth vibrator is retracted aggregate is discharged through the supply tube attached to the vibrator. The depth vibrator is then re penetrated to compact the aggregate in lifts. To avoid blockages forming in the installation tube the aggregate particles suitable are smaller than for the top-feed method and particle shape becomes more important (i.e., more rounded particles are preferable to avoid the installation tube becoming jammed) **Aggregates to be graded 8-50mm**	• In soils where an open bore would not be stable • This approach has largely superseded the wet bottom-feed technique in the UK	www.keller.com (Serridge, 2005) (Serridge and Slocombe, 2012)

2.2.7 Plant and Equipment used for Granular Column Installation

Each of the installation methods utilise a depth vibrator, see Figure 2.2, (also known as a vibrating poker or vibroflot) which comprises an eccentric weight which rotates inside a tubular steel casing (see Figure 2.3). The end of the depth vibrator is tapered to aid penetration (BS EN 14731:, 2005). Vibration frequencies usually set at 30 Hz or 50 Hz depending on electricity source (Greenwood and Kirsch, 1984). The depth vibrator is usually equipped with instrumentation which measures the power output and the depth of penetration (Greenwood and Kirsch, 1984).

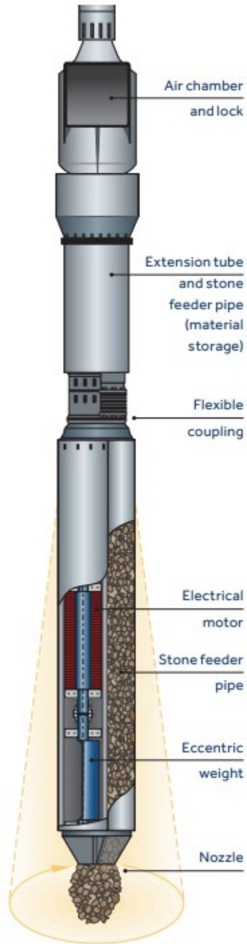


Figure 2-1: Schematic of a Depth Vibrator (Keller, 2022)

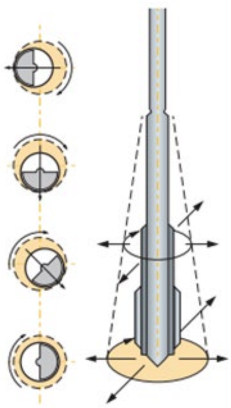


Figure 2-2: Illustration of Movement of a Depth Vibrator During Column Installation (Keller, 2022)

2.2.8 Quality Control

During column installation the vibrating poker, depth vibrator (or vibroflot) is able to measure the resistance to penetration and feed this information back to rig, enabling the quantity of backfill material to be adjusted accordingly (McCabe, McNeill and Black, 2007).

Due to the depth vibrator creating a bore which is larger than its diameter it tends to hang like a pendulum within the bore, ensuring that bores created are always vertical (Greenwood and Kirsch, 1984). Verticality is an important factor in achieving successful construction and functionality of stone columns, a maximum deviation of 1 in 20 is typically specified (Serridge, 2013).

Following column installation there are two main types of tests that are used to confirm the quality. These tests are detailed in the ICE specification for Ground Treatment (1987) and include plate bearing tests and zone tests. Plate bearing tests comprise the application of a load, that is 3 times working load, to a single column, in 5 equal increments. The settlement is measured at 5-minute intervals and the next load increment is not applied until zero change in settlement is recorded over a 10-minute period. The settlements measured during these type of tests may not be representative of the foundation behaviour due to the difference in duration that loads are applied and the depth to which the ground is stressed (McCabe, McNeill and Black, 2007). The second type of test is the zone test where loads of up to 250 % working load are applied across a larger area (usually a slab over a number of columns) in 3 steps that are increased when the settlement rate is less than 0.5 mm/hr. Zone tests are considered to provide a closer reflection of column behaviour due to the application of load to a full scale section of a foundation, however, due to the high costs associated with these tests they are usually limited to marginal sites (McCabe, McNeill and Black, 2007).

2.2.9 Granular Column Design

When granular columns are loaded end bearing resistance and side friction (similar to piles) are generated to some extent (Sivakumar *et al.*, 2004). Granular columns can either toe into firmer strata (known as 'end-bearing') or terminate within soft soil (known as 'floating'). However, due to being unbound and their compressibility, granular columns deform laterally into the surrounding soil (described as 'bulging'), and form a composite soil-granular column system where the lateral support provided by the soil prevents further expansion (Black, Sivakumar and McKinley, 2007). The increased lateral stresses within the clay lead to an increase to further consolidation and enhanced resistance to bulging, a process that continues until equilibrium is reached (Sivakumar *et al.*, 2004; Black, Sivakumar and McKinley, 2007). The phenomenon of bulging means that stresses can be transferred to the soil in the upper region of the column rather than transferring to deeper strata (as is the case with piled foundations). This type of load transfer makes GCs ideal for relatively light loads but less effective for heavier loads due to the lower extent of load transfer to deeper, more competent strata (Hughes, J.M.; Withers, 1974; Sivakumar, Bell and Black, 2011).

The extent of bulging is very dependent on the strength of the in situ soil, with soils of greater strength offering increased passive resistance to support the stone column (Hughes, J.M.; Withers, 1974). The increase in lateral stress induced within the clay (as a result of granular column installation) leads to an increase in the rate of consolidation and an increased resistance to column bulging until equilibrium is reached (Black, Sivakumar and McKinley, 2007). This phenomenon is echoed in the pressuremeter test, the results of which show that during cylinder expansion the soil reaches a limiting value where indefinite expansion occurs (Hughes, J.M.; Withers, 1974). Gibson and Anderson (1961) developed a relationship which enables the limiting stress to be estimated (assuming the soil acts an elasto-plastic material (Eq. 3) (Hughes, J.M.; Withers, 1974):

$$\boldsymbol{\sigma_{rL} = \sigma_{ro} + c\left[log_e \frac{E}{2c(1+\mu)}\right]} \text{..Equation 2.3}$$

Where σ_{ro}= Total in situ lateral stress, E = Elastic modulus, μ = Possion's ratio, c= undrained shear strength of the soil

This equation has been approximated using pressuremeter data to Eq. 4 (Hughes, J.M.; Withers, 1974):

$$\boldsymbol{\sigma_{rL} = \sigma'_{ro} + 4c + U} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \text{..Equation 2.4}$$

Where U = Porewater Pressure

The ultimate vertical stress which a single column can support, assuming it bulges laterally, is described by eq. 5 (Hughes, J.M.; Withers, 1974).

$$\boldsymbol{\sigma'_v = \frac{(1+\sin\ \phi')}{(1-\sin\phi')}(\sigma_{ro} + 4c - U)} \quad \text{......................................Equation 2.5}$$

Where σ'_v = ultimate vertical stress, φ' = angle of internal friction of column material.

2.2.10 Consolidation

As mentioned previously, one of the ways in which GCs work as an effective ground Improvement method is to increase the rate of consolidation by increasing the number of drainage paths in a soil (Sondermann et al., 2016; Serridge and Slocombe, 2012). In laboratory research on granular columns within kaolin, by Sivakumar et al. (2020,) it was shown that the presence of GCs more than halved the time taken for primary consolidation to occur.

Secondary consolidation (or creep) occurs once primary consolidation is complete and can be a significant contributor to the overall settlement in applications such as embankments (Serridge and Slocombe, 2012). The ability of GCs to reduce long-term creep settlements is an area in which there has been limited research to date (Cimentada et al., 2009; Sivakumar et al., 2021).

2.2.11 Failure Modes

Various modes of failure have been observed for GCs and these largely depend on the geometric configuration of the columns (Muir-Wood, Hu and Nash, 2000). These modes of failure have been summarised by Barksdale and Bacchus (1983) and Muir-Wood (2000) and are presented below in Figure 2.3. It is important to note that the failure mechanisms of single columns differ from column groups as they do not have the influence of surrounding columns such as additional confinement.

According to Barksdale and Bacchus (1983), small scale studies indicate that the settlement behaviour of single columns is significantly affected by how the load is applied. For example, applying load through a rigid foundation that is of a greater diameter than the column increases the bearing area and the lateral support offered by the host soil is also increased due the greater vertical and lateral stresses induced in the soil, leading to greater load capacity of the column.

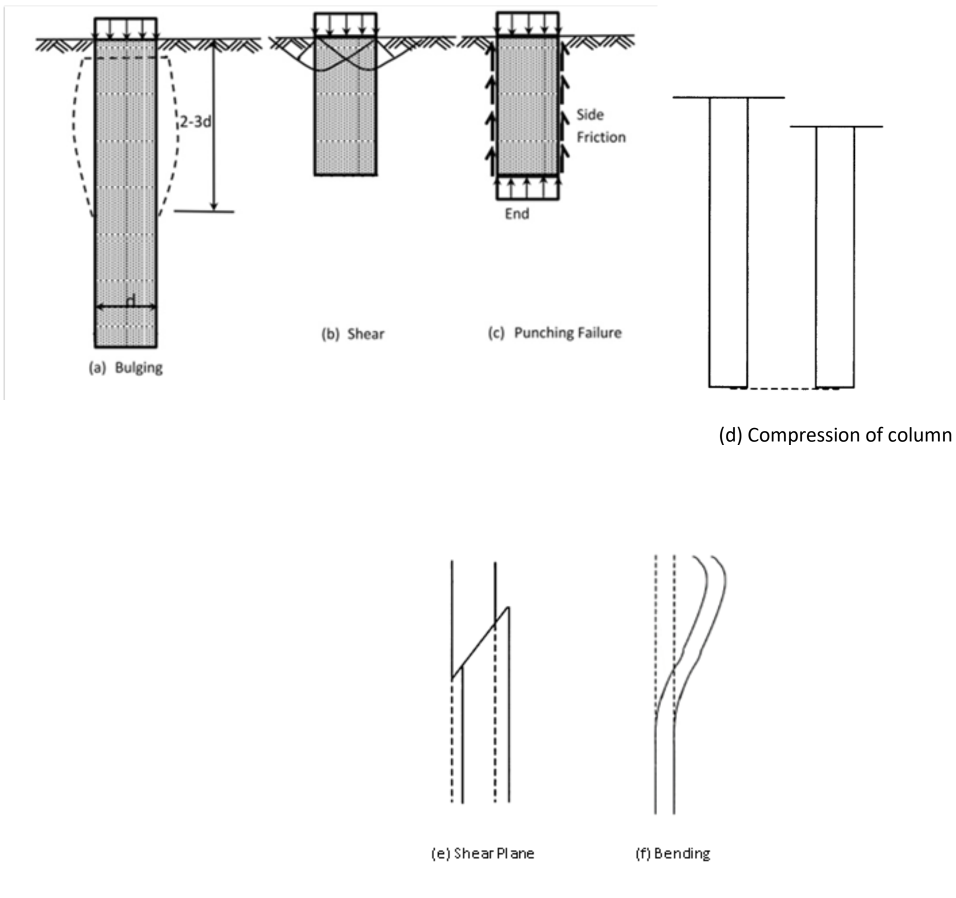


Figure 2-3: Possible failure modes of granular columns (Barksdale and Bachus, 1983; Muir-Wood, Hu and Nash, 2000; Al-Obaidy, Jefferson and Ghataora, 2016)

(a) Bulging: If load is applied to a column and the lateral pressure exerted by the column exceeds the passive pressure provided by the host soil (either through the presence of a group of columns or simply the shear strength of the soil) then bulging will occur. Bulging occurs until an equilibrium is reached and generally occurs to a depth up to 4 times the column diameter (Hughes, J.M.; Withers, 1974; Sivakumar *et al.*, 2004).

(b) General shear failure: This type of surface failure is observed when the column length is short and columns bear onto a firm strata (Barksdale and Bachus, 1983; Al-Obaidy, Jefferson and Ghataora, 2016)

(c) Punching: If columns are short (defined by McKelvey (2004) as less than 6d in length) then there is potential for punching failure to occur (i.e., the column deflects vertically into the soft soil below the toe). Penetration into is more prevalent with higher area replacement ratios. In the case of short, end-bearing columns punching failure can occur before bulging happens (Barksdale and Bachus, 1983; Amini, 2015). In work conducted by McKelvey et al. (2004) it was found that short, sand columns had penetrated the underlying clay by 10 mm.

(d) Compression of Column: As column length increases the potential for punching failure reduces as more of the foundation settlement is absorbed throughout the length of the column (Muir-Wood, Hu and Nash, 2000).

(e) Shear Plane: If columns experience high stress ratios within soil that offers little lateral restraint shear planes may form through the column (Muir-Wood, Hu and Nash, 2000). This type of failure as observed in work by McKelvey et al. (2004) in longer columns (i.e., L/d >6).

(f) Bending: As granular columns are unbound they offer relatively little resistance to lateral loads and so bending failure can occur as a result of horizontal loading, especially if columns are slender (Muir-Wood, Hu and Nash, 2000).

2.2.12 Design Factors and Granular Column Geometry

2.2.12.1 Diameter and Area Replacement Ratio

The diameter of a granular column plays a major part in increasing the bearing capacity and reducing the corresponding settlement of the stone column (Greenwood, 1975). The diameter of columns formed in practice is dependent on the in situ soil; in softer soils a larger dimeter will be formed by the depth vibrator than in stronger soils (Greenwood and Kirsch, 1984). In practice GCs typically have a diameter of between 0.6 – 1.2m (BS EN 14731).

The area replacement ratio can be defined as the fraction of soil within the unit which is replaced by the column material, see equation 2.6 (Barksdale and Bachus, 1983).

$A_r = A_c / A$ ……………………………………………….**Equation 2.6**

Where A = total area within the unit cell, Ar = area replacement ratio, Ac = area of the stone column after compaction.

The area replacement ratio has a significant effect on the degree of improvement accomplished, i.e., in general increasing the area replacement ratio leads to an improvement in the overall response of a granular column.

The recommended area replacement ratio within the literature varies. In work conducted by Muir-Wood et al. (2000) on model columns within clay a minimum A_r of 25 % was recommended, however, the optimum replacement ratio for settlement control was found to be 30-40 % by Black *et al.* (2011), which concurs with the finding of Barksdale and Bachus (1983). The optimum A_r identified by Black et al. (2011) is based on the finding that for floating columns in particular there was a significant increase in the settlement reduction of columns with increase in area replacement ratio but a threshold of 30-40 % was observed.

2.2.12.2 Critical Length

The critical length can be defined as the shortest column length that can carry the ultimate load, not considering settlement (Hughes, Withers and Greenwood, 1975). It has been found that no significant effect on load bearing capacity is found when column length exceeds five times column diameter (Hu, 1995; Sivakumar *et al.*, 2004).

Additional research has found that columns with a ratio of L/d < 6 (i.e., column length/column diameter) tend to fail in end bearing and columns with L/d > 6 fail in bulging (McKelvey 2002). This concurs with the work of Hughes and Withers (1974) which found that no increase in load carrying capacity was achieved beyond a column depth to diameter ratio of 6.3.

2.2.12.3 Stress Concentration Factor

The stress concentration factor (*n*) can be defined as the ratio of stress within the column to the stress in the surrounding soil (Dheerendra Babu, Nayak and Shivashankar, 2013). The magnitude of the stress concentration depends on the relative stiffness of the column and surrounding soil and decreases along the length of the column (e.g., values of n at the surface are typically 3-4 but overall values of 2-6 are expected) (Dheerendra Babu, Nayak and Shivashankar, 2013). Other factors that affect the stress concentration factor are the time of consolidation and the area replacement ration. Research has indicated that *n* is greater for end bearing columns than for floating columns (Fattah et al., 2011).

2.2.13 Laboratory Modelling

Over the past decades there have been numerous laboratory studies into GCs (Muir-Wood, Hu and Nash, 2000; McKelvey, Sivakumar, Bell and Graham, 2004; Weber, Laue and Springman, 2006; McCabe, McNeill and Black, 2007). Some of these studies utilise a centrifuge and others, more commonly, adopt a 1 g approach. There has been some doubt over the relevancy of laboratory studies (due to their inability to fully represent reality) but it has been argued that for research and development purposes they offer a relatively low cost, effective and viable approach to preliminary investigation (Black *et al.*, 2006; Serridge, 2016). Ideally these small scale studies would be supported by further field based (i.e. full scale) tests (Serridge, 2016).

An additional benefit of laboratory scale tests is that they are not hindered by the difficulties faced when attempting to instrument a full scale column (Hughes, Withers, 1974). In order for model tests to represent reality in terms of stresses induced due to gravity a centrifuge is required, however Hughes and Withers (1974) suggest that 1 g tests provide sufficient data to enable some analysis of column behaviour under loading.

An important consideration when modelling columns in the laboratory is the boundary conditions. When model columns are tested under lateral restrained conditions, i.e., under the confinement of a test cell, a non-uniform pressure distribution can be created, therefore making analysis difficult (Black, Sivakumar and McKinley, 2007). In practice this can be avoided by utilising an appropriately sized tank (i.e., where the tank size exceeds boundary effects) and applying lubrication to the sides of the tank therefore reducing vertical friction generated. Boundary conditions are further discussed in section 3.4.1.

2.2.13.1 Unit Cell concept

The unit cell approach is the most common simplification adopted for GC design (Sondermann, Daramalinggam and Yohannes, 2016). Barksdale and Bachus (1983) stated that around each stone column there is an area of soil associated with each column, an equivalent diameter (D_e) which represents a treatment with a particular area replacement ratio. This is most closely represented by a hexagonal shape but can be approximated to a circle with an effective diameter, depending on the shape of the grid pattern (triangular or square), as detailed below. This is known as the unit cell.

In the case of a large grid of GCs the unit cell can be considered as representative of the whole treatment area (Sondermann, Daramalinggam and Yohannes, 2016).

D_e = 1.05 s (Triangular grid pattern)

D_e = 1.13 s (square grid pattern)

Where s = grid spacing of the columns

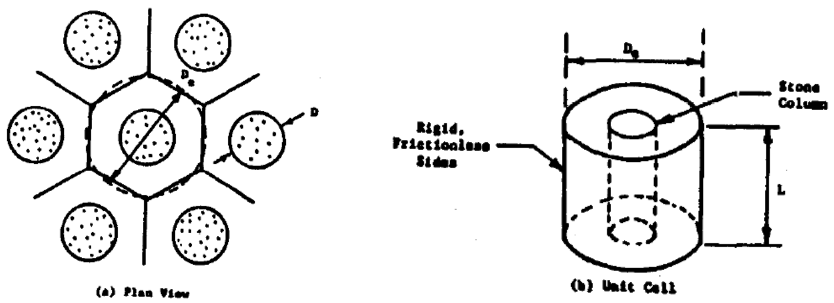


Figure 2-4: Sketch of the unit cell in plan view (a) and from the side (b) (Barksdale and Bachus, 1983)

2.2.13.2 Monitoring and Instrumentation

The key item monitored during laboratory model tests of granular column is the vertical displacement (settlement) of the footing (Sivakumar *et al.*, 2004; Black, Sivakumar and McKinley, 2007; Sivakumar, Bell and Black, 2011; Amini, 2015). In many studies an adaption of the triaxial cell was utilised to test model columns and so the cell pressure was known (Sivakumar *et al.*, 2004; Sivakumar, Bell and Black, 2011). Where the triaxial apparatus was not used and the confining pressure of the cell was unknown, the sides of the test cell were monitored using LVDTs to enable any movement of the sides of the test cell to be measured (Al-Obaidy, Jefferson and Ghataora, 2016). In work by Hu (1995) the surface of the clay was monitored during loading, using a combination of LVDTs and photography, to enable the area of influence of the column to be identified.

In a number of studies pressure cells were utilised to monitor contact pressures at the centre of the foundation and in the surrounding soil. Research carried out by Sivakumar et al (2011) showed that the use of pressure cells along the vertical sides of column has a stiffening effect and significantly alters results when compared to a tests without pressure cells. Sivakumar *et al.* (2011) found that measured settlements without the use of pressure cells were almost double that of the sample which did include pressure cells. In work by Hu (1995) miniature pressure transducers were used to monitor the surface of the surface of the loaded area and within the clay mid-way between the columns. The results were scattered and so of limited success (Hu, 1995), however, some useful general observations were able to be made, such as the stress within the columns being greater than in the surrounding soil.

In addition to the monitoring conducted during the load tests on granular columns other parameters, such as the shear strength and water content of host soil, PSD of column aggregate, density of column as installed , were measured prior to the commencement of the test (Hu, 1995; Muir-Wood, Hu and Nash, 2000; Amini, 2015; Al-Obaidy, Jefferson and Ghataora, 2016).In many of the studies post-load test models of the columns were created in order to investigate the deformed shape and draw conclusions on failure mechanisms (Hu, 1995; Muir-Wood, Hu and Nash, 2000; McKelvey, Sivakumar, Bell and Graham, 2004; Amini, 2015). In work by Amini (2015) the PSD tests on the aggregates were conducted post-test to determine the extent of particle breakage, an aspect that is considered important when investigating AA.

2.3 Conclusion

In conclusion, there is a clear need to increase the adoption of recycled and secondary aggregates within the construction and industry and specifically within geotechnics. The ground improvement technique, granular columns, is one potential application for these alternative aggregates.

There are many published studies on granular columns generally and also the use of AA in this application. However, the secondary aggregate created via the carbonisation of air pollution control residue (APCr) has not yet been tested geotechnically and the use of waste glass (both container and window) has not yet been studied in depth within the context of granular columns and so both of these material merit further research.

3 Methodology

3.1 Introduction

This chapter details the approach taken to investigating the suitability for AA to be utilised within GCs. The overall aim of the research was to compare the performance of recycled materials against a primary aggregate when used in granular columns, and this is reflected in the approach described here.

In this chapter the materials adopted for the research are introduced and the index tests carried out are detailed. The main focus of the project was a series of single-column model load tests in soft cohesive soil (Kaolin) designed to investigate the short-term behaviour of GCs formed using AA. The methods that have been adopted for all aspects of the research are drawn from the literature and the supporting research underpinning this is discussed. The benefits and limitations of the methodology chosen are also detailed within this chapter.

The alternative aggregates adopted were container glass (sourced from Viridor waste, Sheffield), window glass (sourced from a demolished University of Birmingham building) and APCr (sourced from the University of Greenwich). This chapter outlines the fundamental principles and practices that underpinned the research methodology of the model granular column tests along with validation/quality assurance of the testing. Testing was split into two phases: the first was a preliminary analysis of the four materials using classical geotechnical tests (e.g. particle size distribution, shear box and proctor compaction)) and 'industry standard' aggregate tests (such as the aggregate impact value, aggregate crushing value and the Los Angeles Abrasion) to try to determine how the AA behave when compared to the granite; the second was the main phase of testing and this comprised static load and constant rate of strain testing of granular columns in a compacted clay soil (English China Clay), again to compare relative performance of the AA to granite.

The nature of standard tests means that the methods are well established, hence these have not been reported herein (aside from a brief description of the methods/standards followed when undertaking these tests). Therefore, this chapter predominantly focuses upon the development and validation of the bespoke granular column tests (which were developed from previously reported approaches in the literature) to investigate the behaviour of the AA with respect to the granite.

It should be noted that the main phase of study commenced in November 2019, which was just prior to the first Covid-19 national lockdown in March 2020. The laboratories remained closed to the author until November 2020 and when access was granted it was limited to practices that met the covid-safe working policies adopted by the University. This clearly had a disruption to the research programme, hence the number of tests that could be undertaken were reduced. This was not only due to lost time due to the lockdown but also due to implications of the social distancing and covid-safe working practices adopted within the laboratory. These had unexpected consequences upon the experimental phase; for example, only a limited number of people could use the soil mixers at any time, limiting the amount of clay that could be prepared in a week and hence limiting the number of tests that could be undertaken. Despite the restrictions on the second phase of testing, a dataset was gathered, using the methodology developed for the project, that clearly indicates differences in response to the load applied for the four aggregate types (considered in more detail in the Results and Discussions chapters).

3.2 Phase 1: Standard Testing to Consider Various Geotechnical Engineering/Aggregate Parameters of the Four Aggregates

The ‘standard’ test undertaken; to consider the relative performance of various geotechnical engineering parameters of the aggregates, are described in Table 3.1. Outcomes from these tests appear in chapter 4.0.

Table 3.1: Summary of aggregate index tests

Test	Brief Description	Comments	Reference
Particle Size Distribution (PSD)	The aim of PSD tests was to determine the initial grading of the materials both to understand how this affects column behaviour and also to enable extent of particle breakage post-test to be measured.	Sieve time was 10 minutes, as recommended by Head (2006)	BS 1377:Part 2:1990:9.3
Loose Bulk density	Measurement of the density of the material in an uncompacted state		BS EN 1097-3:1998
Particle Density	Particle density is a measure of the average density of the solid particles in a sample	The large pycnometer method was adopted	BS 1377: Part 2: 1990: 8.4
Proctor Compaction	The aim was to understand the relationship between compaction curve and to obtain the optimum moisture content and the maximum dry density.	For all aggregates a fresh sample of material was used for each water content due to the potential for particle breakage. This meant that there was a limit to the number of tests that could be conducted due to limited amounts of the aggregates being available.	BS 1377- 4:1990
Direct Shear	The direct shear tests, conducted using the shear box, enables a value of the angle of friction (ϕ’) to be determined.	During the period of time that this research was conducted only a 60 x 60 mm shear box was available. The author acknowledges that this is not the most appropriate size of apparatus to use for the	BS EN ISO 17892-10:2018

		materials used and these results are indicative only.	
Aggregate Crushing Value (ACV)	ACV gives a relative measure of the resistance of an aggregate under compressive load which is applied over 10 minutes in equal increments until 400kN is reached. The resulting fines enable an estimation of the ACV. ACV = M2/M1 x 100 Where M2 is the mass of material passing the 1.18mm sieve, M1 = mass of the test sample.	The standard range of particles sizes for the test is 14 -10mm. As the particles of the materials to be tested were smaller (9.5-1mm), the alternative test procedure (for particles sized 6.30 -5.0mm) was followed. This alternative procedure involved using a 1.18mm sieve.	BS ISO 20290-2:2019
Aggregate Impact Value	The aggregate impact test is designed to provide a relative measure of the resistance of an aggregate to a sudden shock or impact. The test comprises the repeated dropping of a 13.5-14 kg hammer onto a sample of fixed size. $m = M_2/M_1 \times 100$ Where M2 = is mass of particles passing the 1.18mm sieve, M1 is the total mass of the sample	The standard range of particles sizes for the test is 14 -10mm. As the particles of the materials to be tested were smaller (9.5-1mm), the alternative test procedure (for particles sized 6.30 -5.0mm) was followed. This alternative procedure involved using a 1.18mm sieve.	BS 812-112:1990

Los Angeles Abrasion (LA)	The Los Angeles (LA) test has replaced the Ten Percent Fines value (TFV) in determining the resistance to fragmentation of aggregates for use within granular columns (Serridge, 2005)	An alternative method, detailed in appendix B.1 of the standard, was adopted due to the particle size range of aggregates used in research being smaller than the standard range reference range of 10/20 mm. The standard notes that the results given for the alterative size fractions may not be identical to the standard test, however, given that all materials were testing using the same method the results obtained are directly comparable. Caution just needs to be exercised when comparing results to those published in other research.	BS ISO 20290-2:2019
Flakiness Index	The flakiness index is measure of the thin and flat particles (greater than 6.3 mm) within a sample. Flakiness index = (M3)/(M2) Where M3 = Mass of all particles passing through the gauge and M2 = Total mass of particles, any size fractions which represent less than 5 % of the sample are to be discarded	In this research the test method detailed in British Standard BS 812: Part 105.1: 1989, which has been superseded by BS EN 933-3:2012, was followed. The author notes that is not best practice follow an outdated method, but the revised procedure requires the use of bar sieves which were not available in the laboratory at the time of research. The flakiness gauge, detailed in the original test procedure, was available and so was made use of.	BS 812: Part 105.1: 1989 BS EN 933-3:2012

It is important to note that the use of the ACV and AIV test is no longer recognised within the newer European Standards relating to GCs and these tests have been replaced by the LA value (Serridge and Slocombe, 2012).

However, these tests are still adopted by researchers and may still be used to classify materials within industry and so have been included within this research to enable comparisons to be drawn (NHBC Foundation, 2012; Serridge, 2013; Amini, 2015; Mitchell, 2015; Shahverdi, 2020).

3.3 Phase 2: Development of a Bespoke Static and Dynamic Granular Column Testing Methodology

Despite the impact of the Covid-19 pandemic on the research programme, 44 model columns tests were successfully conducted, and these can be split into two categories in terms of the method of loading: constant rate of strain (CRS) (36 tests) and static loading (8 tests). CRS is a common approach used within this field of research (Niroumand, 2011; Lee *et al.*, 2012; Amini, 2015; Ayothiraman and Soumya, 2015; Mekkiyah and Al-Saadi, 2016; Moradi *et al.*, 2018), so was adopted to allow for comparison between the results obtained herein and those published in the literature. However, constant rate of strain does not reflect loading conditions of granular columns *in situ;* in such instances static loading, is recommended (ICE, 1987). However, this method does not appear to have been widely adopted for small scale testing within laboratories, hence it was investigated herein.

To create a bespoke testing arrangement a number of conditions must be identified (such as boundary conditions, scaling issues, type of column: floating or end bearing, diameter to length ratio, diameter of load cap, etc) and these issues are considered in sections 3.4 to 3.6. Section 3.5 considers the creation of the soft host soil and Section 3.7 provides details on the test arrangement for the static and constant load tests.

The methodology adopted for this research is similar to that adopted by Amini (2015). However, Amini (2015) conducted tests on different materials, adopted end-bearing columns that were of different diameters to those adopted within this study, used a large test cell for a number of tests and attempted to measure changes in pore water pressure. Aspects that are considered within this work such as monitoring the tank sides, movement of the surface of clay, moisture content of the aggregates post-test and static load tests were not considered in the work of Amini (2015).

3.4 Phase 2: Test Philosophy

3.4.1 Unit Cell Concept and Sizing of Test Cell

The unit cell concept, previously explained in Chapter 2.0, is an idealisation of a singular granular column within a group. The unit cell identifies an area of influence surrounding a column and this approach was first adopted by Balaam (1978) in research on column groups. It has since been used in studies on both single and groups of columns (Cimentada *et al.*, 2010; Ng and Tan, 2014; Amini, 2015; Mekkiyah and Al-Saadi, 2016). In this research the effect of loading on a single column and the surrounding soil was studied utilizing the unit cell concept.

Hughes and Withers (1974) identified that only the surrounding cylinder of clay up to 2.5 D (where D is the column diameter) is affected when columns are loaded; if columns are spaced at greater than 2.5 D then they are considered to act independently (assuming the footing used to load the column has the same diameter as the column). In addition, Sivakumar et al, (2004) and Balaam and Booker (1975) note that a cell should have an internal equivalent diameter of 5 times the diameter of the column to limit boundary effects. A container with an internal diameter of 349 mm was chosen for this study with a column diameter of 60 mm and a load plate of 100 mm in diameter, to meet the finding of Hughes and Withers (1974). However, the requirement to have a test cell greater than five times the foundation plate diameter was found later, and so additional tests were conducted using a smaller load plate (60 mm in diameter) to meet the requirements stated by Balaam and Booker (1975) and Sivakumar et al. (2004).

Contrary to the recommendation of a cell size to foundation diameter ratio of greater than 5, researchers have also found that ensuring that a distance to the edge of the container that is equal to diameter of the foundation plate (i.e. 100 mm in this research) provided good results which were not negatively impacted by the effects of boundaries (Muir-Wood, Hu and Nash, 2000; Hu, 1995). In addition, many researchers have made use of the triaxial cell for these types of tests and this method clearly has a benefit in that the boundary is flexible and the lateral pressure can be known and also controlled (Black *et al.*, 2007), although due to constraints on equipment in the laboratory this type of testing was not viable for this research.

The test cells adopted throughout this research were 0.52 m in height and 0.349 m in internal diameter. These containers were selected for practicality i.e., they could be filled, and emptied, with compacted clay in a relatively short period of time (it took approximately 3 days per test), and the container would fit on the load frame available. In the planning stages it was envisaged that larger scale tests would be conducted (using a test cell 0.605 m in diameter and 1 m in height) to enable the identification of any boundary effects related to the test cell size. However, due to the Covid-19 pandemic and the requirement for social distancing measures, and covid-safe working practices, within the laboratory, these tests were abandoned (due to the logistical limitations in mixing and compacting the volume of soil required with time constraints imposed on the communal equipment). This is something recommended for further work.

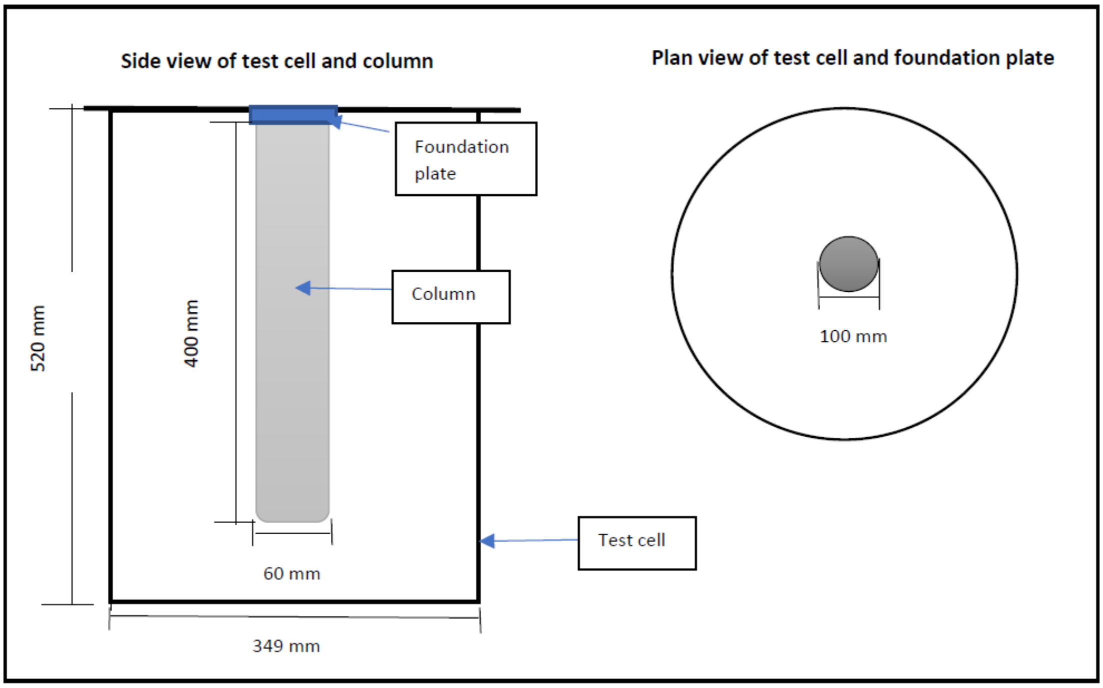


Figure 3-1: Experimental apparatus setup

To further investigate the potential boundary effects an additional set of tests were undertaken with a foundation diameter of 60 mm (for container glass and granite), to complement those using a 100 mm foundation although in both cases the column diameter remained 60 mm. However, the use of a smaller foundation plate alters the area replacement ratio to 100 % and the possible impact of this on column behaviour cannot be ignored.

The author notes that where samples are tested under constrained lateral conditions (i.e., the sides of the test cell are rigid) frictional resistance between the host soil and the side wall of the test cell creates a nonuniform pressure distribution, making analysis difficult (Black, Sivakumar and McKinley, 2007). To mitigate this the sides of the container were greased using DC44 Silicone Grease (manufactured by Dow Corning) as recommended by Togon et al. (1999) and a single sheet of polythene was used to line the container. The accepted best method of reducing friction between the soil and the side of the container is to use two sheets of polythene with grease in between them (Togon et al., 1999). This method was trialled but the inner sheet of polythene became damaged during the process of compaction and so it was deemed that applying a layer of grease to the inside of the container and lining it with a single sheet of polythene was the most practical solution.

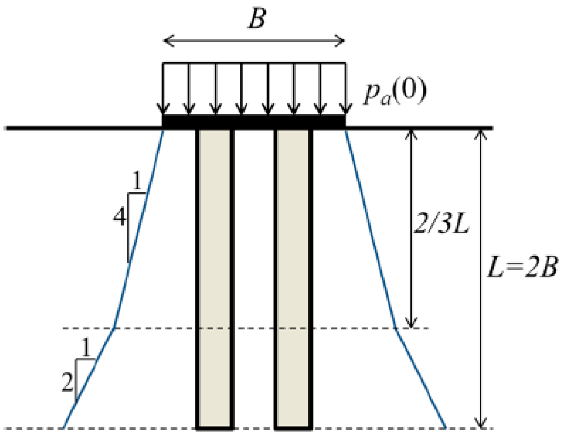


Figure 3-2: Stress distribution beneath floating columns (Castro, 2017)

It was previously stated that the internal diameter of the cell should be at least five times that of the GC diameter to avoid boundary conditions. However, Castro (2017) presented an idealised model for the volume of soil that experiences a change in loading with a floating GC system (Figure 3.2). This model considers the mobilised volume of soil with respect to the diameter of the foundation plate and not the column diameter, and as one of the foundation diameters adopted for this research (100 mm) exceeded the GC diameter, a check was undertaken to ensure that this larger plate would not invalidate the testing. The researchers define the critical length of a floating column as twice the size of foundation width based on the depth of the pressure bulb for settlement reduction and the depth of the failure mechanism for bearing pressure. Given the depth of floating column used in this study (400 mm), the diameter of potentially mobilised soil was 117 mm using Castro's approach, leaving an approximate 57 mm annulus of material between the mobilised soil and boundary (test cell side). This is less than the suggested cell diameter to foundation plate diameter ratio (5), which suggests that the internal diameter of the cell of 349 mm would be acceptable.

3.4.2 Horizontal Deflections of Container Sides

In order to enable any lateral movement of the test container to be detected three LVDTs were placed at equal intervals around the circumference of the tank at a depth of 200 mm (the overall depth of the tank was 520 mm). This location was chosen due to the depth of maximum column bulging being estimated to be 3-4 D (180-240mm) and the centre of the tank (between flanges) being 250mm, this was considered to be the most vulnerable region of the tank to lateral deflection.

Of all the materials the maximum lateral deflection measured was 0.37 mm, overall the deflections were much lower than this (i.e., less than 0.1 mm as shown by figure 3.3). Even considering the worst case scenario the deflection would equate to 0.11 % of the tank diameter; this value of movement is considered insufficient in comparison to the vertical deflections measured during test (Al-Obaidy, Jefferson and Ghataora, 2016).

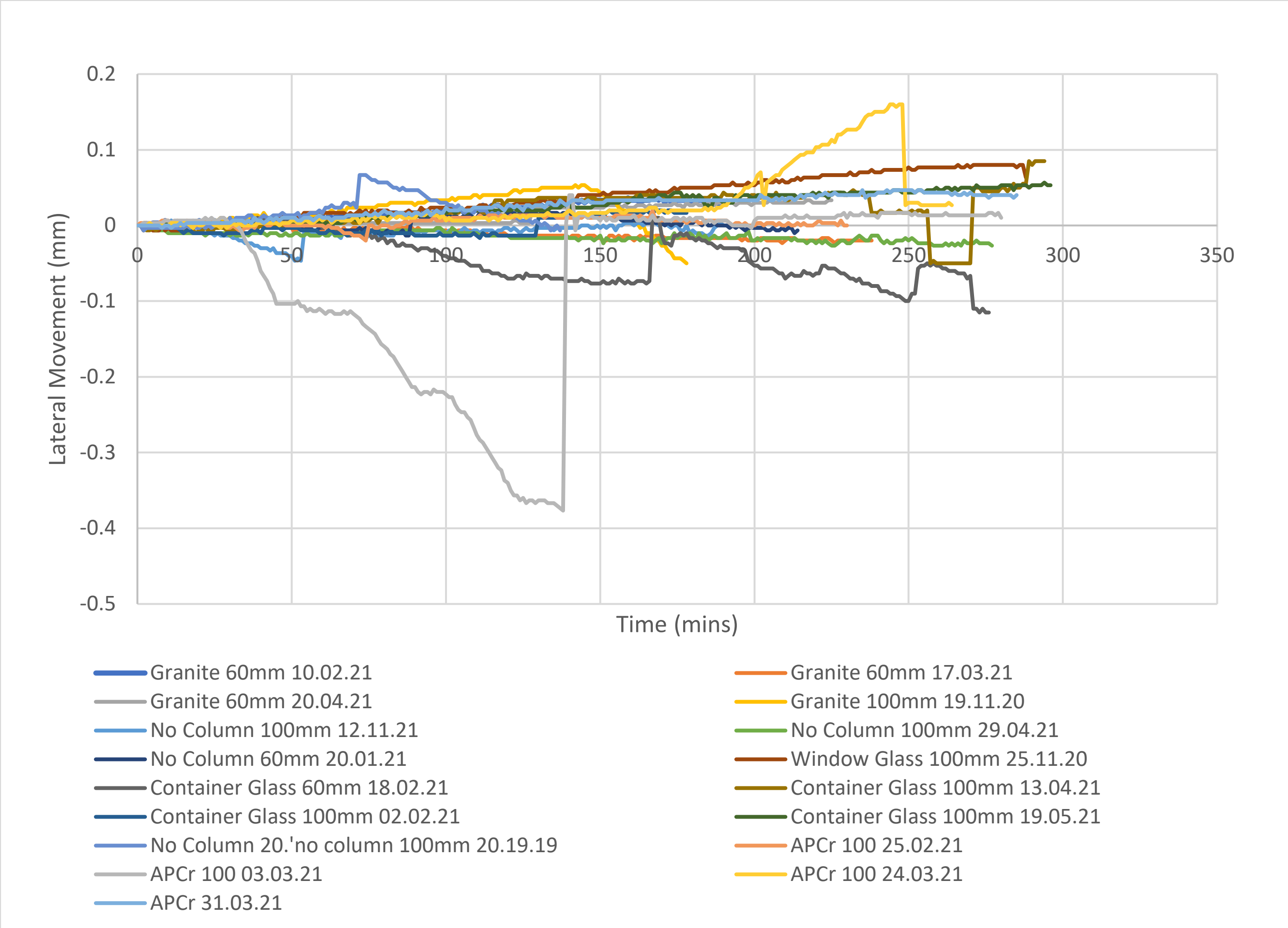


Figure 3-3: Time Vs. Lateral movement of tank sides (where positive lateral movement values indicate outwards movement of the sides of the tank)

The measurements appeared to be both positive and negative, indicating that movements measured could be due to background noise in the laboratory. For the CRS tests the constant upward movement of the test apparatus could have led to disturbances in the LVDTS. In addition, the very small values of lateral movement recorded make it is plausible to attribute this measurement to noise; this was seen in the initial few minutes prior to the start of the test where the LVDT measurements tended to fluctuate by 0.1 mm. The largest reading of - 0.37 mm, indicates an inwards movement of the container; if there were larger outward movements measured this could be attributed the container sides flexing, however given the very small, recorded movements it is more likely due to the movement of the test cell or an accidental touch to the LVDT.

However, for a number of the tests a general trend can be observed that does indicate an increase in lateral deflection with vertical column stress, see Figure 3.3. This would be expected for the 100 mm load plate tests, given the increase in load over time, but not for the 60 mm tests where the ratio of load plate to container size is greater than 5 as recommended to avoid boundary conditions. This pattern of increasing movement over time, with increased load, was observed for both the 100 mm and 60 mm plates, see Table 3.2.

Table 3.2: Average value of lateral deflection (mm) at the end of the test

	Load Plate D (mm)	
Material	**60**	**100**
Granite	0.01	0.05
Container glass	-0.06	0.05
APCr		0.06
Window glass		0.08

The results shown in Table 3.2 suggest that there was no significant difference in the lateral deflection of tank between the 60 mm and 100 mm load plate. This result is unexpected as it is generally understood that a test cell with diameter 5D (where D=foundation diameter) is sufficient to avoid boundary effects. In order to fully determine this, tests would need to be conducted in much larger test cells, exceeding 5D.

However, as stated previously given that overall the horizontal deflections were less than 0.1 mm, which was deemed acceptable based on the findings of Al-Obaidy, Jefferson and Ghataora (2016), these readings are most likely due to noise.

An average of the 3 readings taken from the 3 LVDTs has been presented to enable comparisons to be drawn on the relative tank movement between each of the tests. It would have been difficult to present the 3 separate plots of each of the LVDTs for all of the tests on one graph without it becoming very difficult to interpret. The author appreciates that this is an imperfect method of presenting the data but given that the movements detected were mostly in the order of 0.01 mm (which could well be attributed to noise), and no patterns were detected in the data after analysing each of the tests separately, this aspect was not considered to be critical to the thesis and so was given limited attention.

3.4.3 Surface Movement

In order to determine the effect column loading of the surface profile of the clay bed, two 2 LVDTS were placed at 1.5D and approximately 2.7D (90 and 160 mm) and from the centre of the column (where D is the column diameter). According to the work of Hughes and Withers (1984) the area inside 2.5D is affected by a single column, at distances greater than 2.5D the area is not influenced.

However, as mentioned previously, there is some conjecture over what the boundary conditions should be based on. Many researchers have adopted container sizes of greater than 5D, where D describes the foundation diameter, as recommended by Sivakumar (2004) whilst others have used containers of 3D (Hu, 1995). The placement of an LVDT at 2.7D was to enable any possible boundary condition effects to be detected via heave (as discussed in the work of Hu (1995)). The two anticipated outcomes are illustrated in figures 3.4 and 3.5.

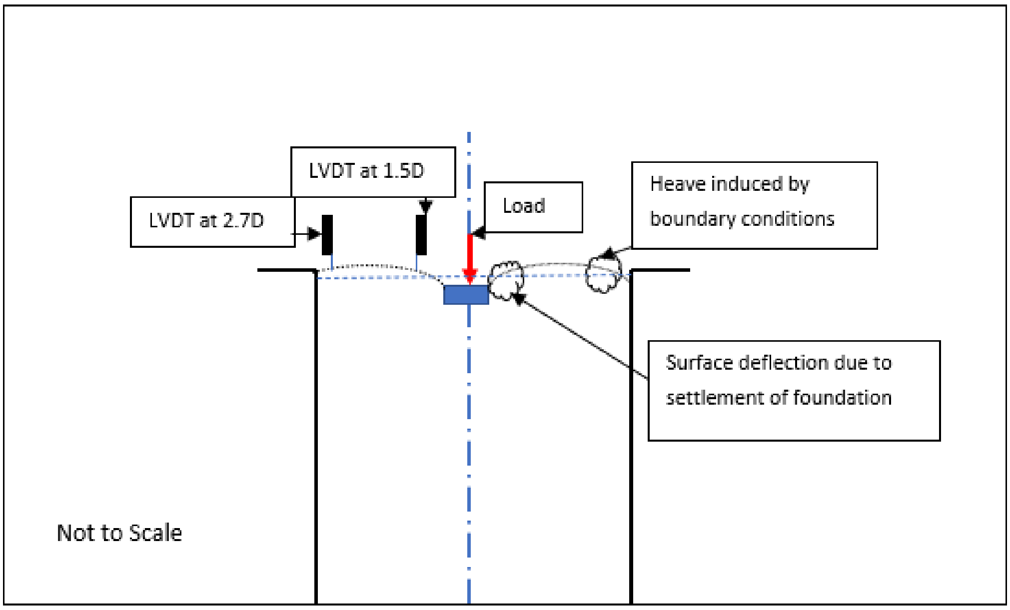


Figure 3-4: Potential shape of clay surface during/after load test if boundary conditons influence behaviour (settlement of foundation refers to the downward movement of the foundation plate into the clay sample under load)

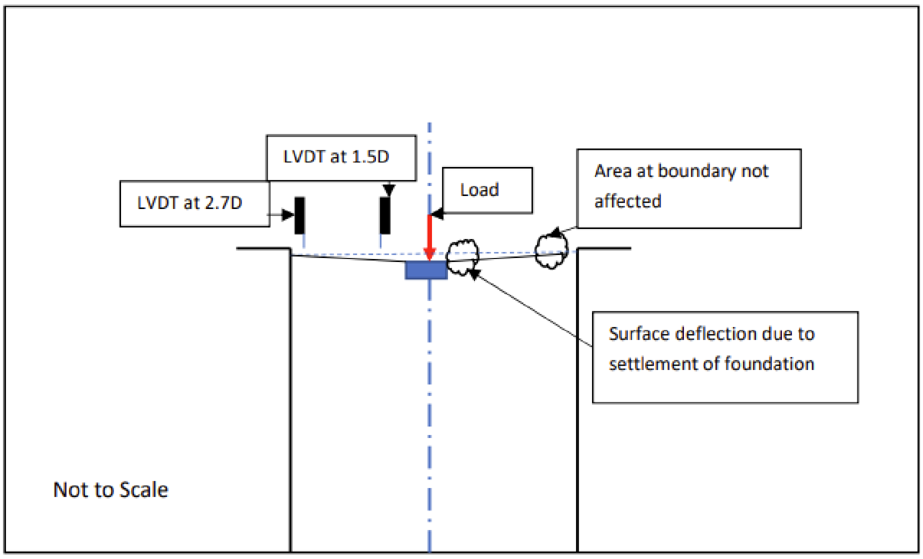


Figure 3-5: Potential shape of clay surface during/after load test if boundary conditons do not influence behaviour (settlement of foundation refers to the downward movement of the foundation plate into the clay sample under load)

The use of LVDTs to obtain the surface movement measurements did present some problems, mainly due to the self-weight of the LVDT leading to the tip being pushed into the soft surface of the clay. To mitigate this, thin strips of metal were placed under the LVDT tips to prevent them sinking into the soft surface of the clay. However, it is recognised that this issue may have affected the results collected.

However, as shown by figure 3.6 and 3.7, generally greater movements were observed at 1.5D than 2.7D, which was anticipated. The comparatively low level of movement detected at 2.7D (from the centre of the column) confirms that at distances greater than 2.5D the host soil is unaffected (Hughes and Withers, 1984).

For each of the tests the time elapsed and the vertical displacement of the foundation plate measured are equivalent. To enable comparisons to be drawn between tests which achieved different load capacities time was adopted for the horizontal axis.

In future work it would be of interest to confirm this finding and to undertake further analysis of surface movement. Perhaps a more appropriate method would time-lapse photography, as adopted in the work of Hu (1995).

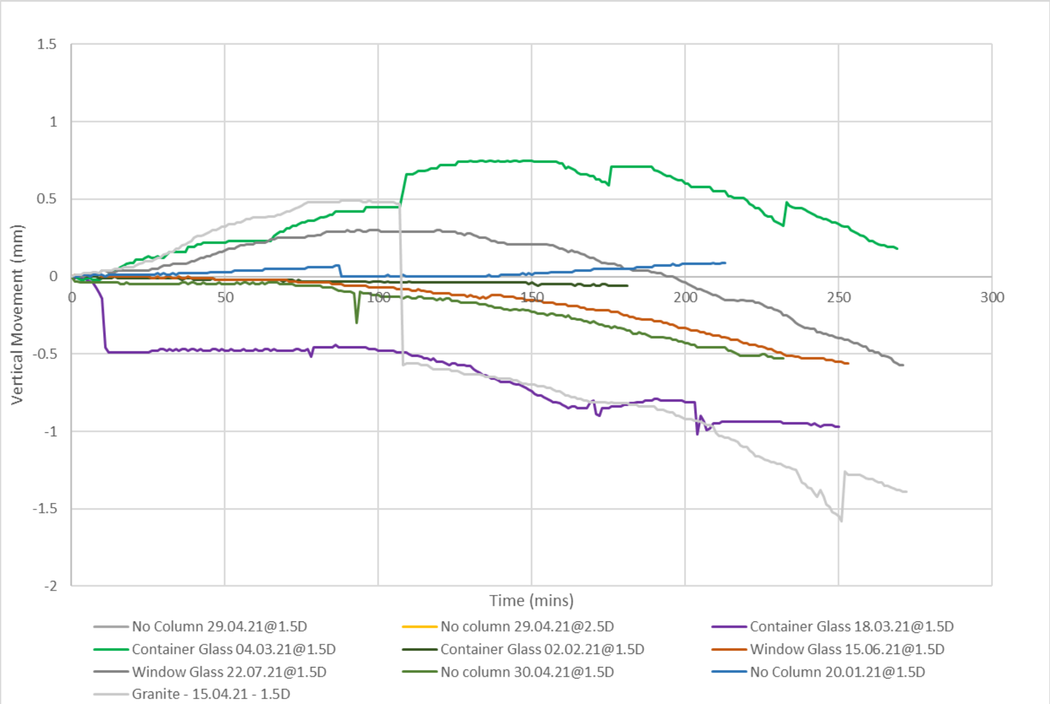


Figure 3-6: Vertical movement of clay surface measured at 1.5D from column centre

(Negative readings reflect downward movement of clay surface and positive readings indicate upwards movement)

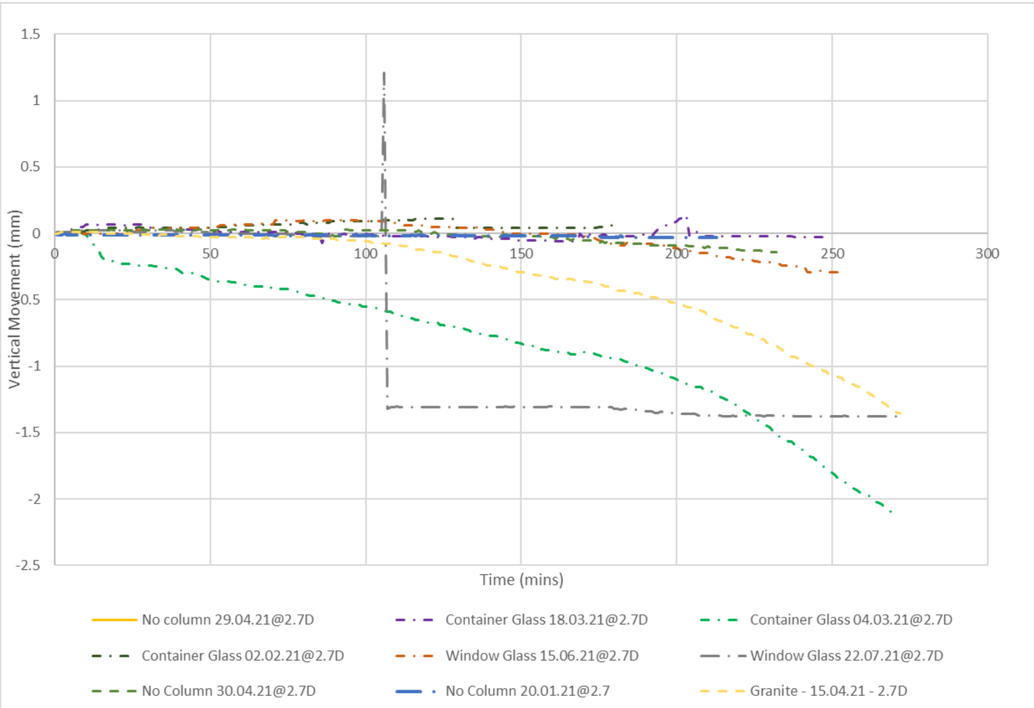


Figure 3-7: Vertical movement of clay surface measured at 2.7D from column centre.

The result for window glass 22.07.21 appears to follow the general trend until just after 100 minutes, it is very likely that this LVDT was knocked during the test. The results for container glass 04.03.21 and granite 15.04.21 appear to show a gradual downwards vertical movement and due to the location of these LVDTs it is more likely that this is due to the gauge sinking into the clay (an issue previously noted) rather that indicating true vertical displacement. (negative readings reflect downward movement of clay surface and postiice readings indicate upwards movement)

3.4.4 Assumptions

3.4.4.1 Single Column

Research has shown that the failure mechanisms and modes of singular columns is different to column groups (Muir-Wood, Hu and Nash, 2000; McCabe, McNeill and Black, 2007). It is recognised that granular columns are most commonly installed in groups in practice, however, due to the focus of this research being the relative performance of RA and SA compared to a PA material, a singular column was modelled throughout. This approach enabled the focus to be on column behaviour under loading without the added complication of the potential influence of group behaviour. It would be of interest in the future to model column groups constructed using the recycled aggregates.

3.4.4.2 Short-Term vs Long-Term Loading/Deformation Behaviour

The focus of this study was to consider the relative behaviour of granular columns containing the PA with those containing AAs. Initially, long-term testing was considered a desirable parameter to consider, along with the short-term response more commonly associated with constraint rate of strain tests. However, as noted above, the experimental programme was disrupted (due to the global pandemic) and choices had to be made regarding tests undertaken. Very little is known regarding the response of the AAs, when compared to that of a PA, in granular columns hence it was decided to focus on short term testing. This would allow for multiple tests to be conducted using the four aggregates with the experimental cells available (for static and dynamic conditions). Load testing typically took around 4-5 hours for the constant rate of strain tests and up to 24 hours in the static loading tests (although this did not include preparation time, hence the actual experimental time was in the order of three days for the CRS tests and four days for the statically loaded tests. It would be of interest to study the columns using the alternative aggregates over a longer period, to assess the extent of material degradation over time (particle breakage, creep, etc.), and this is recommended for future work.

3.4.4.3 Application of Load

The constant rate of strain (CRS) is a common method used in the laboratory setting, although, as noted above the author recognises that this does not represent the loading of granular columns in practice. However, it would be very difficult to accurately replicate the loading scenarios experienced by full scale columns *in situ* within a simple laboratory model and, as the focus of the research was to compare the behaviour of the four different aggregate materials, a repeatable method of loading was deemed to be the most important aspect. However, to address this issue further, and determine to what extent the loading impacts column behaviour, a second method of loading, static loading, was also used for 8 tests. The static load tests comprised adding weights to the column in intervals and measuring surface deformation.

In the case of the constant rate of strain tests, the load applied to the column was measured by a digital load cell as the test cell was raised at a rate of 0.18 mm/min. The apparatus featured a maximum travel distance of 300 mm, but tests were stopped at 50 mm of settlement (in some cases 50 mm was not reached due to time constraints) as this was deemed to be beyond failure, which using the least conservative value can be deemed to be reached when settlement reaches a maximum value of 36 mm (see criteria listed below).

- 10 % of foundation in vertical displacement (Zakariya, 2001)
- 58 % of column diameter in vertical displacement (Hughes and Withers, 1974)
- 60 % of column diameter in vertical displacement (Al-Mosawe et al., 1985)

In addition, a maximum settlement of 25 mm is commonly adopted in industry (Serridge, 2005; Wehr and Sondermann, 2012; Sivakumar *et al.*, 2021).

An example plot of the deformation with time is shown by Figure 3.8.

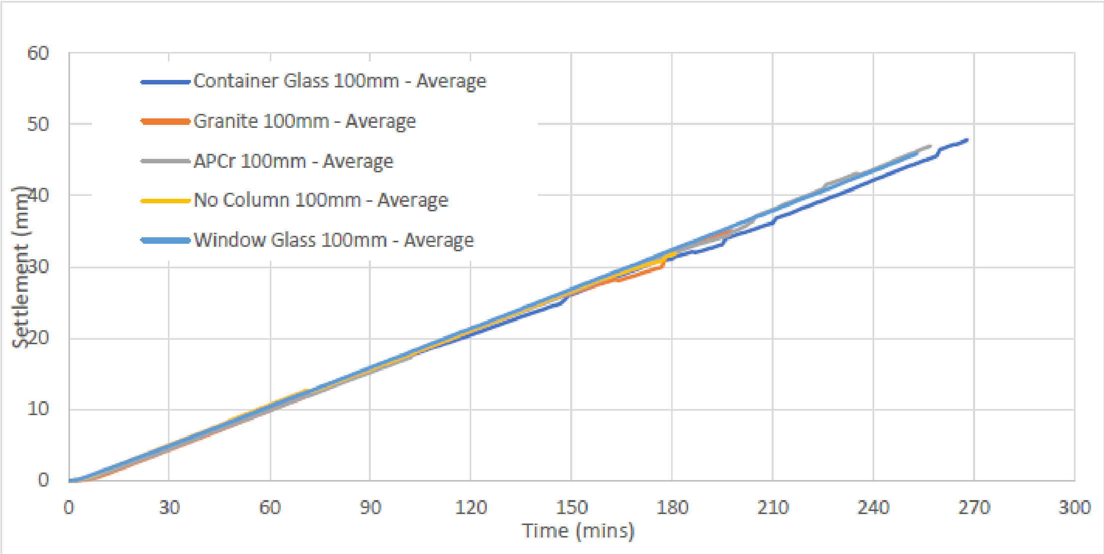


Figure 3-8: Time Vs. settlement for CRS tests

3.4.4.4 Floating vs. End Bearing Columns

In practice granular columns can be installed so that they toe into hard strata, known as end-bearing columns, or they can be 'floating' i.e., the toe of the column lies within weak soil and is not bearing onto a strong foundation soil. There have been numerous studies both of these types of granular column (Paper and Kharagpur, 2010; Amini, 2015; Fattah, Al-Neami and Shamel Al-Suhaily, 2017; Moradi *et al.*, 2018, 2019; Shahverdi, 2020) In this research the columns were floating, i.e., the column did not extend to the base of the container. This approach was chosen as the focus of the research was to identify how the recycled materials transmit the load (e.g., to what extent does bulging occur, does particle breakage occur?) into the surrounding soft host soil without the potentially beneficial effect of the hard surface of the container base.

3.4.4.5 Column Diameter and Foundation Size: Scaling Issues

There are examples of previous research into granular columns, where scaled models have been adopted (e.g., reducing the aggregates within the experiment so that they are one-sixth the size of those use in situ (Black et al., 2007); or limit the particle sizes so that they are a function of column diameter such as in work by Al-Shaikhly (2000) and Fattah et al. (2016) where a maximum particle size of 14 % of column diameter was recommended for optimal column performance). This approach is commonly adopted when the aggregate sizes used on site would be too large for bench scale laboratory tests (and boundary conditions would become an issue). In this study the concept of scaling, in terms of aggregate size or aggregate size to column diameter, cannot be applied as the glass cullet is supplied in the size fractions produced when crushed at the recycling centre, and would be adopted for full scale granular columns (i.e., a stated range between 1 – 9.5 mm; although PSD tests undertaken in the laboratory indicate the maximum particle size was normally around 10 mm with very few instances where this was exceeded (all particles passed a 14 mm sieve, see Figure 3.9), a column diameter could be produced that did not invalidate the boundary conditions (Section 3.4.1).The column diameter was taken as 60 mm (with a maximum particle size to column diameter ratio of 6; as recommended by Black et al. (2007), giving an internal cell diameter to column diameter ratio of 5.8 exceeding the distance of 2.5D suggested by Hughes and Withers (1974) The author is not attempting to state that using an unmodified particle size distribution of the aggregates in a bench scale test would result in accurate predictions of settlements *in situ*, as Hu (1995) and Serridge (2016) note, bench scale experimentation do not accurately replicate *in situ* response. However, this investigation is a comparative study, and if the AA considered herein prove potentially viable alternatives to PA, then larger scale testing would be recommended for further work.

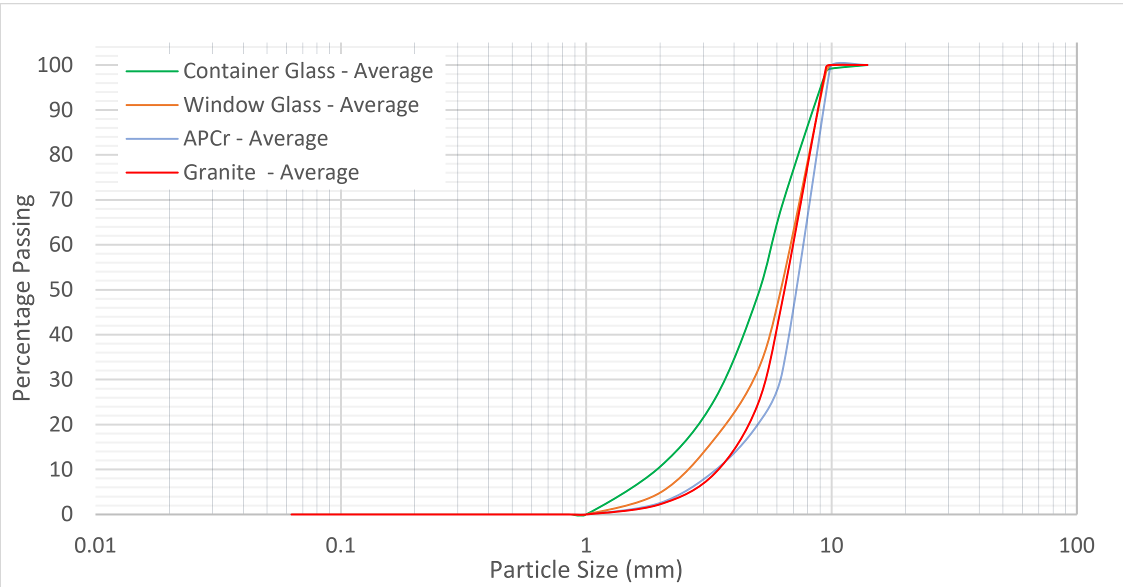


Figure 3-9: PSD of all materials before testing

Muir Wood, Hu and Nash (2000) noted that based on typical full-scale column diameters (D) of 0.6 to 1 m and aggregate particle sizes of 25-50 mm (d) the ratio of D/d is usually in the range of 12 to 40. The maximum particle size of the aggregate used in this study was 9.5 mm, so based on a column diameter of 60 mm, the ratio of D/d is 6, which is outside of the typical range applicable for full scale columns. It is important to note that the researchers state that if the column is sufficiently long that then ratio of particle sizes should not be significant (Hu, 1995; Muir-Wood, Hu and Nash, 2000). This was taken into account when designing the length of the columns (i.e., columns were designed to have an L/D ratio of less than 6 are classified as short, the L/D value in this research is 6.7).

The minimum particle size used within this study was 1 mm. This was due to health and safety (dust is known to be hazardous to health and so minimising exposure to it was of great importance and this is relevant both in the laboratory and in practice) and also the recommendations that fines content (for materials used within GCs) should be less than 5 % (Serridge, 2005).

In practice there is no strict grading criteria appliable to GCs. In the guidance document 'Specifying vibro stone columns (BRE, 2000) it is recommended that aggregates with a 'grading appropriate for compaction to form a dense column fully interlocked with the surrounding ground' are adopted. In addition, uniformly graded aggregates (which can be used in all three installation methods) or aggregates graded between 75 -20 mm (dependant on installation method) are deemed suitable (BRE, 2000; Serridge, 2005)

3.4.4.6 Area Replacement ratio (A_r)

The area replacement ratio (i.e., the ratio of column diameter to foundation plate diameter) is known to be a key parameter that influences granular column performance (Hu, 1995). The optimum area replacement has been found to be between 30-40 % for settlement control (Sivakumar, Bell and Black, 2011). In this research two A_r values were modelled, 100 % and 36 %. In the context of this study tests adopting 100 % Ar are known as 'axially loaded' and those with an A_r of 36 % are categorised as foundation style loading.

3.4.4.7 Column Length to Soil Height Ratio

The ratio of column length to soil height (H_c/H_s) in this research was 0.8 (based on a column length of 400 mm and a soil height of 500 mm). This is based on work by McKelvey (2004) which found that the optimum ratio of H_c/H_s is >6 but <10 and this is confirmed by work conducted by Sivakumar (2004) where it was found that there was no benefit, in terms of performance, in H_c/H_s being greater than 0.8.

3.5 Host Ground (Soft English China Clay)

3.5.1 Material

The host soil used to fill test cell was Kaolin (also known as English China Clay). English China clay was chosen as it is an ideal material to work with in the laboratory due to its consistency and ability to produce repeatable samples and could be purchased in bulk (McKelvey, Sivakumar, Bell and Graham, 2004; Black, Sivakumar and McKinley, 2007; Andreou *et al.*, 2008; Ayothiraman and Soumya, 2015).The most important aspect of the host soil for the purpose of this research was consistency to enable comparison between tests whilst limiting variation in the host soil (as this could unduly influence the results). The strength of the clay needed to be within the range of 15-25 kPa as in soils with strengths greater than 25 kPa the extent ground improvement achieved is reduced and in soils with strengths lower than 15kPa the lateral pressure required to support columns as they bulge is insufficient (Wehr, 2013).In order to determine the required moisture content of the soil, trial compactions were carried out and the strength of the material was then tested using the hand shear vane apparatus (with a small number of cores taken from the soil sample and tested in the triaxial under unconsolidated undrained conditions).

3.5.2 Water Content of the Host Soil

The proctor compaction test was used to determine the required water content to achieve a clay strength within the desired range (15-25 kPa).

An Initial target water content range of 38.5 to 41 % was selected as hand shear vane tests conducted during the proctor compaction tests indicated that the strength achieved was within the desired range (see Figure 3.10, below).

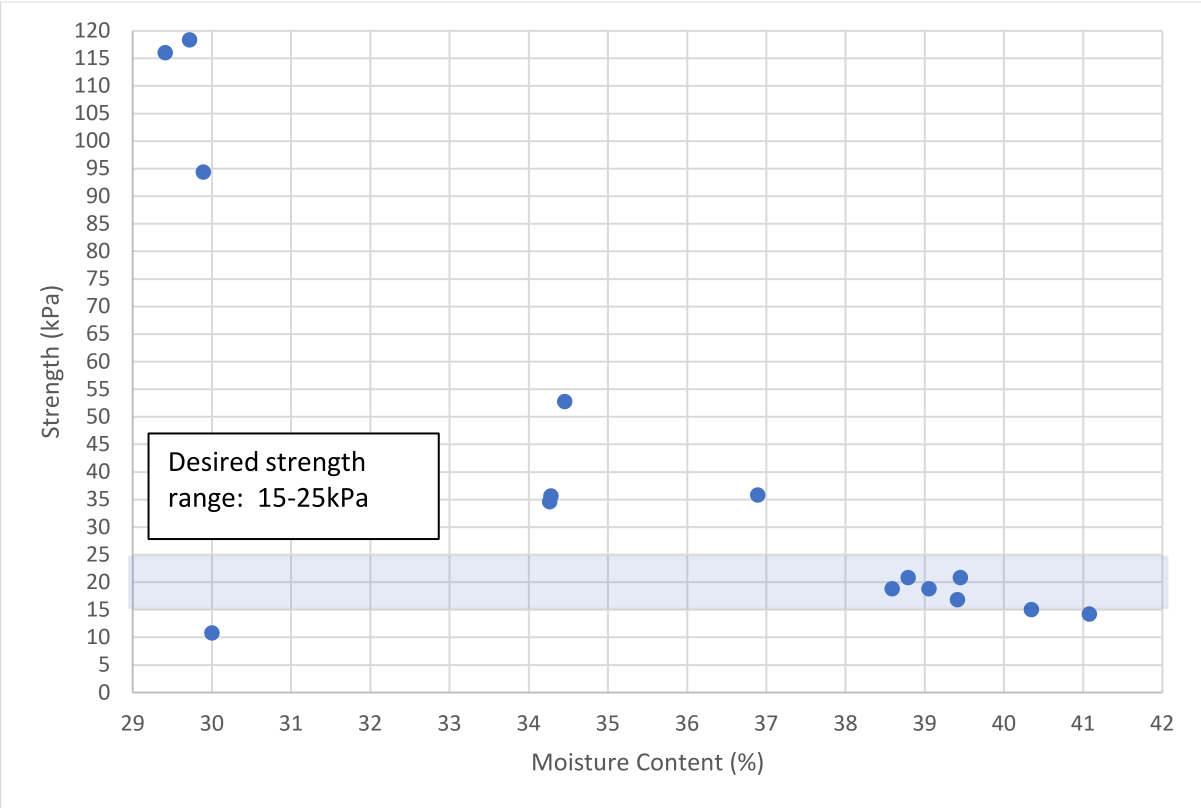


Figure 3-10: Standard Proctor Compaction Test - Kaolinite water content (%) Vs. strength (kPa)

This moisture content was trialled within the test cell, and it was found that the strength remained within the desired range. This was increased to 41 % (the upper value of the original target water content range), after the trial model column tests, when a new batch of kaolin arrived in the lab and the plasticity index differed and so the target moisture content was adjusted.

Table 3.3: Properties of Kaolin

Date of Test	Plastic Limit	Liquid Limit	Plasticity Index (%)
Nov-20	37.9	59.5	22
Mar-21	35.0	65.0	30

3.5.3 Preparation of the Host Soil

GC are used in soft soils to improve strength and consolidation times; hence it would have been ideal to consolidate the host soil from a slurry directly in the test cell. This practice has been used in previous research (McKelvey, Sivakumar, Bell and Graham, 2004; Black *et al.*, 2006; Black, Sivakumar and McKinley, 2007; Shahu and Reddy, 2011). However, due to time, and equipment, constraints the compaction method was adopted for this research. It is recognised that this method of preparing the soil bed does not replicate the natural process under which clay consolidates. Whilst this is less than ideal, and will likely impact upon the results, the study aimed to compare responses between AA with PA to investigate the potential suitability of these AA for use in GC. If it were found that they were suitable then more sophisticated testing could be undertaken (as further work).

To prepare the soil it was mixed from powered with water (to achieve a water content of 38.5 – 41 %) using an industrial sized mixer for a period of 10 minutes. This duration was selected through visual inspection, that material appeared to be well mixed after this mixing time was also adopted by Amini (2015). The mixer was capable of mixing more than one bag at a time, but it was found that the material was not well mixed when more bags were added. The mix time of 10 minutes was kept constant throughout the entire research project as other researchers anecdotally noted that variations in mix duration led to changes in the consistency of the material.

The clay was compacted in 10 layers (each 50 mm thick and weighing 8 kg) using a circular plate, with diameter the same size as the test container to spread the compaction energy more evenly across the surface of the clay, and then left overnight to homogenize (Head, 2006). The placement and removal of the 80 kg of clay required to fill each test cell was physically very demanding and was a factor when deciding on the size of test cell to be adopted.

Falling weight hammers were initially trailed but abandoned in favour of a vibrating compactor (using a Kango hammer) when densifying the clay. These are described in the following subsections.

3.5.3.1 Proctor Hammer

Initially 3600 g of Kaolin was placed at the base of the container. The circular plate placed on top and 45 blows, using the 2.5 kg hammer, were executed in a circular manner, originating at the centre. This level of compactive effort, which was based on a scaling up of the proctor compaction test, resulted in a layer thickness of 35 mm. When removing the plate some of the clay, which had adhered to the plate, was also removed, voiding the compaction process.

A second trial was undertaken, which utilised the same process as above, also resulted in a 35 mm layer although it was noted that it appeared 'patchy' in some areas, indicating poor compaction. Prior to placing the plate onto the soil, a layer of cling film was placed on top of the sample, this successfully prevented the clay from adhering to plate and resulted in a good surface finish.

Reducing the number of blows per layer (90 blows) resulted in a layer thickness was 70 mm uniformly across the container, hence it did not achieve the desired 50 mm thickness.   Once compacted the shear strength was estimated at various positions within the layer (Figure 3.11 shows a plan view with approximate locations).

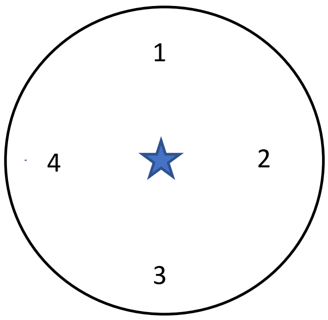


Figure 3-11: Plan view of test container and approximate locations of hand shear vane test locations

Readings were taken at the centre of each layer of clay using the hand shear vane and the average value was taken.

The results of Hand shear vane testing for the layer are shown in Table 3.4:

Table 3.4: Measured undrained shear strength of trial 3

Position	kPa
Centre	15
1	20
2	12
3	18
4	14

The results shown in Table 3.4 reflects the variation in compaction, with two of the locations not achieving the minimum strength of 15 kPa.

A sample was taken by pushing the 60 mm tube into the clay and pulling it out, the soil plug was then carefully extruded from the tube. Whilst this is a disturbed sample and is likely not a complete representation of the *in-situ* soil it was apparent that there were defined layers within the clay (see Figure 3.12). It is possible that the extrusion process led to the formation of the smaller, surface cracks however, on closer inspection the larger crack (highlighted in figure 3.12) extended across the sample and a define layer was apparent, suggesting another cause, such as the installation process. The presence of defined layers was not desirable due to the increased flow paths for water within the material. Also, the presence of layers could potentially affect the lateral resistance offered to the column and so affect column behaviour.

Figure 3-12: Visible layers within soil structure

To create a more uniformly compacted sample, i.e., without defined layers a different method of compaction, vibration, was trialled using the vibrating hammer.

3.5.3.2 Vibratory Compaction (Kango Hammer)

Vibrating Hammer Specifications:

$240v$
$3.9A$
$900w$
$25 - 60Hz$

Figure 3-13: 900 w Vibro-compaction hammer

To assess the feasibility of using vibration to compact the clay and to determine a suitable length of compaction of each layer a number of trials were conducted. Prior to starting the trials, the safe working time for using the equipment (to avoid the risk of hand-arm vibration syndrome, HAVS) was calculated as 20 minutes (daily limit). To further mitigate the risk of HAVS anti vibration gloves were worn when using the equipment. In addition, hearing and eye protection were worn along with an appropriate face mask.

The clay mixed for the proctor compaction trial was broken up and placed at the bottom of the container, see Figure 3.14. The circular plate was placed on top of the clay and four sets of approximately five seconds of vibration from the Kango hammer were applied to the surface of the clay. The square plate attachment for the vibrating hammer was used and the vibration was applied with light pressure to achieve compaction of the clay.

Figure 3-14: Clay prior to compaction using the vibrating hammer

Figure 3-15: Sample after approximately 20 secs of compaction

The results after 20 seconds were comparable to those achieved after 90 blows of the proctor hammer.

Table 3.5: Shear strength after 20 seconds of vibration

Position	kPa
Centre	16
1	15
2	19
3	16
4	14

This quick trial proved that using the vibrating hammer to compact the clay was a viable method although the degree of vibration needed to be increased in order to create a more uniform sample.

3.5.3.3 Trial 2

6000 g of Kaolin, mixed to 39 % water content, was placed in the container. This created a layer that was approximately 100 mm thick prior to compaction (this is a very rough measurement). Polythene sheeting was placed on top of the sample, to prevent the plate from sticking to the clay, and then the circular plate was placed on top. Four sets of 15 seconds of vibration were applied at four different points around the circular disc at the centre of each quadrant, totalling in one minute of vibratory compaction. Vibration was applied to each of the four quadrants, rather than just the centre, to more evenly spread the compactive energy and to avoid a denser central core being created within the sample. This method resulted in what appeared to be a well compacted sample with a layer thickness of about 50 mm. The surface was smoother than previous attempts and was level, see Figure 3.17.

Figure 3-16: Plate with handle attached

Figure 3-17: Sample after 1 minute of vibratory compaction

The edge of the sample appears to be disturbed as despite the layer of polythene there was some difficulty in removing the plate post compaction. To prevent this occurring again a handle was attached, see Figure 3.15. The handle was designed to be removable to allow the freedom to apply compactive energy across the whole plate.

Table 3.6: Shear strength of clay after 1 minute of vibration

Position	kPa
Centre	18
1	18
2	17
3	20
4	22

The shear strengths achieved were within the desired range.
As the tests continued and the method of compaction was repeated it became apparent that the best method of applying uniform compaction was to constantly move the vibrating hammer in circular motions around the plate. This method spread the compactive energy more evenly and when the clay was removed from the cell no visible layers were present.

Whilst every effort was made to maintain a consistent host soil for each test (in terms of water content and density) there were differences in soil strength between the tests. Whilst most of fell into the target range (set to 15-25 kPa due to the difficulties in creating a soil of the same strength each time), this is quite a large range and could potentially have affected column performance. In addition, during the research clay from two different batches was used and differences between the characteristics of the two (plastic limit and plasticity index) were noted, leading to the target water content being altered to ensure the compacted soil strength was within the target range.

3.5.4 Hand Shear Vane

It is acknowledged that, although used since the 1950s to estimate shear strength, the hand shear vane is not a perfect tool and the results can be affected by factors such as the speed of rotation (Perez-Foguet et al., 1999). Some research suggests that the use of the hand shear vane apparatus can lead to the over estimation of undrained shear strength when compared to other test methods such as the triaxial (Perez-

Foguet et al., 1999). However, for the purposes of this research it was utilised as simple method of measuring the approximate undrained shear strength of the host soil. It was not feasible to measure this in any other way due to the difficultly in measuring the *in-situ*, undisturbed strength. In order to vindicate the (approximate) measurement of the shear strength of the host soil using the hand shear vane, a number of samples were taken from a fully compacted test cell, and these were measured using the triaxial apparatus, in unconsolidated undrained conditions. Due to the soil being very soft it was quite difficult to obtain and set up the test without disturbing the sample. However, care was taken to disturb the soil as little as possible, by using a coring tube with sharpened cutting ring that was lubricated internally and the sample gently extruded using a hand tool and the results obtained indicated that the hand shear vane was providing measurements that were close to those obtained from the triaxial.

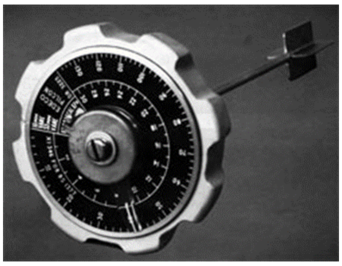


Figure 3-18: Hand shear vane apparatus (Impact Test Equipment, 2022)

Table 3.7: Triaxial test results

	Confining pressure (kPa)					
Depth (mm)	**10**	**20**	**50**	**100**	**150**	**200**
25-100			12	13	18	
25-100			11			
30-100	8	10	12			
225-300			10	13	15	17
225-300					15	
325-400			12	13		16
325-400						17
Avg (kPa)	***8***	***10***	***11***	***13***	***16***	***17***

Table 3.8: Results of hand vane shear test

Depth (mm)	Strength (kPa)	Strength (kPa) with correction* according to Bjerrum (1972)	Strength (kPa) with correction** according to Larssen et al. 1984 (BS ENISO 22476-9)
25-100	18	17	15
30-100	17	17	15
225-300	18	18	16
325-400	20	20	17

The results shown in tables 3.7 and 3.8 indicate that the hand shear vane apparatus gives a satisfactory approximation of the shear strength of the clay.

The column was placed into the host material dry (Section 3.7), yet post-test investigation indicated that there was water within the columns and the water content varied with depth (no drainage of water from the test cell was permitted). This suggested that the soft clay host soil was consolidating (partially at least) in response to the constant rate of strain and static loading environments. Whilst the hand shear vane is not a precise tool, it was used before and after testing to give an indication into the change in strength of the host soil (both radially within the compacted layers and with depth). These changes are reported in both the Results and Discussion Chapters.

3.6 Factors studied

Andreou et al (2008) suggested a number of influential parameters that impact/control granular columns behaviour, these are as follows:

- Drainage conditions
- Particle size of column aggregate
- Confining pressure of the soil (could be either due to imposed boundary conditions or soil strength)
- Rate of deformation

In this research some of these factors are considered, others are not (either being a function of the test or difficult to vary independently of other parameters. For example, the rate of deformation in the constant rate of strain tests is considered to be a constant (see Figure 3.7, previous section 3.4.4.3). In the static loading it is understood that the rate of deformation will be non-linear and is not a parameter that is controlled. When undertaking this test, settlement of the load plate is monitored and when the settlement becomes approximately constant (not allowing for longer term movements from secondary consolidation) then the load was changed. This was akin, in principle, to the methodology used in oedometer testing. See Figure 3.19 below which illustrates settlement with time.

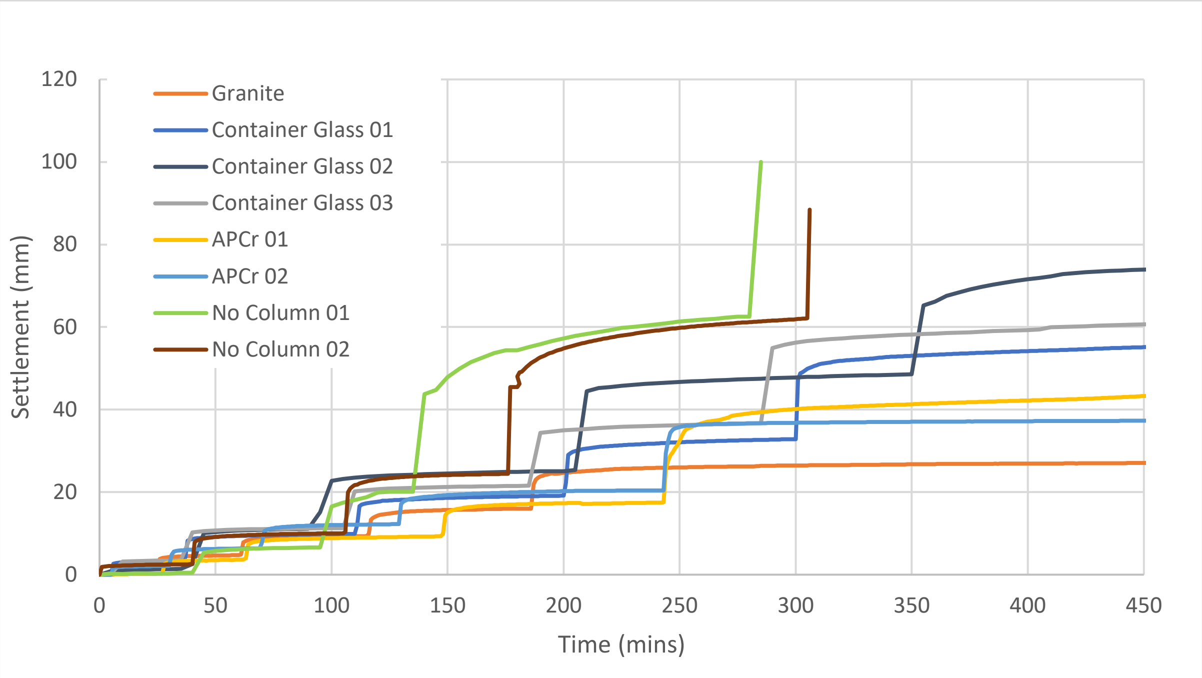


Figure 3-19: Static Loading – Settlement Vs. Time

The drainage conditions will vary based on factors such as PSD and column density, host soil water content and density, as well as load applied to the host soil via the deformation of the GC. This will be considered to a degree (it was noted previously that water contents development was observed in the GC and the host soil appears to increase in strength on occasion) as separating out the influence of these individual parameters will likely prove problematic due to inherent heterogeneity within the soil and column).

The presence of a rigid cell wall could influence the confining pressure in the host soil (the host soil is not acting as an infinite medium in this case); steps were taken to minimise this by choosing an internal diameter sufficient to limit boundary conditions along with the lubrication of the cell wall-host soil boundary (Hughes and withers, 1974, Togon, 1999). In addition, whilst every effort was made to ensure consistency in the host soil, it is acknowledged that there may well be variation in the properties of this soil which will impact upon the confining pressure. Furthermore, the confining pressure applied is also a function of how the GC deforms into the host soil and the shear strength response it mobilises within it (whilst considered initially undrained it is also likely unsaturated, due to compaction, hence it might exhibit undrained angle of internal shearing resistance (ϕ_u>0) and the soil at the host-soil-GC interface could start to drain). In essence, it is not possible to control the confining pressure in this experimental arrangement and it is also difficult to characterise it with any degree of certainty (a limitation of the test, although as noted previously the aim of this investigation was to study the relative change in response when changing the aggregate type within the GC).

The particle size is understood to vary due to the variable nature of the AA and this is a variable within the investigation that can be considered.

The factors identified above do not take into account any possible effects of the column installation process which is clearly and important stage and merits exploration (in subsequent subsections).

3.6.1 Column Installation

Various methods have been adopted to create model granular columns in a host soil. These include:

- Pre-bore holes into the host soil and install frozen columns (Black, Sivakumar and McKinley, 2007; Cimentada *et al.*, 2010)
- Thin open-ended tube pushed into soil, soil from inside tube removed and aggregate placed in lifts whilst the tube is withdrawn in increments (Ayothiraman and Soumya, 2015; Shahverdi, 2020)
- Pour granular material freely into pre-augured hole (Hu, 1995)

Whilst these methods have enabled the construction of a model column these approaches do not closely simulate the stress situation that occurs during the installation process of full scale granular columns (Weber, 2004). The author recognises that it is very challenging to model the installation process on a small scale, however, attempts have been made to simulate this as closely as possible in this research.

Figure 3-20: From left to right: (a) Centralising frame and installation tube, (b) imaging showing how centralising frame is fixed to container, (c) Empty bore

Initially a centralising frame, which could be attached firmly to the test container, was placed to ensure that the column would be inserted vertically (figure 3.19a and 3.19b). Next, a tube with external diameter of 60 mm (internal diameter 54 mm) was pushed into the clay (Figure 3.20) and the material inside the tube was removed using an auger (Figure 3.21). It is acknowledged that a guide tube would not be required to install full scale columns due the scale of the plant (i.e., vibroflot) utilised, however, for these model columns the use of a guide tube is necessary to produce the bore.

Figure 3-21: Chamfered edge of installation tube

The tube was not greased, as adopted by other researchers (Ayothiraman and Soumya, 2015; Hu, 1995), to avoid any influence on the clay surrounding the column. It is noted that the process of installing the tube is likely to cause disturbance to the soil, in an attempt to minimise this, without the use of grease, the bottom edge of the tube was chamfered, as shown in Figure 3.21. The tube was then removed fully before installing the GC aggregates, which differs from the installation process others have followed where the tube was removed in stages (Ayothiraman and Soumya, 2015). This approach was taken as when the tube was removed the bore remained stable (this could be examined with the use of a torch).

Figure 3-22: Auger used to remove clay from inside tube

The aggregate was installed in lifts of 50 mm and vibrated for a period of 20 s using a 25 mm vibrating concrete poker/agitator (Figure 3.23). It is recognised that the vibrating frequency of the small concrete poker, with an operating vibration frequency of 50 Hz, differs to those used for full scale columns, where the operating frequencies are likely to range from 20-30Hz, as this is thought to be the range of resonance of soils (Bell and Kirch, 2012), and this will have an impact on the extent of densification and potential particle breakage during installation. In addition, the ratio of the poker size to the bore in this research may be different to the equipment used in full scale construction of granular columns. However, the processes and equipment adopted in this study were consistently used for all of the GC tests.

Figure 3-23: Details of Concrete vibrator used during column installation

Table 3.9: Details of concrete vibrator used for column installation

Parameter	
Hz	50
W	500

The duration of vibration of each lift was chosen as during trials it was found that at 20 seconds approximately 10 mm for settlement was observed for each of the materials, additional vibration did not appear to cause any further movement of particles.

Initial trials indicated that the vibrating poker could force the GC aggregates into the host soil during compaction. This was undesirable, so the method adopted avoided this (it is accepted that might well occur in actual installations but it was difficult to control and its effects were problematic to quantify on the observed behaviour). The modified method involved holding the poker lightly on top of the layer of aggregate and vibrated as a whole (instead of driving the poker into the aggregates and then vibrating the mass during withdrawal). The author notes that this differs from the process on site, however repeatability was prioritised due to the main purpose of the test being comparison between each of the model tests rather than trying to exactly model full scale granular columns, something which the author feels is near impossible to do at small scale.

Once the column had been compacted into place the upper surface was levelled prior to placement of the load plate. A granular blanket was not included in this research, as per the recommendation of Bell and Kirch (2012).

A perceived disadvantage of AA is the (perceived) poor performance when compared PA with respect to indicative aggregate performance tests. This includes breakage of particles during installation and handling. Therefore, trial installations, using each material, were conducted and the materials were sieved before and after installation (sieving also took place after loading tests). Variation between the pairs of particle size distributions would allow an assessment of the extent of particle breakage during column installation (and so enable particle breakage due to loading to be estimated, if only approximately). The results of these tests are discussed in Chapter 6.0.

3.7 Column Load Tests

3.7.1 Constant Rate of Strain (CRS)

The constant rate of strain load tests were conducted using a proprietary Clockhouse load frame mounted with a digital load cell which was connected to a datalogger, enabling the applied load to be measured at 60 s intervals. The platform on which the test cell was positioned moved upwards at a rate of 0.18 mm/min and this displacement was recorded by an LVDT.

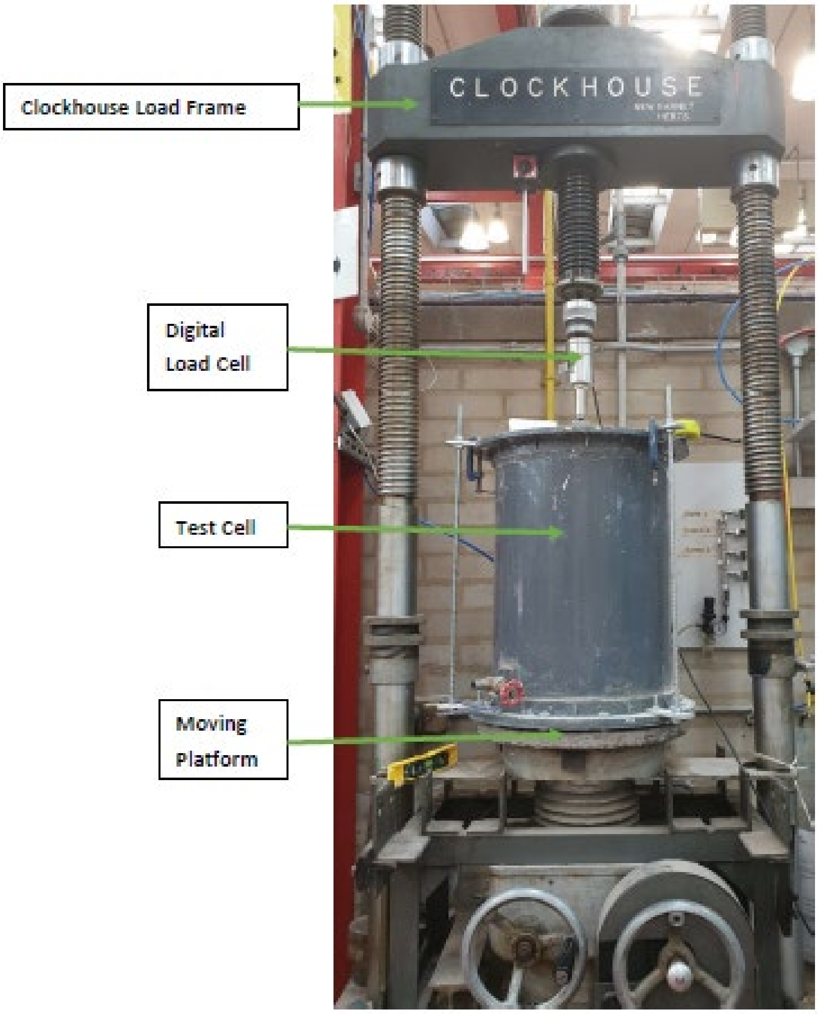


Figure 3-24: Constant Rate of Strain test set-up

During the planning of this work it was envisaged that only three repeats of each test would be required. However, due to variation in load/settlement behaviour observed during the work additional tests were documented, as detailed in Table 3.10 below.

Table 3.10: Details of CRS tests conducted

Test date	**Material**	**Load Plate size (mm)**
25/11/2020	Window Glass	100
15/06/2021	Window Glass	100
22/07/2021	Window Glass	100
24/01/2021	APCr	100
25/01/2021	APCr	100
03/03/2021	APCr	100
24/03/2021	APCr	100
08/04/2021	APCr	100
31/03/2021	APCr	100
27/05/2021	APCr *(crushed)*	100
21/01/2021	Container glass *(particles above 6.3mm removed)*	100
19/10/2020	Container Glass	100
27/01/2021	Container Glass *(with fines)*	100
03/02/2021	Container Glass	100
02/02/2021	Container Glass	100
16/02/2021	Container Glass	60
18/03/2021	Container Glass	60
04/03/2021	Container Glass	100
13/04/2021	Container Glass	100
18/02/2021	Container Glass *(prepped night before)*	60

21/04/2021	Container Glass	60
19/05/2021	Container Glass	100
19/11/2020	Granite	100
09/12/2020	Granite	100
15/04/2021	Granite	100
10/02/2021	Granite	60
13/01/2021	Granite	60
17/03/2021	Granite	60
20/04/2021	Granite	60
21/10/2020	no column	100
14/10/2020	no column	100
12/01/2021	no column	100
29/04/2021	no column	100
30/04/2021	no column	60
08/12/2021	no column	60
20/01/2021	no column	60

3.7.1.1 Instrumentation

The deflection of the foundation plate was measured using 2 LVDTS. It is recognised that ideally this would be measured in 3 locations (to enable the measurement of settlement in 3 directions), however, due to two limiting factors – the plate being too small (and being obstructed by the load cell) to accommodate an additional LVDT and only 5 LVDTS being allocated to the research. Whilst the researcher acknowledges that there are limitations to this approach to instrumentation it is believed that the best use of the allocated resources was made.

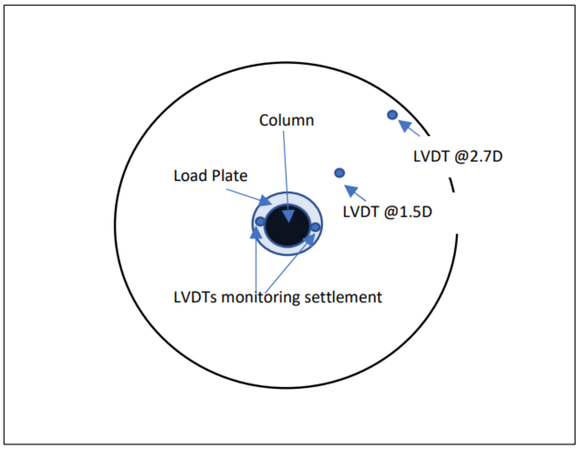


Figure 3-25: Plan view of test setup showing locations of LVDTs

In order to gain a better understanding of the influence of the column and any boundary conditions the sides of the test cell were monitored using 3 LVDTS fixed horizontally. The LVDTS were positioned at a depth of 200mm from the surface of the test cell as this was deemed to be the area of the cell most vulnerable to flexing due to the bulging of the columns (the point of maximum bulge was estimated to be at approximate 120mm below the surface (due to bulging known to typically extend up to 4D along the length of the column), however, the LVDTs were placed lower to avoid the influence of the stiffening ring at the top of the container.

LVDTs were used to monitor the surface of the clay at horizontal distances 1.5D and 2.7D from the column centre in an attempt to detect any heave induced by the column and going further to determine the extent of influence of the column (see section 3.4.3).

Due to limitations in the number of available LVDTS these two types of measurement (i.e., sides of the test cell and surface of the clay) were conducted on alternate tests. Whilst the author recognises that it would have been optimal to monitor both areas on all tests it is believed that the best compromise was made in terms of collecting useful information with the equipment available.

3.7.2 Static Load Tests

In total 8 static load tests were conducted, as detailed in Table 3.11 below. The aim of these tests was to compare the column response under CRS and static loading.

Table 3.11: Summary of static load tests

Test date	Material	Load Plate size (mm)
16/06/2021	No Column 01	100
21/07/2021	No Column 02	100
24/06/2021	Container Glass 01	100
09/07/2021	Container Glass 02	100
15/07/2021	Container Glass 03	100
06/07/2021	APCr 01	100
06/07/2021	APCr 02	100
22/06/2021	Granite	100

To ensure that the static load tests could be conducted safely a different set up was required. The load was applied to column in fixed increments rather than continuously as was the case for the CRS tests. This loading was achieved by applying physical weights to the column. The main consideration was to enable the load to be applied safely, without the risk of toppling to occur as not only could this potentially cause an accident the results of the test would not be of great use. The following set up, as detailed in figures 3.25 and 3.26, was created.

The loading sequence was based on the zone tests that are carried out for full scale columns. It is acknowledged that generally these zone tests are applied to a group of columns over a longer period of time, but the approach was adapted to suit the limitations of the model test and time available. The ICE Ground Improvement Specification (1987) states that, in the case of zone tests, load should be applied in increments not exceeding 25 % of working load in 3 stages. For the purpose of this research the load was applied in 5 stages for two main reasons:

- Practicality: adding weights to the sample takes time and so the fewer weights that needed to be added at each increment the shorter the disruption to the measurement of settlement
- the inclusion of two additional load steps facilitated closer investigation of the column at various stages of loading for comparison with the constant rate of strain tests. It is difficult to compare the two directly as one has a constant load (per load step) and non-linear settlement whereas the other has constant displacement rate and varying load. However, the settlements observed in the static load tests would approach an equilibrium (ignoring long term secondary consolidation settlements), akin to the Taylor root-time plots in one dimensional oedometer compression tests. Taking the load and deformation as the trend approaches equilibrium allows for an approximate comparison with the CRS tests at the same strain (show a figure to highlight this). To identify this 'equilibrium' the recommendation in the ICE specification (ICE, 1987), the subsequent loading increment was not applied until the settlement rate under the preceding load was less than 0.5mm/hr (or less than 0.04 mm every 5 minutes which is the recommended interval at which to take readings of settlement).

The GC model tests did not have a defined working load (as required in the ICE specification), therefore the load at which 30 mm of settlement was reached (i.e., 50 % of column diameter deemed to be failure in this research) during the CRS granite column tests was selected. This value was selected to enable comparisons to be drawn between the static and constant rate of strain tests in terms of settlement with load. Another consideration was the reasonably practicable maximum load that could be applied to the column. This selected working load of 187 kPa could safely be applied through the stacking of weights. The loading stages were split into 5 roughly equal stages of 6, 37, 75, 112, 149 and 187 kPa.

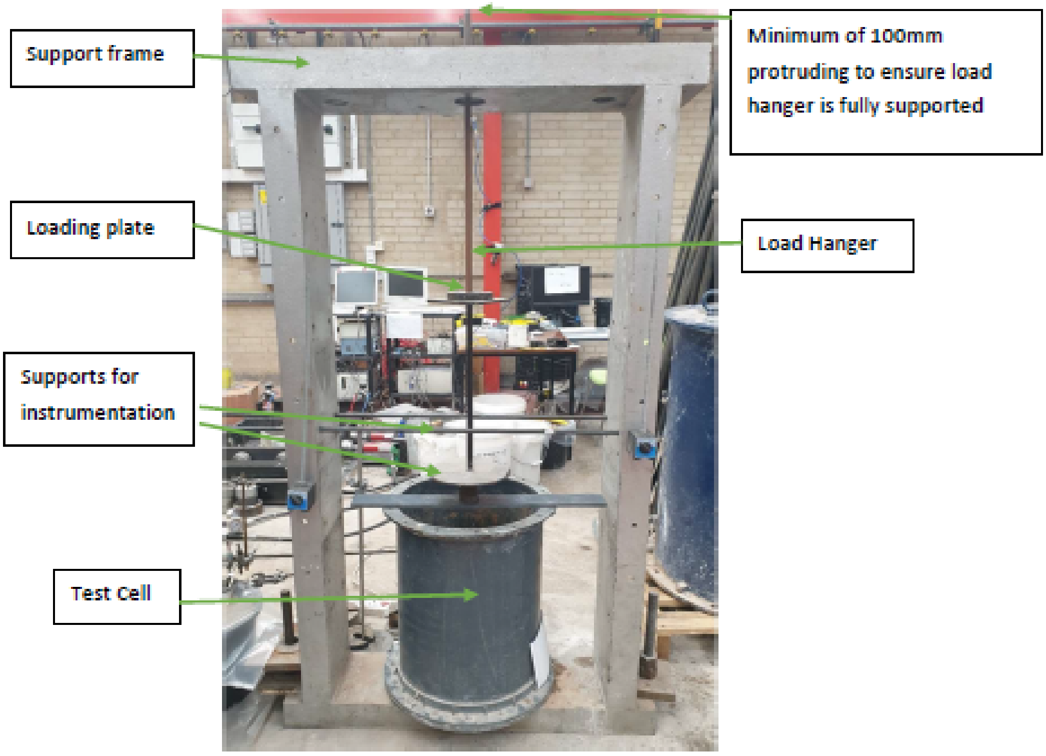


Figure 3-26: Static load test setup

Figure 3-27: Static load test in progress

3.7.2.1 Instrumentation

The same instrumentation that was utilised for the constant rate of strain tests was used for the static load tests. However, the settlement of the foundation plate was measured at 3 points since there was space for 3 LVDTs within this test set up.

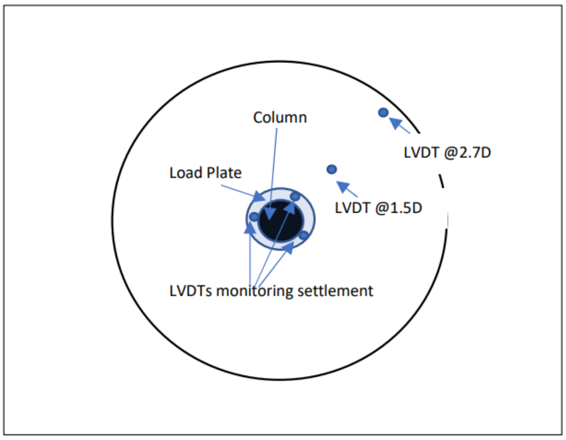


Figure 3-28: Plan view of test setup and location of LVDTs

3.7.3 Test preparation

The test preparation process, see sections 3.5.3 and 3.6.1, was kept the same as for the constant rate of strain tests to enable comparisons to be drawn between the two types of tests without installation factors becoming a variable.

3.7.4 Phase 2 Pre-Load Tests

Prior to the commencement of the column loading the following tests were conducted:

- PSD of column aggregate (to enable an assessment of particle breakage during loading)
- Water content of host soil (for the no column tests this measurement is taken from the material prior to compaction, for all other tests this is measured from the material removed from the core from a depth of 0 – 400 mm prior to column installation)

- Undrained shear strength of host soil (measured at the centre of the test cell prior to the core being removed, no measurements were able to be taken for the no column tests).
- Column density (estimated by measuring the weight of material placed in the bore and the approximate volume of the bore)

3.7.4.1 Strength and Water Content

The strength of the host soil is a critical aspect of the test as greater strength soils offer increased lateral resistance to the column, potentially enhancing performance. It was difficult to measure this before the test without disturbing the sample, although the clay compacted in the location of the GC would clearly have to be removed and disturbing this (with the hand shear vane) was possible if care was taken not to disturb the surrounding material. Therefore, the clay strength of the core was measured, at 100mm vertical layers (to enable pre-test base values to be established) before it was removed (water contents were also taken from the same sample points). This was not practical for ‘no column’ tests, in which case strength was only estimated post loading.

3.7.4.2 Column Density

The column density was estimated using the weight of aggregates compacted into the void together with an estimation of the volume of the bore. The author notes that the calculated density is only an estimation as the volume of the bore is only an approximation and could vary. Another factor is the potential for aggregate particles to be pushed into the sides of the bore during installation, despite attempts to avoid this. Despite these limitations it is hoped that having an understanding of the approximate density of the column will help interpretation of the observed behaviour (denser columns can lead to an increased stiffness and enhanced interactions between the column and the surrounding clay (McKelvey, Sivakumar, Bell and Graham, 2004; Sivakumar *et al.*, 2004).

3.7.5 Post-Test Measurements

3.7.5.1 GC Aggregate Removal and PSD Test

The GC aggregates were removed from the apparatus using a vacuum cleaner (used solely for this task); with a tube (approximately 30 mm in diameter) attached to enable the aggregates to be removed without disturbing the sides of the bore. Whilst every effort was made to recover all material there were two main ways in which aggregates were lost:

- Firstly, due to the column 'bulging', under load, into the host soil. After each test there were a number of particles that had been pushed into the side of the bore and so could not be removed (without disturbing the host soil, which was undesirable).
- Secondly, it is acknowledged that some fine particles, produced as a result of aggregate breakage or crushing during the test, could have been lost during the removal process. Generally, a minimum of 70 % of column material was recovered.

When removing the aggregate, the sample was split into two sections: the top 150 mm from the column and the bottom 250 mm to enable any difference in water content to be detected. It was thought that due to the bulging occurring in the top half of the column, potentially causing additional drainage to occur, that the water content would be greater than for the bottom. The material was weighed and left in the oven overnight to determine the water content of the material. This was measured to determine the extent of drainage from the host soil into the column.

3.7.5.2 Post-Test PSD of Aggregates

The dried GC aggregate sample was sieved to determine the post-load test PSD. The aim of this was to provide an indication of particle breakage during the load test for each of the materials. The potential for particle breakage is one of the concerns relating to the use of AA in geotechnical applications so this simple test was of great interest.

It is understood that the loss of aggregates into the host soil impacts on the ability to consider the complete impact of particle breakage. Therefore, this approach is only indicative. However, this was not the only method used to understand the potential for breakage, compaction tests were undertaken (standard and modified Proctor hammer as well as a vibrating hammer – with greater energy input than the poker used in phase two testing) and PSD curves were obtained before and after the application of compactive effort (previously described in Section 3.2).

3.7.5.3 Column Shape and Penetration into the Host Soil

After the removal of the GC aggregates the depth to the toe of the column was measured (using the rim of the test cell as a datum) to assess whether there has been any penetration of the column into the soil. This is failure mechanism column common with short columns and so was not expected during this research, checking allowed this expectation to be qualified. This is only an approximate measurement.

In order to evaluate the deformation of the column, and to assess the extent and location of bulging (which is an important indication of how the column transfers load into the surrounding soil), Plaster of Paris was poured into the empty bore (after vacuum removal of the GC aggregates). This enabled a model of the column post-loading to be created and observed. Measurements were taken of the model in order to enable comparison of the extent of bulging between each of the types of test and GC material. The author acknowledges that these measurements of the model only give an indication of the extent of bulging as the aggregate particles embedded into the sides of the bore during loading become part of this model and could distort the recovered shape of the column. However, it was deemed that the models could be used to give estimate of the extent of bulging.

As can be seen in Figure 3.28, bulging occurred at the top of the column (i.e., closest to the surface of the clay) which is to be expected given that the overburden pressures are the lowest (McGabe, McNeill and Black, 2007). The extent of bulging occurring for each of the columns will be discussed further in Sections 6.2.8, 6.3.2.1 and 6.4.7.

Figure 3-29: Example of Plaster of Paris models of post load column shapes

3.7.5.4 Soil Strength and Water Content

In an attempt to assess the impact of the column on the surrounding soil (i.e., any stiffening) the strength of the soil was measured using the hand shear vane apparatus at various distances from the centre of the column at 100 mm vertical intervals from the surface to the base of the column. Measurements were taken at 1.5D, 2D and 2.5D to determine the extent of influence of the column. These values were chosen based on the work of Hughes and Withers (1974) who identified that at distances greater than 2.5D a single column has no influence on surrounding soil.

Water content samples were collected at distances 1.5D and 2.5D from the centre of the column at vertical intervals of 100mm to assess the variation in moisture content caused by the column being loaded. These values were compared to the measurements collected prior to the test.

3.8 Summary

In this chapter the methodology used to assess the performance of recycled/secondary aggregates within granular columns. Three types of waste materials were utilised as AA (waste container glass, waste flat glass and APCr) to form granular columns and these were compared against a PA, granite.

Two types of loading scenario were modelled, static loading and constant rate of strain, to determine to what extent the loading style impacts on column behaviour.

In additional two different sized loading plates were used to apply load to the column to simulate axial and foundation style loading. The primary reason to model a column where the boundary effects, due to the size of the test cell, were limited. It is recognised that in order to eliminate all potential boundary effects a very large test cell would be required.

4 Results and Discussion – Phase 1: Aggregate Index Tests

In this chapter the results of the aggregate index tests conducted are discussed. The main aim of the index tests (conducted on granite, APCr, window glass and container glass) was to investigate the overall quality of the materials and determine their suitability for use within granular columns based on the existing guidance (this would be assessed further when considering the outcomes of Phase 2 testing). The relevance of the recommended index tests for aggregates to be used within granular columns has been called into question by many researchers, hence comparing the performance of the materials in Phase 1 (index tests) and Phase 2 (model column tests) would help assess the validity of these index tests when considering the suitability of materials for use within granular columns.

4.1 Aggregate Index Tests

Four materials comprising three alternative aggregates (window glass, container glass and APCr) and one primary aggregate (granite) were investigated in this research. There are a number of established index tests that have been deemed appropriate for determining the suitability of materials for use within granular columns BRE (2000). In the most recently published guidance on the construction for VSCs (BRE, 2000) these properties are as follows:

- PSD and particle shape (in particular flakiness)
- Aggregate Impact Value (AIV)
- Aggregate Crushing Value (ACV)
- Los Angeles Abrasion Value (LA)
- Angle of internal friction (as determined by the shear box test)

4.1.1 Particle Shape

The particle shape and PSD clearly have an impact on the engineering response of coarse-grained soils (Holtzs, Kovacs and Sheahan, 2016). Figure 5.1 provides a general overview of possible particle shapes.

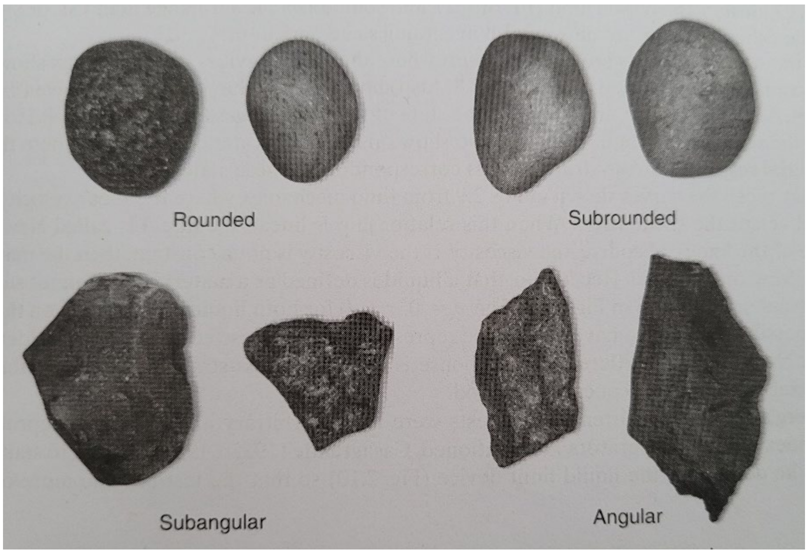


Figure 4-1: Typical shapes of coarse-grained material (Holtzs, Kovacs and Sheahan, 2016)

The images below (figures 4.2-4.5) show the materials that were used within this research. Figures 4.2 and 4.3 present samples of the container glass and window glass. The container glass was sourced from the Viridor waste glass processing centre in Sheffield. At the time there were no materials washing facilities at the centre and so the material was washed twice with clean water in the laboratory prior to use for health and safety reasons. The window glass was sourced from a building that was being refurbished on campus, this glass was also washed prior to use.

Both samples of glass included larger fractions (i.e., greater than 9.5 mm) that were not included within the column aggregate (section 3.4.4.5). From the images it can be seen that the overall shape (in term of angularity) of the particles of the two materials are similar and (based on Figure 4.1) these particles can be classified as sub-angular. The main difference between the two types of glass is that the larger particles of container glass clearly show the curvature of the original container, this was less prevalent for the smaller fractions used within the testing, and the thickness of the particles. Due to the roller crusher being used at the waste treatment plant to break the container glass the particles did not have sharp edges. The window glass, which was delivered large panes of glass and was broken using the LA machine, also features particles that are rounded but, whilst difficult to quantify, to a lesser extent that the container glass. The PSD of the two types of glass was made to be as close as possible and so the main difference between the two materials was the particle thickness (and potentially chemistry of the glass, although this was not assessed in this study).

Figure 4-2: Sample of container Glass

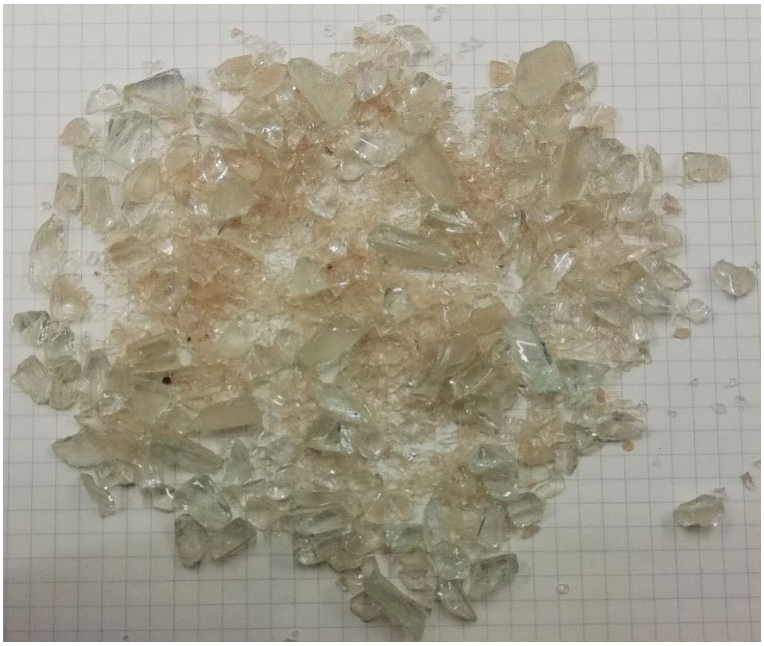


Figure 4-3: Sample of window glass

The APCr was sourced from the University of Greenwich. The material has spherical particles, related to how the material is created in the accelerated carbonation process (using rotating drums), of the APCr are shown in Figure 4.4. Whilst the larger particles (great than 9.5 mm) shown in the image were not used within the columns (these were sieved out pre-investigation: 3.4.4.5) the poorly-graded nature of the material can be seen along with the roundedness. Perhaps more difficult to determine from the image is the surface roughness of the particles (again a function of the accelerated carbonation process). Figure 4.5 presents an image of the granite used within this research, which can be classified as angular. The granite was sourced from materials suppler ' www.thestonewarehouse.co.uk'.

Figure 4-4: Sample of APCr

Figure 4-5: Sample of granite

4.1.2 Material Properties

In addition to the properties listed, water content (as received), loose bulk density and particle density were also measured for each of the aggregates (Table 4.1).

Table 4.1: Summary of properties of aggregates

Material	(As received) Water Content (%)	Loose Bulk Density (kg/m^3)
Granite	0.29	1470
APCr	5.60	985
Container Glass	0.04	1316
Window Glass	0.09	1309

The natural water content of the APCr was much higher than for the other materials and the water content of the granite was approximately three times greater than that of the glass. These results were expected due to the impermeability of the glass and the (relative) greater porosity of the APCr and granite. The APCr has a water absorbency value of 18.8 % (EN 1097) (Tota-maharaj, Hills and Monrose, 2017). The water absorption of granite is significantly lower than the APCr at 0.2 – 0.4 % (Jamshidi *et al.*, 2016; Kim *et al.*, 2018). Some published values of water absorbency for glass exist (0.5 % (Arul. Arulrajah *et al.*, 2014), 1.0 % (Arulrajah *et al.*, 2013)) , which seems incorrect due to the hydrophilic nature of glass (Almesfer and Ingham, 2014). However, if waste glass samples include other materials such as paper (from labelling) this would explain the water absorption values (Arulrajah *et al.*, 2013). It is of interest to explore the impact of the different relationships with water of the materials with regards to column behaviour and this will be expanded on in Chapter 6.0. The value of loose bulk density for both types of glass are very similar which is not surprising given that they are the similar materials (with different chemical compositions), and the PSDs were designed to be comparable. The low value of loose bulk density of the APCr (also encountered by Tota-maharaj et al.,2017), means that it is defined as a lightweight aggregate (aggregates with bulk densities less than 1200 kg/m^3 are defined as lightweight (BS EN 13055:2016 BSI, 2016).The particle density of each of the aggregates are reported in Table 4.2, along with published values.

Table 4.2: Particle density measured for each of the aggregates studied during this research together with some published values

Material	Particle density measured during this research (Mg/m^3)	Published Particle density results (Mg/m^3)	Reference
Container Glass	2.48	2.50	(M. Disfani, Arulrajah, Ali, *et al.*, 2011)
		2.48 – 2.49	(wartman, J; Strenk, 2004)
		2.51-2.52	(Ooi *et al.*, 2008)
Window glass	2.52	*No data available specifically for window glass*	
APCr	2.17	1.94	(Tota-maharaj, Hills and Monrose, 2017)
Granite	2.62	2.65	(Joel, 2010)
		2.9	(Ubi *et al.*, 2020)
		2.63	(Theodoridou, 2009)

The information in Table 4.2 above indicates that there is variation in the previously published particle density values for each of the materials. The result for granite obtained in this research is within the range of the reported values. There does appear to be a difference in the published particle density value of APCr compared to the result reported within this research; this is not surprising given the variation in source materials that the material is created from (Gunning *et al.*, 2011). The result for container glass matches the lower values reported by other researchers. There are no published values specifically on window/glass which is likely to have a different chemical composition to glass due to the additional requirements of optical clarity. The particle density of the window glass measured in this research lies on the upper bound of the reported values for container glass.

4.1.3 Proctor Compaction

The proctor compaction test is suitable for both cohesive and granular soils, however, vibration compaction is normally considered the most suitable approach for granular materials (as detailed in BS 1377: Part 4: 1990: 3.7). However, when this method was trialled, the materials crushed excessively (Figures 4.6 to 4.9) with the APCr experiencing the greatest proportion of breakage; indeed, the breakage was so significant that questions were raised if the material should not be used any further (although was, with surprising results). The process for conducting this test with particles susceptible to crushing is detailed in BS 1377: Part 4: 1990: 3.7 and comprises using a fresh sample of aggregate for each stage of the test. According to BS 1377: Part 4, the material is to be placed in 3 layers (using a CBR mould) and each layer is to receive 60 seconds of vibration. The APCr in particular was reduced to a large amount of "dust" (very fine sand or silt size or less – this was not quantified) and so for this reason the test was not adopted to determine maximum dry density and optimum water content. Figures 4.6 to 4.9 present the PSDs for each material before and after vibration.

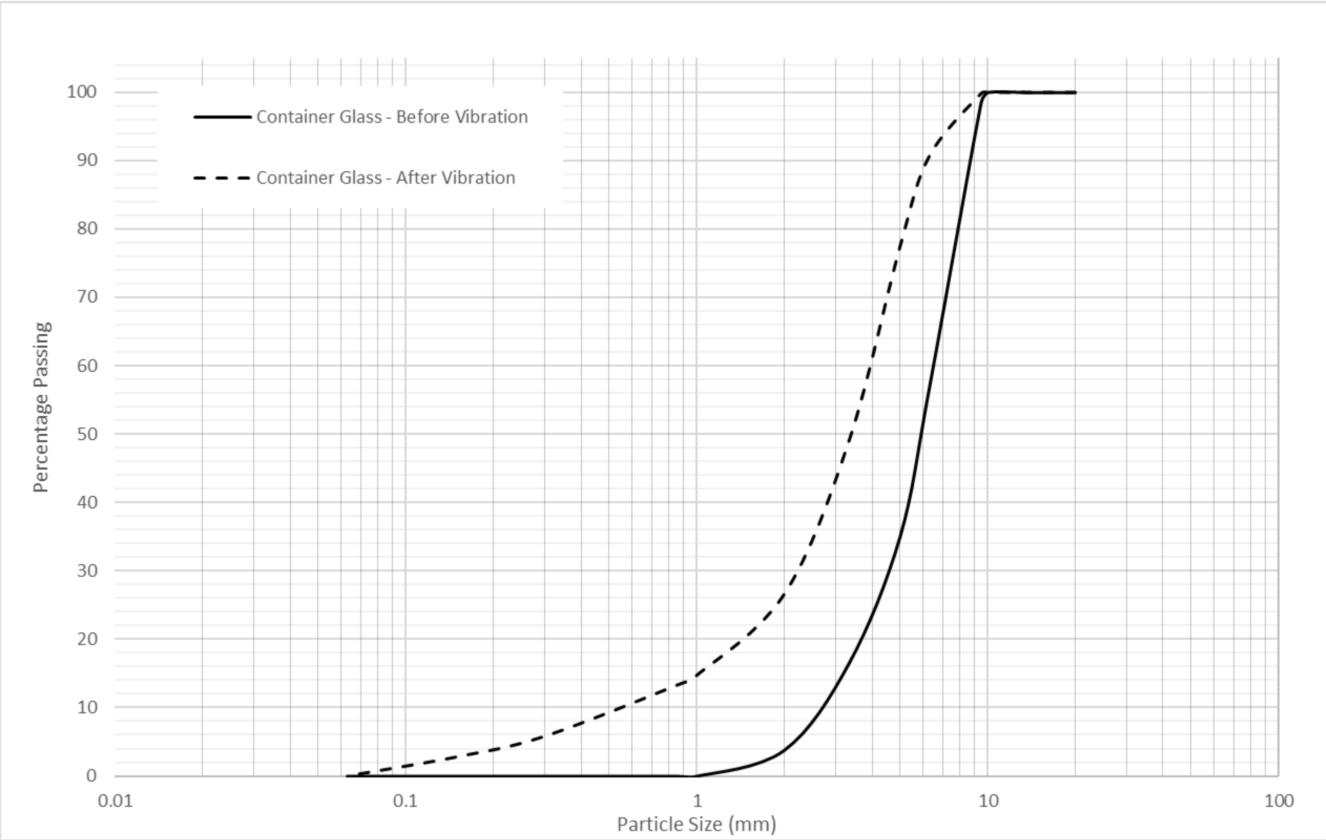


Figure 4-6: Container glass - PSD before and after compaction via vibration

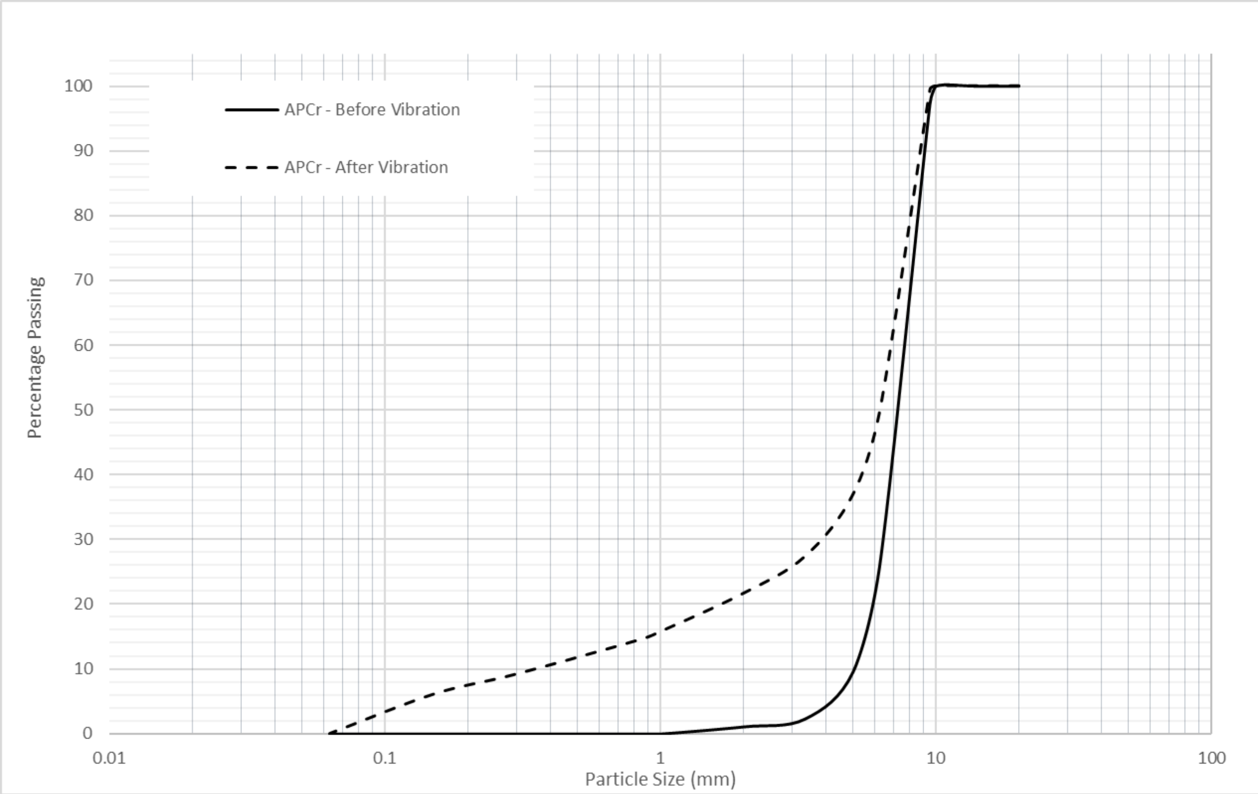


Figure 4-7: APCr - PSD before and after compaction via vibration

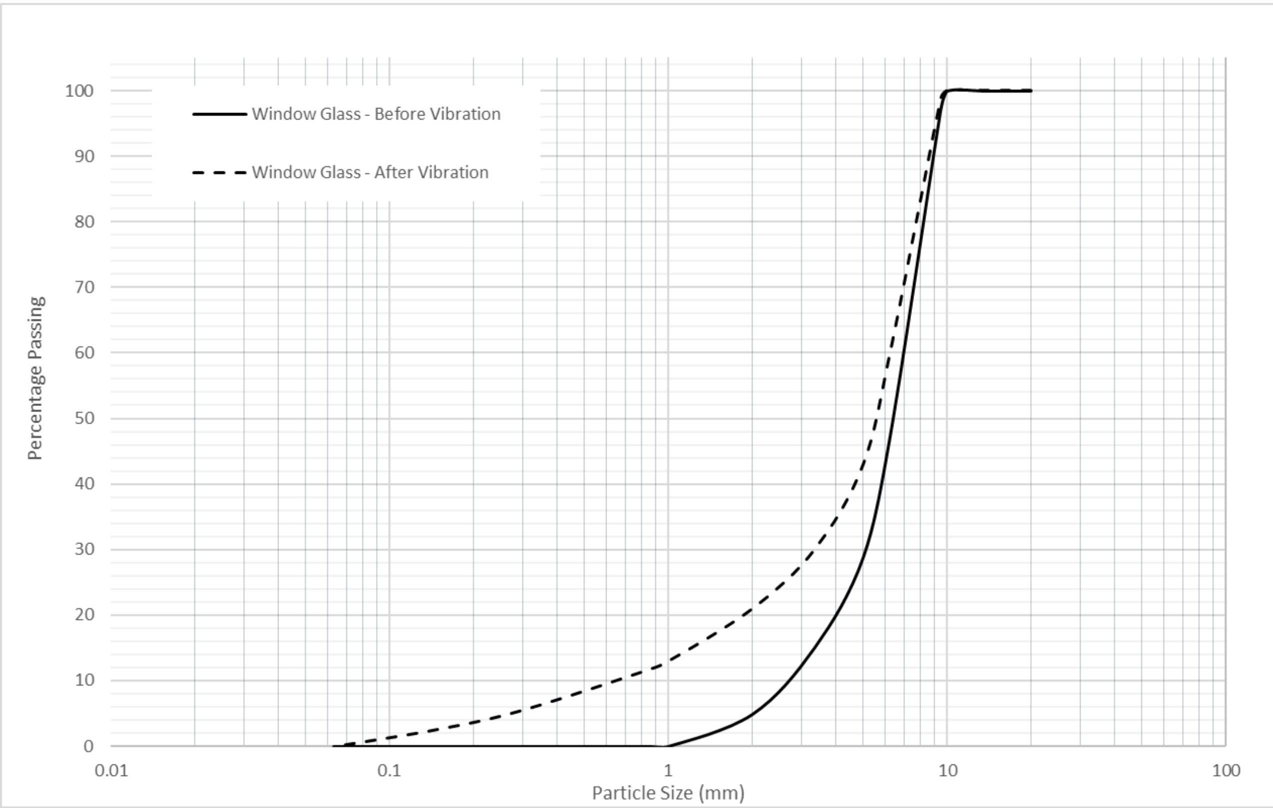


Figure 4-8: Window glass - PSD before and after compaction via vibration

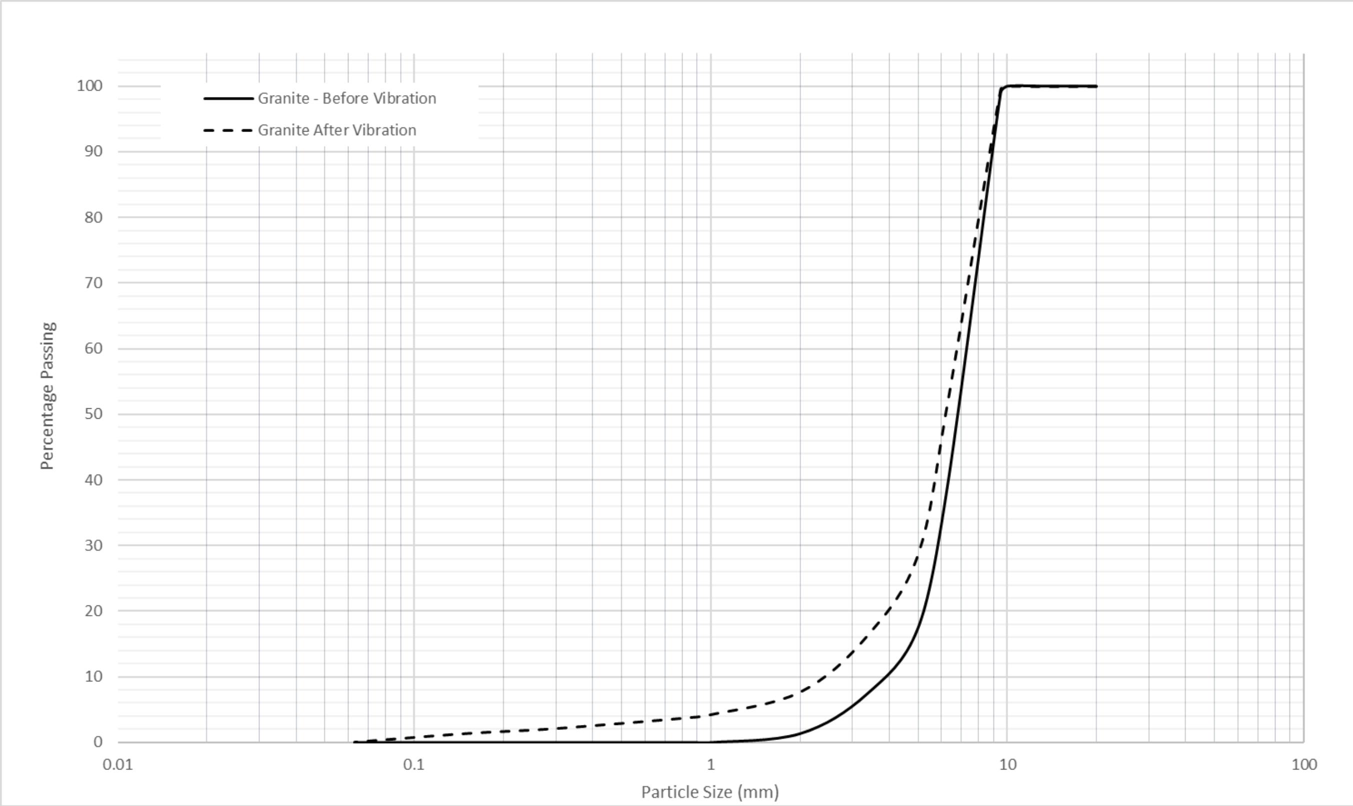


Figure 4-9: Granite - PSD before and after compaction via vibration

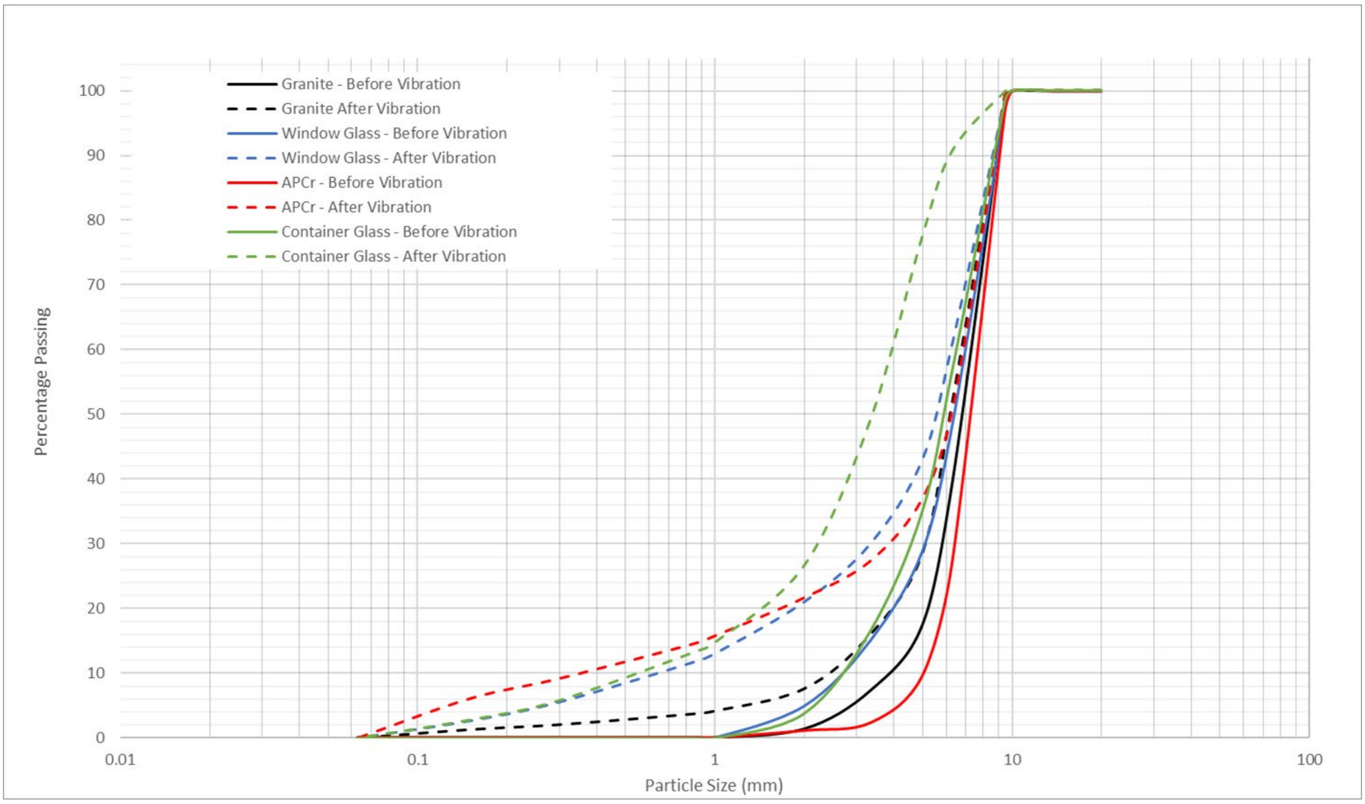


Figure 4-10: Plot comparing/contrasting the PSDs of all materials before and after vibration.

The comparison of results shown in Figure 4.10 shows that all materials are affected to some extent by the application of vibration and highlights that APCr is most affected.

In addition to the vibro-compaction, the standard Proctor compaction test was undertaken for each material; the modified Proctor test was not used due to the extent of particle breakage observed. Due to the potential for particle crushing a fresh sample of material was utilised for each stage of the test (as recommended by Head, 2006), this meant that there was a limit to the number of repetitions of each test that could be carried out.

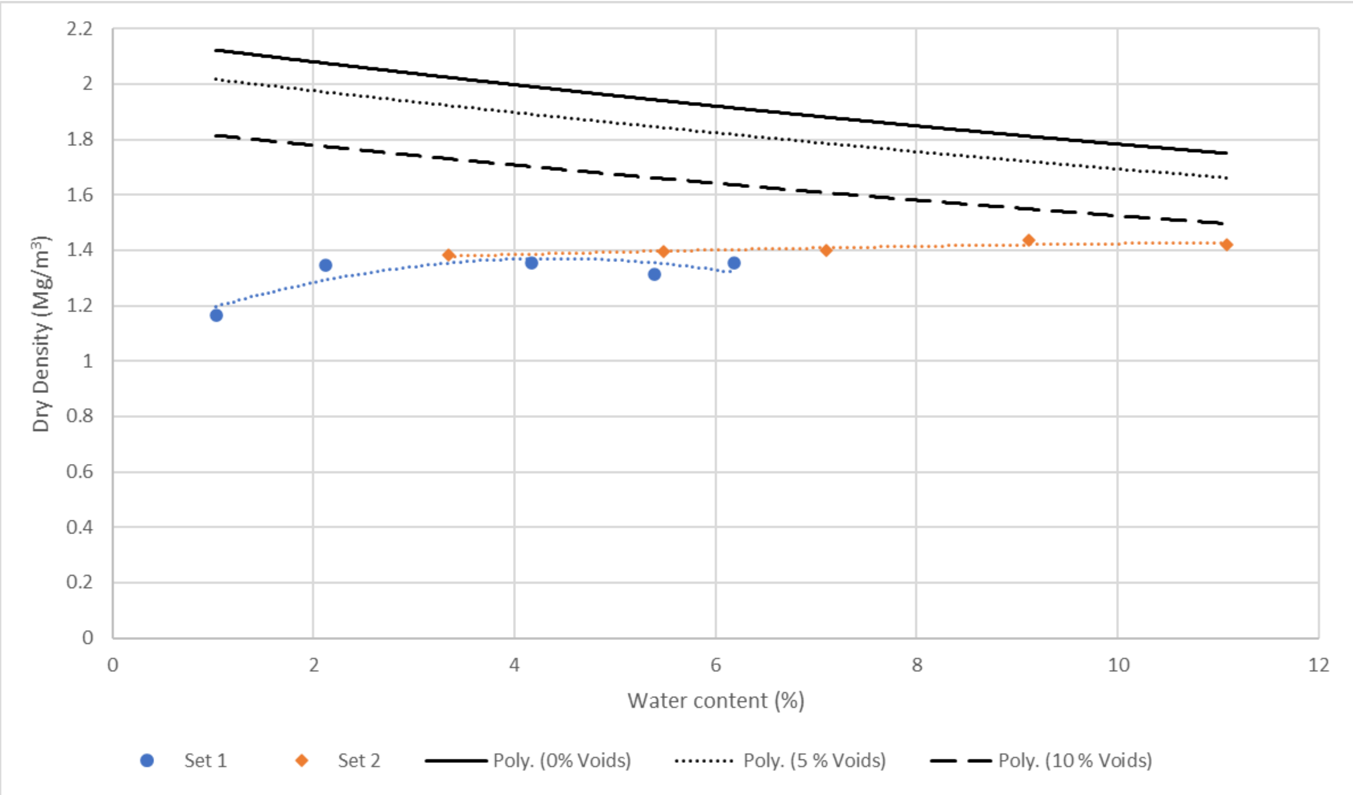


Figure 4-11: APCr: Standard proctor compaction results (note that a set refers to 5 samples of material, each tested at a different water content according to BS 1377:1990 part 4)

The proctor compaction results for APCr (as shown by fig. 4.11) indicate that the material is insensitive to water content, which is consistent for poorly graded materials. However, due to the relatively high absorbency of the material (18.8 %), which is likely to increase as the surface area of the material increased due to crushing, these results may not be representative. The lack of data to draw comparisons to is a challenge (as a way of determining validity) but in Phase 2 the column material was installed dry.

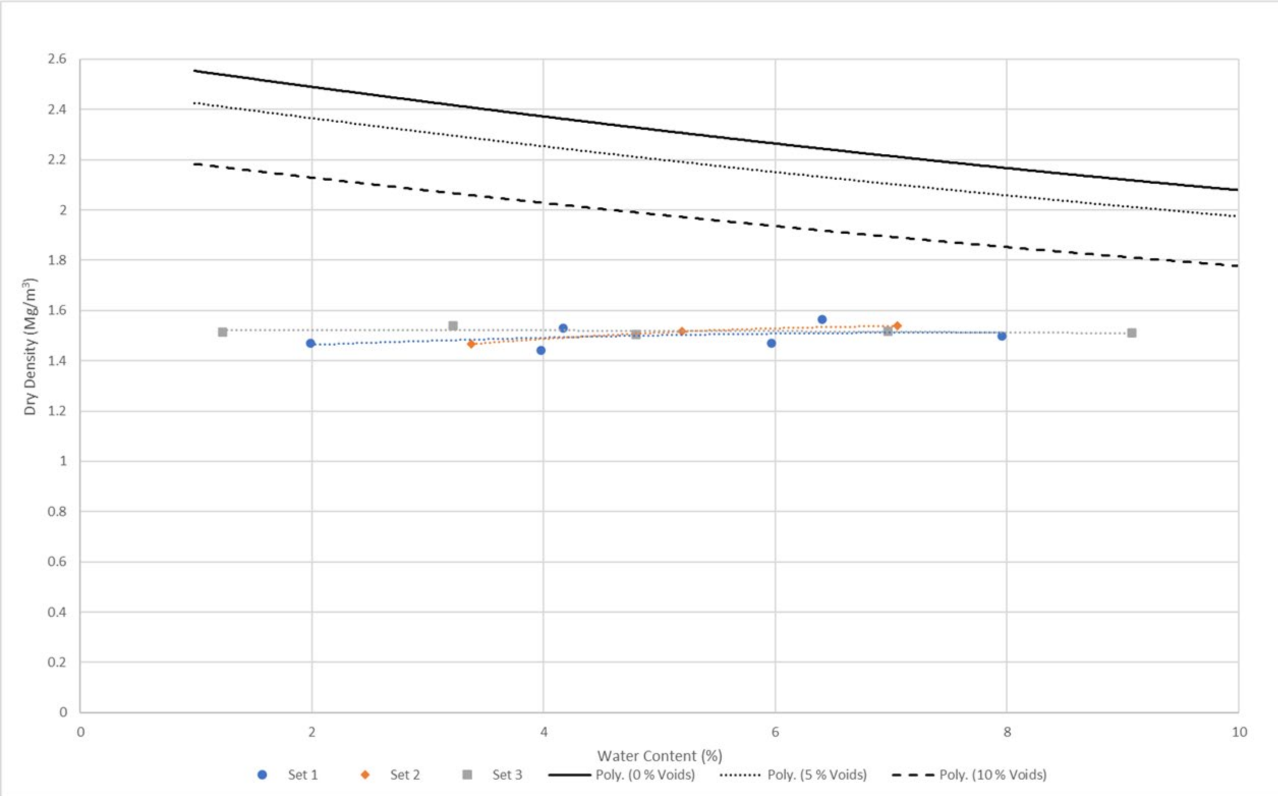


Figure 4-12: Granite: Standard proctor compaction results

The results of the standard proctor test conducted on granite (Figure 4.12) do not reflect a strong correlation between water content and dry density. Whilst no specific data on granite was found for comparison one study found that when the proctor compaction tests were conducted on gravel there was no correlation between density achieve and water contents (between 5 – 8 %) (Connelly, Jensen and Harmon, 2008).

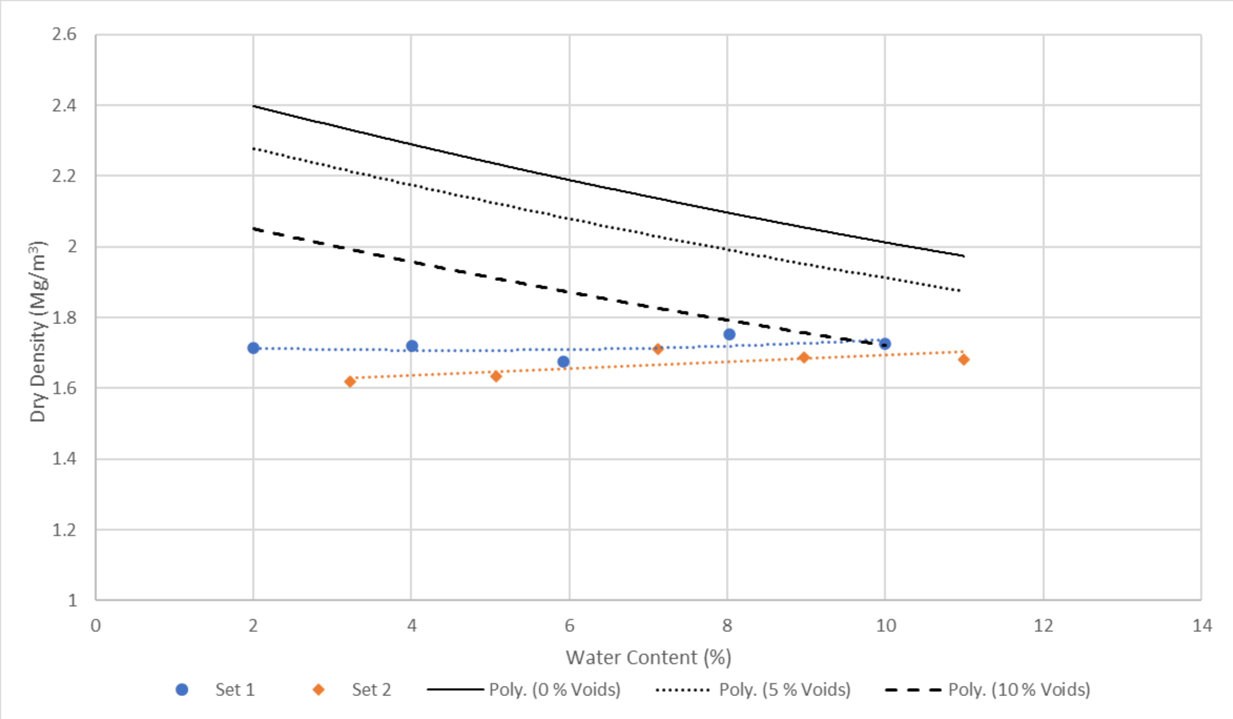


Figure 4-13: Window Glass: Standard proctor compaction

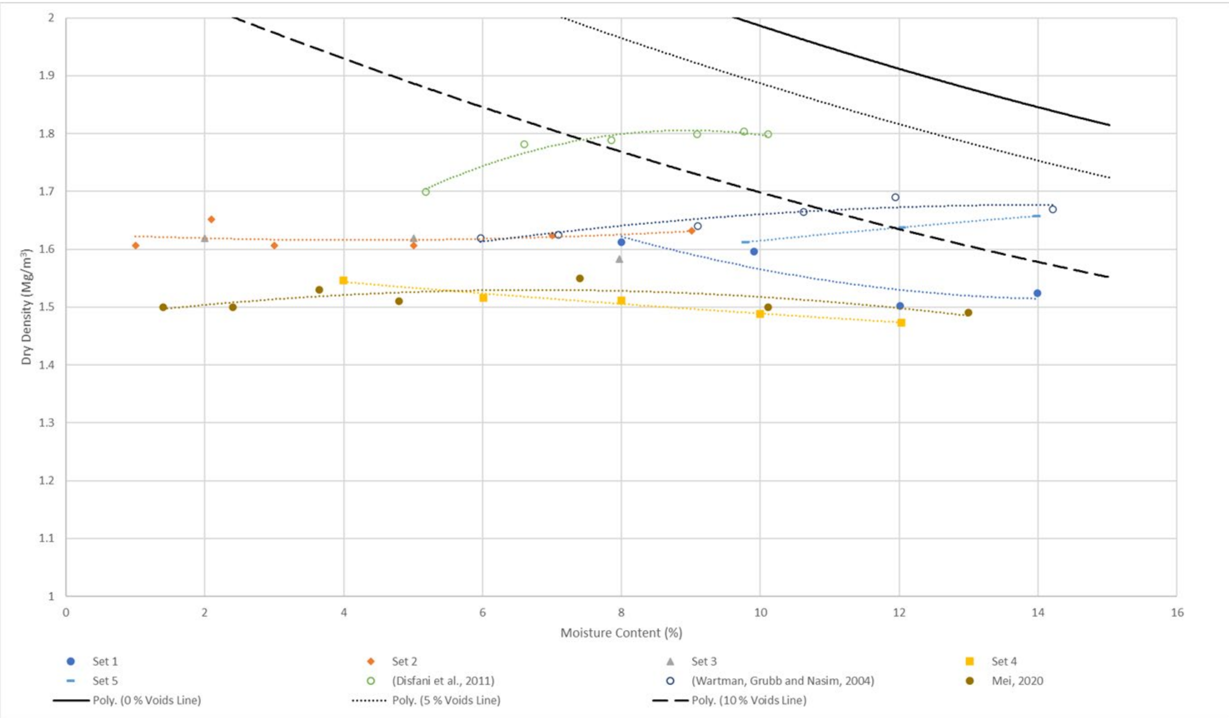


Figure 4-14: Container Glass: Standard proctor test results including data from the literature

Overall, it can be seen that there is no strong correlation with water content and the maximum dry density achieved for both types of waste glass. Many other researchers have made the observation that glass is insensitive to water content (CWC, 1998; Wartman, Grubb and Nasim, 2004; So *et al.*, 2011;

Arulrajah *et al.*, 2013) indicating stable compaction and good workability over a wide range of water contents. In the case of the waste container glass there also appears to be a variation in maximum dry density, which is likely due to the variation in PSD of the material tests. In addition, the poorly graded nature of the materials is likely to have had an effect on the density achieved. The data presented in Table 4.3 indicates that the optimum water content of container glass is between 8 to 10 %. The container glass was tested at water contents within that range during this research, but no obvious optimum water content could be determined. It would be of interest to understand how Ooi et al. (2008), Disfani et al. (2011) and Dyer (2014) overcame the issue of water draining from the proctor mould at water contents greater than 7 % (as observed during this research).

Table 4.3: Additional results for optimum water content and maximum dry density for medium waste glass (i.e., particles less than 9.5 mm)

Max dry density (Kg/m^3)	**Optimum water content (%)**	**Reference**
18.5	9.7	(Ooi *et al.*, 2008)
18.0	9.0	(M. Disfani, Arulrajah, Bo, *et al.*, 2011)
19.5	8.8	(Dyer, 2014)

4.1.4 PSD: Particle Breakage During the Proctor Compaction Test

The proctor compaction test (including both the standard and modified) was used to determine the relative crushing potential of the aggregates (Figures 4.15 to 4.18). Smaller samples were collected from the bulk material according to BS 1377- 4:1990.

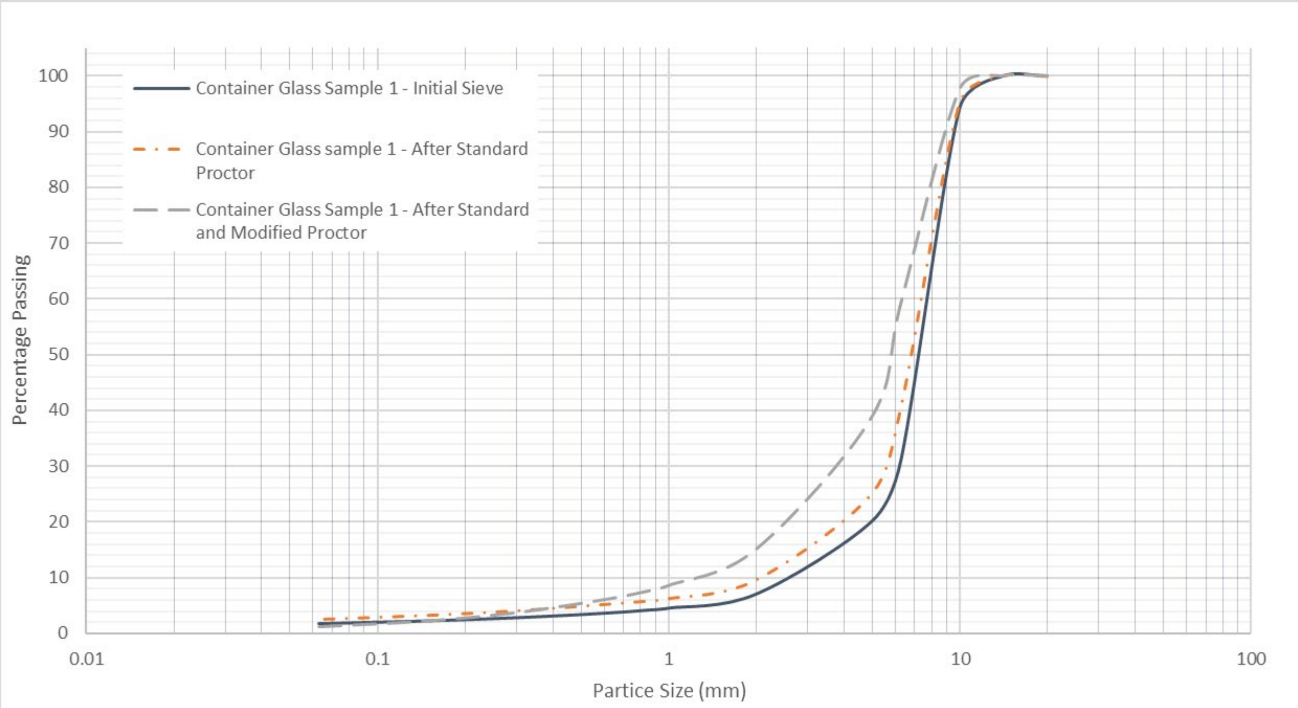


Figure 4-15: Container Glass sample 1 - Impact of standard and modified proctor hammer

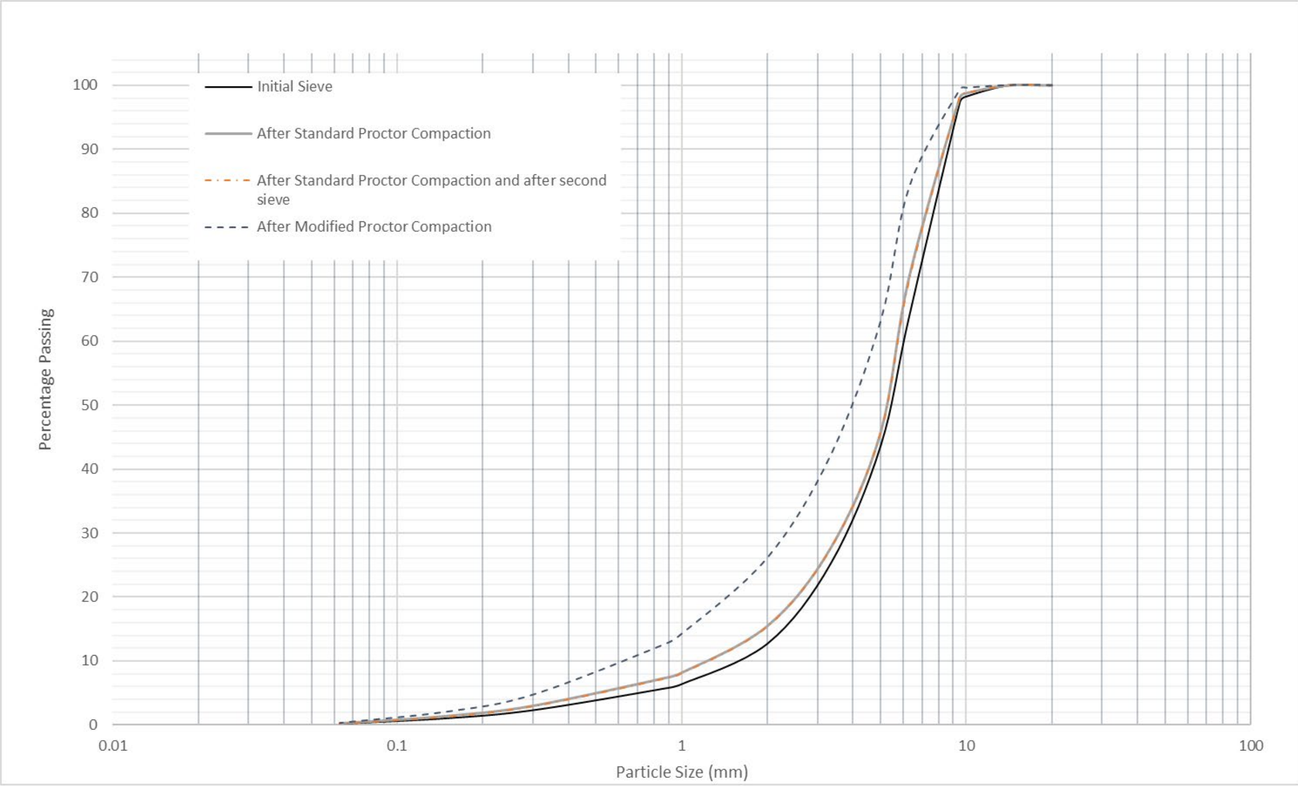


Figure 4-16: Container Glass sample 2 - Impact of sieving and standard and modified compaction

Figures 4.15 and 4.16 clearly show the impact on PSD of container glass after the impact of the standard and modified Proctor compaction. The compactive effort of the modified Proctor hammer appears to have a greater effect on particle breakage (approximately three times greater than the standard Proctor), unsurprising due to the increased weight of the compaction hammer (4.5 kg rather than the 2.5 kg of the standard Proctor). Sample two was sieved twice (after the standard proctor compaction) to assess the impact that sieving may have on particle breakage. Schouenberg (2004) noted that the mechanical sieving of AA could lead to particle breakage, however, on inspection of the PSD curves (Figure 4.15) of the material it appears that sieving has no impact on particle breakage. In the case of sample 1 and 2 the modified Proctor compaction was applied after the standard proctor and so there was potential for weakening of the material. To assess whether this was a factor the modified proctor compaction test only was conducted for two additional samples. This observation, that crushing occurs during standard proctor compaction and to a greater extent during the modified proctor compaction test, has also been made by (Disfani, Arulrajah, Bo, *et al.*, 2011).

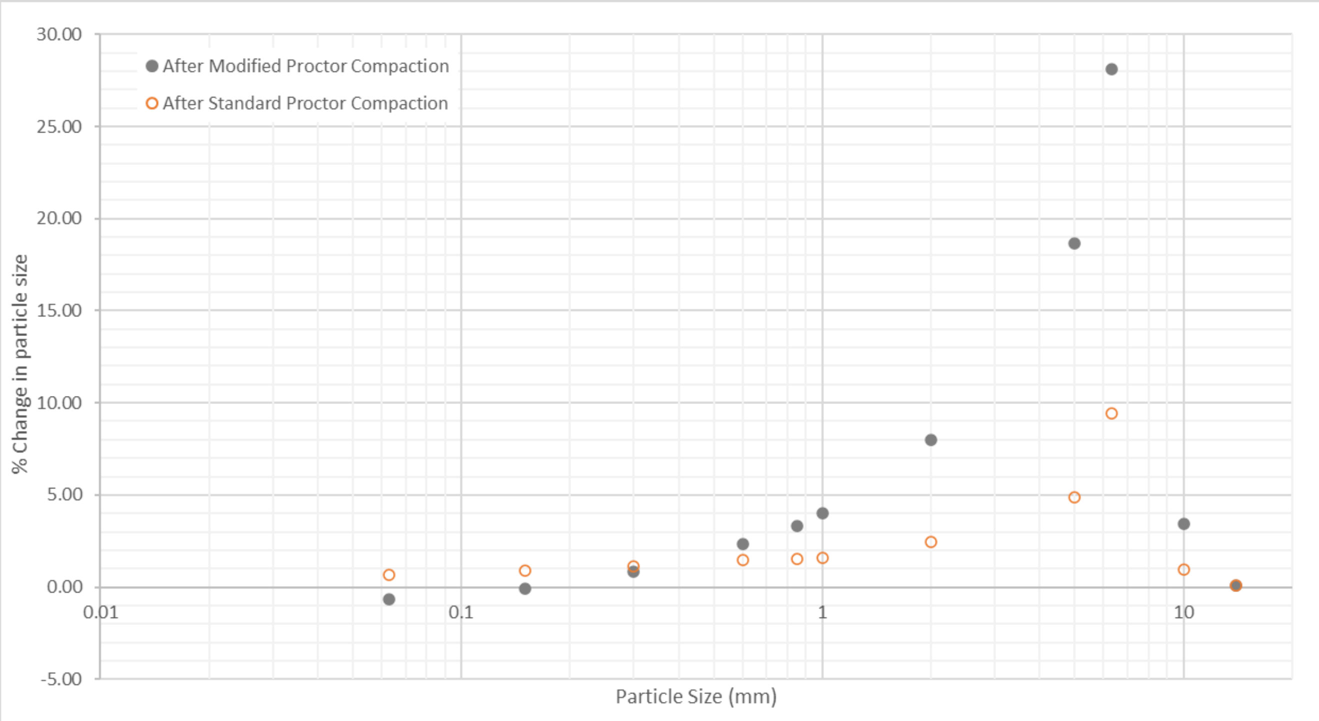


Figure 4-17: Percentage change in particle size for container glass samples 1 and 2 (where sample 2 is sample 1 post modified proctor test)

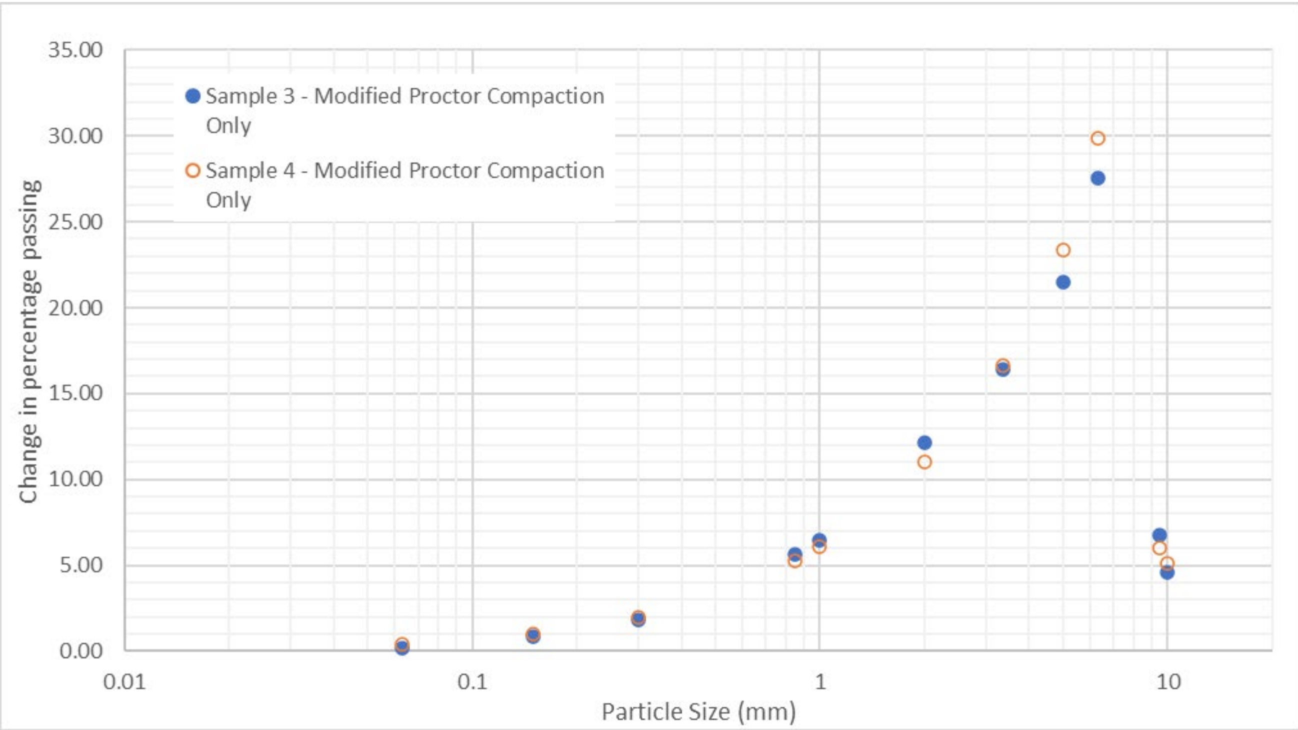


Figure 4-18: Container glass samples 3 and 4 - Change in percentage passing after modified proctor compaction only

Comparing the results of change in percentage passing for samples 1 (standard followed by Modified Proctor) with 3 and 4 (modified proctor only) (figures 4.13 and 4.14) the change in percentage passing was similar. These results indicate, in the case of these tests, that weakening of the material (via the compactive effort of the standard Proctor hammer) was not so significant that the extent of particle breakage was affected. This does not mean that weakening of the material via impact will never occur.

4.1.4.1 Window Glass

Schoenberg (2004) noted that in the case of more sensitive AA mechanical sieving can cause the cause particle breakage. In order the assess the impact of sieving on the PSD a sample of window glass was sieved twice. The results (Figure 4.18) indicate the impact of sieving on PSD is minimal.

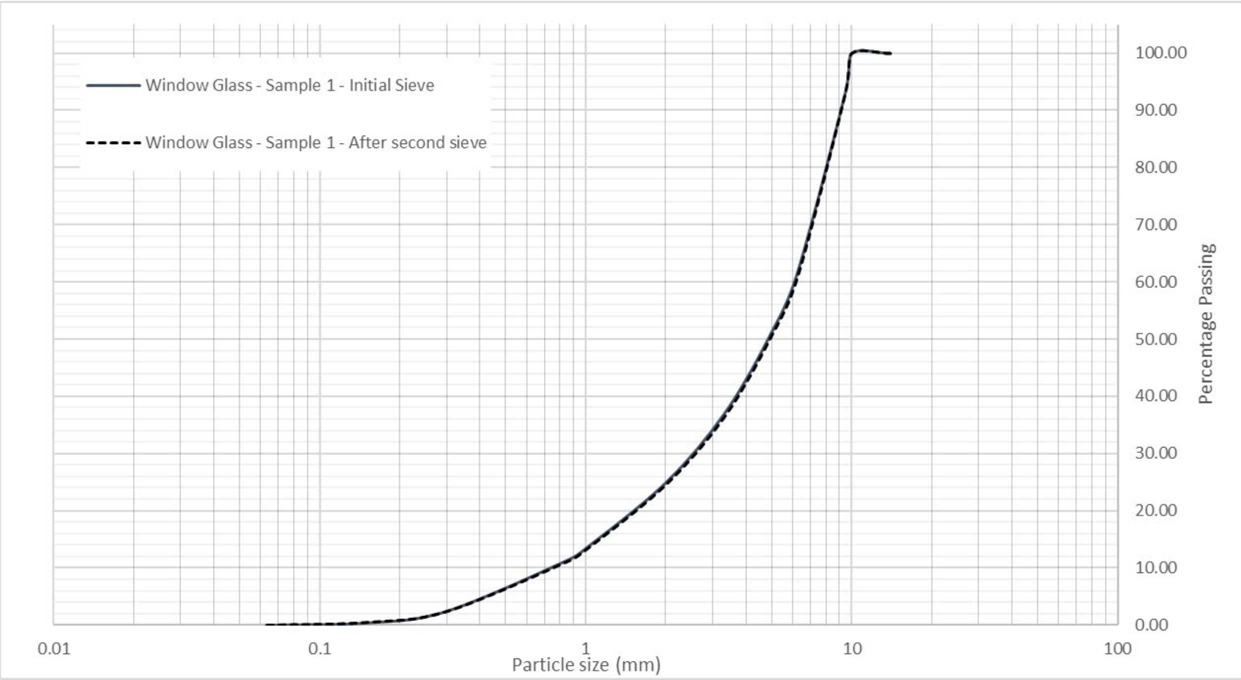


Figure 4-19: Window Glass PSD after first and second sieve

The PSD curves shown in Figure 4.20 below indicate that window glass is also subject to particle breakage similar to cullet and that all size fractions are affected. The impact of the standard Proctor hammer appears to be greater in the case of the window glass when compared to container glass. It follows that the impact of the modified hammer, whilst greater than the standard hammer, is less notable in comparison to the container glass.

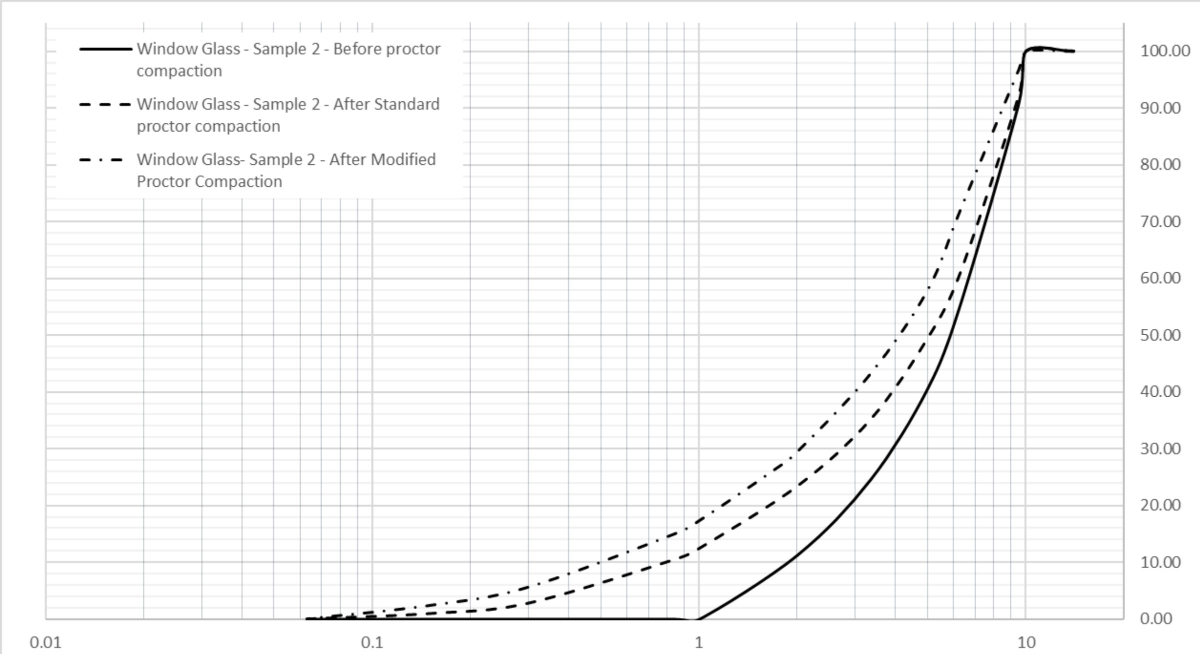


Figure 4-20: Window Glass - Sample 2 - PSD before and after both standard and modified proctor compaction

4.1.4.2 Granite

Figure 4.20 shows the PSD of granite before and after the impact of the standard and modified Proctor compaction test. The extent of the change in PSD is minimal in comparison with the other aggregates tested in this research. In particular, the modified Proctor compaction does not appear to have had a notably greater impact on PSD compared to the standard hammer. The low particle breakage of granite observed, compared to the AA, is unsurprising given the superior performance exhibited in the other tests such as AIV, ACV and LA. Granite is a natural (primary) aggregate and is expected to show a greater resistance to particle breakage.

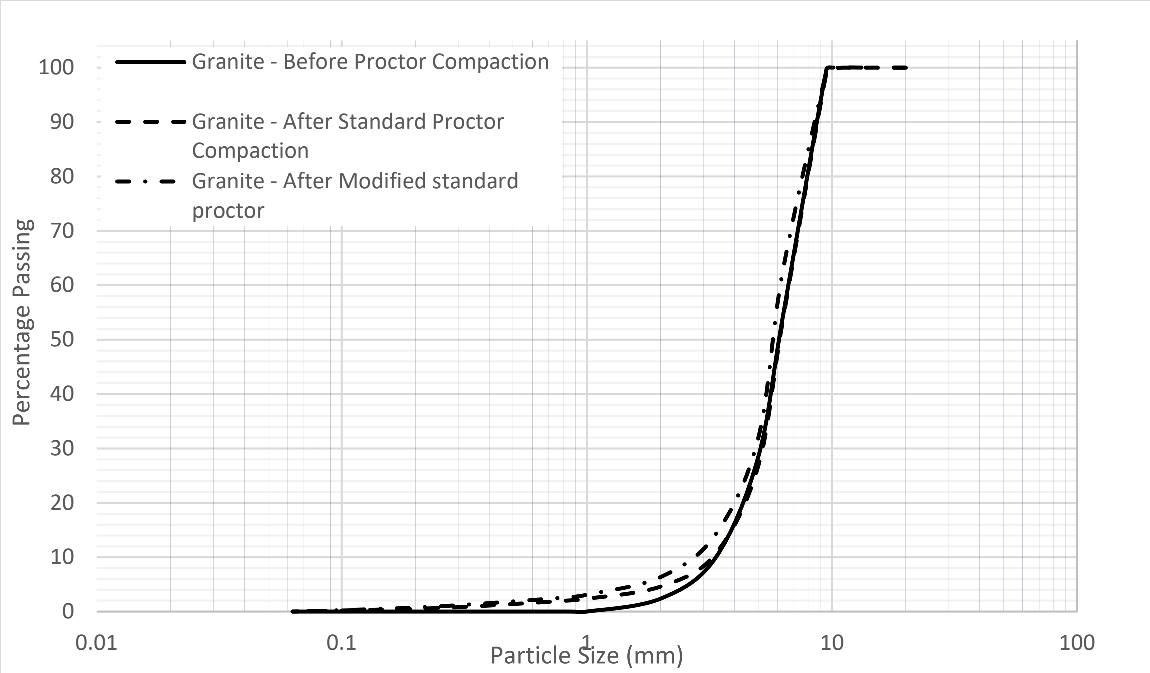


Figure 4-21: Granite PSD before and after both standard proctor compaction and modified proctor compaction

4.1.4.3 APCr

Due to the relatively high particle breakage potential of APCr (as observed during both the compaction via vibration and during the standard proctor compaction) new samples of material were used to assess the impact of the standard and modified proctor compaction (rather than conducing the standard compaction test followed by the modified test on the same sample).The material crushed significantly under the impact of both the standard and modified Proctor hammer and the change in PSD observed was the greatest of all the aggregates (figures 4.22 and 4.23). The impact standard Proctor hammer does not appear to have generated a significant amount of fines whereas the opposite is true of the modified proctor compaction. This suggests that the standard Proctor hammer led to particle breakage/weakening and the modified Proctor compaction led to the actual crushing of particles.

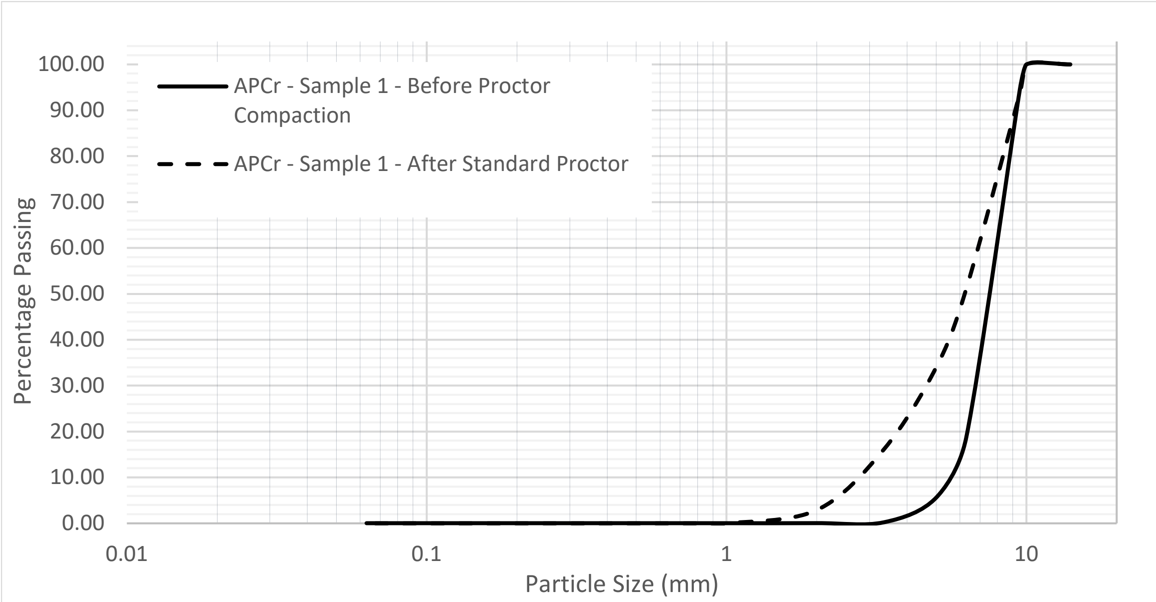


Figure 4-22: APCr Sample 1 - Before and after Proctor Compaction

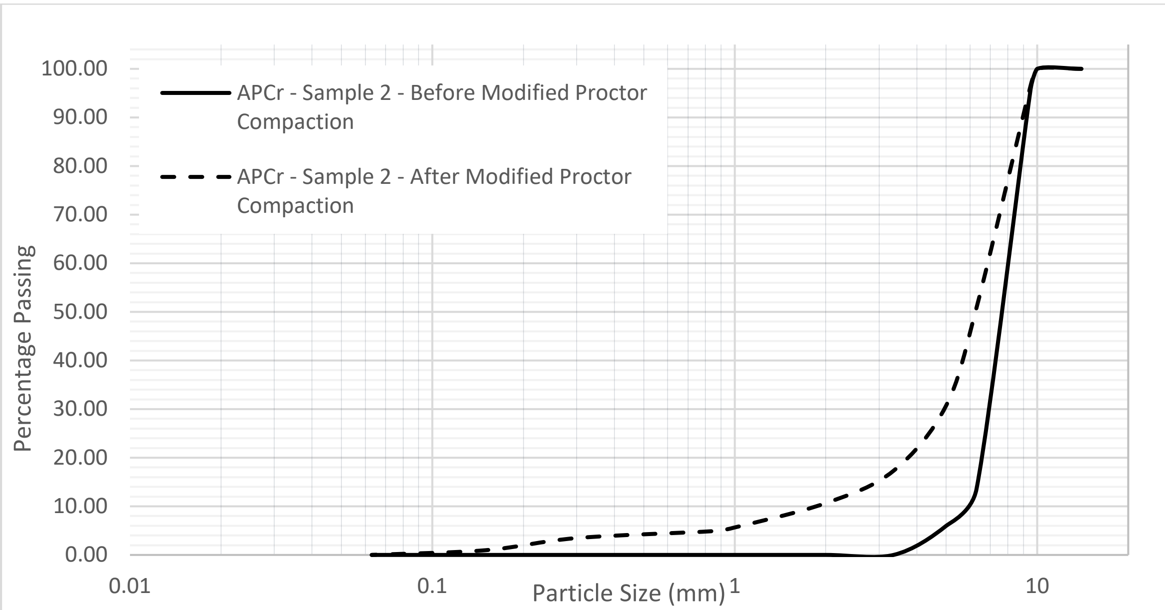


Figure 4-23: APCr - Sample 2 - Before and after Modified Proctor Compaction

APCr has high water absorption compared to the other materials at 18.8 %. In order to assess the effect of water absorption on the susceptibility of APCr to crushing, which could be a concern when using the material within granular columns, a sample of the material was soaked in water for a 24-hour period and then subjected to the standard proctor compaction test. The results shown in Figure 4.24 indicate that the absorption of water does not increase the crushing of the material. This is a positive finding considering that as the surrounding soil begins to drain into the granular column under load the material is likely to increase in water content.

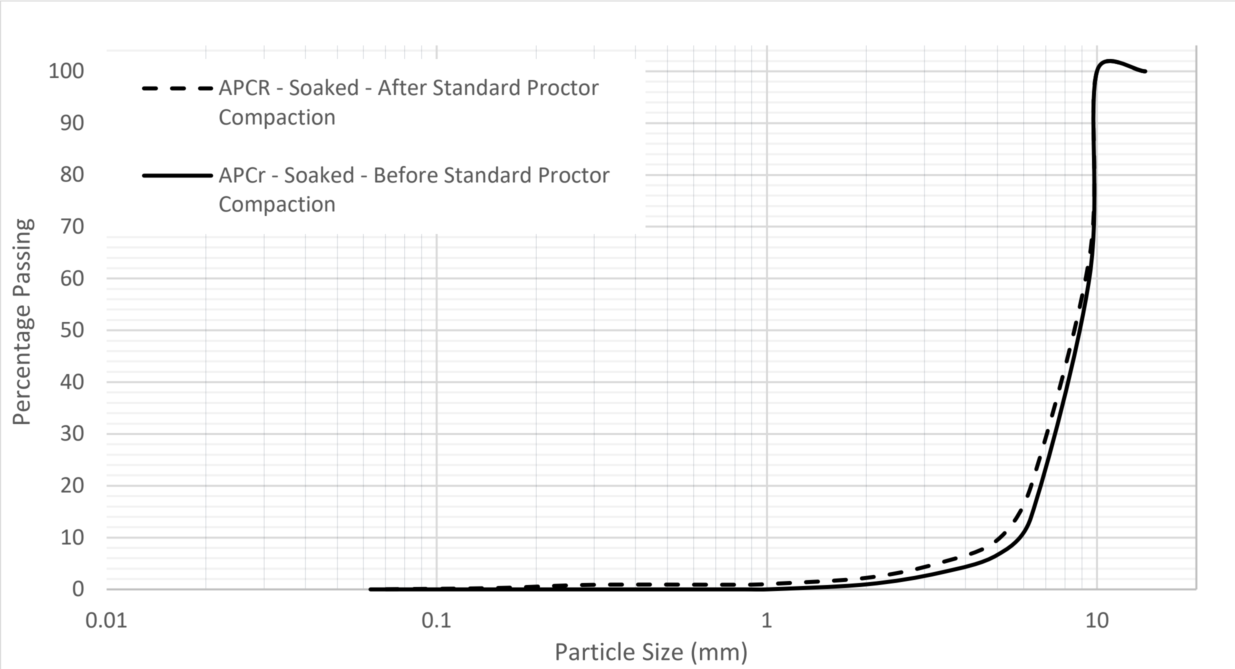


Figure 4-24:APCr PSD before and after standard proctor compaction (soaked)

4.1.5 Aggregate Impact Value (AIV)

The AIV test was conducted three times for each material and the average result is presented in Table 4.4. There were no published values available for both types of glass and the APCr. The test is designed to give an indication of the susceptibility for particle crushing under repeated and sudden forces and the greater the AIV value the stronger the aggregate (Mitchell, 2015). All materials meet the requirements issued in the 'Specifying Stone Columns' guidance published by the BRE (2000), with the granite being the most suitable (i.e., least susceptible to crushing).

Table 4.4: Summary of AIV results

<table>
<tr><th>Material</th><th>Actual Average AIV (%)</th><th>Value Reported by Amini (2015) (%)</th><th>Recommended Value (%) (BRE, 2000)</th></tr>
<tr><td>Granite</td><td>7</td><td>12.7</td><td rowspan="4"><30</td></tr>
<tr><td>Window Glass</td><td>16</td><td rowspan="3">-</td></tr>
<tr><td>Container Glass</td><td>19</td></tr>
<tr><td>APCr</td><td>22</td></tr>
</table>

It is surprising to note that both types of glass meet the quality criteria for crushing as this is one of the main concerns and barriers against the use of this material within granular columns (Sivakumar et al., 2004; Serridge and Sarsby, 2010; Zukri and Nazir, 2018). This test is an impact test (a 13-14 kg hammer is dropped repeatedly onto the sample); hence it is questionable how much this tells us about the performance of the material in situ within a granular column (this is illustrated with outcomes of Phase 2 testing; Chapter 6.0).

4.1.6 Aggregate Crushing Value

The aggregate crushing value (ACV) test is a measurement of the resistance of an aggregate to crushing by compressive force (Mitchell, 2015) and the higher the value the weaker the aggregate. This test perhaps better models the *in situ* response of the aggregates within a column under load (to qualify this statement it is clearly relative, as the test is undertaken in a metal mould which will create 'rigid' boundary conditions that the column would not likely experience *in situ* and hence the load transfer through the aggregates is not the same as it would be *in situ*), compared to the AIV and LA test (at least), as the test comprises the application of a steadily increasing load over a 10-minute period. The confined conditions (due to the container) do not allow for 'bulging', etc., so is still not an ideal test.

The results of the recycled and secondary materials are less favourable than those obtained for the AIV as the APCr exceeds the recommended value and both types of glass only just meet the criteria. Unsurprisingly, the granite performs well and falls well under the maximum recommended value. It is not clear to the author why the value obtained by Amini (2015) is so different to that observed during this study.

Table 4.5: Summary of ACV results

Material*	ACV (%)	Recommended Value (%) (BRE, 2000)	Value Reported by Amini (2015) (%)
Granite	13	<30	40
Window Glass	29		
Container Glass	29		
APCr	33		

4.1.7 Los Angeles Abrasion Value (LA Value)

The LA test is a measurement of the resistance of an aggregate to attrition (Mitchell, 2015). It was initially developed to consider the suitability of aggregates in the wearing course of a bound road (considering abrasion resistance of the aggregate due to passing vehicles), hence it could be argued that this test does not directly relate to aggregates in confined conditions. It also can result in some unusual findings, for example shale resulting in a better LA value than granite (Perkins *et al.*, 2021); this was attributed to energy dissipation between sedimentary layers resulting in improved additional resistance. Furthermore, the LA test comprises the placement of a sample of material along with steel balls into a drum and the drum is rotated numerous times. Therefore, this test does not seem to bear any resemblance to the experience of aggregates within a granular column, so it is difficult to place any confidence in the LA value when determining the suitability of a material for GCs.

The results obtained for all materials meet the requirements set out by the ICE (1987) and are all less than 50 %. However, the APCr only just meets this criterion with a value of 49 % and does not meet the more onerous criteria recommended by Serridge and Slocombe (2012) as is the case for the container glass. It is of interest to note that the LA value obtained for container glass within this research (42 %) indicates a much poorer performance than observed in other work for glass with similar sized particles (25 %) which is puzzling as the LA equipment is standardised. It should be noted that the author adopted an alternative option, included within the standard, for certain size fractions and some difference may be expected to the results of material tested using the standard test but not to the extent observed. This difference in LA values can only be attributed to the quality of the source materials; the values published by Arulrajah et al. (2014) indicate that the performance of the recycled glass is quite similar to waste rock.

The LA values measured in this research for the APCr are around 20 % greater than values recorded in other research (Gunning et al., 2014; Tota-Maharaj et al., 2017). The source material used to create the APCr aggregate affects the quality of the material and so variation between materials produced in different areas, using different source material, is to be expected.

Table 4.6: Summary of LA results

	Material	**Actual LA Value (%)**	**LA Value (%) Recommended by ICE Specification (1987) for GCs**	**LA Value (%) Recommended for GCs** (Serridge and Slocombe, 2012)
Materials tested within this research	Granite	18	<50	< 30 - 40
	Container Glass	42		
	APCr	49		
Other AAs, as measured by: *Arulrajah et al, 2014*[a] Ooi *et al.*, 2008[b] Tota-maharaj et al., 2017[c]	Waste Rock	21[a]		
	Crushed Brick	36[a]		
	Fine recycled glass	25[a]		
	Medium recycled glass	25[a], 27-33[b]		
	Recycled Concrete Aggregate	28[a]		
	APCr	39[c]		
	Typical Quarry material	<40[a]		

4.1.8 Flakiness Index

The Flakiness Index is a measure of the quantity of flat and elongated particles within a sample and is a property that can be largely influenced by the processing of a material (Arulrajah *et al.*, 2018). It is important to note that the flakiness index is only (currently considered) relevant for particle sizes between 6.3 and 63 mm (BS 812: Part 105.1: 1989). There doesn't appear to be any published guidance on whether there is an allowable percentage of flaky particles, it can only be assumed that the higher the percentage of flaky particles the greater the potential negative impact on performance. Guidance from the BRE (2000) indicates flakiness is undesirable for use within granular columns due to issues with particle packing and the potential for particle breakage. In addition, flakiness can affect ACV and AIV, causing variations of between 30 – 60 % (BRE, 2000), although Tam et al. (2013) found that if aggregates were flat and elongated (i.e., flaky) then the ACV was actually lower (which is in contrast to what has previously been reported).

The flakiness index of container glass and window is 94 % and 100 % respectively (Table 4.7). This due to the thickness (i.e., the smallest dimension) of the particles being 2-3 mm for container glass, with the occasional 4 mm thick particle, and approximately 4.5 mm for the window glass). In contrast to the glass, APCr does not include any flaky particles, which would be deemed a very positive attribute (when considering the guidance above). However, when the other index test results (such as ACV and AIV) are also considered this throws the suitability of APCr for adoption within GCs into doubt. This, again, calls in to question the suitability of the recommended index tests for GCs. The granite does include some flaky particles; however, the flakiness is more than 50 % lower than for the waste glass materials.

Table 4.7: Summary of flakiness tests

Material	**Flakiness (%) (Particles >6.3mm)**	**% of Flaky particles out of all particle sizes for an average column sample**
Granite	24	14
Window Glass	94	46
Container Glass	100	32
APCr	0	0

4.1.9 Small Direct Shear

The small shear box (60 x 60 mm) was used to determine the internal angle of shearing resistance of each of the aggregates investigated in this research. It is acknowledged that ideally a larger apparatus, considering the size fraction of the particles (typically the 60 x 60 mm shear box is used for sand size particles and the larger 305 x 305 mm for gravels up to 37.5 mm particles (Head, 2011), would have been used but unfortunately this was the only size available during the period of the laboratory work.

Attempts were made to utilise the triaxial apparatus but due to issues with creating a sample and the membrane becoming pierced no useful data was collected. The use of the large shear box has been recommended by various researchers (Steele, 2004; Chidiroglou *et al.*, 2008) to determine the angle of shearing resistance for AA. However, the test does not exactly model the experience of the particles within GCs (Amini, 2015) which is a clear limitation. In addition, the drawbacks of the direct shear test are well known (i.e., forcing a sample to shear along one plane that is not necessarily the weakest, the deformation that can be applied is limited by the length of travel of the apparatus, the area of contact between the two halves of material decreases as the test progresses (Head, 2011)). These limitations together with the incorrect size of apparatus being used mean that these results are indicative only. It is noted that the use of a large 305 x 305 mm shear box would not have reduced the inherent limitations of the shear box test, however, the potential impact of the small size of the equipment relative to the particle sizes would have been diminished (Alias et al., 2014). However, the values obtained at least provide a relative indication of angle of shearing resistance. Three values of normal stress were applied to the samples, 50, 100 and 200 kPa, each of the aggregates and three repeats of each set were conducted for each material. The results are presented in the figures below.

The granite achieved the greatest value of internal friction which is unsurprising given the angularity of the particles and the fact that it is a primary aggregate. Interestingly the APCr achieved an average ϕ' value of 35 degrees. The APCr particles are spherical and so a lower value of ϕ' would be expected, however, the surface of the particles is rough and so this is likely to generate friction between the particles enabling resistance to failure (Table 4.8).

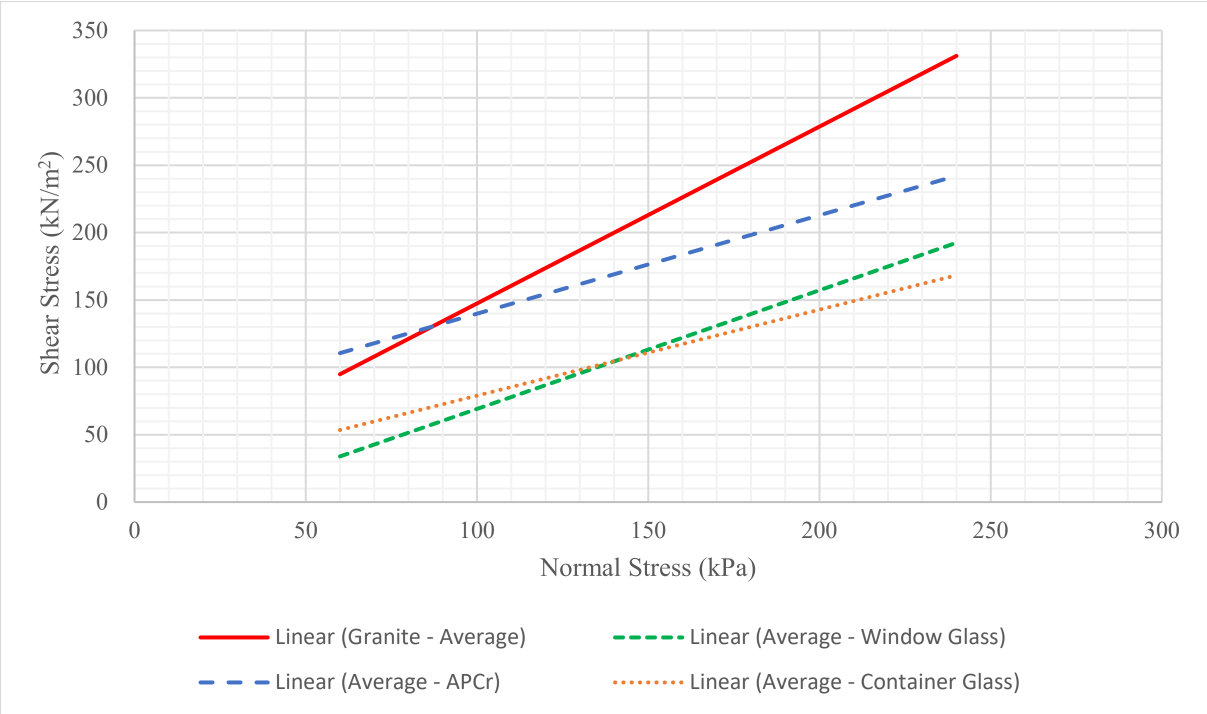


Figure 4-25: Average results of the direct shear test

Table 4.8: Summary of average angle of internal values determined from the results shown in Fig. 4.24

Material	**Average value of Internal Angles of Shearing Resistance (ϕ') (°)**	**Variation (+/- °)** ***(Observed across the three tests conducted for each material)***
Granite	51	3
Window Glass	40	3
APCr	35	7
Container Glass	31	4

Other researchers have investigated glass (container) and granite and the internal angles of shearing resistance (ϕ') are reported in tables 4.9 and 4.10. No data was available for comparison for the APCr and window. There is significant variation of the values reported, and some values seem higher than would be expected based of the experiences gained during this study (glass having an ϕ' of 63° seems questionable and it is not clear under what conditions these values were derived).

Table 4.9: Container Glass: published values of ϕ'

Container Glass		Reference
Direct Shear	**Triaxial Shear (CD)**	
41		(Ooi *et al.*, 2008)
40 - 47	35 -50	(M. Disfani, Arulrajah, Ali, *et al.*, 2011)
47- 63	47- 48	(Wartman et al., 2004)

Table 4.10: Granite: published values of ϕ'

Granite	Reference
Direct Shear	
47	(Amini, 2015),(Ooi *et al.*, 2008)

The results of the direct shear tests undertaken for both types of glass during this research indicate an angle of internal friction that is lower than the granite which is to be expected given the lower angularity and surface roughness of the particles. The window glass exhibited superior performance when compared to the container glass and achieved an average angle of internal friction that was higher (by 9 °). This is an interesting finding given the similarity in the two materials in terms of particle smoothness. However, perhaps this difference can be explained by the difference in shape (due to the different processing techniques). The greater thickness of the window glass particles may have also been a factor by enabling a stronger interlock between them and so achieving greater resistance to failure. The results of direct shear conducted for container glass indicate an able of friction that is lower than values presented in the literature (Table 4.10). This is likely to be due to differences in source particle such as angularity, shape, PSD and possibly cleanliness. In the work of Disfani et al. (2011) it was found that with an increase in normal stress in the direct shear test the internal angle of friction was reduced. However, the opposite was found in the work of Wartman et al. (2004), highlighting the variability of this material that is most likely to due to variations in the source material and particle size and shape. Disfani et al. (2011) observed that value of internal angle of friction obtained when using the triaxial apparatus is lower; these values are closer to the results of the direct shear tests conducted as part of this research. Given the known drawbacks of the direct shear test it is prudent to adopt a more conservative value of angle of shearing resistance.

4.1.9.1 Variation in φ' Values Measured During this Research

The value of ϕ' for the granite varied between 50 and 53 °. This was a very small variation when compared to the container glass and APCr (is to be expected given that the processing method and source material are constants). The value of ϕ' for the APCr varied between 28 to 35 ° and for the container glass it varied between 28 and 32 °. This variation in angle of internal friction for the recycled materials is not surprising given the variation in material properties such as particle shape/size and PSD. The variation observed for the container glass is also smaller than APCr and it is interesting that the variation in the results for the window glass is the same as for granite. The variation in the value of angle of internal friction, particularly in the case of the APCr, could be significant considering that a reduction in 10 ° can potentially lead to a 50 % reduction in the overall bearing capacity of a granular column (Mckelvey *et al.*, 2002). This means that based on the observed variation of 7 ° the capacity of the APCr columns could vary by 35 %. It is recommended that for use within granular columns materials should have an internal angle of friction value between 40 and 45 ° (Serridge, 2006). Based on this criterion only the granite and window glass, which only just met this criterion, would be deemed suitable.

4.2 Summary

The LA, AIV and ACV tests are all designed to give an indication of particle hardness/abrasion resistance. Harness is an important property in the context of GCs as a lower hardness would tend to lead to breakage of particles (particularly during installation: Serridge, 2005) and a potential reduction in angle of shearing resistance, reducing bearing capacity and settlement reduction (Priebe, 1995). The further implications of particle breakage have been identified as reduction in drainage and consolidation rate of the surrounding ground, which could negatively affect column performance due to the potential for stiffening of the surrounding ground being reduced. It is interesting that of the three index tests, the ACV test (which comprises the application of 400 kN in equal increments over a 10 minute period) appears to model the experience of particles within a granular column the most closely (it is not an exact model due to the presence of a rigid boundary and speed of load application) and this where the AAs perform most poorly.

When considering the granite (primary aggregate) the index tests indicate that this would be suitable material for use within GCs which is unsurprising given that it is a primary aggregate. If primary aggregates were not finite (and valuable resources) then the granite would be recommended in preference to the alternatives (as is often the case with current practice). However, primary aggregates are finite and valuable and better/more selective use of these materials is clearly desirable if these resources are to be used sustainably (and the volume of wastes that are landfilled is limited by utilising them productively in other applications). It is clear from the findings herein that the alternatives considered could be suitable (although design changes may be required to minimise breakage during handling, etc.); the obvious exception to this would be the APCr which (in Phase 1 testing at least) did not perform well (although as demonstrated in Phase 2 testing (the modelled column tests) APCr performs very well; arguably as well if not better than the granite (which was very surprising). This contrast in behaviour observed during index tests and model columns highlights the insufficiency of index tests to determine suitability of materials in applications such as granular columns.

The window glass appears to outperform the container glass in Phase 1 testing and this is attributed to the flakiness of the two materials (with the container glass having thinner walls than the window glass) and at this stage of the study the window glass would appear more suitable than the container glass. However, both materials have issues (with ACV and AIV) that would need consideration before use. The APCr appeared to have a reasonable internal angle of shearing resistance but performed relatively poorly in LA, AIV and compaction tests and failed the ACV criteria; Figure 4.25 illustrates three samples of APCr (before testing, after LA testing and after AIV) and it is apparent that after both tests the APCr has experienced significant changes. The LA testing has created a significant change in PSD, with the larger particles ground-down and the AIV indicates that the particles have crushed and merged together for form a compressed, and interlocking structure (that required chipping with a tool to loosen it before the material was vacuumed from the mould). Taking the ACV, AIV, LA and compaction performance into consideration as a whole and the APCr appeared unusable. As noted previously, at the end of Phase 1 testing it was questioned if the APCr should be used in Phase 2; it was and the outcomes of this phase of testing justify the decision to continue with it.

Figure 4-26: Change in APCr with testing: (a) Sample of APCr before testing (previously used in Figure 5-4) (b) sample of APCr after LA testing (reproduced in Figure 6.37) and (c) sample of APCr after AIV testing (reproduced in Figure 6.33)

Questions must be raised regarding the performance criteria used to determine the suitability of aggregates in granular columns. AIV, LA and compaction in a stiff metal mould, do not replicate conditions faced *in situ*. Aspects such as flakiness must be considered with caution; Chidiroglou et al. (2009), which investigated the crushing of concrete and bricks under shear using the direct shear box apparatus, found that particle shape (i.e., elongation/flakiness) was not the dominant factor in particle breakage and it was postulated that individual particle strength had greater influence on crushing potential. Furthermore, Chidiroglou et al. (2009) noted that the difficulty in characterising recycled/secondary aggregates (particularly given the variability in their composition) utilising the existing index tests remains a challenge to the increased adoption of these types of materials. It is evident that as index tests they are satisfactory but as a method of determining suitability for applications such as granular columns, they are lacking. Geotechnical tests which more closely model the in-situ experience of materials would be far more useful. This conclusion has been drawn by many other researchers (Schouenborg, 2005; Amini, 2015; Shahverdi, 2020) and is a challenge that needs to be addressed to enable the increased utilisation of AA. Outcomes of Phase 2 testing will reinforce this message (Chapter 6.0).

5 Results

This chapter presents the results of the various bench-scale column load tests and associated data. No extensive analysis of these results will be included in this chapter, this will be presented within the discussion, Chapter 6.0.

5.1 Constant Rate of Strain Tests

5.1.1 Particle Size Distributions

PSD tests were conducted before and after the load tests in order to assess the extent of particle breakage that occurred during loading.

Figure 5.1 shows PSD of the materials prior to the column installation process. This test was conducted to assess the variation in PSD across different samples of the aggregates. Figure 5.1 (b) reflects the wide variation in PSD between samples for APCr in contrast to the minimal differences between samples of window glass shown in Figure 5.1(d).

The PSD of a material impacts the way in which particles pack together and it follows that variation in original PSD (i.e., prior to column installation or loading) could affect overall column behaviour under load. Based on Figure 5.1 it could be expected that window glass columns perform more uniformly than those formed of APCr.

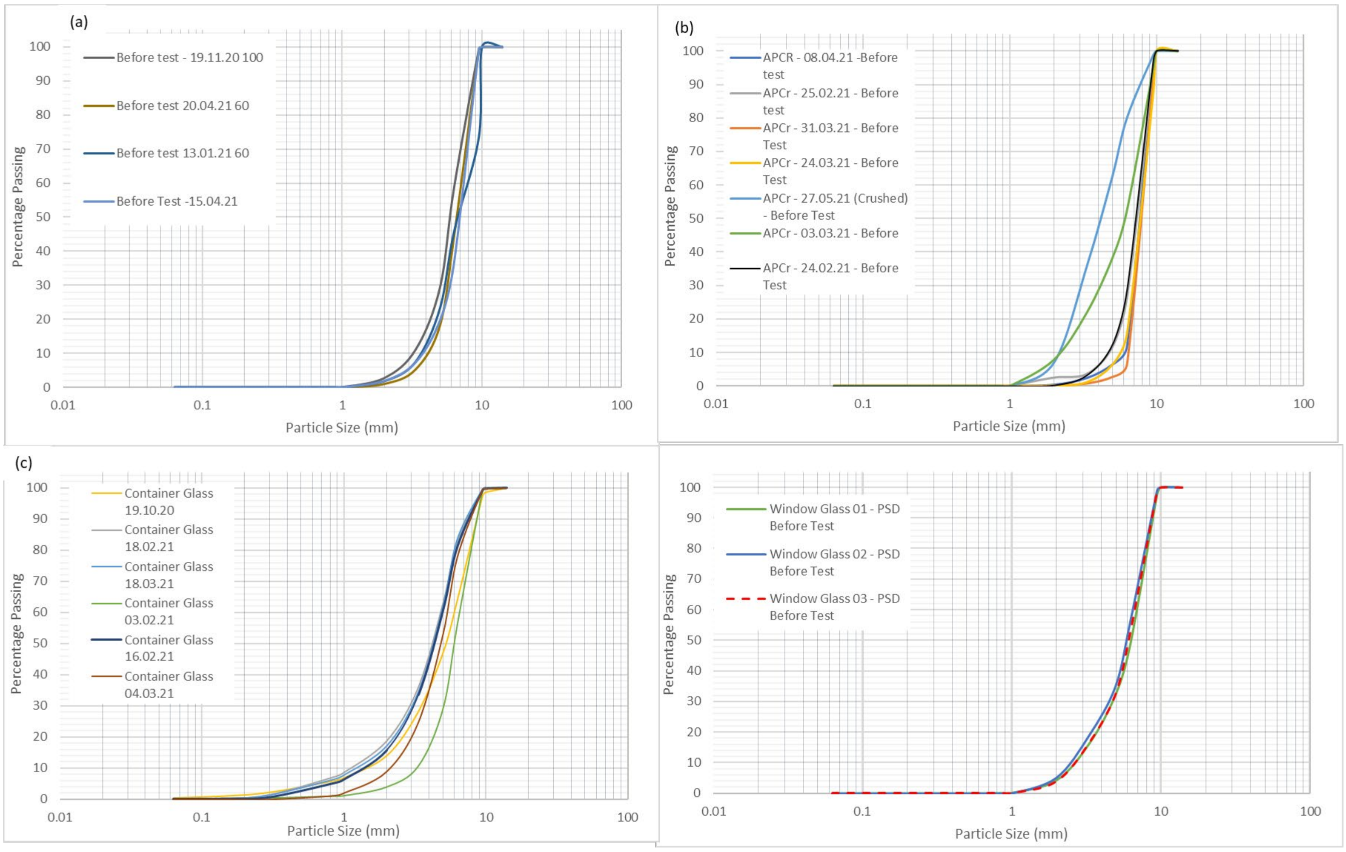


Figure 5-1:PSD Before Installation and Load Testing (a) Granite, (b) APCr, (c) Container Glass, (d) Window Glass

Figures 5.2, 5.3, 5.4 and 5.5 shows PSD before and after column loading. These tests were conducted to assess the extent of particle breakage that occurred due to column loading. One of the concerns around the adoption of AA withing GCs is the extent of particle breakage and the impact this may have on column capacity under load.

Figure 5.3, showing the results for APCr, shows that in some cases, extensive breakage occurs during load testing whereas Figures 5.2 and 5.4 showing results for granite and window glass, indicate minimal breakage. This may be expected for the PA, granite, but is more surprising in the case of the window glass as being a brittle material, particle breakage could be expected. The container glass results (Figure 5.5) indicate that some particle breakage does occur under load, however this does vary as indicated by 5.5(b) where minimal breakage can be observed, whereas 5.5(e) reflects more extensive breakage. These results highlight the variable nature of the material. It should also be acknowledged that due to the difficultly in removing the full sample of aggregate post-test the results may be skewed.

The relationship between column performance and particle breakage will be explored further in Chapter 6.0.

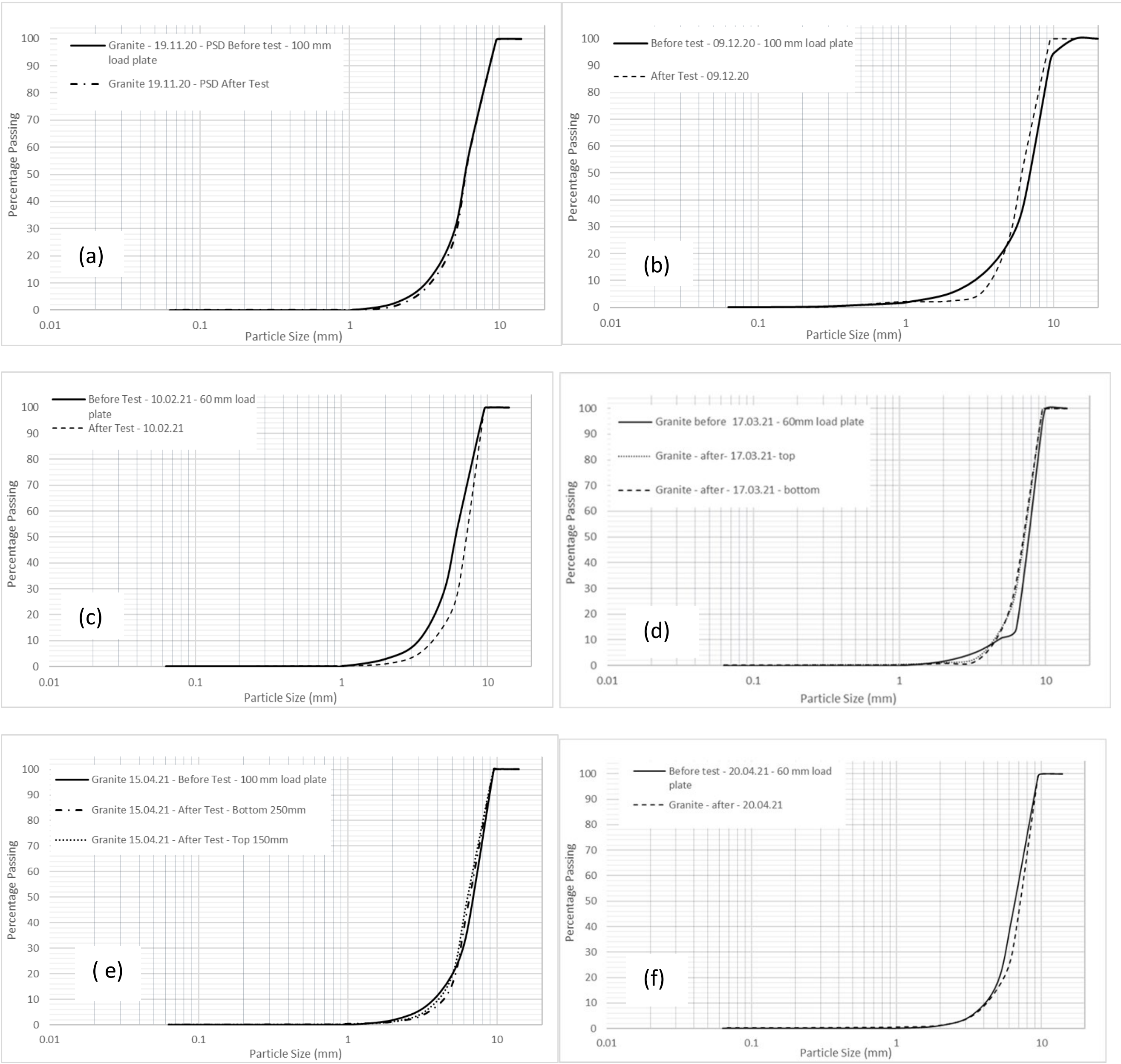


Figure 5-2: PSDs for granite columns measured before and after load testing

(a) Column tested 09.11.20 using 100 mm load plate, (b) Column tested 09.12.20 using 100 mm load plate, (c) Column tested 10.02.21 using 60 mm load plate, (d) Column tested 17.03.21 using 60 mm load plate, (e) Column tested 15.04.21 using 100 mm load plate, (f) Column tested 20.04.21 using

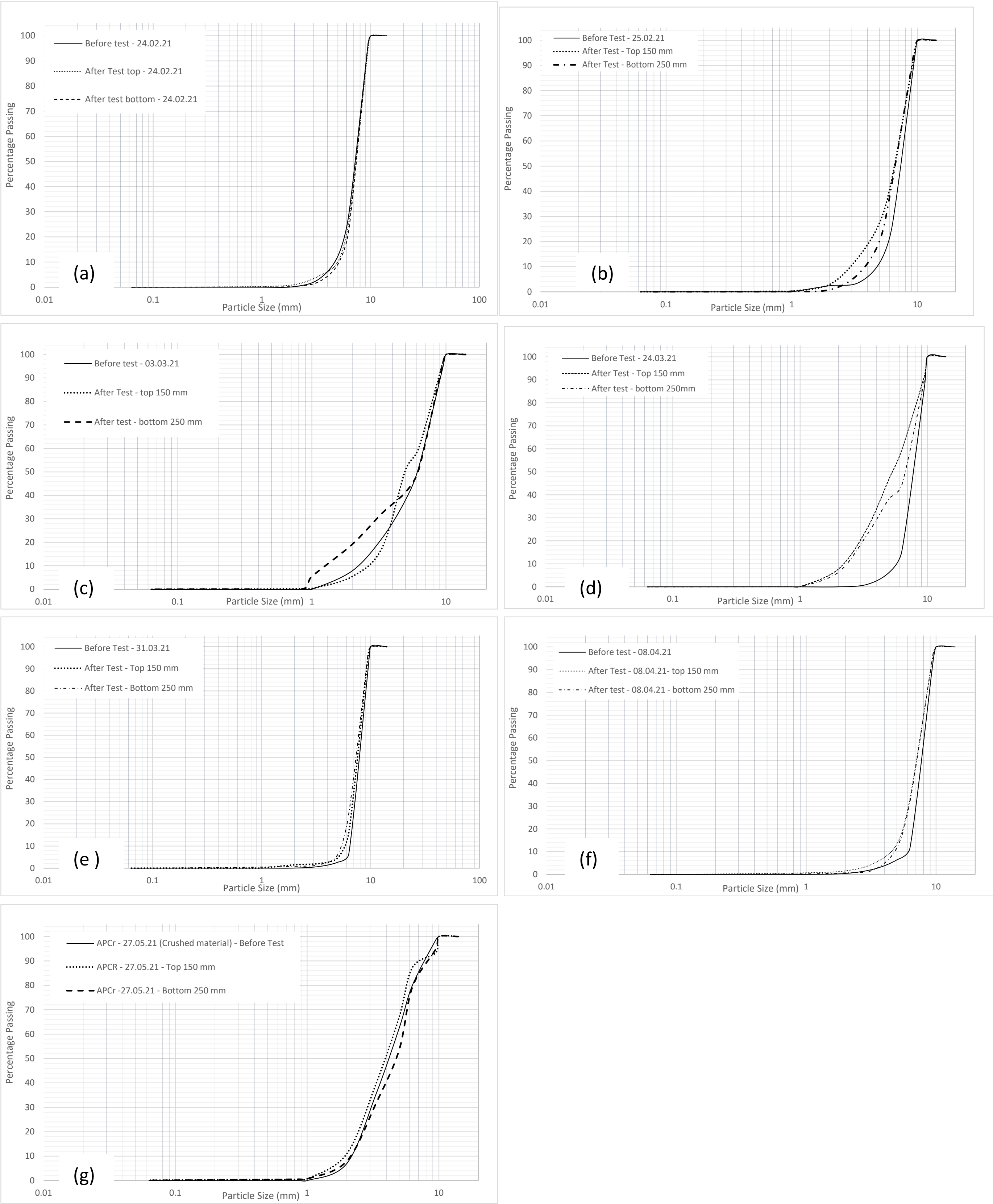
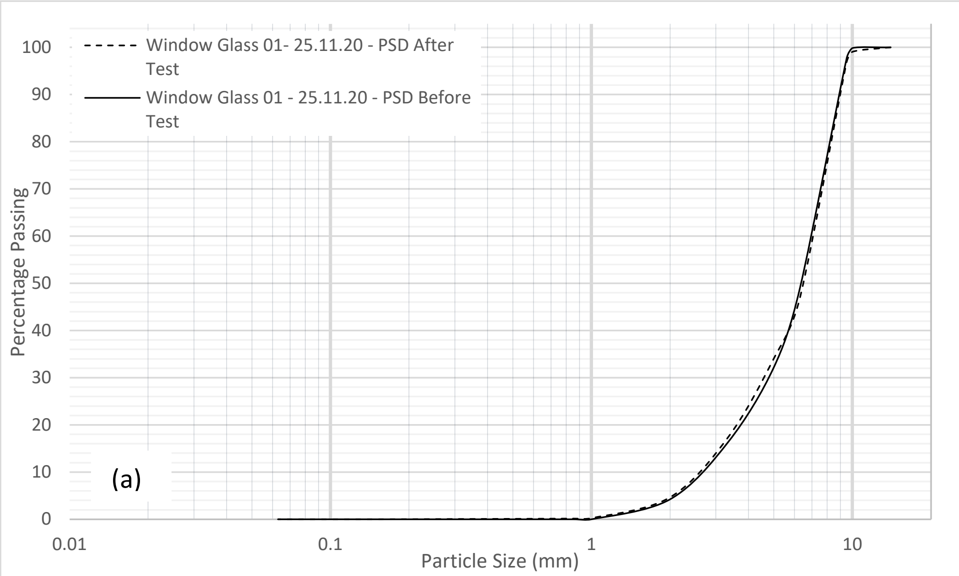
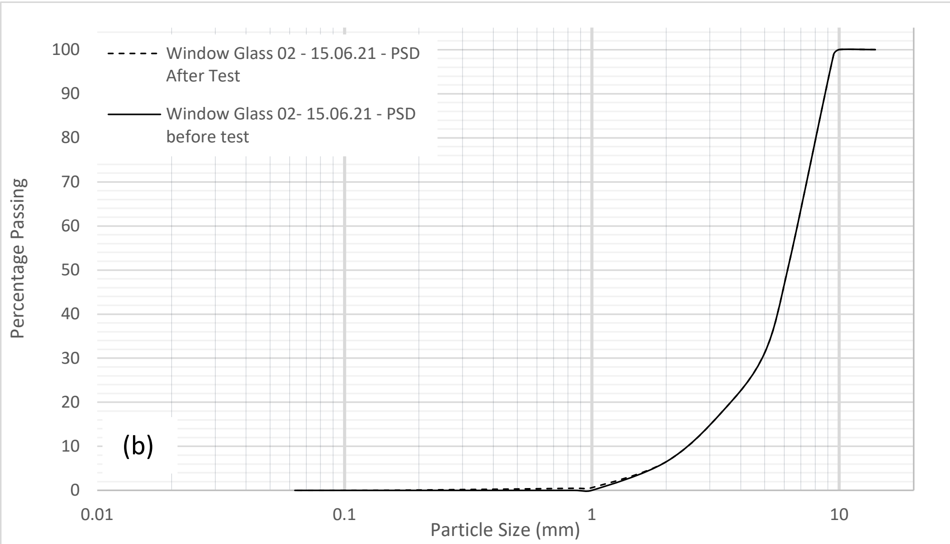


Figure 5-3: PSDs for APCr columns measured before and after load testing:

(a) Column tested 24.02.21 using 100 mm load plate, (b) Column tested 25.02.21 using 100 mm load plate, (c) Column tested 23.03.21 using 100 mm load plate, (d) Column tested 24.03.21 using 100 mm load plate, (e) Column tested 31.03.21 using 100 mm load plate, (f) Column tested 08.04.21 using 100 mm load plate, (g) Column tested 27.05.21 using crushed material and a 100 mm load plate





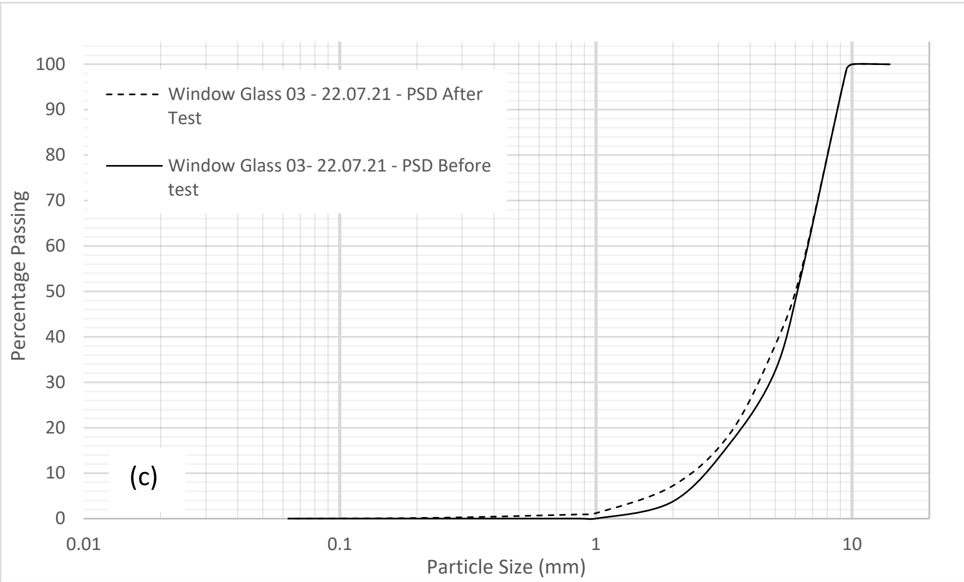


Figure 5-4: PSDs for window glass columns measured before and after load testing:

(a) Column tested 25.11.20 using 100 mm load plate, (b) Column tested 15.06.21 using 100 mm load plate, (c) Column tested 22.07.21 using 100 mm load plate

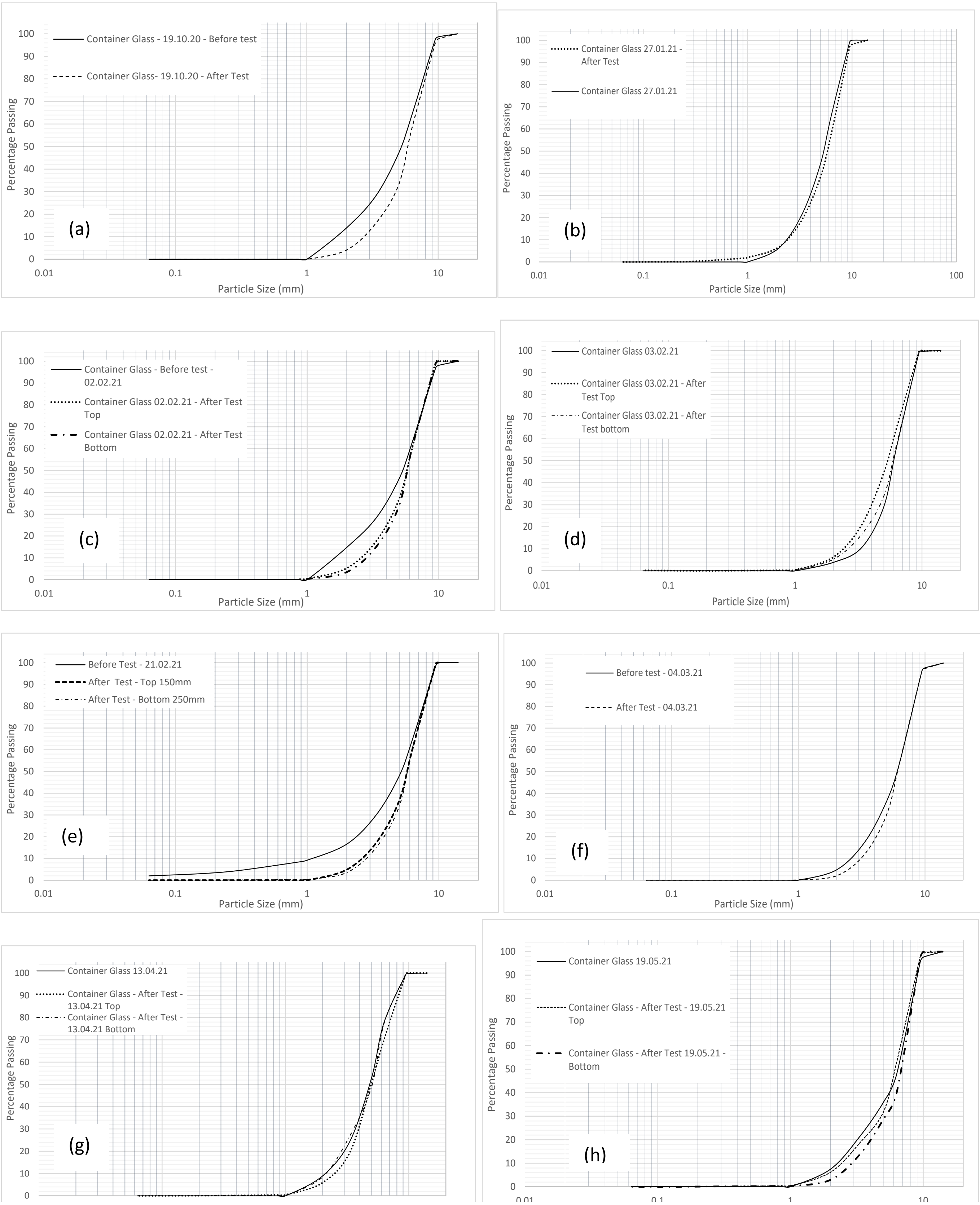


Figure 5-5: PSDs for container glass columns measured before and after load testing:

(a) Column tested 19.10.20 using 100 mm load plate, (b) Column tested 27.01.21 using 100 mm load plate, (c) Column tested 02.02.21 using 100 mm load plate, (d) Column tested 03.-02.21 using 100 mm load plate, (e) Column tested 21.02.21 using 100 mm load plate, (f) Column tested 04.03.21 using 100 mm load plate, (g) Column tested 13.04.21 using crushed material and a 100 mm load plate, (h) Column tested 19.05.21using crushed material and a 100 mm load plate

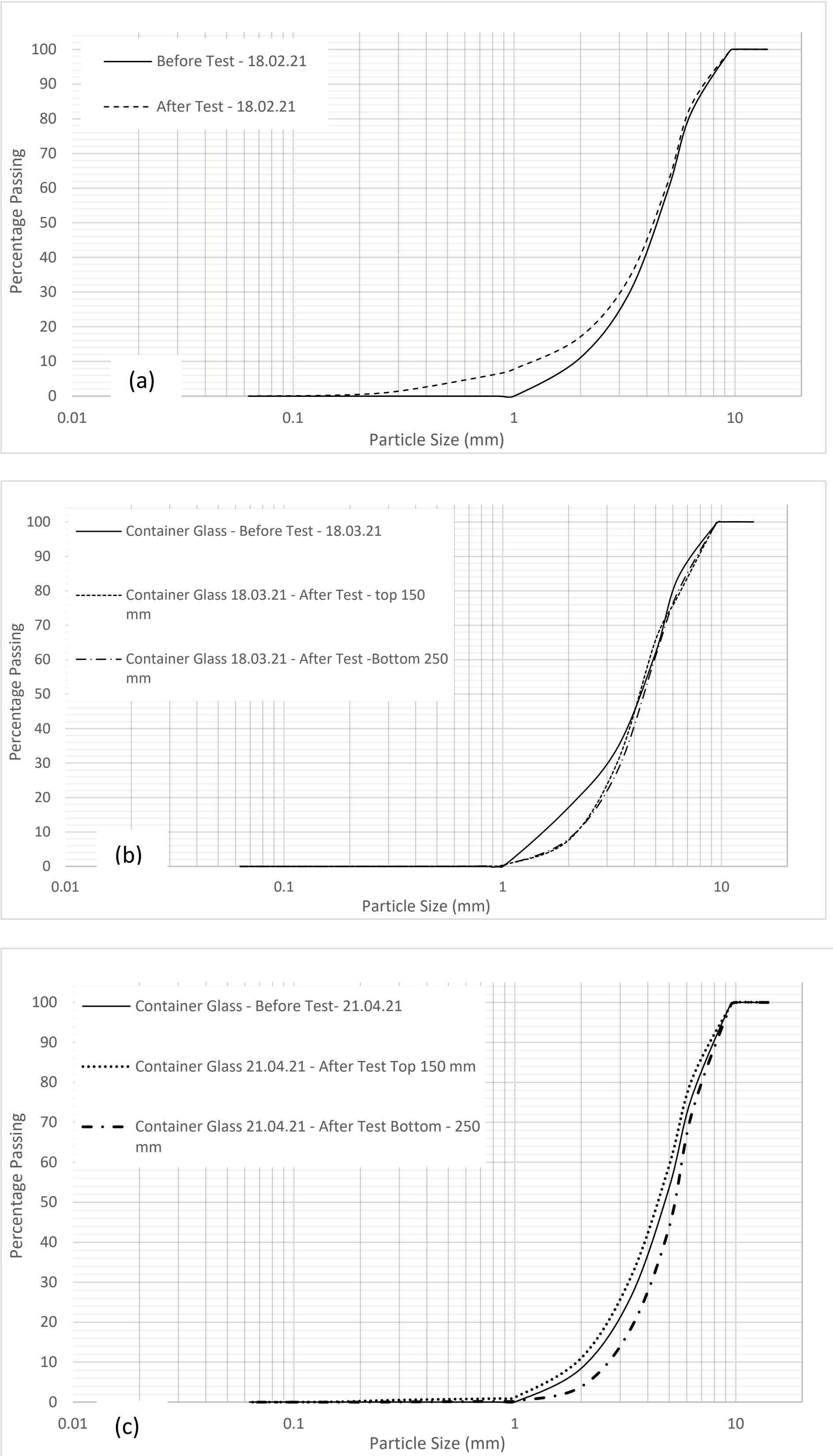


Figure 5-6: PSDs for container glass columns measured before and after load testing:

(a) Column tested 18.02.21 using 60 mm load plate, (b) Column tested 18.03.21 using 60 mm load plate, (c) Column tested 21.04.21 using 60 mm load plate

1.1.1 Stress-Settlement Plots

Stress-settlement plots for each test conducted for each material are presented in figures 5.7 to 5.10. For ease of comparison and to avoid plots becoming too cluttered the results for 60 mm and 100 mm load plates, for granite and container glass, have been presented on separate graphs.

In addition to the to the stress-settlement plots, supplementary data including column density, clay strength before and after the load tests and aggregate water content before after testing is presented in tables 5.1 to 5.4. This data has been included to enable any links or patterns between these properties and column behaviour to be identified, this is discussed in detail in Chapter 6.0, section 6.2.1.

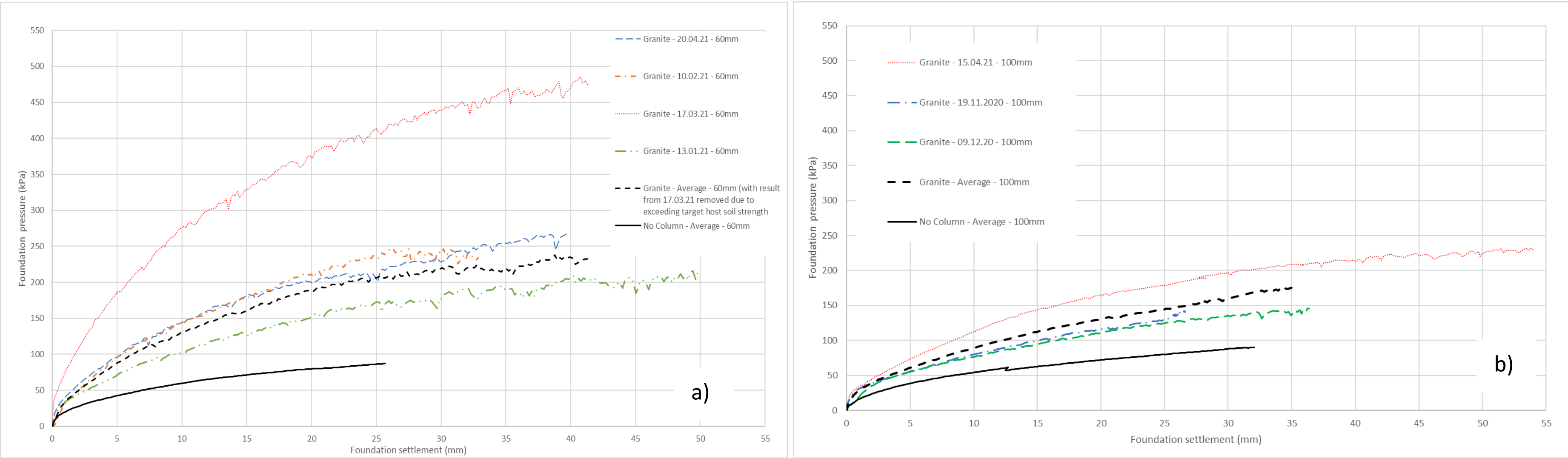


Figure 5-7: Granite – Stress-settlement for all tests - (a) 60 mm load plate, (b) 100 mm load plate

Table 5.1:Granite - Summary of pre and post-test results

Test date	Column Material	Load Plate size (mm)	Column Density (kg/m³)	Clay Water Content After Test (%)	Difference in Clay Water Content Before and After Load Test (%)	Clay Strength Before Load Test (kPa)	Clay Strength After Load Test (kPa)	*Increase in Clay Strength Post-test (kPa)*	*Average soil strength at 1.5D (kPa)*	Water Content of Aggregate Post-Test - Top 150mm (%)	Water Content of Aggregate Post-Test - Bottom 250mm (%)	Difference in Water Content Between Top and Bottom Aggregate (%)	Vertical Movement at Toe (mm)
19/11/2020	Granite	100	1621.79	39.90	0.48	17	18	*1.16*	*18*	4.7			
09/12/2020	Granite	100	1584.83		38.77		24	*23.64*					
15/04/2021	Granite	100	1365.55	41.50	0.32	21	25	*3.98*	*26*	1.1	1.5	0.4	0
10/02/2021	Granite	60	1702.25	38.13	1.79	24	24	*0.07*	*25*				
13/01/2021	Granite	60	1615.95	37.60	2.20		21		*21*				10
17/03/2021	Granite	60	1643.98	39.79	0.13	30	31	*0.65*	*31*	1.63	1.63	0.00	0
20/04/2021	Granite	60	1380.581	40.00	1.54	21	23	*1.47*	23	1.06	1.32	0.26	0

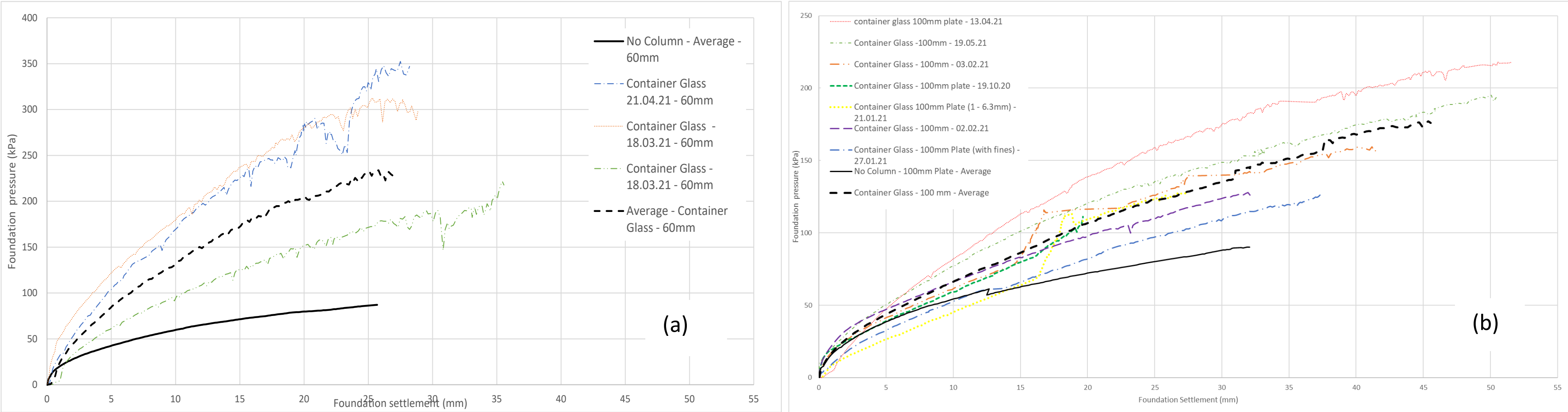


Figure 5-8: Container Glass – Stress-settlement for all tests - (a) 60 mm load plate, (b) 100 mm load plate

Table 5.2: Container Glass - Summary of pre and post-test results

Test date	Column Material	Load Plate size (mm)	Column Density (kg/m^3)	Clay Water Content After Test (%)	Difference in Clay Water Content Before and After Load Test (%)	Clay Strength Before Load Test (kPa)	Clay Strength After Load Test (kPa)	*Increase in Clay Strength Post-test (kPa)*	*Average soil strength at 1.5D (kPa)*	Water Content of Aggregate Post-Test - Top 150mm (%)	Water Content of Aggregate Post-Test - Bottom 250mm (%)	Difference in Water Content Between Top and Bottom Aggregate (%)	Vertical Movement at Toe (mm)
21/01/2021	Container glass *(Particles 1-6.3mm)*	100	1693.05	38.88	0.18	18	20	*2.34*		4.9			19
19/10/2020	Container Glass	100	1482.49	38.14	-0.27		22		*22*				5
27/01/2021	Container Glass *(with fines)*	100	1641.95	39.22	0.63	20	21	*1.18*	*21*	0.3			3
03/02/2021	Container Glass	100	1605.78	38.65	0.09	19	21	*2.53*	*22*	0.4	4.7	4.3	0
02/02/2021	Container Glass	100	1604.06	38.68	0.55	20	22	*2.07*	*23*	0.4	2.4	2.0	12
16/02/2021	Container Glass	60	1679.35	38.12	1.12	20	22	*1.78*	*23*	1.0	4.9	3.9	10
18/03/2021	Container Glass	60	1588.28	39.08	1.63	25	27	*1.76*	*28*	0.2	0.2	0.0	0
04/03/2021	Container Glass	100	1685.63	38.27	1.69	20	23	*2.98*	*22*	0.2	0.4	0.2	15
13/04/2021	Container Glass	100	1626.22	41.68	-0.85	22	22	*0.07*	*23*	0.1	0.3	0.2	5
18/02/2021	Container Glass *(prepped night before)*	60	1637.62	39.53	-0.02	19	22	*2.80*	*22*	1.0	5.1	4.1	0
21/04/2021	Container Glass	60	1561.487	40.40	-0.25	22	23	*0.89*	*24*	0.2	0.1	-0.1	0
19/05/2021	Container Glass	100	1482.55			21	23	*1.38*		0.1			0

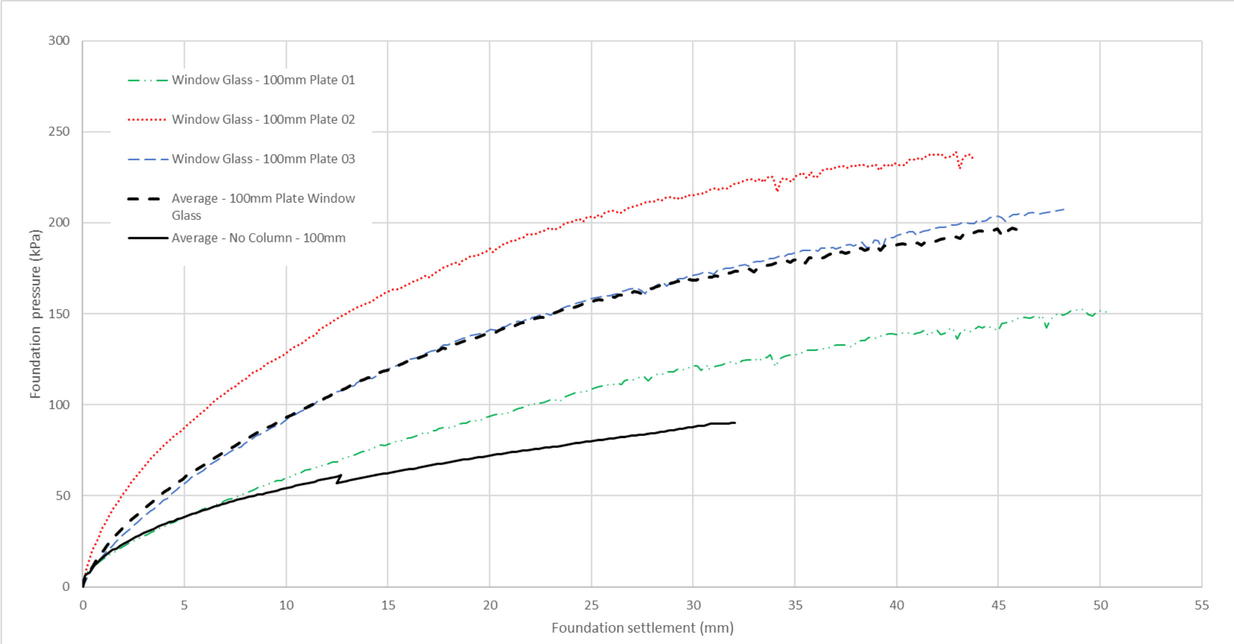


Figure 5-9: Window Glass – Stress-settlement for all tests (100 mm load plate)

Table 5.3: Window Glass - Summary of pre and post-test results

Test date	**Column Material**	**Load Plate size (mm)**	**Column Density (kg/m^3)**	**Clay Water Content After Test (%)**	**Difference in Clay Water Content Before and After Load Test (%)**	**Clay Strength Before Load Test (kPa)**	**Clay Strength After Load Test (kPa)**	***Increase in Clay Strength Post-test (kPa)***	***Average soil strength at 1.5D (kPa)***	**Water Content of Aggregate Post-Test - Top 150mm (%)**	**Water Content of Aggregate Post-Test - Bottom 250mm (%)**	**Difference in Water Content Between Top and Bottom Aggregate (%)**	**Vertical Movement at Toe (mm)**
25/11/2020	Window Glass	100	1640.53	39.55	-0.89	17	19	*1.56*	*18.00*	1.6			
15/06/2021	Window Glass	100	1448.86	41.98	-0.18		21	*20.60*	*21*				0
22/07/2021	Window Glass	100	1444.07	38.76	1.15	21	21	*-0.13*	*20*	0.2	5.8	5.6	0

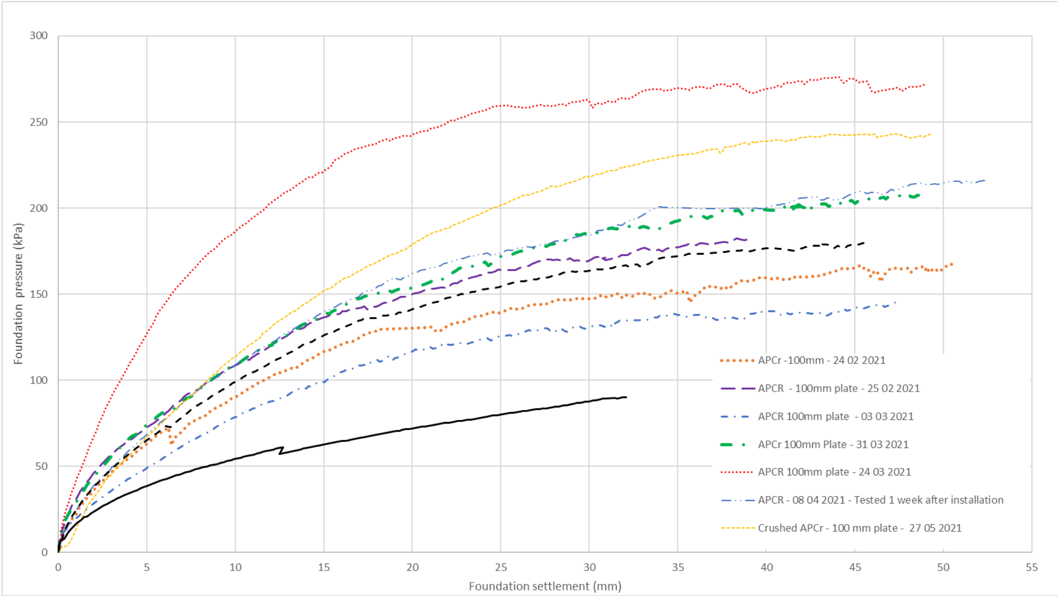


Figure 5-10: APCr – Stress-settlement for all tests (100 mm load plate)

Table 5.4: APCr - Summary of pre and post-test results

Test date	Column Material	Load Plate size (mm)	Column Density (kg/m^3)	Clay Water Content After Test (%)	Difference in Clay Water Content Before and After Load Test (%)	Clay Strength Before Load Test (kPa)	Clay Strength After Load Test (kPa)	*Increase in Clay Strength Post-test (kPa)*	*Average soil strength at 1.5D (kPa)*	Water Content of Aggregate Post-Test - Top 150mm (%)	Water Content of Aggregate Post-Test - Bottom 250mm (%)	Difference in Water Content Between Top and Bottom Aggregate (%)	Vertical Movement at Toe (mm)
24/02/2021	APCr	100	1337.87	37.75	1.93	22	26	*4.44*	*25*	11.7	14.7	2.9	0
25/02/2021	APCr	100	1454.76	38.78	-1.03	20	23	*2.85*	*23*	10.4	13.7	3.3	4
03/04/2021	APCr	100	1416.30	39.09	1.40	21	22	*0.98*	*22*	11.5	24.1	12.6	0
24/03/2021	APCr	100	1362.26	38.19	1.15	30	31	*0.99*	*33*	8.9	11.7	2.7	0
08/04/2021	APCr	100	1334.96	40.98	-0.53	21	24	*3.45*	*24*	17.0	17.5	0.5	0
31/03/2021	APCr	100	1341.68	40.42	0.31	22	24	*2.50*	*25*	11.7	13.4	1.7	3
27/05/2021	APCr (crushed)	100	1143.02	40.26	1.54	20	24	*3.91*	*25*	33.0	15.0	-18.1	0

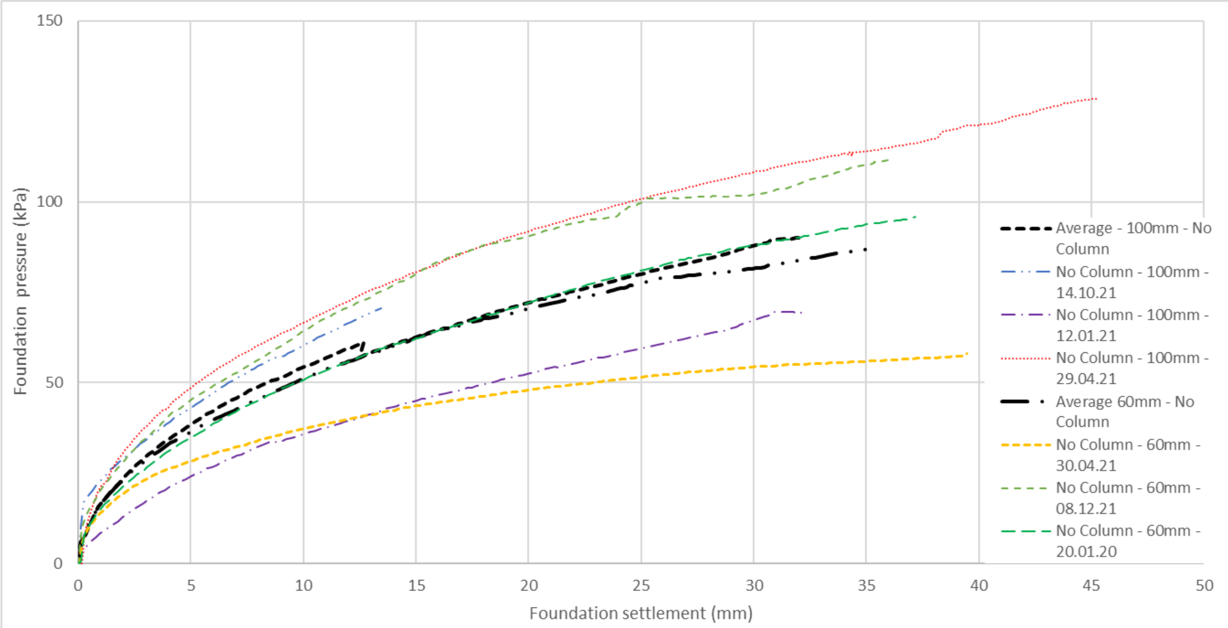


Figure 5-11: No Column test - Stress-settlement for all tests (100 and 60 mm load plate)

Table 5.5: No Column tests - Summary of pre and post-test results

Test date	Column Material	Load Plate size (mm)	Column Density (kg/m^3)	Clay Water Content After Test (%)	Difference in Clay Water Content Before and After Load Test (%)	Clay Strength Before Load Test (kPa)	Clay Strength After Load Test (kPa)	*Increase in Clay Strength Post-test (kPa)*	*Average soil strength at 1.5D (kPa)*	Water Content of Aggregate Post-Test - Top 150mm (%)	Water Content of Aggregate Post-Test - Bottom 250mm (%)	Difference in Water Content Between Top and Bottom Aggregate (%)	Vertical Movement at Toe (mm)
21/10/2020	No Column	100		39	-1.00		21		20				
14/10/2020	No Column	100		38	-0.04		22		20				
12/01/2021	No Column	100		40	0.45		20		19				
29/04/2021	No Column	100		40	2.14		23		23				
30/04/2021	No Column	60					21		21				
08/12/2021	No Column	60		42	-3.00		23		22				
20/01/2021	No Column	60		39	0.39		20		20				

5.2 Bulging Behaviour

Plaster of Paris casts of the post-load column shapes were created to assess the extent of bulging occurring during loading. Two measurements of the post-load diameter were taken using digital callipers (the average of the two is presented in figures 5.12 to 5.15) at 25 mm vertical intervals.

Generally, figures 5.12 to 5.15 show that bulging does occur and that this predominately affects the top third (i.e., closest to the clay surface) of the columns) which is consistent with the literature.

The extent of bulging is discussed in further detail in Chapter 6.0, section 6.2.8.

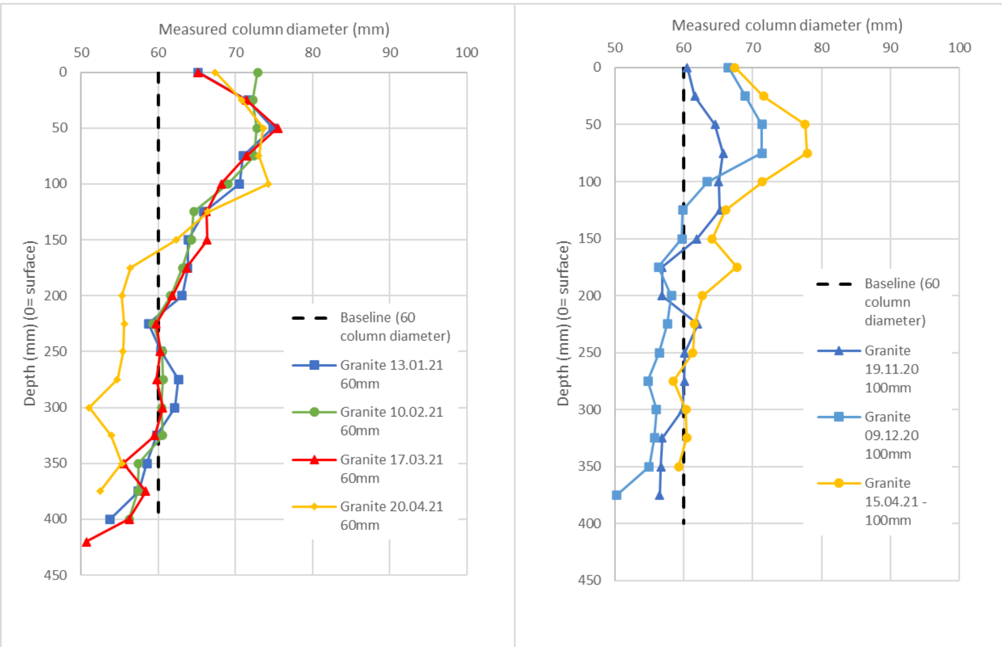


Figure 5-12: Granite – 60 mm load plate (left), 100 mm load plate (right)

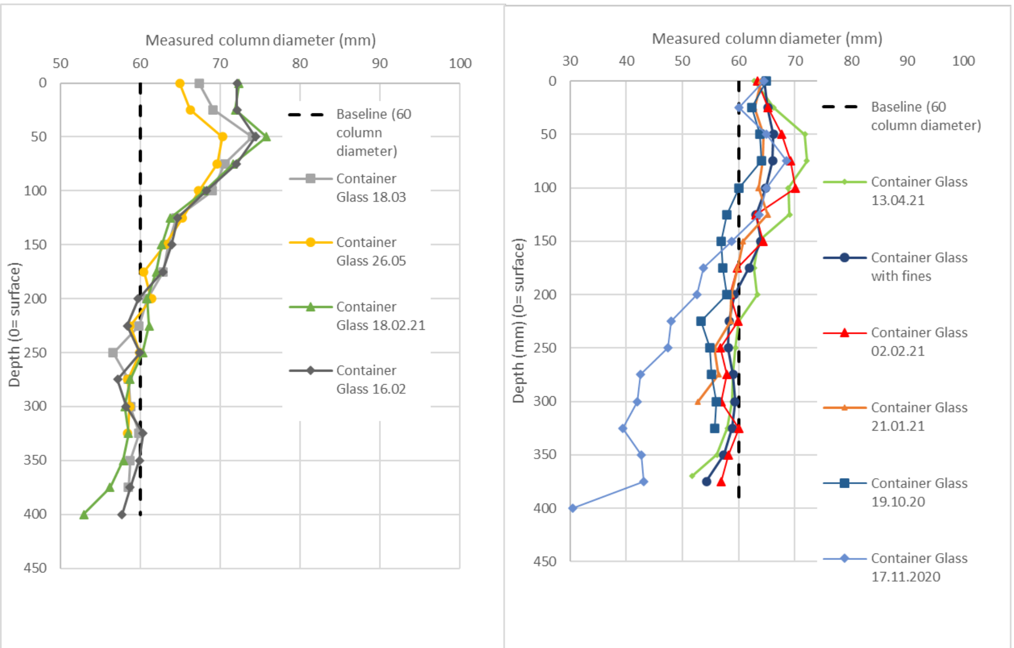


Figure 5-13: Container Glass 60 mm load plate (left), 100 mm load plate (right)

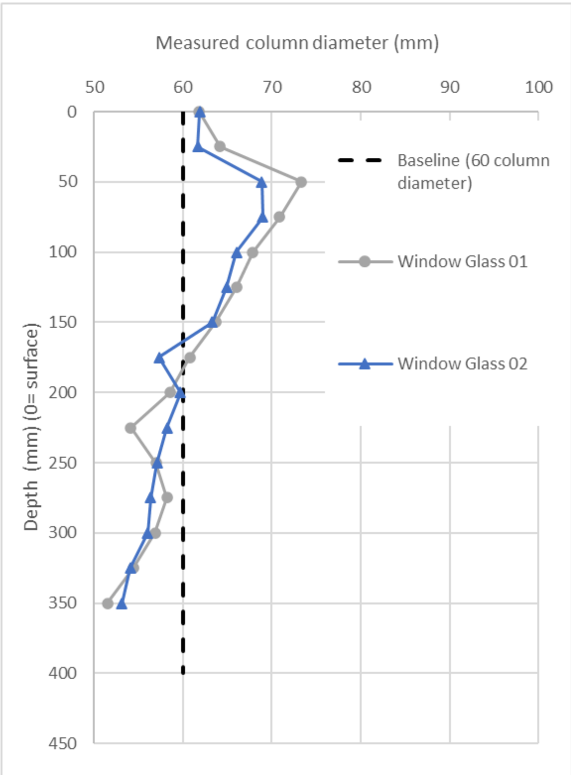


Figure 5-14: Window Glass

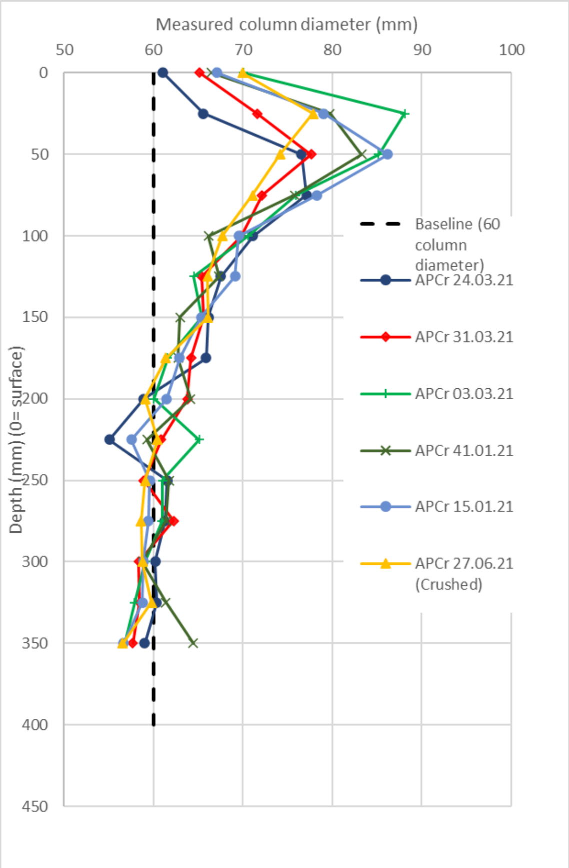


Figure 5-15: APCr

5.3 Static Loading

The static load tests were conducted after the CRS tests to enable comparison between the type loading and column behaviour to be drawn. The same test sample preparation methodology was used (i.e., container and host soil) and the same approach to column installation was adopted as for the CRS tests.

5.3.1 Particle Size Distributions

PSD tests were conducted before and after the load tests in order to assess the extent of particle breakage that occurred during loading. The results for APCr, shown in figure 5.18, are interesting as there appears to be minimal breakage, particularly for test APCr 02, which contrasts with the behaviour observed during the phase 1 testing.

These results are discussed in more detail in Chapter 6.0, section 6.3.

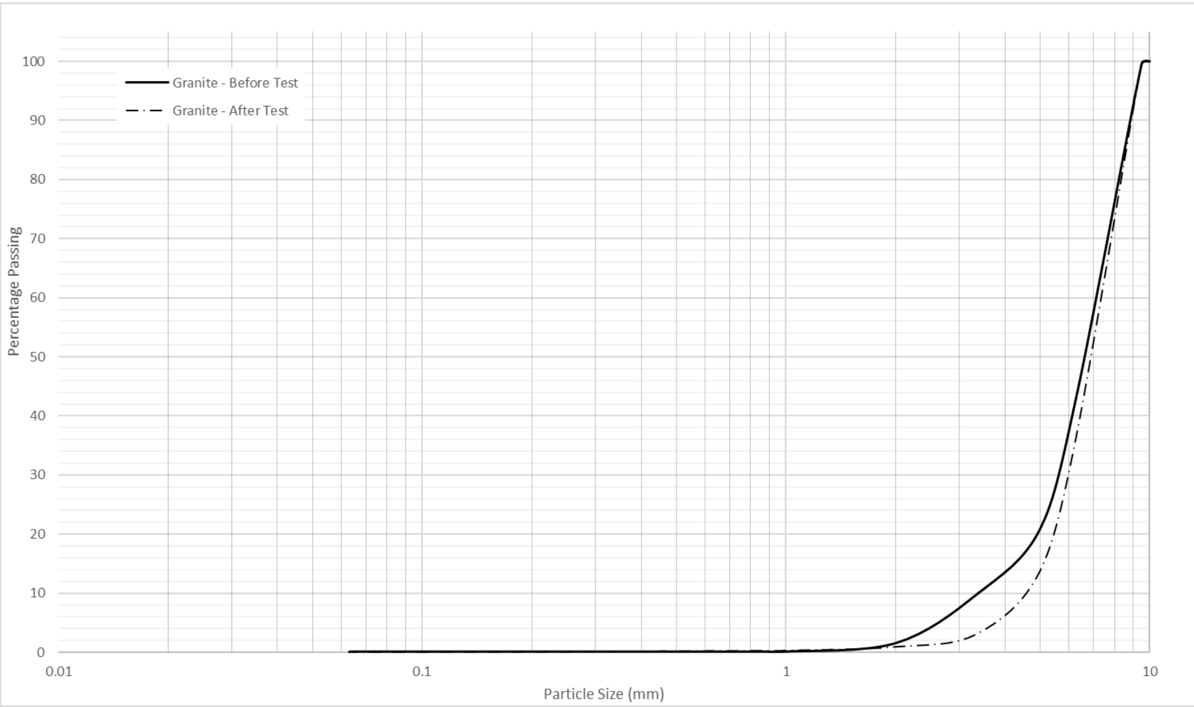


Figure 5-16:Granite PSD before and after test

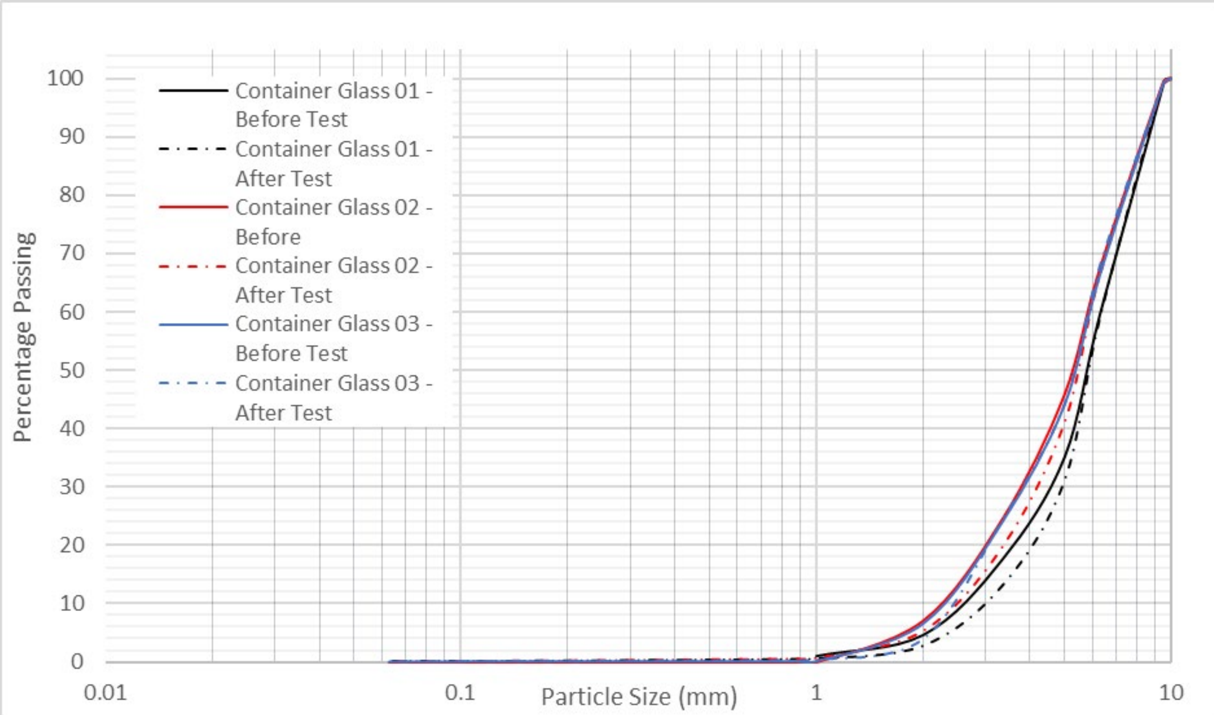


Figure 5-17: Container Glass PSD before and after test

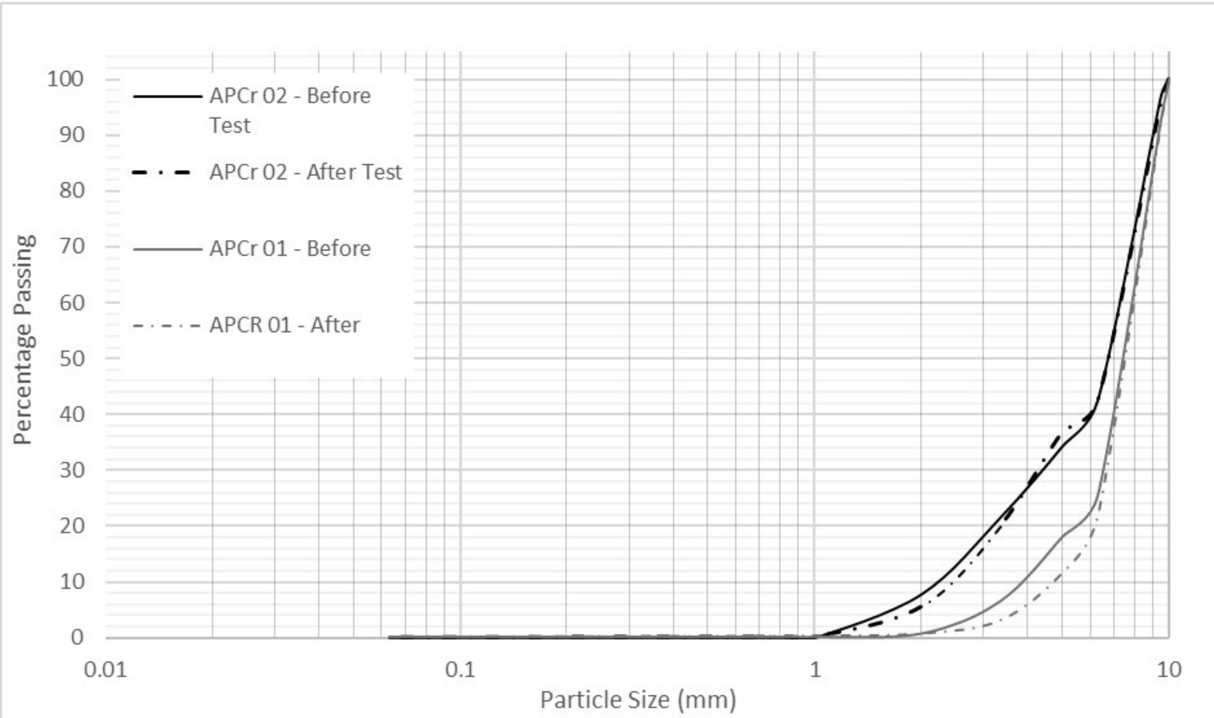


Figure 5-18: APCR PSD before and after test

5.3.2 Stress-Settlement Plots

The stress-settlement plots for each of the materials are presented in figures 5.19 to 5.21. Due to the nature of the test, i.e., load being applied in increments and not applied at a continuous rate as for the CRS tests, settlement against time has also been presented. Figures 5.19 and 5.21, which present the results for granite and APCr, clearly show how the installation of the columns significantly reduces settlement over time. Figure 5.20 shows that the installation of container glass columns does reduce settlement but to a lesser extent than seen for granite and APCr.

These results are discussed in more detail in Chapter 6.0, section 6.3.1.

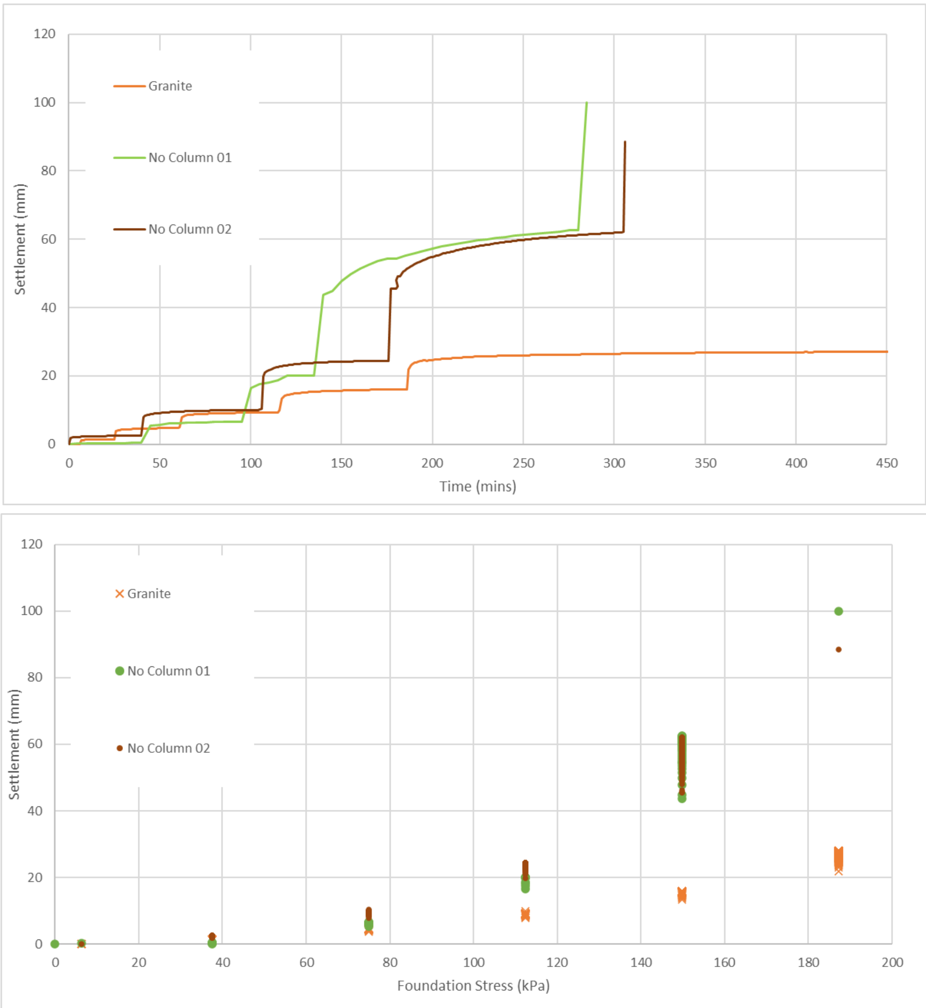
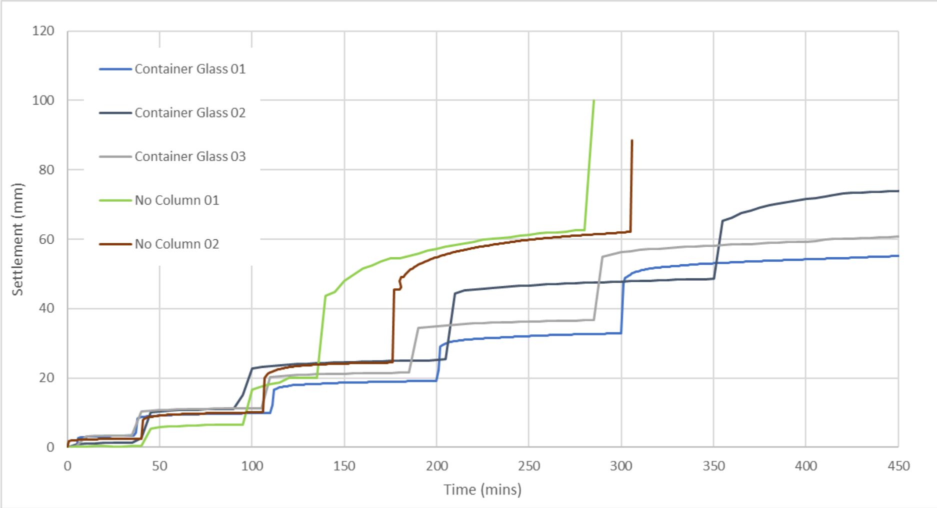


Figure 5-19: Granite – Settlement vs. time (top) and settlement vs. foundation stress (bottom)



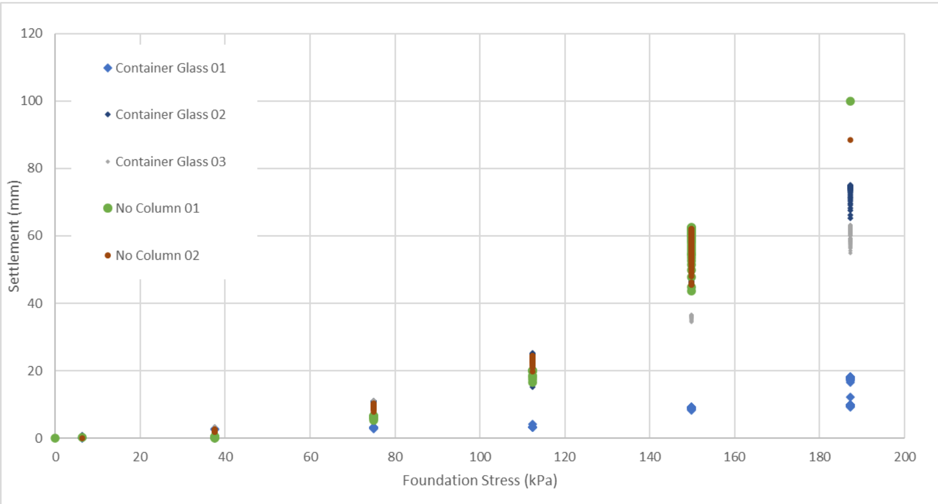


Figure 5-20: Container Glass – Settlement vs. time (Top) and settlement vs. foundation stress (Bottom)

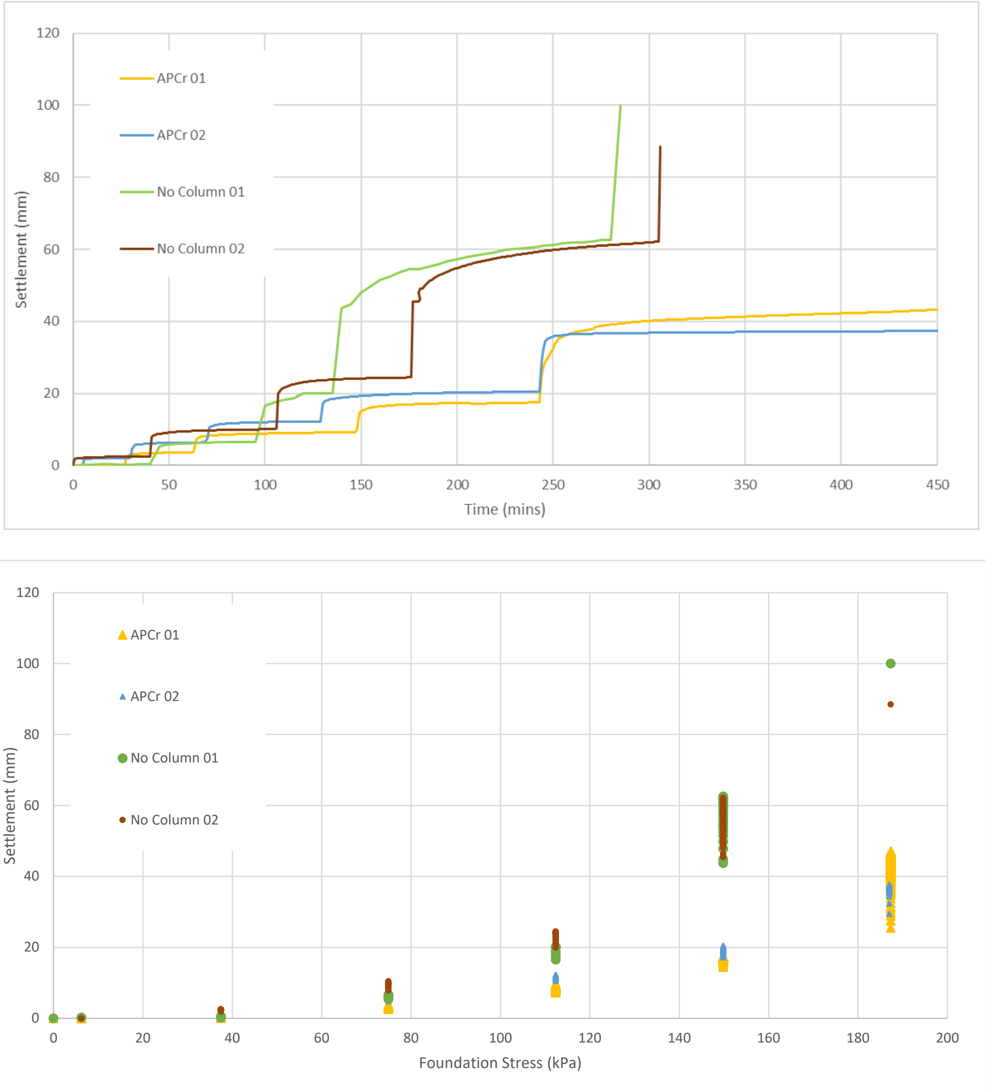


Figure 5-21: APCr – Settlement vs. time (left) and settlement vs. foundation stress (right)

5.3.3 Bulging Behaviour

The post column shape was able to be assessed by reviewing the Plaster of Paris models that were created. Two measurements of the post-load diameter were taken using digital callipers (the average of the two is presented in Figure 5.21) at 25 mm vertical intervals.

Figure 5.22 presents the post-load column diameters for all materials. The extent of bulging varies between tests but generally it can be observed that bulging occurs in the top (i.e., closest to the surface of the clay) third which consistent with the literature.

These results are discussed in further detail in Chapter 6.0, section 6.3.2.2.2.

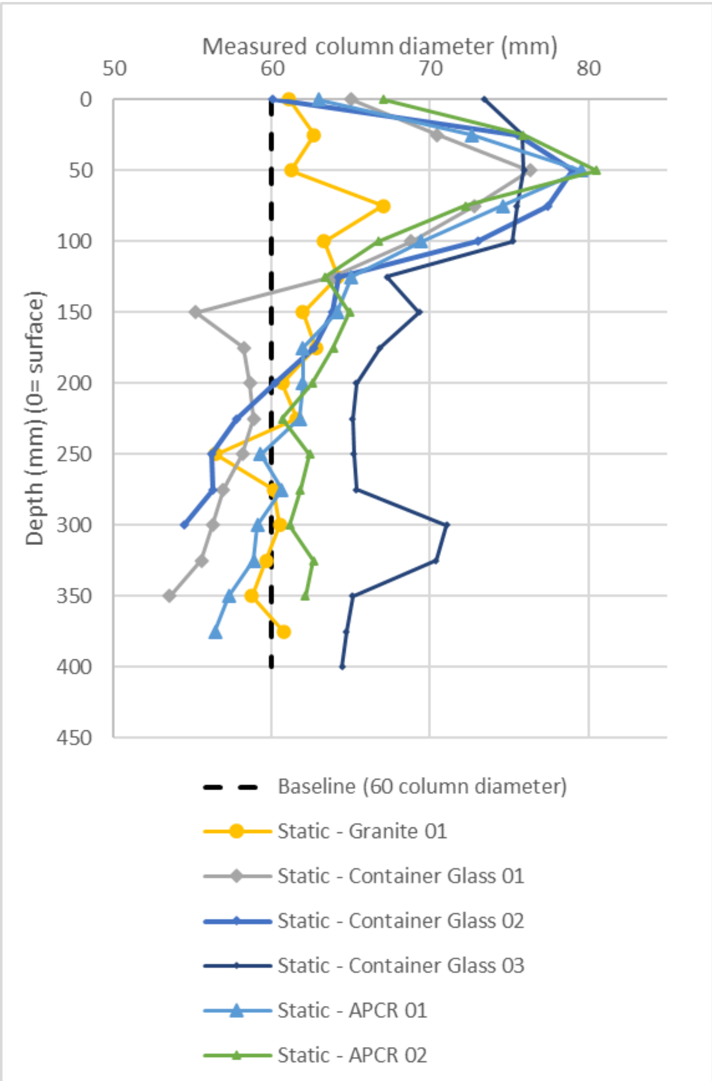


Figure 5-22: Summary of bulging behaviour observed from measuring casts of columns post-load.

(The measurements for granite may not be reliable due to problems creating the cast of this column)

5.3.4 Summary of Results

Figure 5.23 presents the stress-settlement results for all materials on the same plot for ease of comparison. As mentioned previously, the presence of granular columns of all materials clearly lead to reduction in settlement but the container glass columns achieve this to a lesser extent than the granite and APCr columns. See section 6.3.1 for further discussion on these results.

Table 5.6 presents the supplementary data to the column load tests to enable any patterns or links between column behaviour and properties such as initial clay strength, column density and water content of the aggregate to be determined. This data is discussed in more detail in Chapter 6.0, section 6.3.

Table 5.6: Static Load Tests - Summary of pre and post-test results for all materials

Test date	Test/Material	Plate size (mm)	Column Density (kg/m^3)	Clay Water Content of Core Before (%) **for no column tests this taken as the moisture content as mixed*	Clay Water Content After Test at 1.5D (%)	Clay Water Content after Test at 2.5D (%)	Clay Strength Before Test (kPa)	Average Increase in Clay Strength Post-Test (kPa)	Average Soil Strength at 1.5D after Test (kPa)	Average Soil Strength at 2.5D (kPa)	Water Content of Aggregate Post-Test Top 150mm (%)	Water Content of Aggregate Post-Test Bottom 250mm (%)	Combined Water Content of Top and Bottom Aggregate (%)	Movement at Toe (mm)
16/06/2021	No Column 1	100		41.80	42.65	42.84			25	23				
21/07/2021	No Column 2	100		42.23	40.21	40.54			25	23				
22/06/2021	Granite	100	1503.82	40.88			23	3	26	24	0.91	0.97	1.88	0
24/06/2021	Container Glass 1	100	1524.43	42.14			23	2	24	23	0.05	0.24	0.29	0
09/07/2021	Container Glass 2	100	1439.64	42.50	42.98	42.34	23	-2	21	21	0.09	0.23	0.31	0
15/07/2021	Container Glass 3	100	1445.57	42.83	42.59	40.58	20	2	23	21	0.22	0.72	0.94	5
06/07/2021	APCr 1	100	1435.91	42.54	40.26		23	3	26	23	10.62	12.33	22.95	0
15/07/2021	APCr 2	100	1479.31	43.08	41.67	42.10	21	2	24	23	16.43	15.44	31.88	0

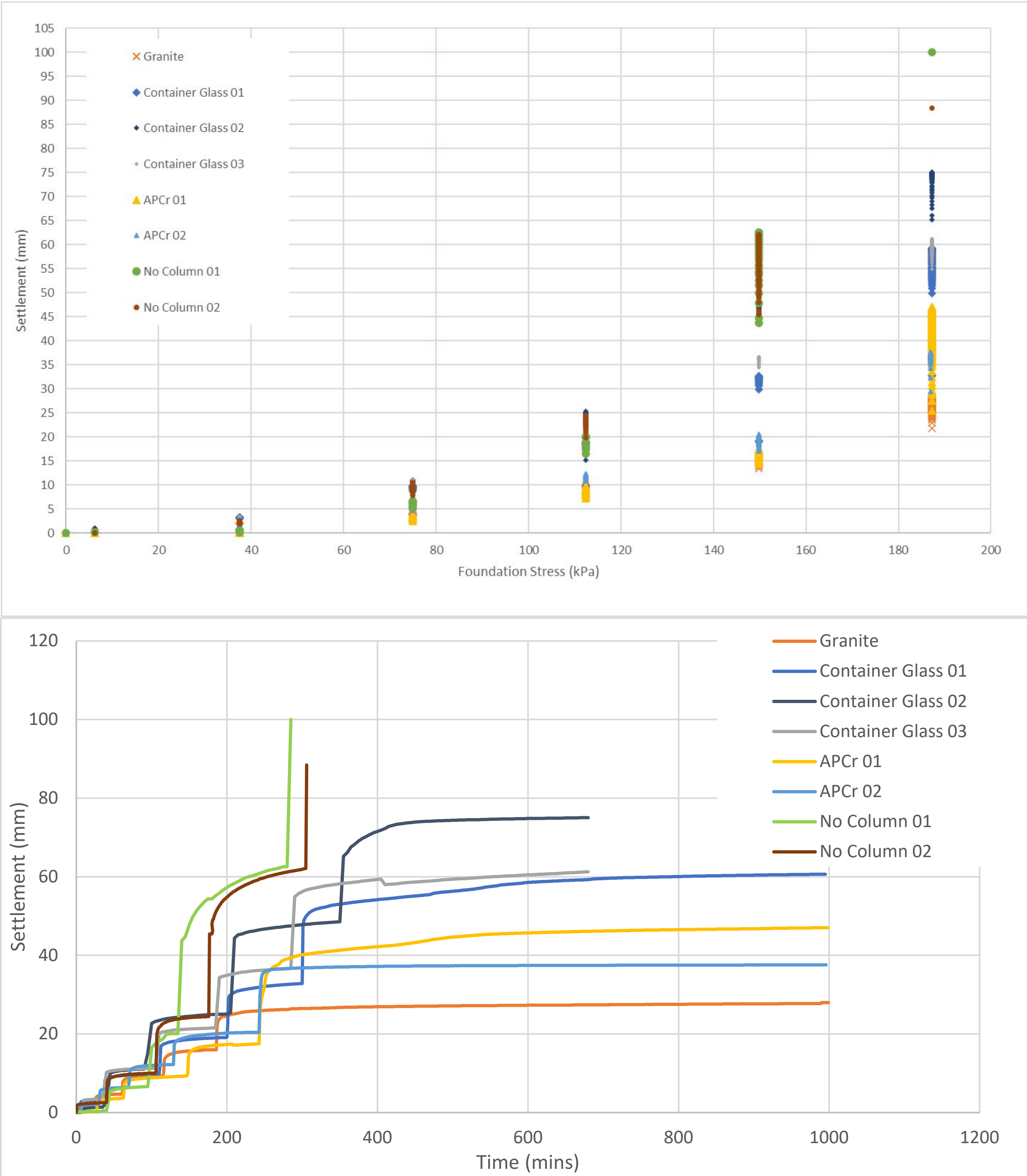


Figure 5-23: Results for all columns combined - Settlement vs. foundation Stress (top) and settlement vs. time (bottom

6 Discussion - Phase 2: Model Granular Column Tests

In this chapter the results of the phase 2 testing, i.e., model column load tests, are discussed together with the results of the pre and post load tests that were conducted on all samples. The first section details the impact of the column installation process on the PSD of the aggregates. In Section 6.2 the results of the constant rate of strain tests (CRS) are reviewed and in Section 6.3 the results of the static load tests are discussed with comparisons being drawn to the CRS tests. Section 6.4 draws comparisons between the results of the two types of load test.

The primary intention of the model column tests was to compare the performance of the alternative aggregates (container glass, window glass and APCr) to a primary aggregate (granite) and to determine the feasibility of their use within full scale granular columns. The factors influencing column behaviour, including host soil strength, column density, the properties of the column aggregate, extent of column bulging, foundation plate size and type of loading will be explored.

The test programme included a total of 36 CRS tests and 8 static load tests, as detailed in the table below. The data has been presented in the Chapter 4.0 and this will be drawn upon in this section to support the discussion.

6.1 Aspects of the Column Testing

6.1.1 Installation Effects on the Aggregates

One common reason cited for the lack of usage of RA/SA is the potential for particle breakage during installation and under loading (and these concerns underpin the approach adopted for index tests that consider the relative suitability of potential aggregates, e.g. AIC, ACV and LA; these were investigated in Phase 1 testing in this study and are reported in chapter 4.0. To assess the impact of the column installation process on particle breakage, Particle Size Distribution (PSD) tests were undertaken before and after the column installation process for each of the materials. For the purpose of these tests a new batch of clay was compacted into the test cell for each test and a sample of aggregate was installed. The compacted clay and aggregate were then discarded (and not used for further column load tests) to avoid any potential influence on column behaviour that may have occurred from the bore being enlarged or from aggregates being affected by going through the installation process more than once.

The column installation process, which was the same for all tests (as detailed in chapter 3.0), comprises the aggregate being placed in the bore, using a centralising tube, in 50 mm layer (confirmed by measuring the distance between the top of the layer and the top of test cells), that are each compacted for a duration of 20s using a small vibrating concrete poker/compactor.

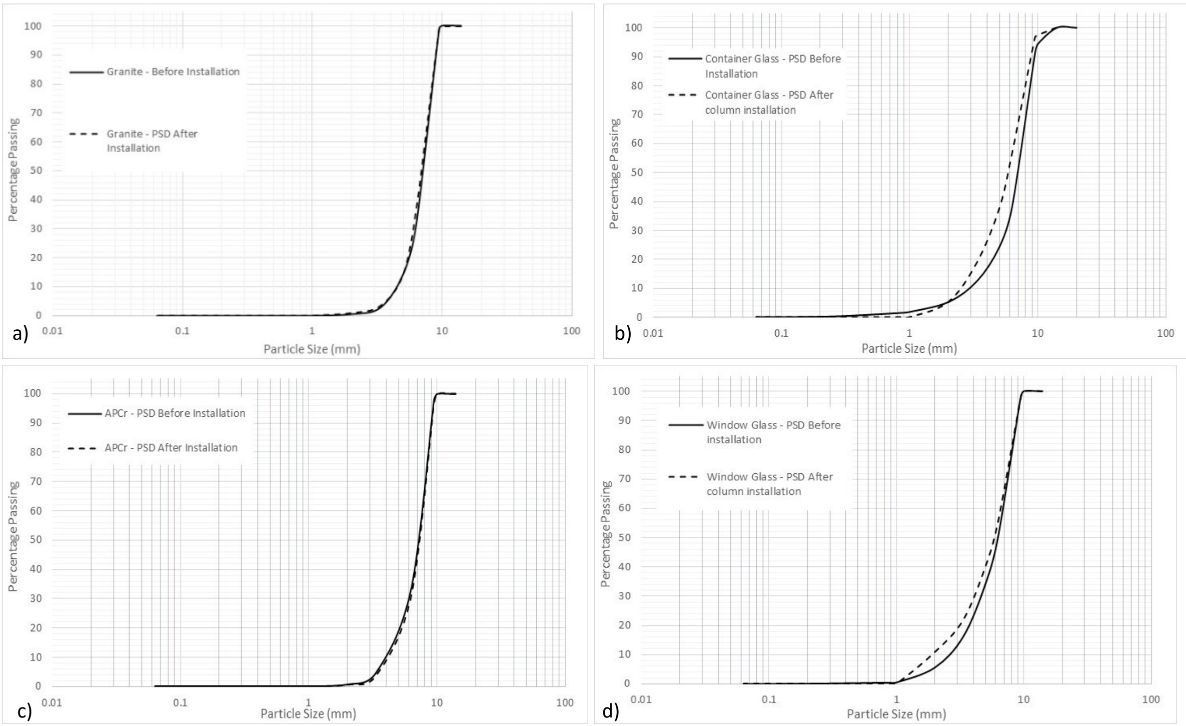


Figure 6-1:PSD curves for the four aggregates before and after compaction into the test cells (to form the granular columns): a) Granite, b) container glass, c) APCr and d) window glass

The results show that both types of glass experienced some particle breakage due to the installation process which is not surprising due to the performance of glass in phase 1 tests. For both types of glass, breakage occurs between particles sized 1 to 10mm (i.e., across the whole sample) which contrasts with the literature where it has been stated that particles less than 6.3 mm do not break (Ooi *et al.*, 2008). The extent of breakage, and the number of fine particles created, appears to be greater in the case of the container glass compared to the window glass. The thickness of the container glass particles was an average of 2 mm, with very occasional particles 4 mm thick, whereas the thickness of the window glass was generally 4.5 mm (small fluctuations were noticed). It is likely that due to the container glass particles being thinner that they were generally more susceptible to breakage and for particle edges being broken off than the thicker window glass particles.

The results of the PSDs for granite and APCr show that breakage occurring during installation for these materials is negligible. This apparent lack of breakage of the granite was unsurprising due to it being a primary aggregate and superior performance was expected (this was reflected in the characterisation tests); this finding has also been observed by Amini (2015), a previous researcher using the same installation process and material. However, the lack of breakage of the APCr was surprising given the poorer performance of the material during the index tests.

The APCr performed very poorly in Phase 1 testing (see chapter 4.0) and at the transition between Phases 1 and 2 of testing it was questioned weather to abandon this material and concentrate on the three other materials. Eventually it was decided to preserver with the APCr in Phase 2 and this decision has clearly highlighted some very interesting findings. It will be demonstrated later in this chapter (section 6.2 and 6.3) that APCr performed very well in the column tests (comparable, if not better, than the granite), which was wholly unexpected and was very surprising when first encountered during Phase 2 testing. This response (and the others observed) might be attributed to the energy input during installation and interactions (during installation and loading) between the APCr and the host clay (as well as water seeping into the column from the host soil with loading); interactions not commonly investigated in the (Phase 1) indicative performance tests when assessing aggregate suitability. These interactions are considered in more detail in subsequent sections of this discussion.

It is appreciated that the energy input in the column installation is likely to be less than that of the falling weights (i.e., proctor compaction and AIV) or vibro-compaction tests (as explained in the Methodology), which might make direct comparison between Phase 1 and 2 outcomes problematic. Furthermore, the author acknowledges that the installation process adopted for this research does not exactly resemble the process(es) used in practice. It would be very difficult to replicate the actual process in the laboratory due to scaling factors (such as ratio of bore to vibroflot and the vibrating frequency of the vibroflot). Therefore, it cannot be said that APCr will not crush during the installation process of a full-scale granular column without testing the material in this context. However, this lack of breakage during installation was the first of several unexpected responses (for APCr) encountered during Phase 2, hence it raises into question the suitability of index testing (such as the LA test) when assessing the suitability of these materials. If they do not replicate the energy input used *in situ,* nor the mechanisms that control load transfer between the column and the host soil, what purpose do they serve? Testing is clearly required (even if only at bench scale) that reflects (albeit approximately) loading conditions/column-host soil mechanisms if more appropriate assessments are to be made regarding the suitability of the alternative aggregates (to primary variants). Clearly this statement is limited to the findings of this study, which was focused upon laboratory testing; the real proof would be full scale trials *in situ,* and this is a recommendation of this study. Full scale testing was outside the remit of this study, and yet would be unlikely to take place at all if the standard indicative tests were undertaken alone when assessing aggregate suitability for APCr (as the material would likely be rejected as unsuitable very early on). A contribution of this work, therefore, is undoubtedly the need to consider the geotechnical response of these aggregates as well as their 'indicative aggregate' response when assessing the suitability of these materials and the need to undertake full scale testing. Full scale testing would comprise the installation of a granular column with a diameter in the order of 0.6 to 1 m which would then be tested using the plate load test as detailed in the ICE Specification for Ground Improvement (1987). To date there is no evidence of full-scale testing of GCs constructed using alternative aggregates in the literature but given AA have been used in industry it is known that these tests have been conducted (Keller, 2022; Balfour Beatty, 2022).

The apparent (minimal) breakage caused during the installation process for all four materials was unexpected as potential breakage during installation is cited as one of the key challenges when selecting recycled/ secondary materials within GCs (BRE, 2000; Serridge, 2005). However, as noted previously the installation method adopted in the laboratory differs to the methods used in practice. In addition, it is difficult to make direct comparison with other laboratory research for the materials studied, other than granite, as they are unique to this research (this is the first study, that the author is aware of, that considers both APCr and Window Glass) and experimental conditions vary. Another important aspect to consider is the particle breakage caused by column loading (rather than installation) and this will be discussed later in this chapter.

6.2 Constant Rate of Strain (CRS) Column Testing: 100 and 60 mm Load Plate Tests

Initially it was planned to only undertake CRS tests using a 100 mm diameter load plate, and the 60 mm foundation plate tests (introduced to investigate possible boundary effects within the cell when using the 100 mm foundation plate) and static load tests were included as the project progressed. The aim of these tests was to assess the load/settlement performance of the columns and to investigate the factors influencing behaviour such as: boundary conditions (Chapter 3.0), host soil strength, PSD and density of the column material, water content of the aggregate post-load and extent of column bulging when exposed to CRS loading conditions. The three AAs were tested alongside baseline tests that did not include columns (i.e., unreinforced samples of clay).

6.2.1 Load-Displacement Relationships

The load-displacement relationships are key to understanding the behaviour of the columns formed of each of the aggregates. Figure 6.2 shows the average CRS stress/strain curves for all materials tested; these plots are reproduced in figures 6.2 to 6.3 to separate out the trends (to improve clarity on the observed behaviour) but also present them as stress vs settlement (instead of strain). This is because presenting settlement in millimetres (rather than as a strain) is useful when considering Improvement Factors (a convention often used when considering stone columns and these are discussed later). From figures 6.2 to 6.3, it can be seen that all materials increase the load bearing capacity of the host soil using the no column tests as a baseline. It also shows that the columns tested under a 60 mm plate (granite and container glass) exhibited performance that exceeded the 100 mm plate tests. This isn't unexpected due to the higher area replacement ratio (100 % compared to 36 %) which is known to improve column performance and this difference in behaviour will be discussed throughout the chapter.

The results show that APCr outperformed the primary aggregate, granite, and the two types of waste glass (when the strength of the host soil were comparable, as illustrated in Figures 6.3 and 6.4, increased shear strength of the clay, outside of the desired 15 to 25 kPa range, could have a noticeable impact upon the performance of the column). However, in the case of the APCr column tested on 24.03.21, the host soil strength exceeded the target range (15-25 kPa) at 30 kPa. It is unsurprising that this column performed more favourably than those within the target soil strength range due to the additional lateral support offered by the clay. This result highlights the impact that clay strength has on column performance. When this result is removed from the average, as is shown in Figure 6.3, the performances of APCr, window glass and granite are more similar. One of the 60 mm granite column tests also featured an out of strength host soil (30 kPa) and the average results both including and excluding this out-of-range result have been plotted in Figure 6.4 to highlight the influence soil strength has on column performance.

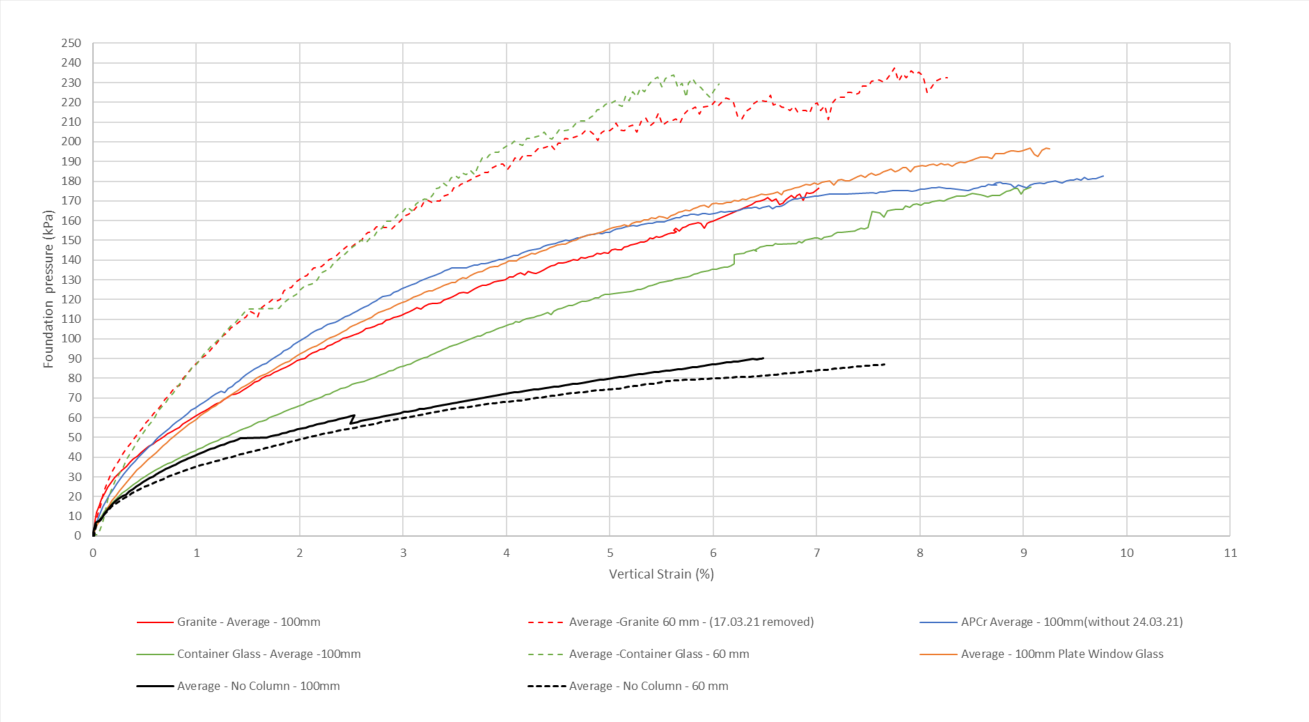


Figure 6-2:Mean Stress/Strain plots for all materials using both the 100 mm and 60 mm load plate.

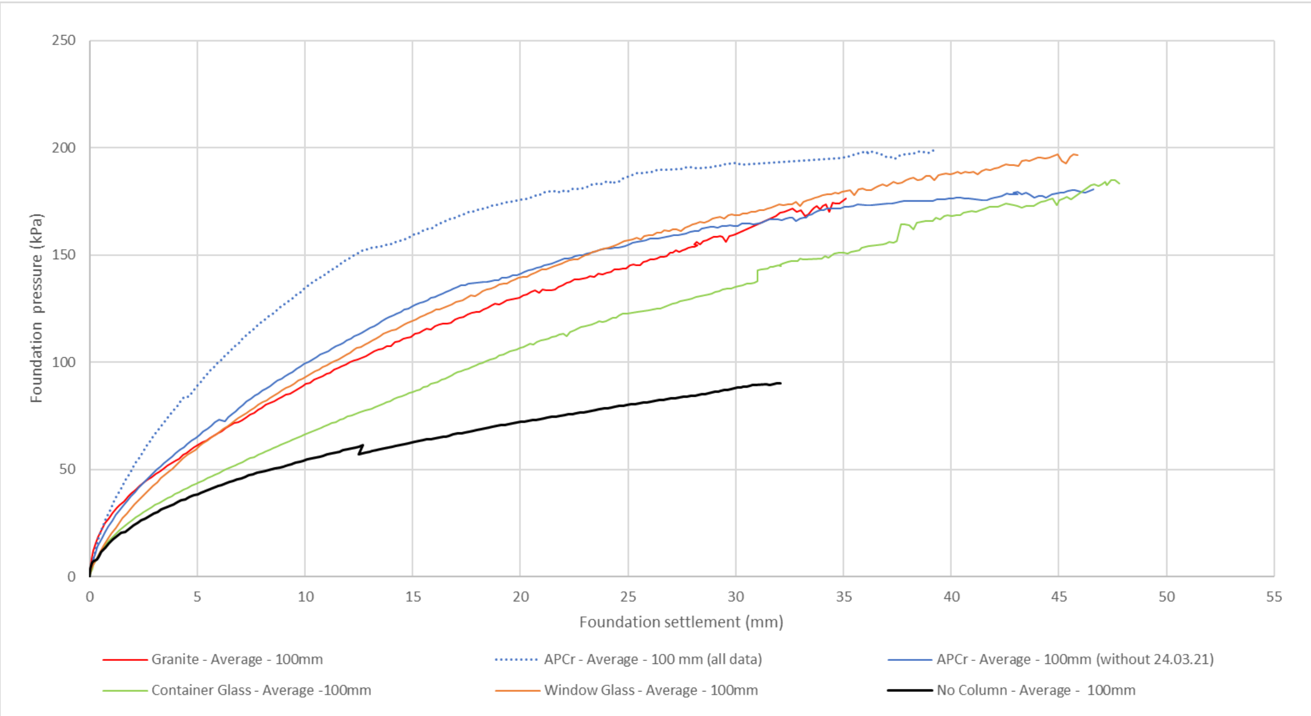


Figure 6-3: Average Stress/settlement results of all materials (100 mm plate)

Please note there was an issue with the test undertaken on the 24th March 2021 as the host soil was stronger than those used in the other tests (in the 30 kPa range rather than the desired 15-25 kPa range), hence a modified average trend for the APCr has been developed to present the average of APCr test results with data from the 24.03.21 removed

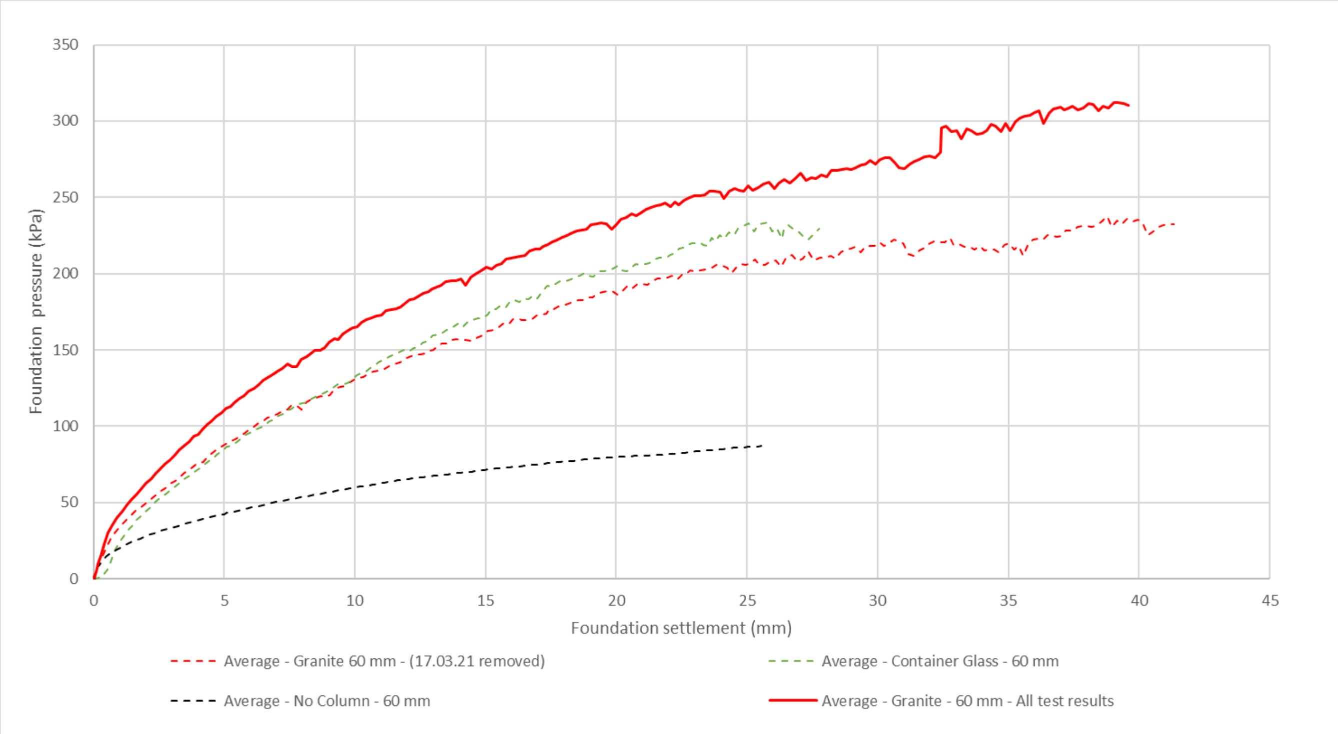


Figure 6-4: Reproduction of the CRS tests (from Figure 6.2) using the 60 mm load plate; expressed as Stress/Settlement plots.

Please note there was an issue with the test undertaken on the 17th March 2021 as the host soil was stronger than those used in the other tests (in the 30 kPa range rather than the desired 15-25 kPa range), hence two average trends are presented for the granite: the average of all test results and the average of test results with data from the 17.03.21 removed.

6.2.1.1 Shape of the Load-displacement Plots

Considering the 100 mm load plate tests: the initial gradient of the load-displacement plots for APCr, granite and window glass appear to be similar up to a settlement of 5 mm, whilst the container glass performance is closer to the unreinforced clay (referring to Figure 6.2, the trends for Granite, APCr and Window Glass appear stiffer than the recycled Container Glass). It is postulated that (during this initial loading period) if the particles of the column were not tightly packed then some rearrangement of the particles would be expected (that did not result in bulging into the host soil; mobilising the passive strength of the clay) as the application of the load compresses the column, causing particles to move into void spaces. Conversely, if the columns were already tightly packed, i.e., close to maximum dry density, then it might be surmised that a dense response to shear would be expected, resulting in localised bulging of the column into the host soil (mobilising the passive strength of the clay); one of the key mechanisms associated with granular column ground improvement. This initial behaviour suggests that the granite, APCr and window glass may have been installed at a density that is closer to maximum density than container glass (attempts to verify in situ dry density proved problematic pre-testing and all but impossible post-testing due to: particle loss into the host soil; fouling of the column with host soil; fundamental change in volume of the column with loading). Another explanation for this difference in initial behaviour would be crushing of the container glass under load which will be discussed later in this chapter, although this is not considered compelling due to the relative performance of the container glass under the 60 mm footing.

When reviewing the result for the APCr columns the foundation stress continues to increase with time up to a stress of 156 kPa where it appears to level off and meets the container glass curve; whilst the foundation stress of both types of glass continues to increase over time. The trajectory of the window glass curve suggests that it would intersect the plots for container glass and APCr at some point if the test were to continue. The observed plateau in the result of the APCr columns could be due to APCr particles crushing; in the static load tests (Section 6.3) crushing was audible for APCr at stresses of 149 kPa and greater, and this would appear to approximately correlate with the plateauing in the CRS testing, however there is no data available that enables direct comparison of crushing under both CRS and static loading at specific points in time. The change in behaviour of the window glass with increasing strain, could be attributed to the increased inter-particle contact caused by the compressive load compacting the glass, densifying the material and changing the load transfer mechanism into the host soil. The results of the post-test PSD of the window glass suggests that minimal crushing occurs during loading, further supporting the previous statement. The extent of particle breakage will be discussed in Section 6.2.7.

There is inherent variation with each of the tests undertaken. Figures 5.7 to 5.11 plot the individual trends for each material, along with the mean values used in Figures 6.2-6.4); highlighting the difficulty in undertaking larger scale testing, although it is argued that they are still fundamental, APCr would have been rejected if based on Phase 1 outcomes alone, yet it appears a suitable material, one certainly worthy of additional consideration). However, it the overarching trends do appear to illustrate that two of the three alternatives perform as well as the primary aggregate over a 7 % vertical strain, with the third material (container glass) apparently improving with continued strain (possibly the function of particle repacking within the columns investigated; densification changing the loading mechanism), which was unexpected and once again highlights that these materials could be considered viable alternatives to primary aggregates

6.2.1.2 Variance in the Data

There was a large spread in the data gathered for the CRS tests. This high level of variance could be expected for the alternative aggerate due to greater variance in material compared to a PA which is produced as a more uniform material. However, in this research differences in performance were seen for all materials, not just AA (although the smallest spread in the data was observed for the granite columns, possibly due to the more consistent PSD and particle shape of the material). This suggests that despite care being taken to keep the host soil and column installation process consistent, slight differences (e.g., soil moisture content and strength, column density, PSD of column aggregate) may have affected column behaviour. However, the variance in data is likely to be caused by a combination of factors affecting column behaviour that are unique to each test.

In Figure 6.5 the upper and lower bound results (presented as stress and settlement, akin to Figures 6.2 to 6.4) for all materials are plotted to illustrate the extent of variance in performance.

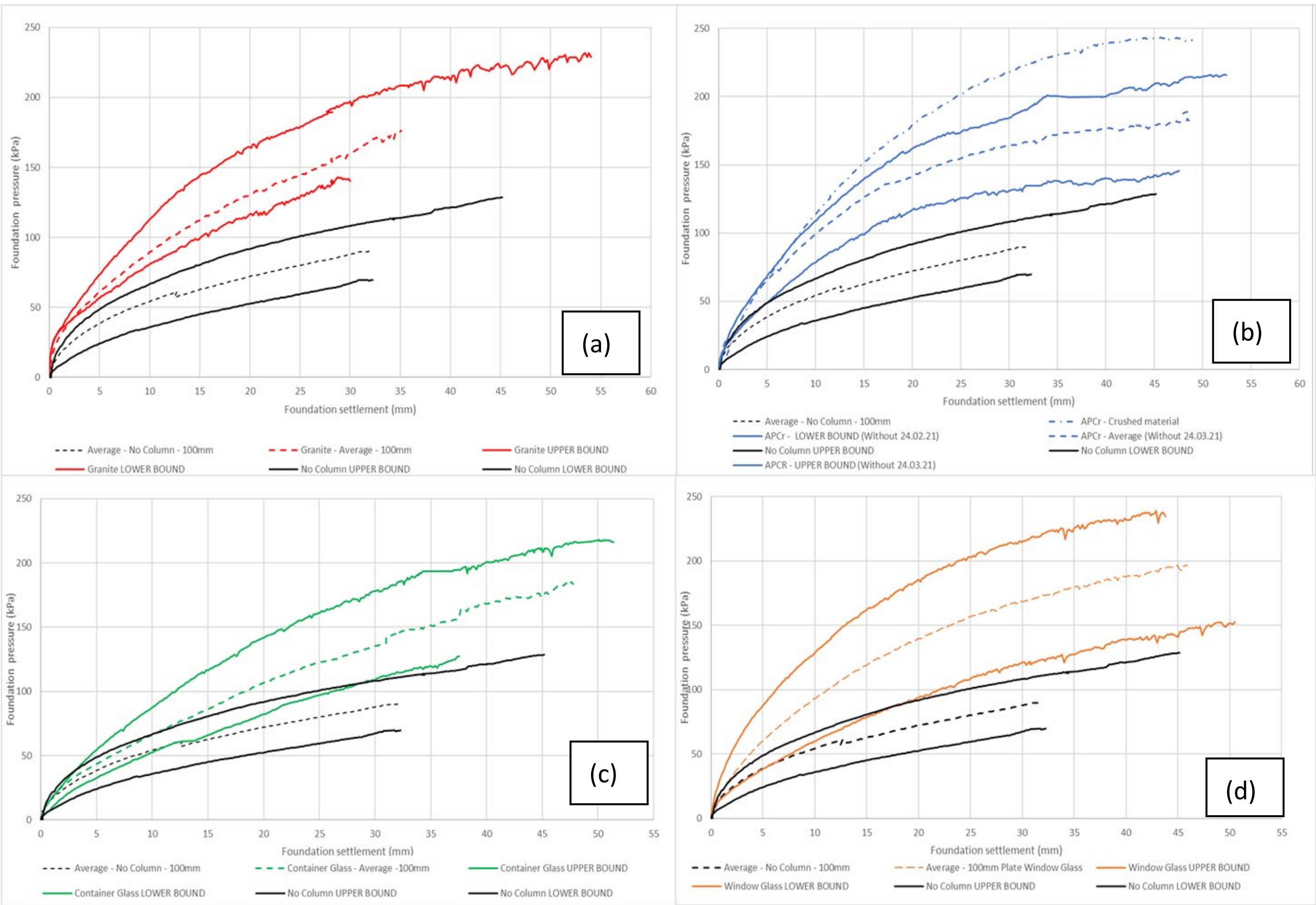


Figure 6-5:Upper and Lower bounds for the 100 mm plate CRS tests, coupled with the mean trend for each of the materials: a) granite, b) APCr, c) Container Glass and d) Window glass

Figure 6.5 shows the upper and lower bounds of the stress/settlement recorded for each of the materials. As mentioned, the difference between the upper and lower bound of the granite columns is the smallest of all the materials, this could be a reflection of the more uniform behaviour expected from PA due to consistent production methods. The container glass exhibited the second smallest variance in data, and this is possible due to it being linked to the it being a whilst the window glass was produced using the LA machine which may have generated a material with a greater variety of particle shapes. It is not surprising that the APCr produced a great variety in behaviour for two reasons: firstly, the samples are formed from a variety of waste streams and it is known that material formed from difference wastes exhibit different properties such as strength; secondly there was a large variety of particle sizes within the samples.

6.2.1.3 Lower Bound Behaviour

Whilst Figure 6.5 highlights the striking difference in the upper and lower bound behaviour of the APCr it also shows that even the lower bound behaviour is comparable to granite, providing confidence that the materials do performance comparably in terms of stress/settlement. Given the performance of the APCr in Phase 1 of testing, this was surprising and is further evidence that practices (such as adopting index tests such as the AIV and LA which do not resemble the experience of aggregates within a GC) must change when considering suitability of alternatives to primary aggregates or viable alternatives (from wastes) will continue to be rejected. It was also surprising to see that the lower bounds for both types of glass were below the upper bound result for host soil (i.e., the no column tests) at settlements below 15 mm, suggesting the potential for zero ground improvement to be achieved with these materials if not handled properly.

However, the fact that the improvement factor (I_f) (which can be defined as I_f = *Load capacity with column/load capacity without column*, measured at a settlement of 25 mm in this study, see Section 6.2.2 for further detail) for both granite and APCr is 1.3 (based on the worst case scenario of upper bound clay behaviour and lower bound aggregate behaviour), which is much lower than would be expected for a primary aggregate, suggests that this apparent poor behaviour is due to a combination of the upper bound clay result being unrepresentative and the lower bound aggregate result reflecting uncharacteristically poor behaviour rather than it being an indication that glass columns perform poorly.

If the lower bound results were used to rank performance, in terms of stress/settlement, then the granite would be first followed by APCr, then the two types of glass (window then container). If the improvement factor was calculated based on the moderate case scenario, i.e., average host soil behaviour and lower bound aggregate behaviour then the granite and APCR would achieve an improvement of 60 % and the window glass and container glass 40 % and 30 % respectively.

There are no obvious indications of as to why there was such a large variation in host soil strength the (i.e., the shear strength was not found to be outside the 15-25 kPa range). The strength of the host soil measured post load (upper bound) was 23 kPa which was close to the target of 25 kPa but was not notably greater than the other samples (the average was 22 kPa) and the moisture content was consistent at 42 %. This indicates the difficulty when attempting to classify the behaviour of these interconnected systems when using coarse tools (such as shear strength derived from hand shear vane; water content; estimated density, etc). Numerical modelling might provide further insight into this (although this was outside the remit of this research). The inherent variation in the samples and the response of the combined column-host soil system is why the approach has been taken to use mean responses and not individual ones.

Whilst there is variation in the performance of aggregate produced from APCr, depending on its source (i.e., waste steam), when monitoring material properties over a 300 day period the compressive strength and density of the aggregate, two key indicators of quality, always exceeded the minimum specified criteria. The compressive strength fluctuated between 0.25 and 0.33 MPa (the minimum required for End of Waste status is 0.1 MPa) and the density varied between 1120 and 1150 kg/m3 (Hills and Gunning, 2014) This indicates that whilst there is variation in quality the difference is not to great that minimum standards are not met.

6.2.1.4 Secant Stiffness

Figure 6.6 presents the secant stiffnesses of the model columns (taking the origin and various points along the stress-strain curves to define the secant stiffness for given vertical strains) and samples without columns. As expected, stiffness is greatest at low strains and decreases with increasing strain and the samples with columns show a greater stiffness than the samples without columns.

The use of a rigid foundation plate ensured that the settlements for the host soil and column were equal. The greater stiffness exhibited by the columns tested using the 60 mm plate is likely due to the load being fully supported by the stiffer column material rather than being shared with the less stiff host soil as was the case for the 100 mm load plate tests.

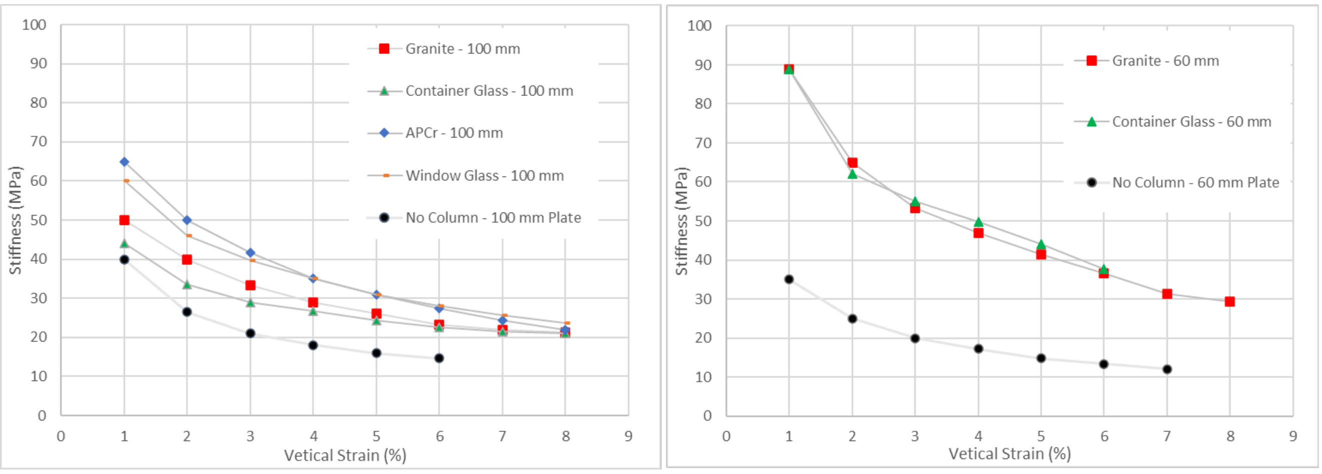


Figure 6-6: Secant Stiffnesses for columns loaded using a 100 mm diameter plate (left) and a 60 mm diameter plate (right)

6.2.2 Comparison Between 100mm and 60mm plate tests

The 60 mm plate was introduced to investigate potential concerns over mobilisation of boundary conditions when using the 100 mm plate (previous research and practice recommends the use of a loading plate that exceeds the diameter of the column i.e., area replacement ratio less than 100 %) (McKelvey, Sivakumar, Bell and Graham, 2004; Black, Sivakumar and McKinley, 2007; Sivakumar, Bell and Black, 2011; Amini, 2015; Shahverdi, 2020); due to the boundary distance to column diameter ratio (one of three parameters considered when attempting to define suitable boundary conditions). However, there did not appear to be notable changes in behaviour in the soil response at given distances from the column when using the 60 mm and 100 mm plates, suggesting these concerns were unfounded (previously considered in Section 3.3.4). However, having datasets for the granite and container glass for both 60 mm and 100 mm plates allows for a comparison of behaviour. For both the container glass and granite columns the capacity of the columns was greater when the 60 mm plate was used to apply load rather than the 100 mm plate. Based on the average results of the capacity of the container glass columns (measured at a vertical displacement of 25 mm; or 6.3% vertical strain) for the capacity of the columns tested using the 60 mm plate was twice that of the 100 mm plate. Similarly, the granite columns loaded using a 60 mm plate achieved a foundation stress 1.7 times greater than achieved for the 100 mm plate. This is in contrast to the results for tests without columns which reflected comparable performance for both plate sizes; suggesting that the enhanced performance observed for the aggregate columns was due particle interaction rather than the confining effect of the container. This finding was also observed in work by Amini (2015) where two load plate sizes were used (54 mm to represent axial loading (or 100 %) area replacement and 100 mm to simulate foundation loading). The results showed that for granite columns the improvement factor was 2.2 times less for the 100 mm plate than for the 54 mm plate. Similarly, for the recycled material columns tested (by Amini (2015), the improvement factor was an average of 1.7 times lower for the larger plate, supporting the statement that the difference in behaviour observed in this study is due to particle interaction rather than container size. Hence, the fact the 60 mm plate tests outperform the 100 mm plate tests is unsurprising as the inclusion of more material that is stiffer than the original soil, i.e., increasing the area replacement ratio, will lead to greater load capacity and this is well documented in the literature (Fattah et al, 2017).

6.2.3 Improvement Factors (Based on Average Column Performance)

The improvement factor is one method of quantifying the extent of improvement a column has on the treated area of ground and it can be quantified in two ways:

I_f *=Settlement with column/ Settlement without column*

Or

I_f*=Load capacity with column/load capacity without column*

CRS tests deform the samples at a stated rate of deformation, hence the settlements measured using this test apparatus are not true settlements but a function of deformation rate and duration of the test. The improvement factor based on the load capacity for a CRS test has been adopted herein as the stress achieved at 25 mm deformation (chosen as this value of settlement is commonly adopted as a maximum allowable value of settlement within industry). Other recognised failure criteria, as discussed previously, are included for comparison in Table 6.1. The IFs herein are calculated using the mean (excluding datasets where the host soil exceeded the desired shear strength range) load capacity/mean response for the host soil without a column.

Table 6.1: Improvement factors for 100 mm CRS tests

Material/Load Plate Size	Improvement Factor (I_f) at 10mm displacement (Zakariya, 2001)	Improvement Factor (I_f) at 25mm displacement	Improvement Factor (I_f) at 35mm displacement (Hughes and Wither, 1984; Al-)
APCr (100 mm)	1.8	2.0	1.9
Granite (100 mm)	1.7	1.8	1.9
Granite (60 mm)	3.2	3.3	3.2
Container Glass (100 mm)	1.2	1.5	1.6
Container Glass (60 mm)	2.5	3.0	-
Window glass	1.7	2.0	1.9

The results of I_f shown in Table 6.1, calculated using the mean results for tests containing columns and host soil only, indicate that all materials have a positive effect on the load carrying capacity of the columns; it is especially encouraging to note that the window glass and APCr exceed the granite in this metric. The improvement factors increase when settlement increases from 10 to 25 mm (i.e., time and load) indicating the need for some vertical displacement to mobilise the full capacity of the column-host soil combined system (passive conditions in soft soils are known to require greater shear strains than active so this would appear to agree with established geotechnical engineering theory). Whilst the container glass has the lowest improvement factor the result still reflects a 50 % increase, at 25 mm of settlement, in load capacity which is not insignificant. The improvement factors for the columns tested using the 60 mm plate are around twice as those achieved by the columns tested using the 100 mm load plate, highlighting the influence area replacement ratio as on column performance.

6.2.4 Column Performance and Performance in Phase 1 Index Tests

Table 6.2: Results of Phase 1 aggregate tests

Material	**Internal friction (°)** *(From small shear box tests)*	**ACV (%)** (<30 Recommended for granular columns (BRE,2000)	**AIV (%)** (<30 Recommended for granular columns (BRE, 2000)	**LA (%)** (<50 Recommended by ICE Specification (1987) for GCs)	**Flakiness (%)****
Granite	51	13	7	18	14
APCr	35	33	22	49	0
Container Glass	31	29	19	42	32
Window Glass	40	29	16	-*	46

**No result available for window glass*

*** Flakiness only applies to particles >6.3 mm, these figures present the extent of flaky particles out of the entire sample*

Based on the results of the index tests, summarised in Table 6.2, it would be a reasonable assumption that the granite would outperform the recycled/secondary materials in the column tests. In fact, due to the poor performance of the APCr, particularly in the LA and ACV tests, the material was very nearly removed from this research due to the lack of perceived benefit in testing it within granular columns. The container and window glass also appear to perform poorly, particularly in the ACV test. Whilst the glass does meet the suggested criteria it appears to be marginal and indicates that the use of this aggregate within granular columns would lead to poorer performance than PA. However, the results of the model column clearly show that the poor performance of the AA shown in the index tests is not reflected in the model column tests. This finding has been observed by other researchers (Amini, 2015) and highlights the need for alternatives to the index tests suggested for granular column aggregates, particularly for alternative aggregates.

Particles that are flaky are not recommended for used within granular column construction due to the potential for them to crush under load and interfere with the packing of particles. There is currently no guidance on an acceptable percentage of flaky particles only that they are not suitable for use within GCs (Steele, 2004; Serridge, 2005; Serridge and Sarsby, 2010, Serridge and Slocombe, 2012). However, the results of the model column tests indicate that both types of glass do have merit for use within granular columns despite the flakiness of the particles. It is suggested this is because once installed, particle breakage becomes secondary to localised shearing between particles (with the exception of APCr in static testing, and possibly dynamic testing, under loads above 147 kPa); this is evidenced by the change in PSD curves pre- and post-testing (Figure 5.3). Hence, if the load at which particle breakage once again becomes dominant is known, and this is above the expected working load then flaky particles could be suitable (this research indicates this is the case) and this is worthy of further investigation.

Angle of friction (ϕ: otherwise known as the angle of internal shearing resistance) is (perhaps) considered the key single parameter when assessing aggregates suitability for granular columns as it indicates the likelihood for dilation with loading (if densification is completed correctly and the particles do not disintegrate during this process) and the angle of friction for each of the aggregates is presented in Table 6.2. Whilst it is recognised that these results are limited (both by the limits of the direct shear test and the size of apparatus used; as discussed in Chapter 4.0) it is interesting to compare these values to observed column performance. For example, as granite had the greatest angle of friction, and is used in practice, it would be expected that granite column would achieve the best performance; yet it is debatable if this is the case (figures 6.2 to 6.4 and Table 6.1). Despite APCr and window glass having lower angles of friction and being more susceptible to particle breakage (Phase 1 testing outcomes), they would appear to outperform granite during Phase 2 testing (dynamic and static testing), indicating that other factors are influencing column performance in these tests. These results suggest that despite apparent poor performance in index tests, suggested for granular columns, recycled and secondary aggregates can perform favourably within the context of granular columns, which is encouraging. Other researchers have made the same observation (Amini, 2015; Serridge, 2005).

6.2.5 Observations from CRS Testing

This section considers observations from the CRS tests in Phase 2 and attempts to consider parameters that might influence behaviour. It is clear that (as this section will show) one parameter for the column or host soil does not appear to control/dominate behaviour – it is very much a combined system that is dependent on a number of interacting mechanisms. It is an outcome of this research that the large bench-scale testing approach is not sensitive enough to isolate key parameters. This was expected (variation in large scale soil tests is not unusual) hence Phase 1 testing was undertaken to try to provide indications of possible material behaviour that could be used to extrapolate/inform the larger scale response. However, as noted previously in this discussion, the index testing adopted in Phase 1 (e.g., AIV, ACV, LA) does not appear to correlate to the behaviour observed in Phase 2 testing and hence did not achieve the initial aim; and also indicates that the standard aggregate index testing might not be fit for purpose when considering confined *in situ* response.

6.2.5.1 Post-test Strength Increase

In work by Hu (1995), where Kaolin was adopted as the host soil, soil strength (measured using the hand shear vane) increases of up to 1.6 times the original strength were found, as shown in Figure 6.7. This increase in strength post-test is expected due to the drainage of water, induced by the loading of the column and facilitated by the granular nature of the column aggregate, causing a stiffening of the clay in the area surrounding the column. The increase in strength post-test, for each of the columns formed of the different materials, observed in this research were similar. The maximum increase in strength was found to be 1.2 times initial strength but the average was 1.09. These results translate to strength increases in the order or 1 to 5 kPa.

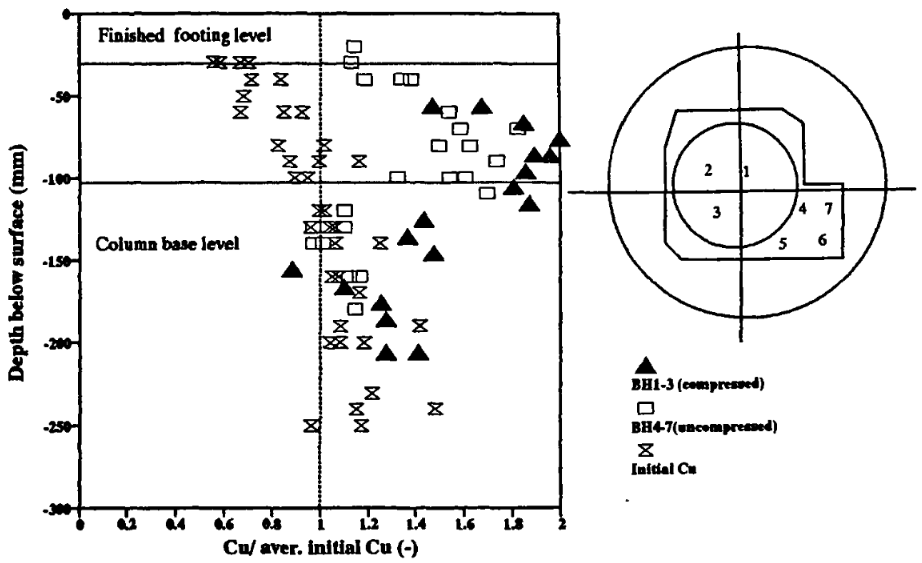


Figure 6-7: Strength increase of host soil post-test (Hu, 1995)

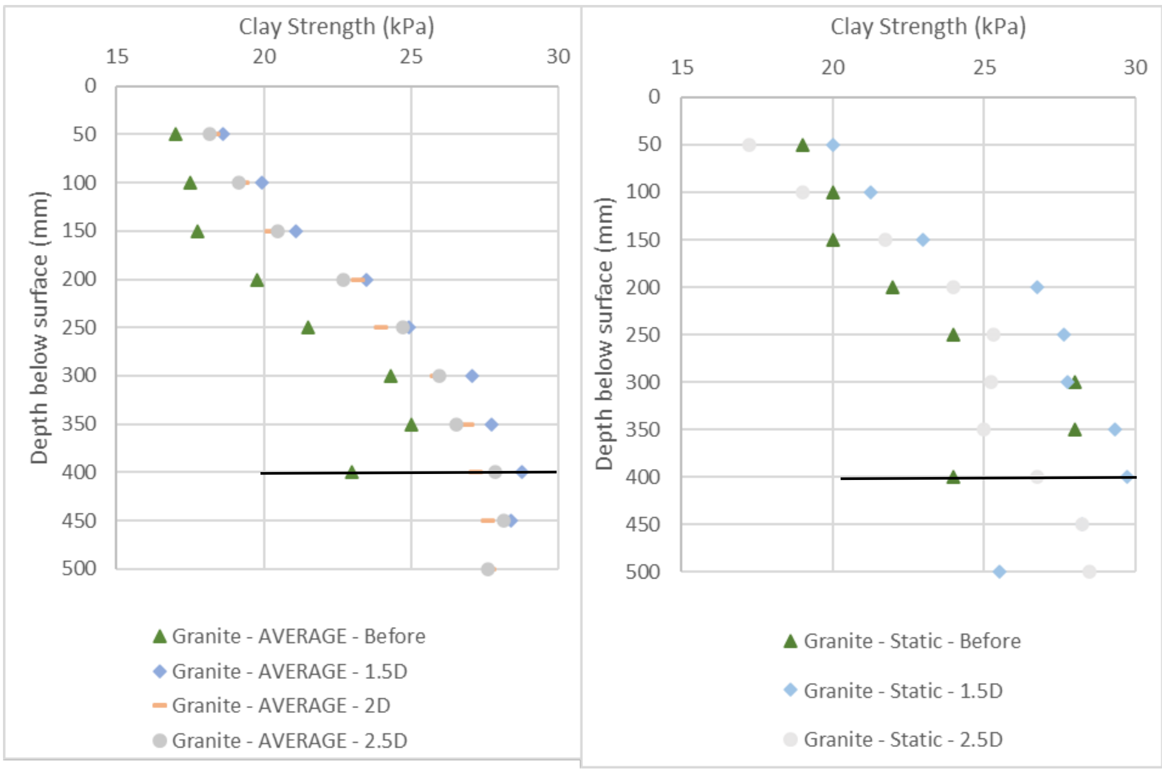


Figure 6-8: Mean clay soil shear strength with granite columns (before testing, after testing at 1.5, 2 and 2.5 times the column diameter from the column outer surface (taken at the start of the test). a) CRS testing, b) static testing (Bottom of column was at a depth 400 mm)

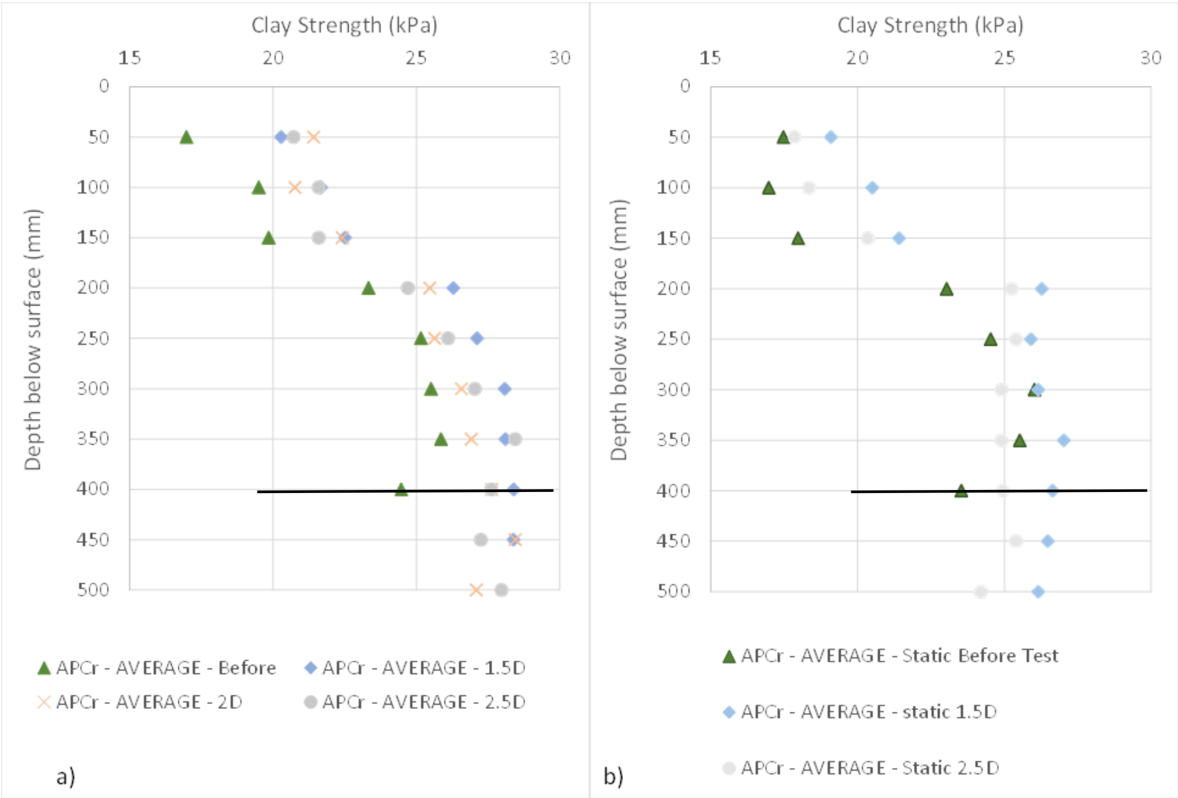


Figure 6-9: Mean clay soil shear strength with APCr columns (before testing, after testing at 1.5, 2 and 2.5 times the column diameter from the column outer surface (taken at the start of the test). a) CRS testing, b) static testing (Bottom of column was at a depth 400 mm)

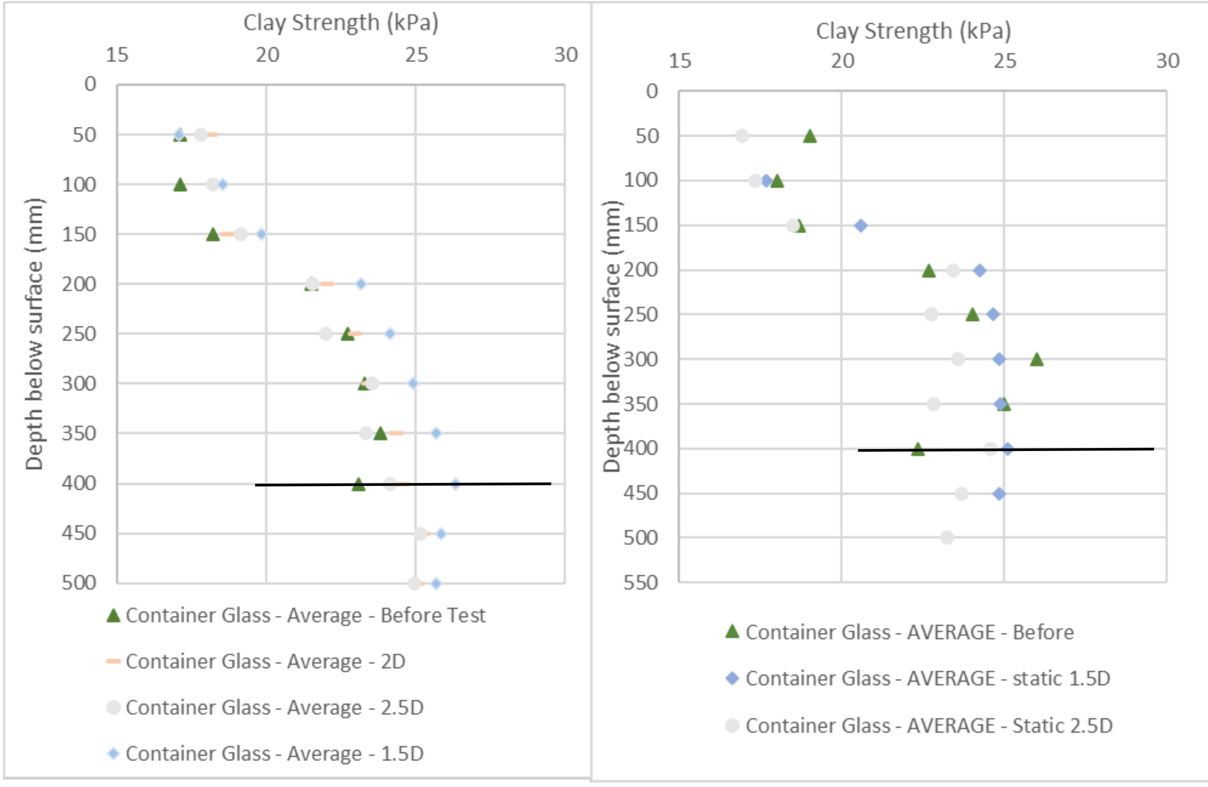


Figure 6-10: Mean clay soil shear strength with container glass columns (before testing, after testing at 1.5, 2 and 2.5 times the column diameter from the column centre (taken at the start of the test). a) CRS testing, b) static testing (Bottom of column was at a depth 400 mm)

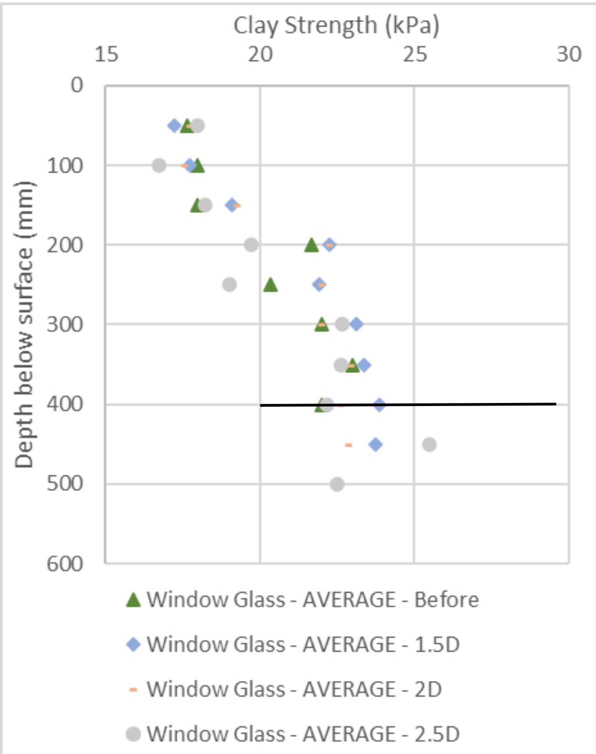


Figure 6-11: Mean clay soil shear strength with window glass columns (before testing, after testing at 1.5, 2 and 2.5 times the column diameter from the centre of the column (taken at the start of the test). (CRS testing only) (Bottom of column was at a depth 400 mm)

Whilst there does appear to be a trend in increase in strength of the clay post-test, even if small, this could be attributed to noise of the hand shear vane. There is most likely is an increase in strength in the area immediately surrounding the column, for the reasons described, but the closest measurement was taken at 1.5D from column centre) due to concerns over the hand shear vane catching on the column; 1.5D (comparatively) a long way from the column/host-soil interface (considering the drainage path) for changes in strength to occur in the host soil over the duration of the test (4-5 hours) and hence might be undetectable within the noise of the hand shear vane. The lack of correlation with change in water content (approximated from before and after tests at, approximately, the same location) would tend to suggest that the observed strength increase is not due to drainage into the column.

The trends of strength increase of the clay post-test can be seen in figures 6.8-6.11, where the initial clay strength is plotted, along with post-test strengths measured at 1.5, 2 and 2.5D, against depth. For all materials the post-test strength increase is greatest at 1.5D. In general, the clay at 2D and 2.5D appears to be less affected, which was expected (based on the findings of Hughes and Withers (1984), that ground outside a radial distance of 2.5D from the centre of a single column. In this research the maximum radial distance from the centre of the column was 174 mm and the diameter of the hand shear vane blades was 25 mm, thus the maximum distance that the reading could be taken was at 150 mm to allow a gap between the container wall and the blades of the shear vane. The close proximity of the container wall to the shear vane may have had some influence on the readings (contributing to uncertainty).

The strength with depth profile of the initial clay strength follows the same pattern for each of the materials, suggesting consistency of clay bed preparation. The lower half of the container tends have a greater strength which is likely due to the rigidity of the container base enabling more of the compaction energy, from the hammer drill, to be directed into the clay rather than being damped by the clay layers below, as was likely the case for the top half of the container.

6.2.6 Initial PSD of Column Aggregate and Column Density

6.2.6.1 Material Grading (of the Columns)

Overall, the aggregate materials can be considered as gravels with a small amount of sands. Table 6.3 presents the values for the coefficient of the coefficient of curvature (Cu), coefficient of curvature (Cc) and Brown's Stability Number (Equation 6.3) for the materials. Cu was between 1 and 2 for all materials, indicating well graded materials (values between 1 and 3 indicate a well graded soil) if taken on its own. However, the values of $C_{c\ were}$ all relatively close to 1, which contradicts the well graded analysis (well graded material requires C_u to be greater than 4). Taking Cu and Cc together suggests the materials can be described as poorly graded (which agrees with the visual inspection of these materials). For all of the materials, the highest percentage of particles were retained on a 6.3 mm sieve. Brown (1977) developed a suitability number for vibrofloatation backfills (i.e., granular column aggregates) based on the relationship in Equation 6.3; the larger the number the less suitable the material: a number between 0-10 is rated as excellent, 10-20 is good and 20-30 is fair). Based on this scale proposed my Brown, all materials studied in this research can be regarded as excellent. Pre- and post-test Cu, Cc and Brown's stability numbers (previously discussed in Section 2.2.5) appear to suggest that that these values have not been significantly changed and the description of the grading remains the same. Hence, whilst some crushing may have occurred during loading, it is not so significant that fundamentally alters the grading of the materials.

$$Brown's\ suitability\ no. = 1.7\sqrt{\frac{3}{D_{50}^2} + \frac{1}{D_{20}^2} + \frac{1}{D_{10}^2}} \quad \text{.....................................Eq. 6.3}$$

Table 6.3: Grading properties of all materials before (left) and after (right) model column tests

Before Testing				After Testing			
Material (and load plate diameter)	Cu	Cc	Brown's Suitability Number	Material (and load plate diameter)	Cu	Cc	Brown's Suitability Number
Granite (100 mm)	2.00	1.23	0.79	Granite (100 mm)	1.76	1.17	0.66
APCr (100 mm)	1.84	1.05	0.74	APCr (100 mm)	2.24	1.00	0.91
Window Glass (100 mm)	2.54	1.29	0.93	Window Glass (100 mm)	2.73	1.20	0.95
Container Glass (100 mm)	2.87	1.20	1.26	Container Glass (100 mm)	2.67	1.34	1.03
Granite (60 mm)	1.85	1.24	0.72	Granite (60 mm)	1.70	1.10	0.73
Container Glass (60 mm)	2.87	1.13	1.28	Container Glass (60 mm)	2.69	1.15	1.27

6.2.6.2 Variation in Initial PSD

The initial PSD of the materials varied, as shown by Fig. 6.12. The aggregate that showed the least variation in initial PSD was window glass which is due to the PSD being manipulated to match the average PSD of the container glass. However, despite this similarity in initial PSD, variations in the results of the model column for window glass were observed, suggesting that initial PSD is not the only influential factor on column performance.

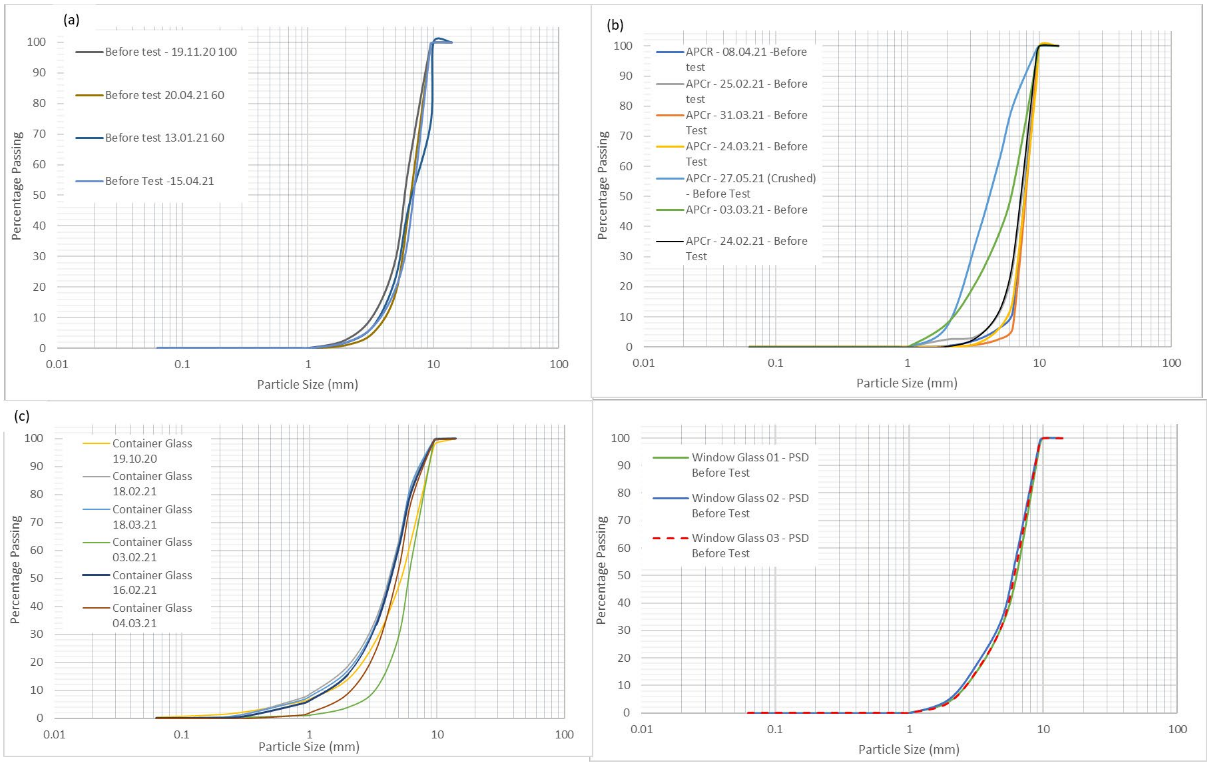


Figure 6-12: Initial PSDs of granite (a), APCr (b), container glass (c) and window glass (d)

Variation was also seen in the initial PSDs of container glass and APCr. This was expected due to the nature of the processing of the two materials. Some variation in PSD was observed for granite but to a lesser extent. This was expected due to the granite being a highly processed product.

6.2.6.3 Column Density

It was anticipated that the variation in initial PSD would have an effect on the column density achieved (the installation process was kept consistent, but it is accepted that small variations could have been incurred due to slight differences in the installation process). However, it was found that the greatest variation in column density was achieved for the granite columns (as shown in Tables 5.1 to 5.4), despite other materials (i.e., container glass and APCr) having greater variations in initial PSD. This observation suggests that other factors, one key example would be particle shape, affect column density and supports the statement that initial PSD and column density are not the only influential factors on column behaviour under load.

6.2.7 Particle Crushing

Figures 5.2 to 5.6 show the PSDs of each of aggregate materials before and after column load tests. The result indicate that the breakage of window glass particles is negligible compared to the crushing that occurred during the installation process. However, the APCr does appear to crush under loading, which contrast with the observations made during the installation process. This was not a constant phenomenon, in some tests there was not an audible crushing noise and the material performed well (for example the upper bound stress-settlement curve for the APCr outperformed most of the other columns in CRS testing), and it appears to occur once a threshold stress is exceeded.

6.2.7.1 Granite

The results of the before and after PSDs of the granite vary from practically no change (column tested on 19.11.20, Figure 5.2a) to more notable changes as seen for the column tested on 09.12.20, Figure 5.2b. It was expected that, due to its particle strength and PA status, minimal breakage would occur within the granite columns. However, the results of the post-test do not prove conclusively that the granite does not break down under loading. In a number of post- tests PSDs there appears to be a change in PSD suggesting that some breakage has occurred and so produced more of a certain size fraction (see PSD results for column tested on 15.04.21 (Figure 5.2e) but the other results appear erroneous and this possibly due to the difficulties faced when trying to remove 100 % of the column aggregate post load.

There does not appear to be a significant difference between the columns loaded using a 60 mm load plate those loaded with a 100mm plate (see Figure 6.13). However, the 60 mm load plate does appear to have generated more fines than the 100 mm plate.

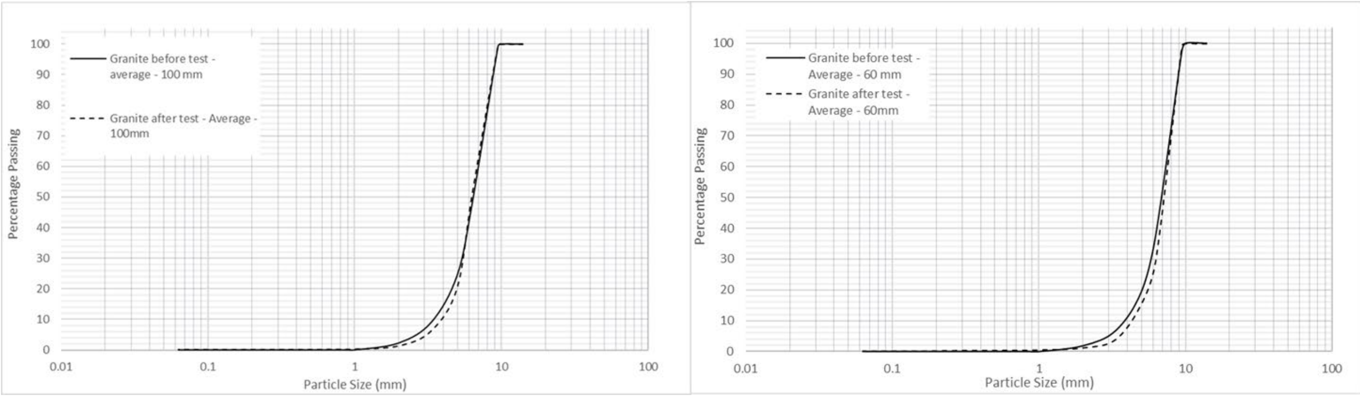


Figure 6-13: Average PSD before after tests for granite columns tested using 100 mm plate (left) and 60 mm plate (right)

6.2.7.2 APCr

In contrast to the results of the granite columns, significant particle breakage can be observed in all but one of the post-test PSDs for APCr. This breakage occurred across all particle sizes, as illustrated in the average post-test PSD curve shown in Figure 6.14. Curiously one sample (24.02.21, Figure 5.3a; and interestingly this is not related to the upper bound stress-settlement trend) showed minimal breakage. There does not appear to be an obvious explanation for this. However, as evidenced by the stress-strain (or stress-settlement) behaviour presented in Figures 6.2 – 6.4, and Table 6.1, the crushing of the APCr during load clearly did not have a negative effect on the carrying capacity of the column which is a commonly cited reason for not utilising materials with the potential to crush under load.

Table 6.4: Change in % retained before and after load testing for APCr

Sieve/Particle Size (mm)	% Retained Before Load test	% Retained After Load Test	Change in % Retained
14	0	0	0
10	0	0	0
9.5	7.14	4.26	-2.88
6.3	69.28	57.25	-12.03
5	10.68	15.03	4.35
3.35	7.44	12.18	4.74
2	3.64	7.29	3.64
1	1.82	3.31	1.49
0.85	0	0.49	0.49
0.3	0	0.1	0.1
0.15	0	0.04	0.04
0.063	0	0.052	0.05

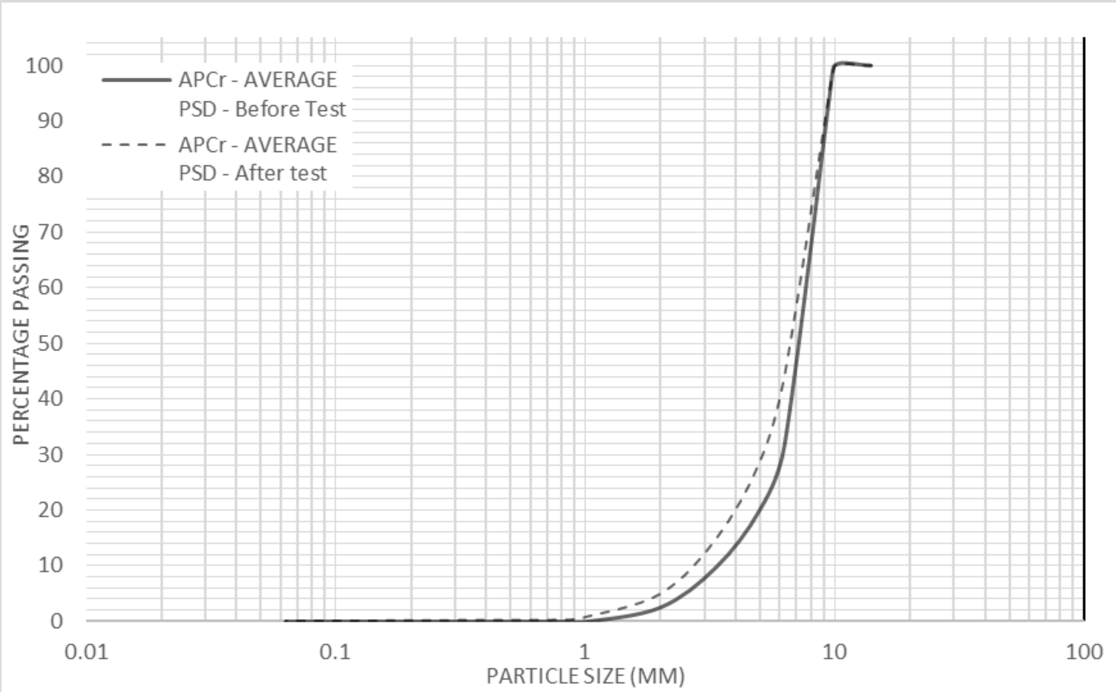


Figure 6-14: Average PSD of APCr before and after load test

6.2.7.3 Window Glass

The post-test PSDs of the window glass columns indicated that a negligible amount of particle breakage had occurred as the post-test PSD curve was almost identical to the pre-test result for all three tests.

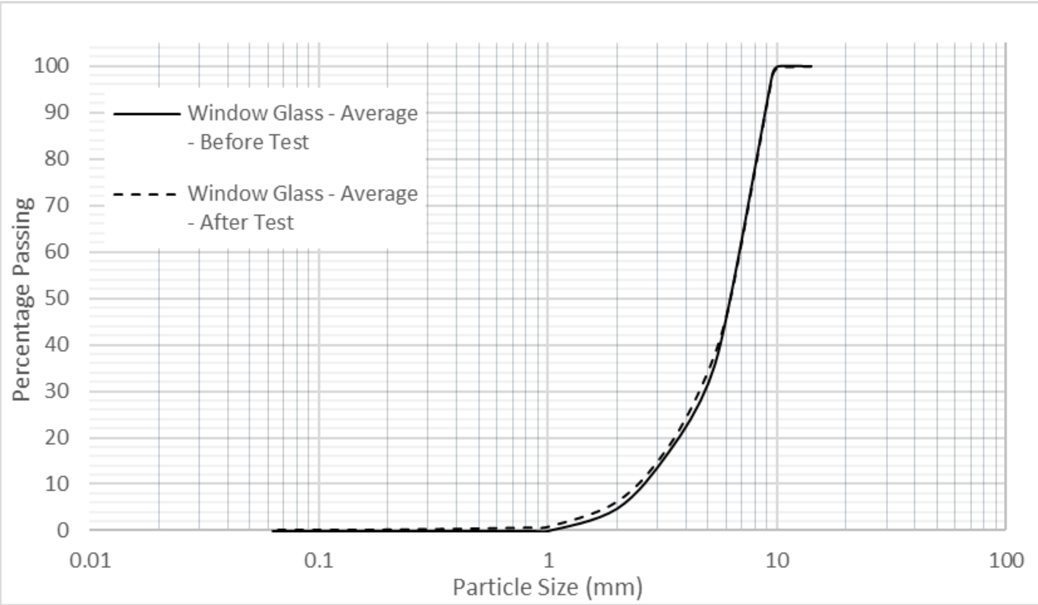


Figure 6-15: Particle Size Distribution - Window Glass Before and After Test

6.2.7.4 Container Glass

The results of the container glass PSD were mixed in terms of the indication of crushing, as was the case for the granite. It was expected that the top half of the column would exhibit a greater extent of crushing due in being directly under the point of application of load, however, the data does not confirm this.

The results for the columns tested on 03.02.21 and 27.01.21 (see Figure 5.5) clearly indicate that some crushing has occurred but the results for the other tests are less clear. The average PSD for both the 100 and 60 mm plate load tests indicates that there are less particles passing the 5 to 1 mm sieves which is mostly likely due to the difficulties in recovering the full sample. It appears that, as was the case for the granite, there was a change in PSDs with an increase in finer particles for certain samples, but this was not reflected in all of the results again suggesting that the inability to recover 100 percent of the material has hampered the particle breakage analysis.

In addition, there does not seem to be a clear difference in the PSD results of the container glass loaded with a 60 mm plate and those loaded with a 100 mm plate. This is surprising as it was anticipated that the 60 mm plate would lead to additional crushing (if was crushing was to occur) due to the increased load concentration on the column.

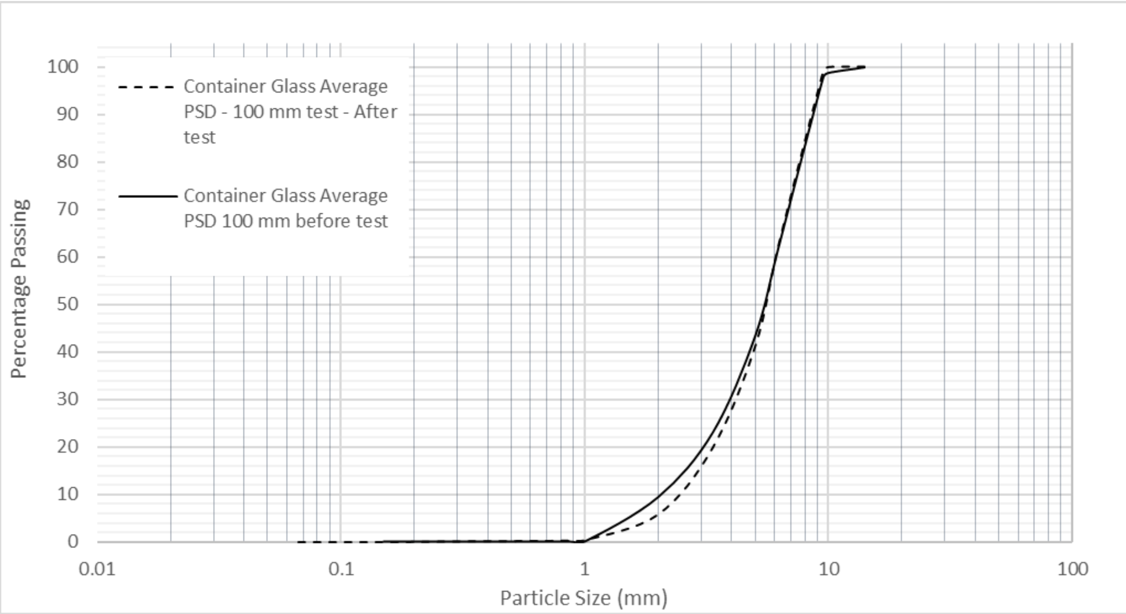


Figure 6-16: Average PSD of container glass before and after testing using a 100 mm plate

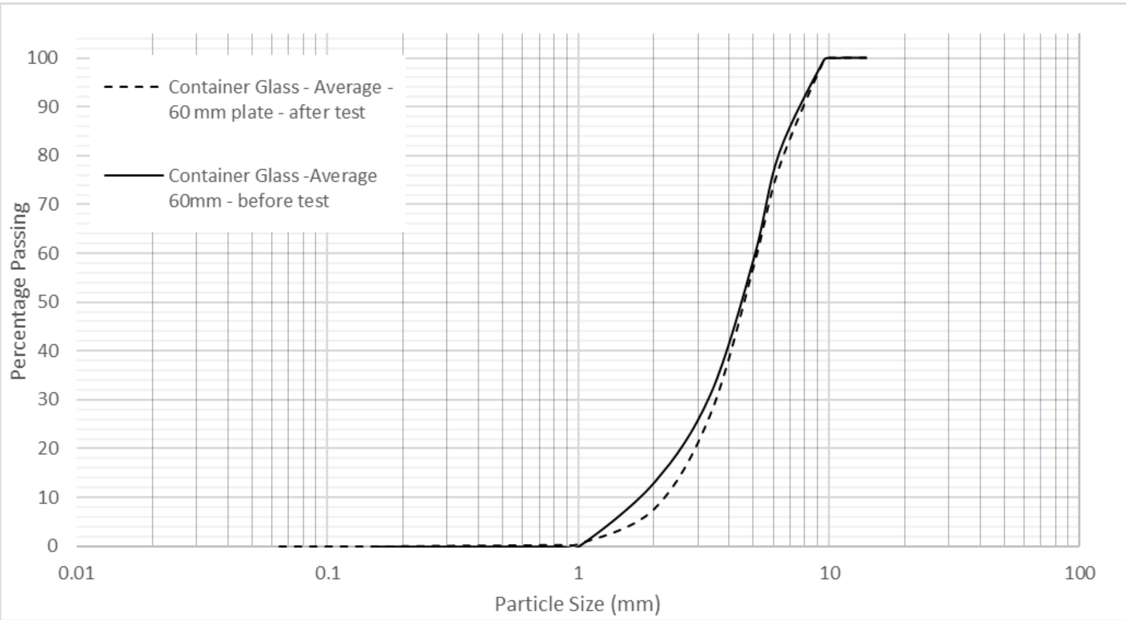


Figure 6-17: Average PSD of container glass before and after testing using a 60 mm plate

6.2.7.5 *Summary of Particle Breakage*

Whilst the analysis of particle breakage was somewhat hampered by the inconclusive results of the granite and container glass. It is clear that the APCr does crush and that this does not have a negative effect on column capacity which is an encouraging finding given that crushing is a key concern in the adoption of AA.

6.2.8 Column Shape and Extent of Bulging

Granular columns are expected to expand radially and bulge into the host soil to mobilise the passive strength of the host soil. However, the columns can also experience shearing (global failure, akin to a shallow foundation ultimate failure) or excessive vertical penetration into the host soil (akin to settlement of a pile). To assess the relative performance of the columns, models of the post-load columns were created by pouring Plaster of Paris into the bore after the aggregate had been removed; this enabled an analysis of the post-load column shape and extent of bulging/vertical penetration to be carried out (Methodology, Section 3.7.5.3).

The diameter of the columns was measured using digital callipers. Three measurements of the column diameter were taken, at 25 mm intervals along the column, and the average of the three was recorded.

These models provided confirmation that, as expected due to geometry of the column and the soft underlying strata, the columns failed through bulging rather than buckling (Afshar and Ghazavi, 2012; Hughes and Withers, 1974; Sivakumar *et al.*, 2004) (the possibility of failing in end bearing will be discussed later in the chapter) (see the photos in figures 6.18 to 6.21). The columns are unbound, hence much of the load carrying capacity is drawn from the lateral strength of the surrounding soil which exerts a confining pressure onto the aggregate as it 'bulges' radially outwards under loading. The shape of the exhumed columns indicated that all columns had bulged under load. The extent of the observed bulging is summarised in Table 6.5.

Table 6.5: Average extent of bulging measured at point of maximum bulge for all materials

(See Chapter 5.0, Figures 5.12 – 5.15 for plots)

Test/Column Material	Average depth of the maximum bulge (mm)	Average maximum column diameter measured post-test (mm)	% Increase in diameter
APCr (100m load plate)	54	82	36
Granite (100m load plate)	75	75	26
Window Glass (100m load plate)	50	72	19
Container Glass (100 mm load plate)	79	68	14
Granite (60 mm Load Plate)	50	74	24
Container Glass (60 mm Load Plate)	50	74	23

Overall, a tapering of the column (i.e., column diameter less than 60 mm), from an approximate depth of 300 mm was observed for granite and container glass. The tapering of the window glass columns appeared to start at a depth of 200 mm from the surface. This tapering is likely not to be due to collapse of the bore prior to or during installation as these were inspected prior to installation and always appeared stable. It is possible that this effect has been caused by the fact that the bottom half of the column is most difficult to recover, leading to more material being left around the perimeter of the bore rather than being captured by the Plaster of Paris.

It is interesting that this tapering of the lower half of the column was not observed as strongly for the APCr columns. Perhaps this was due to APCr particles being easier to recover, or there could be more fundamental process occurring i.e., as the APCr column bulged to the greatest extent the lower half of the column was affected to a lesser extent than for the other materials, leading to the shape of the bore being less affected.

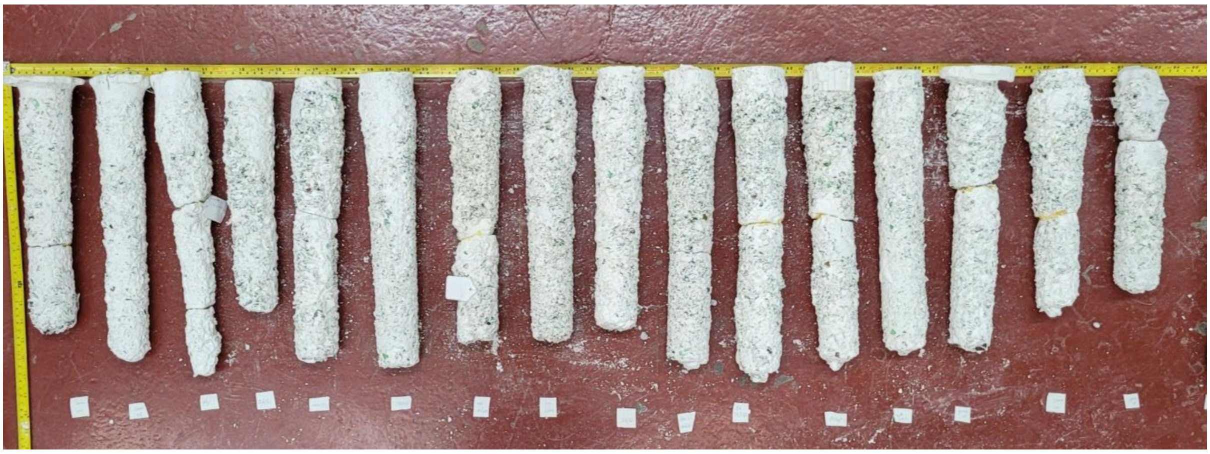


Figure 6-18: All container Glass Column models

Figure 6-19: All granite model columns

Figure 6-20: All window glass column models

Figure 6-21: All APCr column models

6.2.9 100 mm Load Plate Columns

The two materials that exhibited the best performance in the model column tests, APCr and window glass, reached the point of maximum bulge at a shallower depth (54/50 mm respectively) than for container glass and granite which reached this point at a depth of 79/75 mm. This could be due to lower angle of friction of these materials leading to greater horizontal effective stress mobilisation and hence the particles moving outwards more readily than for the granite where the interlock between the particles may support load transfer vertically for a longer duration. However, the similarity in bulging depth of the container glass and granite places doubt over this explanation. The APCr columns experienced the most bulging (the average radial expansion was 36 %). The readiness of the spherical particles, once the friction between particles generated due to surface roughness has been overcome, to roll over each other may have led to the APCr exhibiting the greatest radial expansion.

The container glass columns exhibited the least amount of bulging, which is interesting since these columns reflected the lowest load capacity in the model column tests. This could be explained in three ways: Firstly, the packing of the container glass columns was relatively loose meaning (which is linked to the presence of flaky particles) that under load the particles rearranged leading to compression within the bore rather than bulging outwards; secondly, the container glass may undergo more crushing, leading to a denser packing of particles within the original bore, although the pre-and post-test PSDs would not support this assertion; thirdly, the columns fail via the end-bearing mechanism, hence before sufficient interparticle force building up to cause bulging, the whole column moves downwards (rather than outwards). The potential for failure in end-bearing will be discussed in Section 6.2.11.2.

The window glass showed the second lowest amount of bulging out of all the materials at 14 % in terms of radial expansion which is similar to the 19 % expansion exhibited by the container glass volumes. This similarity would be expected due to the similarity of the two aggregates, however, the difference in performance in the model column tests indicates that bulging behaviour of the two materials may differ. The main difference between the two materials is the particle shape and particle thickness which leads the conclusion that the packing of the column (which is undoubtedly affected by the particle shapes) is likely to be a factor in performance. Another aspect to consider is the lack of particle breakage observed post-test for the window glass, which could be due to the particles having a greater strength (due to a greater thickness of 4.5 mm rather than the 2-3 mm of the container glass). It is possible that the structure of particles within the column was able to withstand a greater load without bulging compared to the thinner, more breakable, container glass particles.

6.2.10 60 mm Load Plate Columns

It is interesting to note that the extent and depth of bulging are almost identical for the granite and container glass columns tested. Compared to the 100 mm columns the depth of the point of maximum bulge for the 60 mm columns was approximately 20 mm shallower. This likely to be due to the increased stress concentration experienced by the columns tested with the 60 mm plate (i.e., 100 % foundation stress is concentrated on the column) leading to bulging to occur earlier in the test. Interestingly, the granite column installed on 17.03.21 in clay (that had an average undrained strength of 30 kPa; which exceeded the target strength of 15 – 25 kPa) did not bulge less than the other columns. This defied expectations as it was assumed the increased lateral support offered by the stronger host soil would reduce bulging.

6.2.11 Water Content of Aggregate Post-test and Toe Movement

6.2.11.1 Water Content of Aggregate Post-loading

The aggregates were compacted into the bore dry, hence the water content post testing was a result of water draining from the clay soil into the column (suggesting the clay was consolidating, although as noted previously the expected strength gain could not be confirmed with the facilities available). The water content within the column was determined by dividing the column into sections (the upper 150 mm and the lower 250 mm. This approach was adopted as it was thought that the top section of the column would bulge, leading to increased drainage (and as these sandy gravels were, relatively free draining, water would seep to the base of the column). Out of the 19 results available for the top 150 mm and bottom 250 mm post-test aggregate water contents 16 columns had greater values in the bottom half (see Tables 6.6 and 5.1-5.4). This is unsurprising as during the test water was able to seep vertically downwards through the column, this was also observed by Amini (2015).

Table 6.6: Post-Test aggregate moisture content and toe movement

Test date	Material	Load Plate Size (mm)	Water Content of all Column Material (%)	Water Content of Top 150 mm of Column Material (%)	Water Content of bottom 250 mm of Column Material (%)	Vertical movement (mm)	Foundation stress at 25 mm (kPa)	Initial Clay Strength (kPa)	Increase in diameter (at point of maximum bulge) (%)
25/02/2021	APCr	100	24.1	10.4	13.7	4	163	20	44
31/03/2021	APCr	100	25.1	11.7	13.4	3	173	22	22
21/01/2021	Container glass	100	4.9			19	122	18	5
19/10/2020	Container glass	100	-			5	-	-	7
27/01/2021	Container glass	100	0.3			3	96	20	10
02/02/2021	Container glass	100	2.8	0.4	2.8	12	109	20	16
04/03/2021	Container glass	100	0.6	0.2	0.4	15	107	20	21
13/01/2021	Granite	60	-			10	173	-	23
16/02/2021	Container Glass	60	5.9	1	4.9	10	141	20	20

In the case of the container glass columns (and to a lesser extent the window glass), on removal of the aggregate, there was clearly a greater amount of free water at the base than was observed for the other materials. During the process of removal, due to the non-absorbent nature of the glass, it is possible that some of this water was not recovered, and the recorded water content of the aggregate may be much lower than in actuality. Another possibility is that the water, that had drained vertically downwards, may have led to base softening. It would have been useful to take undrained shear strength readings of the base to determine whether or not this was the case. This was unfortunately not done and is something for future work. Certainly, the container glass columns experienced basial movements to a greater extent than those for the other materials.

Free water at the base on the column was not observed for the granite or APCr, it is presumed that various mechanisms were 'holding' water within the column. The first might be attributed to surface roughness, both granite and APCr particles were not smooth (indeed APCr have very rough, almost 'cratered', textures), promoting locations for the formations of suctions within the column and reducing the seepage of water towards the base. The second, in the case of the APCr, was the potential for water sorption (with APCr having a water absorption characteristic of 18.8 %). The difference in behaviour between the APCr/granite and the glass could be seen on completion of the test when excavating the column as a wet annulus of material could be seen around the perimeter of the APCr/granite columns, which was not seen for both types of glass. This was apparent along the length of the column and no free water was found at the base. The water content of the APCr post-test was the highest of all the materials. This is due to moisture being absorbed into the particle due to the porous nature of the material. The porosity of granite is very low (<1 %) so the absorption of moisture was likely much lower than for the APCr but the surface of the particles is very textured (unlike the glass) meaning that water was able to be held in these spaces. In the case of these tests, the sorption of the draining water by the outer perimeter of the aggregate may have been beneficial as the inner material was able to stay dry and so friction between the particles was unaffected (Figure 6.23). Whilst, in the case of the container glass it is likely that it was still the outer perimeter aggregate that was mainly affected (the water is more like to move vertically downwards before moving into the column), the presence of this drained water may have detrimentally affected the friction generated between the particles and this is indicated by the smearing of clay on the container glass aggregate post-test (see Figure 6.22) which was not observed for the other materials. In work by McKelvey et al. (2004), on the shear strength of recycled construction materials for use within vibro ground improvement, it was noted that there was a significant reduction in shear strength when particles were smeared with clay.

In addition, water pooling at the interface between column/host soil could result in localised softening (and in more active clays volume change) which might reduce the shear strength of the basal material and potentially result in more load transfer through the column 'shaft' and base, resulting in movements at the toe (considered further in the following subsection) rather than bulging and mobilisation of the passive strength of the host soil.   If these materials were to be used in practice, longer term studies would be required to assess the impact of water on column performance.

Figure 6-22: Clay smeared container glass aggregate after load test

Figure 6-23: Images of the columns post load. Wet material can be seen in the images of the granite (top) and APCr (centre) whilst the container glass appears dry (bottom).

6.2.11.2 Toe Movement

The columns were designed not to be end bearing through the geometry of the column (as they are floating, i.e., had no stronger strata below). However, some vertical movement at the toe was observed. In the 100 mm plate tests toe movement was observed for APCr and container glass, as summarised in Table 6.6.

The results of the container glass, in Table 6.6, show that there was no noticeable trend between base movement and column performance. Additional data is required to confirm this. Out of 6 APCr tests, toe movement was noted for two of the columns; these columns exhibited stress-displacement behaviour that was close to the average (see Figure 5.10, Chapter 5.0) further supporting the lack of correlation between toe movement and performance. However, these measurements were 3 and 4 mm so it is questionable that this was a reflection of actual movement and this could be due to irregularities in the level of the column base or surface of clay. Out of the eight 100 mm plate container glass columns, 3 samples exhibited more significant movements at the column base (12, 15 and 19 mm) and two reflected smaller movement of 3 and 5 mm (which like the APCr could be actual movements or, partially at least, uncertainty within data collection); all of these columns exhibited stress-displacement behaviour that was below average indicating that there may be a link between toe movement and performance.

It appears that generally the container glass columns exhibit greater toe deflection than for the other aggregates (Table 6.6). However, the link between aggregate water content post load test and toe movement is unclear as the column tested on 03.02.21 had a greater water content than other columns (5.1 %) yet did not experience any base deflection, this was also the case for the APCr columns. This lack of strong correlation can be seen in the data for the container glass (tables 6.6 and 5.1-5.4), where overall the column with the greatest water content did have the greatest toe movement (19 mm) but a column with a water content nearly 9 times lower exhibited a base movement of only 4 mm less (15 mm). The initial strength of the host soil for all columns that experience toe movement was 20 kPa so this is unlikely to be an influential factor.

These results suggest that there is a quantity of water that once reached will not lead to increased base softening with additional water (this is clearly only relevant to these specific tests and the associated time frame). Another factor to consider is the potential for the hydraulic conductivity of the individual column (and host soil) which will have affected how long water was able to pool at the column base and lead to softening, this is linked to PSD of the column material which for the container glass did vary (see figures 5.1c and 5.5 and 5.6).

6.3 Static Load Tests

The aim of this research was to compare the performance of model granular columns formed of RA and SA with a primary aggregate, granite. Tests were conducted using columns formed of granite, container glass, APCR along with no column tests to provide a baseline. Due to the limited amount of window glass available it was not tested under static loading. The static load test was based on the granular column tests described in the ICE Specification for Ground Treatment (1985). The test was designed so that the stresses applied were in the same range as those experienced by the columns tested under constant-rate-of-strain (CRS) to enable comparison of behaviour between the two loading types. However, due to restrictions on the amount of load that could safely be applied to the column the maximum foundation stress was limited at 187 kPa. Initially the overall performance of each of the materials, in terms of settlement under loading, will be discussed. Possible explanations of the behaviour identified will then be proposed using quality control data (i.e., strength of the host soil, column density etc.), PSDs, extent of particle breakage and extent of column bulging. [Please refer to Chapter 5.0 for the relevant data, additional data and further interpretation will be included within this section, and Section 3.7.2 for the methodology used in the static tests].

6.3.1 Summary of Column Performance of Tests in Terms of Measured Settlement

The results shown in figures 6.24 to 6.26 indicate that at foundation stresses of 37 kPa the measured settlements for samples with and without columns are similar (note that there is no data for the 'no column' tests after 300 minutes as the settlement exceeded 100 mm which was the limit of the instrumentation). However, as presented in Table 6.7, it can be seen that at the greatest stress value (187 kPa) the installation of columns does lead to settlement reduction, compared to the unreinforced clay, despite columns being made using RA/SA.

The greatest settlement reduction is achieved by the PA, granite, however, the APCr columns achieved an average 55 % reduction. The container glass columns performed less well but still achieved a significant average settlement reduction of 30%. Settlement reductions of 30-50 % following the installation of stone columns are expected (Douglas, et al. 2015). It is apparent that the average performance of all columns indicate that they fall into this range. However, container glass 02 falls short. Further tests are required to determine whether this behaviour is a true reflection of the variable nature of RA or if this dip in performance was by other factors.

It should be noted that just as piles require some displacement to mobilise skin friction GCs do require some load to be applied to gain full capacity of the column. However, this is due to material not being as well compacted towards to the top of the column; Serridge and Slocombe (2012) advise that foundations should be placed at a minimum depth of 600 mm below the level from which granular columns were installed to fully realise the enhanced performance due to the aggregate at the top of the column not being as compact as at greater depths.

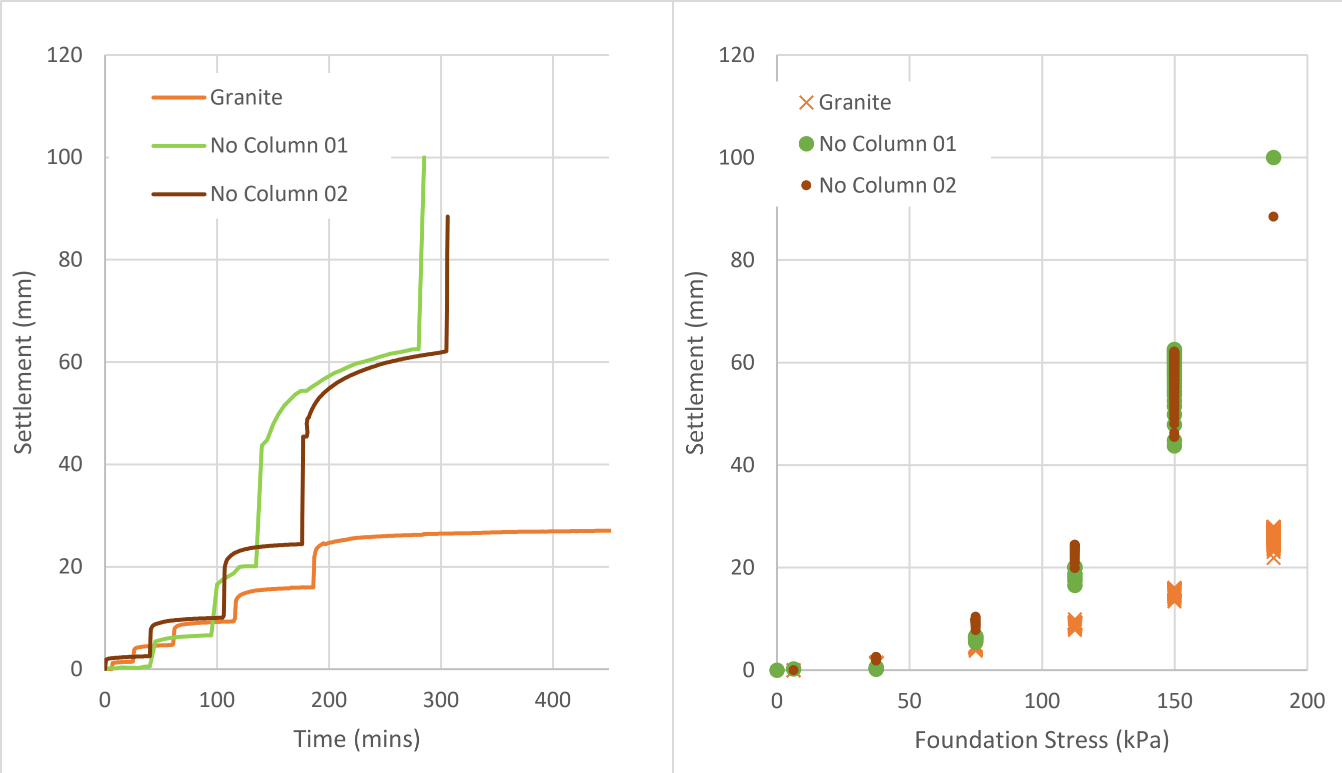


Figure 6-24: Static load test for granite. Relationship between settlement and time (left) for load increments (applied once a maximum settlement of 0.5 mm/hr was reached) and relationship between settlement and load applied (right)

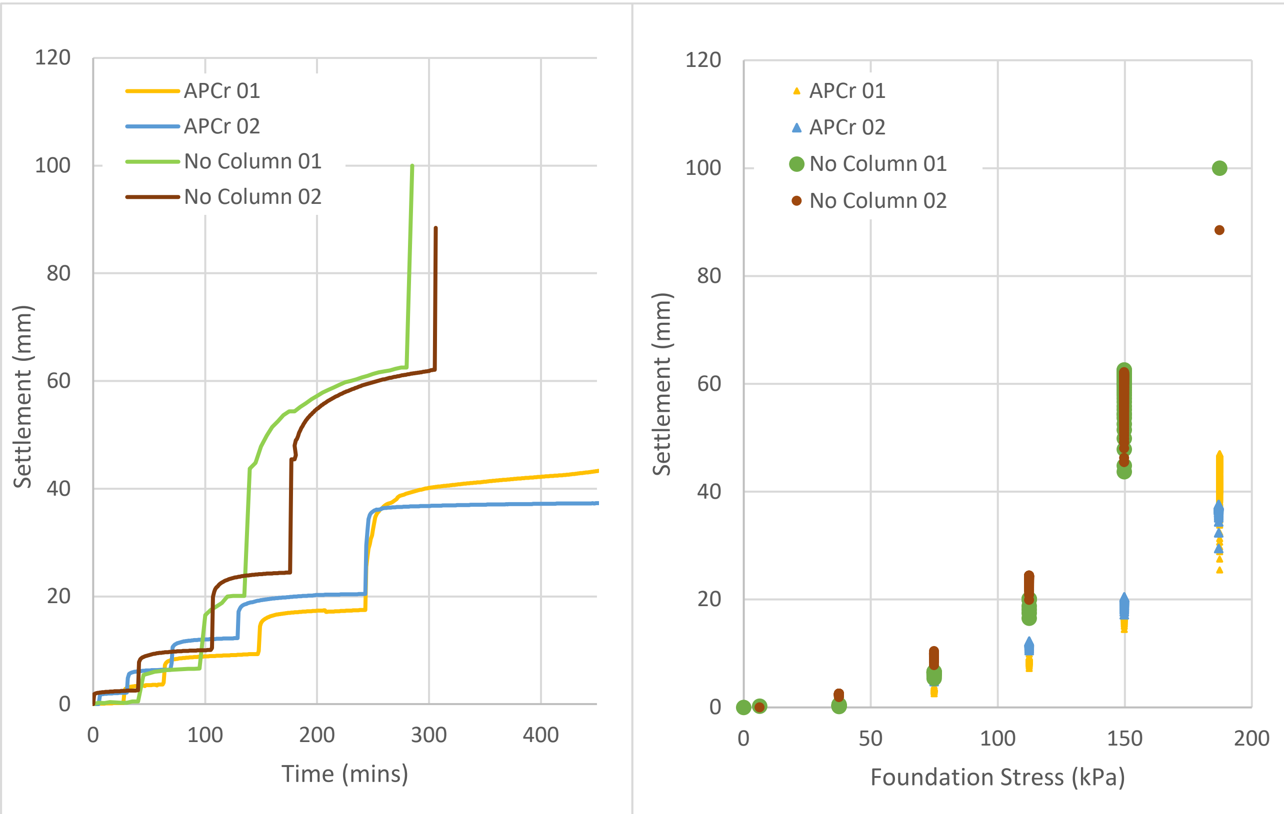


Figure 6-25: Static load test for APCr. Relationship between settlement and time (left) for load increments (applied once a maximum settlement of 0.5 mm/hr was reached) and relationship between settlement and load applied (right)

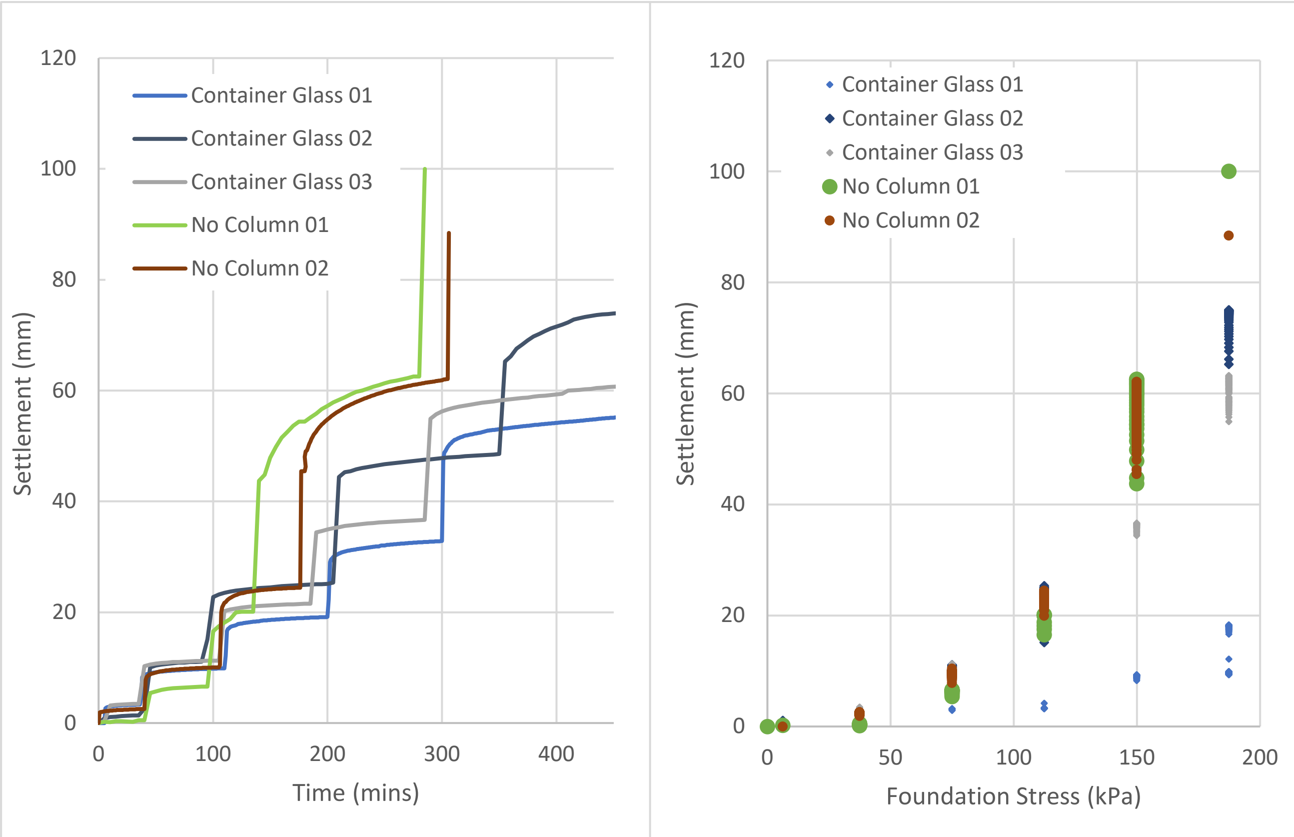


Figure 6-26: Static load test for container glass. Relationship between settlement and time (left) for load increments (applied once a maximum settlement of 0.5 mm/hr was reached) and relationship between settlement and load applied (right)

Table 6.7: Summary of maximum measured settlement for all column materials

Foundation Stress (kPa)	**Maximum measured settlement for load increment (mm)**							
	Material							
	No column 01	No Column 02	Granite	Container Glass 01	Container Glass 02	Container Glass 03	APCr 01	APCr 02
6	0	0	0	0	1	0	0	0
37	1	3	1	3	3	3	0	2
74	7	10	5	10	11	11	4	8
112	20	23	10	19	25	22	10	12
149	63	61	16	33	49	37	18	20
187	100	94	28	61	75	61	47	38

Table 6.8: Settlement reduction of columns

	No column 01	No Column 02	Granite	Container Glass 01	Container Glass 02	Container Glass 03	APCr 01	APCr 02
Settlement reduction (%)*			70	36	20	35	50	60
Improvement Factor			3.4	1.5	1.25	1.5	2	2.5

* based on the most conservative measurement (i.e. lowest value) of settlement for samples without columns (94mm)

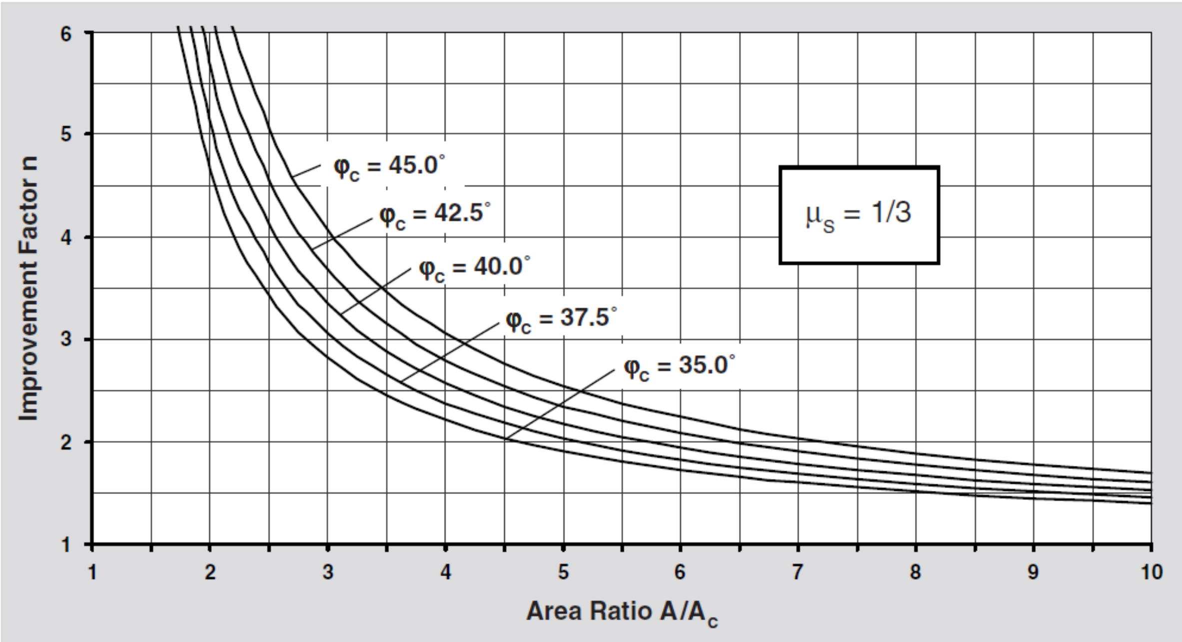


Figure 6-27: Design chart for vibro replacement (granular columns) (Priebe, 1995)

According to Figure 6.27, Priebe (1995) estimated that for an area ratio (A/Ac) of 2.8 (36 % soil replacement) columns formed with a material with an angle of friction of 35°, the improvement factor (I_f), defined as settlement without treatment/settlement with treatment) would be 3 and I_f would be 4.2 for materials with friction angles of 45°. This estimation was based on columns founded on a rigid base and on the assumption that columns are incompressible which does not exactly match the conditions of this research. However, the results for the PA, granite, seem to tie in well with this estimation. The results for the APCr (ϕ=35°), which reflected an average n value 2.2 also fit well with this estimation. The slightly lower friction angle of the container glass, (ϕ=32°), is not represented on the curves drawn by Priebe (2005) and when utilising the curve of the closest value of phi on Priebe's chart (ϕ=35°), Priebe's estimation appears to be overly optimistic, a finding echoed by Sivakumar et al. (2011). It is difficult to say conclusively whether in the case of this research Priebe's prediction of improvement factor is applicable due to the low friction angle of the container glass not being specifically included. Sivakumar et al. (2011) found the improvement factor of a 60 mm diameter column to be 4.8 (using basalt column aggregate), although no direct comparison can be made with this study as Sivakumar et al's column was tested within a 300 mm triaxial cell and was founded on a rigid boundary. Thus, the improvement factor of the statically loaded granite column in this research, of 3.4, could be comparable when factoring in the more favourable conditions of the basalt column in Sivakumar et al's (2011) work (although this should be confirmed with additional research). In work conducted by Barksdale and Bacchus (1984) the improvement factor value for sand columns with the same area replacement ratio as adopted within this research was 1.8, therefore indicating that the container glass columns (in this study the material is a sandy gravel) are more comparable in performance to sand columns.

6.3.2 Difference in the Performance Between Materials

On reviewing the measured settlements, it is apparent that the granite performs the best out of the three materials, followed by APCr and then container glass. In this section the possible reasons for this outcome will be explored.

Table 6.9:Time taken for rate of settlement to reduce 0.5mm/hr

Foundation Stress (kPa)	Time taken for settlement to reduce to 0.5mm/hr (mins)					
	Material					
	Granite	APCr 01	APCr 02	Container Glass 01	Container Glass 02	Container Glass 03
6	10	5	5	10	10	10
37	15	25	25	25	30	25
74	30	35	30	70	50	65
112	50	70	55	85	110	75
149	68	58	74	95	140	95
187	57	300	40	325	115	270

The initial strength of the clay was consistent (20-23 kPa) (Table 5.6), enabling fair comparisons to be drawn between the granite and RA/SA columns.

6.3.2.1 Column Performance: Granite

Only one test was conducted using granite (due to time limitations) it is not possible to discuss reasons for variation in performance due to factors such as column density, initial clay strength etc. However, it is possible to comment on the overall performance of the column under loading. Not only did the granite achieve the greatest settlement reduction it also arrested settlement at the fastest rate, as shown in Table 6.9. This superior performance of the primary aggregate, given the strength of the material, the high angle of friction (51°) and angular particle shape, is unsurprising. This was not the finding of the CRS testing, where window glass and APCr arguably outperformed the granite, suggesting there are some potential limitations in the CRS approach. However, the CRS and static loading approach are compared later in this discussion (Section 6.6) and the observed behaviour in both testing arrangements are very similar. Thus, perhaps it is not a limitation in experimental approach just that there are limited datasets; the static datasets are too small to provide upper and lower bounds or mean responses and this granite test could be a particularly strong column. The impact of the Covid pandemic limited the number of tests that could be completed and additional research in this area could be beneficial.

6.3.2.1.1 Water Content of the Aggregate Post Load

The water content of the aggregate post load was approximately 0.9 %, and there is little difference between the water measured for the top 150 mm and bottom 250 mm of the column suggesting that drainage occurs evenly along the length of column (Table 5.6). The water content is greater than for the glass which can be explained by the ability of granite to sorb water, unlike the glass. The water content is more than 10 times smaller than for the APCr, which is likely to be due to the lower sorbency of granite. Another possible explanation for this is the reduced extent of radial expansion observed for the granite, in comparison to the APCr, leading to less drainage into the column taking place. This will be discussed later in the chapter.

6.3.2.1.2 Column Shape

The post-test model created of the column shape indicates that the extent of bulging was minimal compared to the other materials. This could be due to the high degree of particle interlock of the granite (due to very angular shape, rough texture and relatively high angle of friction). However, as there was only one granite column investigated in static loading, and the casting material used was different than for all the other models, this column shape may be misleading, and this will be compared with the constant-rate-of-strain (CRS) tests later in the chapter to enable firmer conclusions to be drawn.

6.3.2.1.3 PSD

According to Figure 6.27 the material appears changed but as the PSDs show less particles passing, and this cannot be the case, the most logical explanation is that particles being left behind in the bore (e.g., pressed into the sides) have led to a misleading PSD. Unfortunately, as only one granite column was loaded statically there is no other data to draw comparisons with.

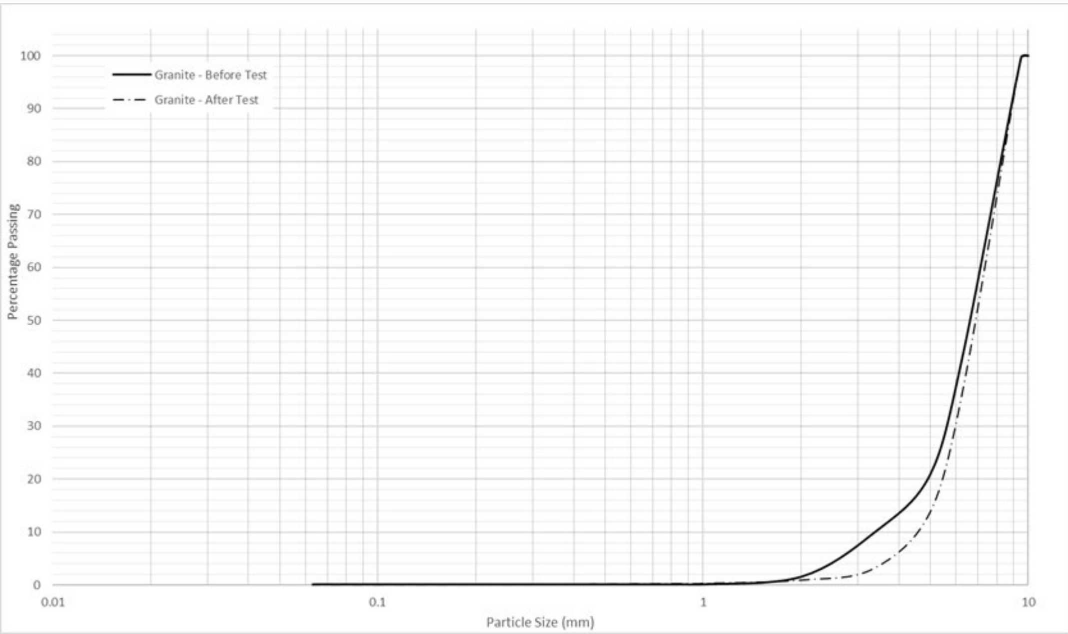


Figure 6-28: Granite PSD before and after static load tests

6.3.2.2 Column performance: Container Glass

6.3.2.2.1 Clay Strength Pre and Post Test

The water content for all three samples of clay used for the container glass tests is consistent at 42 % (the greatest variation between samples is 0.3 %) therefore removing the potential of clay water content being an influential factor for this particular set of tests. However, the initial strength of the clay does vary by 3 kPa, indicating the potential for discrepancies due to the clay compaction process or limitations of the hand shear vane apparatus. Tests 01 and 02 have the same initial clay strength and tests 02 and 03 have very similar column densities (see Table 5.6).

The post load clay strength increase is consistent at 2 kPa for columns 01 and 03 but unexpectedly reduces for column 02. However, container glass 02 column exhibits a greater extent of column bulging than the other two samples and this is a strong possible explanation for the apparent 2 kPa loss of strength post load. To explain, post strength load readings were taken at a distance 1.5D from the column centre i.e., 60 mm from the edge of the column and in the case of column 02 bulging was measured to extend to a radius of approximately 80 mm which could have affected the reading of the hand shear vane. It is appreciated that a 2 kPa difference could well be attributed to noise within the hand shear vane measurements as this is not a particularly accurate method to measure shear strength, hence it if difficult to state with any certainty that the host soil had been strengthened during these tests.

6.3.2.2.2 Column Shape

The post load shape of column 03 appears anomalous as it features a projection at a depth of around 280 to 350 mm (as shown in Figure 6.29) which was not seen in any other column test. This quirk could be explained by the presence of a void in the clay being filled with aggregate, however, the integrity of the bore was checked prior to the installation of column material for every test so it is surprising that this wasn’t noticed. Another explanation could be the presence of a pocket of softer soil which could not have been detected visually. This feature could have contributed to settlement reduction, or this could be attributed to the apparent greater overall diameter of the column.

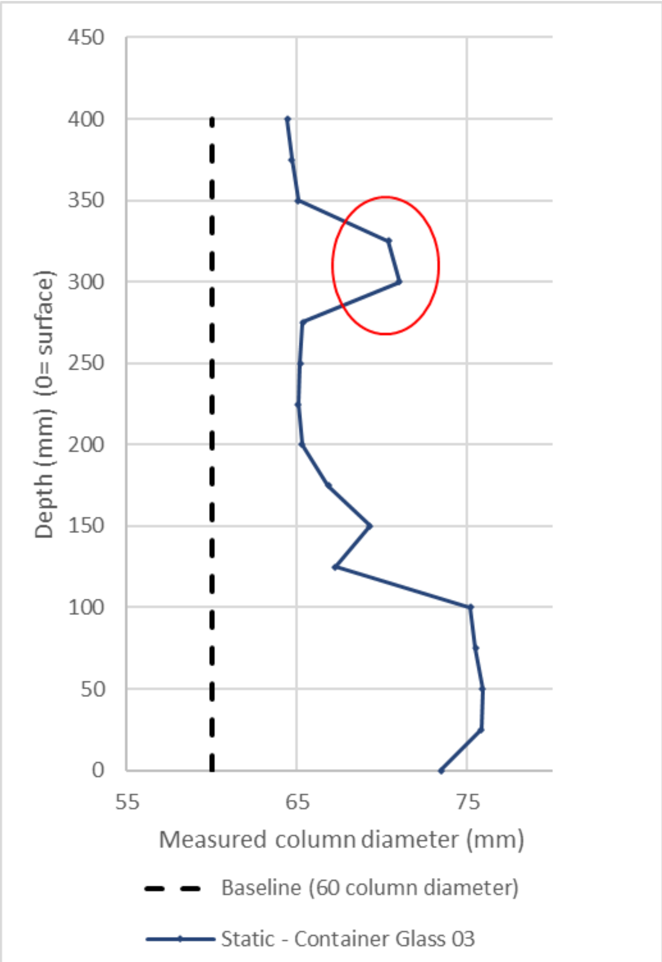


Figure 6-29: Post load column shape of container glass 03

6.3.2.3 Column Density

Of the three container glass columns, column 01 performed the best in terms of settlement reduction. The density of column 01 was 1524 kg/m^3, 6 % denser than columns 02 and 5 % denser than column 03. Perhaps this is the reason for the slightly better performance, but this doesn't offer an explanation with a great degree of confidence as overall columns 01 and 02 perform most similarly and the greatest discrepancy in performance is with column 02.

Based on the analysis of the results clay strength, clay water content and column density do not provide satisfactory explanations for discrepancies in performance, although that said, for this particular set of tests behaviour did not vary greatly. In the next section PSD and extent of particle breakage will be reviewed to determine whether this offers further insight into column behaviour.

6.3.2.3.1 PSDs

As shown in Fig. 6.29 the original PSDs of container glass 02 and 03 are almost identical and the PSD of sample 01 follows a similar curve but features a greater number of smaller particles. However, no obvious link can be made between the initial PSD and column behaviour under loading.

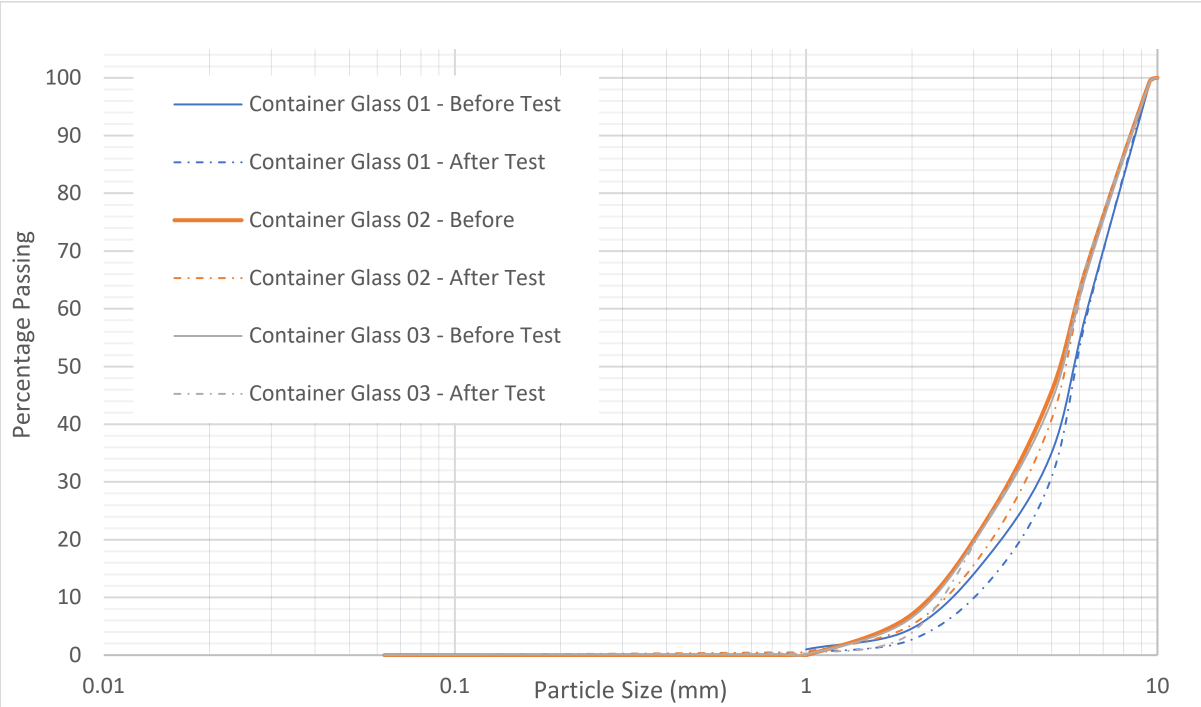


Figure 6-30: PSDs of container glass samples before and after testing

6.3.2.3.2 Particle Breakage

Prior to load testing PSD tests were conducted for all materials and this was repeated after loading. This was deemed an important aspect to consider not only due to concerns in industry over the particle breakage of RA/SA, particularly waste glass (Serridge and Sarsby, 2009) but also to enable some conclusions to be drawn on the interaction and overall behaviour of the aggregate particles under load. Post-load testing, the aggregates were divided into the top 150mm (the region of the column that is most subject to column bulging) and the bottom 250mm.

It is important to note that whilst every effort was made to recover all column material after testing this wasn't possible due to some material being pressed into the clay. Another possible issue is very fine material being lost during the vacuuming process.

During the testing of the container glass columns 01 and 03 what was perceived as particle crushing was heard when stresses of 149 and 187 kPa were applied, this audible crushing was heard after the application of load for approximately 5 minutes. In the case of container glass 02 crushing was only heard at 187 kPa.

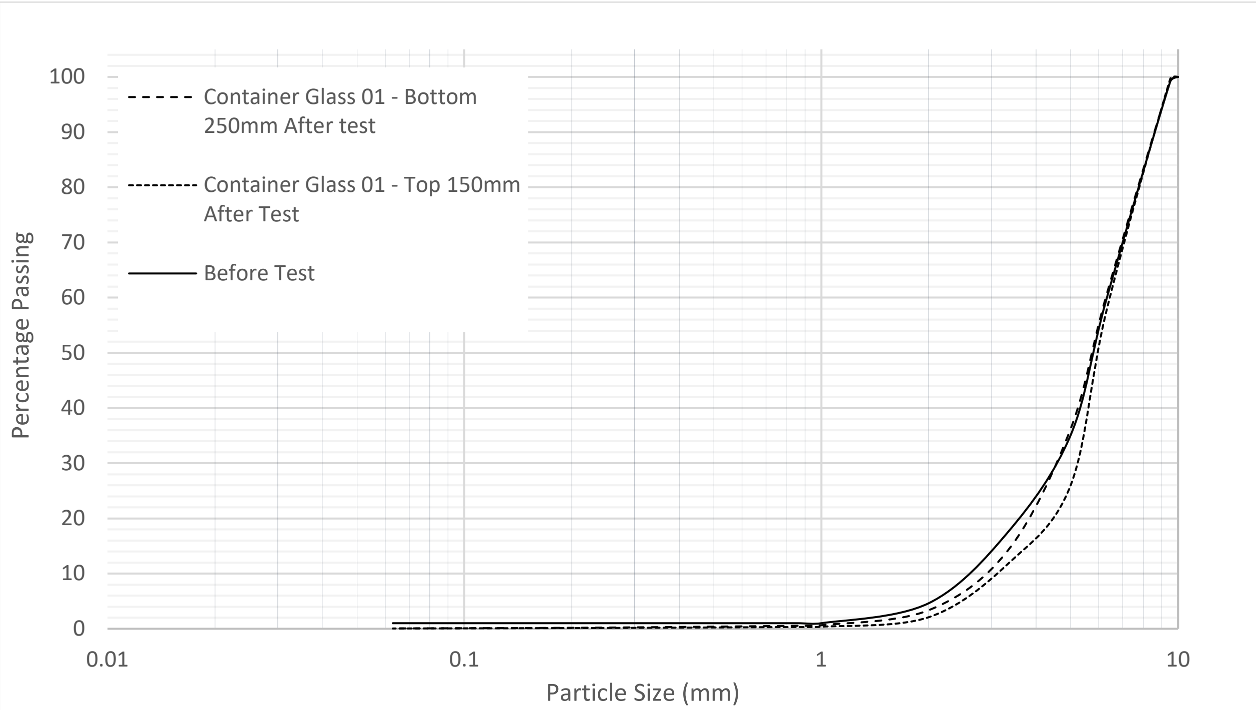


Figure 6-31: PSD of container glass 01 before and after test

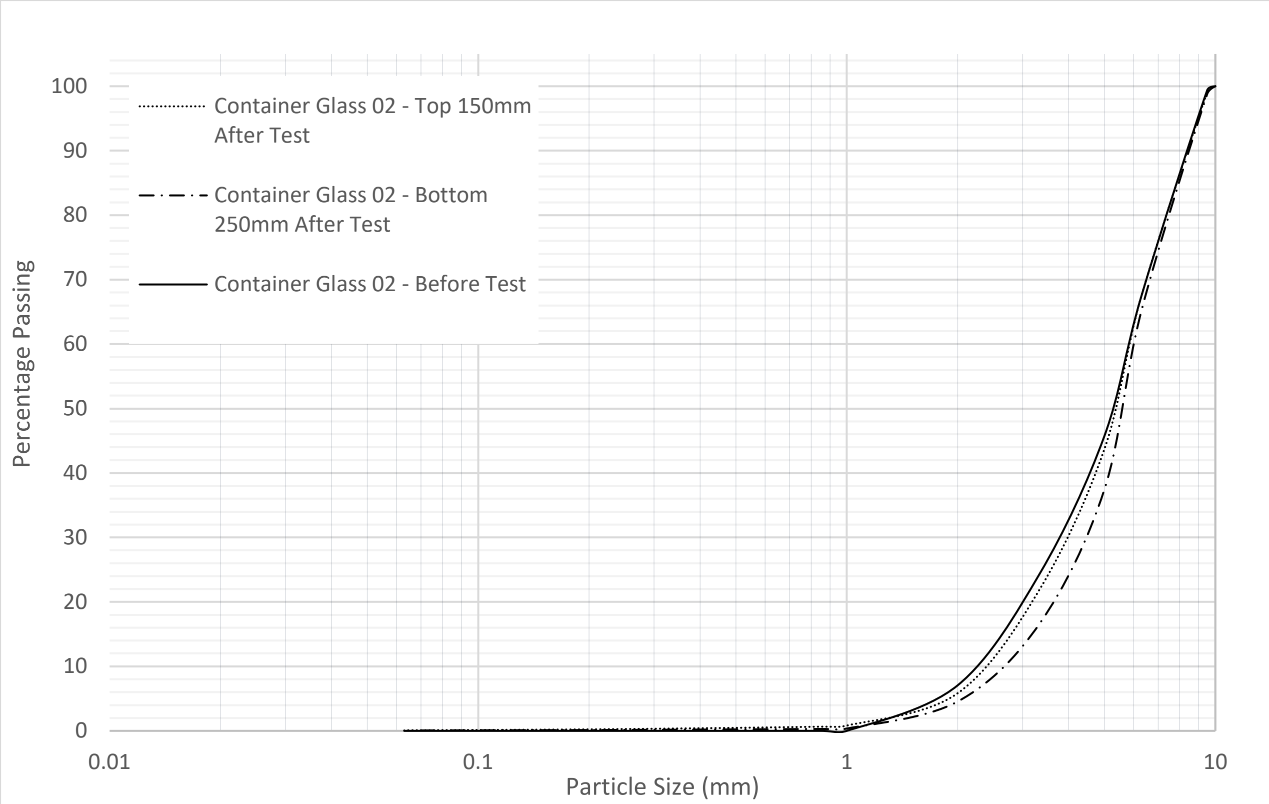


Figure 6-32: PSD of container glass 02 before and after test

The PSD of the top 150mm of aggregate appears to be mostly unchanged after loading (figures 6.31and 6.32). The area directly under the foundation plate (i.e., the top 150mm of aggregate) would be the most likely to break during the transfer of load from the plate to the column but it appears that this does not happen. The particles rearrange and move outwards (i.e., 'bulge') rather than break, as expected, under loading. The PSD of the material in the bottom half of the column appears changed but as the PSDs show less particles passing, and this cannot be the case, the most logical explanation is that particles being left behind in the bore (e.g., pressed into the sides) have led to a misleading PSD. It is disappointing that the data does not enable analysis of the differences between the PSDs of the top 150 mm and bottom 250mm column, potentially leading to some conclusions on load transfer but it is very encouraging that container glass particles do not appear to break en masse despite them all being categorised as 'flaky'.

Some researchers have activity sought to avoid particle breakage in the context of granular columns (Ghazavi and Nazari Afshar, 2013) and this is echoed in industry (Serridge and Sarsby, 2009). This nervousness of particle breakage is understandable in certain contexts but in a confined environment, such as within granular columns, should this be of as much concern? Typically, for the purpose of construction materials, flaky and elongated particles have been avoided due to their poor performance in both index testing and in service (Lees and Kennedy, 1975; BRE, 2000). However, the potential densification, occurring due to the creation of smaller fragments filling the spaces within the aggregate structure, may lead to additional points of inter-particle contact within the structure leading to greater interlock between particles and as long as the additional number of contact points, formed by degradation, cause less impact than the effect of reduced interlock (due the breakdown of particles) stability will be increased (Lees and Kennedy, 1975). Highlighting again the importance of the context of material application and the caution required to be exercised when relying solely on index tests.

6.3.2.4 Column Performance: APCr

In both APCr column tests settlement reduction is comparable up to 187 kPa where column 02 outperforms column 01 by a further 11 mm reduction in settlement and controlling settlement, at the largest application of load, 7.5 times faster.

6.3.2.4.1 Initial Clay Water Content and Strength

The host soil for column 02 has an initial water content that is 0.5 % higher and an initial clay strength that 2kPa lower than that of column 01 and yet still outperforms it. This result suggests that it is something other than the initial host soil condition that have a greater influence on performance, at least for small variations.

6.3.2.4.2 APCr Material Properties

It is well recognised that in the context of granular columns the angle of friction of the material is a key factor in the performance of the material. However, in this research APCr was estimated to have an average value of 35°, over 10 degrees lower than the value estimated for the granite. The cautionary comment made by Serridge (2005) suggested that a 5 ° reduction in friction angle could lead to up to a 25 % reduction in the settlement reduction factor; this does not appear to be the case for APCr.

6.3.2.4.3 Water Content of Aggregates Post Load

For both samples of APCr the water content of the aggregate measured after testing was over 10 times greater than for the other materials. One possible explanation for this is the porous nature of the material which differs greatly to the smooth surface of the glass where water is more likely pass through the material rather than be sorbed. The water content of the bottom 250 mm of the column is slightly higher (in the region of 1-1.5%) for both tests which can likely be explained by the effects of gravity. The similarity in water contents of the top and bottom halves of the column indicates the drainage of water from the clay occurs throughout the whole column. It would be of interest to assess the impact of this absorption of water over time to determine whether this has any negative impact on the capacity of the column. Both columns appear to bulge to the same extent and at the same depth and the behaviour of the material is comparable to both the granite and container glass columns.

Figure 6-30: APCr after aggregate impact (AIV) test

Figure 6.33 shows a sample of APCr after the aggregate impact test. Although the conditions of the aggregate impact test differ (rigid confining container and a total load of 400 kN, applied in one-minute increments of 40 kN) it presents a useful illustration of how the materials performs under compressive loading. The particles, at the surface at least, have clearly been crushed but appear to have formed a denser, more closely interlocking material. This type of behaviour was observed when removing the column aggregate post static load test, the material was found to be so tightly packed it was required to be loosened by gently probing with a rod before it was able to be removed by vacuum.

6.3.2.4.4 PSD

The initial PSD curves for APCr samples 01 and 02, as shown in Fig. 6.34, appear to vary to a greater extent than for container glass and granite. This is consistent with observations of the variability of the material, in terms of the size fraction, made in the lab. The material provided came from multiple waste streams, so variations were expected.

6.3.2.4.5 Particle Breakage

The PSD curves of the material pre and post load, see figure 6.34, indicates that in the case of APCr 02 that the crushing of particles does occur but is not as great as would be expected from the results of the index tests. This is supported by looking more closely at the change in the percentage of particles retained where it is apparent mostly particles sized between 6.3 -1 mm that are affected.

The amount of fines generated post-test appears to be negligible, but this contrary to the observations made post-test where a significant amount of finer particles were seen. This discrepancy is likely due to material being lost during the removal process, either being left behind in the host soil or lost within the vacuum cleaner. The relatively small amount of breakage recorded, that could be heard when stresses of 149 and 187 kPa were added, may have occurred mostly at the surface as the foundation plate transferred load to the column.

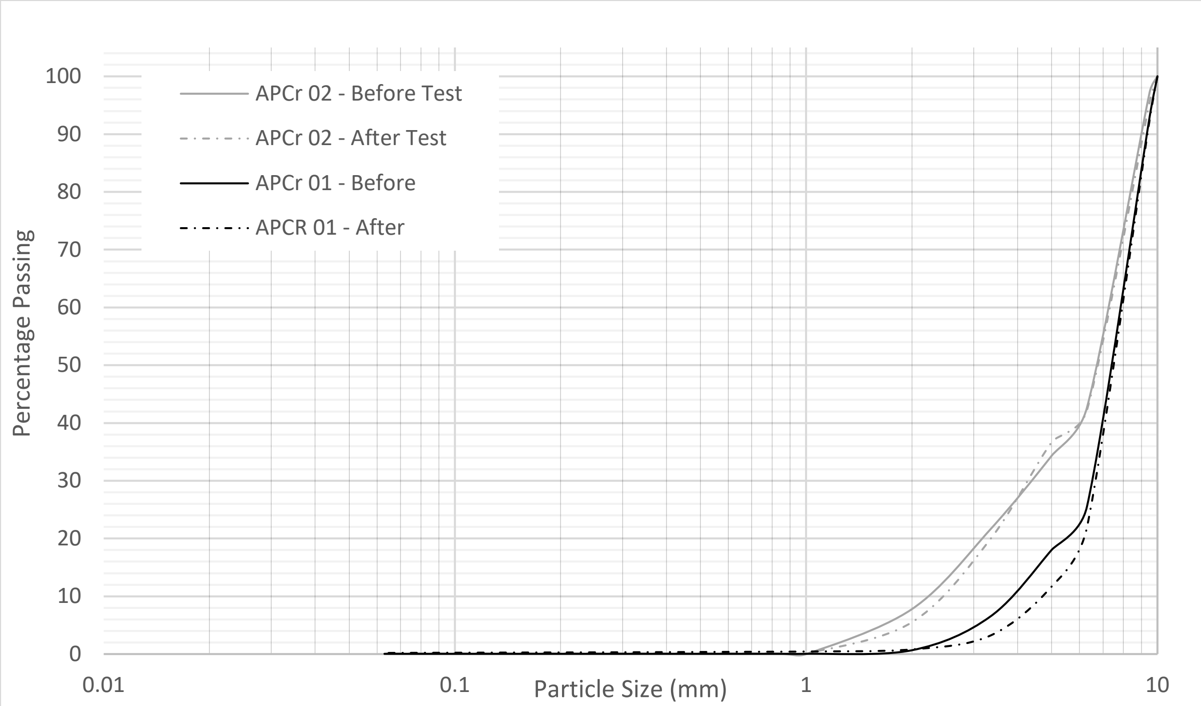


Figure 6-34: PSD curves of APCr column material before and after load test for tests 01 and 02

This apparent lack of mass particle breakage is unexpected when considering the behaviour of the material during the proctor compaction, LA (see Figure 6.35), AIV and ACV tests where the material did appear to crush readily. When considering that the material has an individual pellet compressive strength of 280 kPa (Gunning and Hills, 2014) the lack of apparent particle breakage is not as surprising. In addition, within the context of a granular column the aggregate is able to move laterally into the clay (rather than into rigid boundary such as the steel container of the AIV test) which is less likely to lead to particle breakage.

Figure 6-31: APCr after LA testing

These observations lead to the predictable conclusion that it is the type of loading that the material is to be exposed to that is key and highlights the caution, as expressed by (Schouenborg *et al.*, 2004), that should to be taken when relying solely on the recommended index tests to determine material suitability for granular columns or other applications.

6.3.2.5 General Observations: Clay Water Content and Strength Pre and Post-Test

6.3.2.5.1 Clay Strength

The post-test clay strength was measured at two locations in an attempt to determine to what extent the column influences the surrounding clay strength. It is accepted that within a region of 2.5D the column exerts influence on the surrounding soils (Hughes and Withers, 1984) and it would be logical to assume that the closer to the column the greater the impact. The two measuring locations were selected for practical reasons, 1.5D the closest that could feasibly be measured without disturbing the column in situ (and potentially changing its shape and losing the opportunity to assess post-load column shape) also if soil strength was measured directly on the boundary between the column and the clay this is would not have been representative of the clay strength. The measurement at 2.5D was taken as this was as close to the container boundary that could practically be measured.

Reviewing the pre- and post-test clay strengths (see Table 5.6) there does appear to be a correlation between clay strength and proximity to the column; all 1.5D clay strengths exceed 2.5D strengths by 1-3 kPa. This pattern is not observed in the case of container glass 02 but, as mentioned previously, the 1.5D clay strength may have been affected by the bulging of the column. However, as mentioned in the discussion on the CRS tests these variations in soil strengths may be due to the limitations of the hand shear vane and may be due to noise. Whilst inconclusive due to the possible misleading readings of the hand shear vane apparatus, this finding is unsurprising as it would be expected that some stiffening of the clay surrounding the column would have occurred due lateral bulging of the column and the drainage of water.

It was not possible to take readings throughout the compact clay sample prior to conducting a load test without disturbing it and potentially affecting the column tests; in order to confirm this finding baseline tests would need to be conducted where a test cell was filled with clay using the same methodology as for the column tests and water content measurements taken at 1.5D and 2.5D.

6.3.2.6 Water Content

The water content of the clay surrounding the container glass columns appears unchanged but in the case of APCr columns there is a more notable reduction (approximately 2 %) in water content post-test. This can be explained by the porous nature of the aggregate which could potentially draw water from the clay unlike the container glass which has flat and smooth surfaces. This change in water content is confirmed by the high post-test water content of APCr. Unfortunately, no post-test data was available for the granite; it would have been of interest to note if similar behaviour was observed. This tendency for the APCr to draw water would merit further investigation as whilst in these model tests it performed well and comparably to granite, any potential for material degradation due to water would need to be ruled out before this material could be used in practice.

6.3.3 Summary of Static Load Tests

Based on the analysis of the static load test results and the potentially influential factors of particle shape, angle of friction, initial clay strength, column density and aggregate properties no strong links have emerged. The PA, granite, achieved the best performance in terms of settlement reduction (70 %) (when compared to a clay sample not containing a column) and this was followed by APCR (55 %). This was a surprising finding given the performance of APCr in the index tests. The container glass achieved a settlement reduction of 30 % which whilst less than the other materials is not insignificant.

The analysis of the pre and post-test PSD was hindered to some extent by the challenge faced when attempting to retrieve 100 % of the column aggregate from the clay but some observations were able to be made. A small amount of particle breakage may happen when utilising AAs, as it could when using PAs, but this does not occur to the extent that might be imagined, as reflected in the minimal particle breakage observed within the container glass columns and does not necessarily mean these materials are unsuitable. The index tests currently used to predict to potential for particle breakage appear to be limited in this regard.

The results of the aggregate index tests recommended to assess materials to be used for granular columns such as AIV, ACV and the LA tests, collected for the materials in this research do indicate to some extent how materials will perform. Based the AIV and ACV tests all materials would be considered acceptable for GA construction but the results for the AA are marginal. Considering the results of the AIV, ACV and LA tests the granite outperforms all of the other materials and this is reflected in the overall superior performance of the granite in the column tests. However, the results of the index tests for container glass and APCr are comparable and yet their performance in the model column tests are not (improvement factor APCR = 2.2, container glass = 1.5). This suggests that the index tests do not reflect the full picture which is not surprising considering how different, particularly in the case of the LA test, the tests are to the environment the aggregates experience within granular columns, as echoed by other researchers (Schouenborg, 2005; Chidiroglou, O'Flaherty and Goodwin, 2009; Serridge and Sarsby, 2010; Amini, 2015; Shahverdi, 2020).

The static load test results discussed in this section echoed the comment made by Serridge (2005) on the importance of recognising that some recycled aggregates perform better than would be anticipated from some laboratory tests. These results are encouraging but further work is needed to gain the necessary confidence for them to applied in practice.

6.4 Static Load/CRS Tests

6.4.1 Comparison Between CRS and Static Load Tests

Static load and constant rate of strain (CRS) tests were undertaken on samples with/without a column; and the datasets for these are presented in Chapter 4.0. The methods for these tests clearly vary: CRS are quick (comparatively) but do not appear to replicate *in situ* loading as the load plate is forced into the soil at a constant rate; whereas static loading is perhaps a better proxy for modelling in situ response but is more time consuming. In practice groups of GCs are load tested using the 'zone test' approach, as detailed in the ICE Specification for Ground Treatment (1987), which comprises the increase of load in three steps to a maximum of working load plus 25%. CRS testing was used (as a consistent approach) to allow for the creation of a dataset with the different column materials to allow for a comparison of behaviour and assess potential suitability of the AA in GC. A small number of static tests (limited in number due to the situation faced during this study) to investigate if this approach results in a significantly different response (as the soil/column forms an equilibrium when experiencing an external load that was below the ultimate load).

The question is how to compare them, and two simple approximations were adopted here. The first was to consider the relative 'Improvement Factors' for the host soil with the inclusion of a GC (comprising AA or PA). The second was to convert the static loading into a pair of coordinates (stress and strain) from the settlement with time behaviour (for a constant load). Static loading results in non-linear settlements with time (for a constant load) that approach an asymptote (in the short term; it does not take into account secondary consolidation which was not considered herein) and this was assumed to be the threshold when the settlement was less than 0.5 mm per hour (Section 3.7.2). Taking the load applied and the strain at which this settlement threshold was first observed provides a pair of coordinates, plotting the coordinates for the five load steps applied allows for a 'quasi' stress-strain response that could be plotted against the stress-strain response from the CRS.

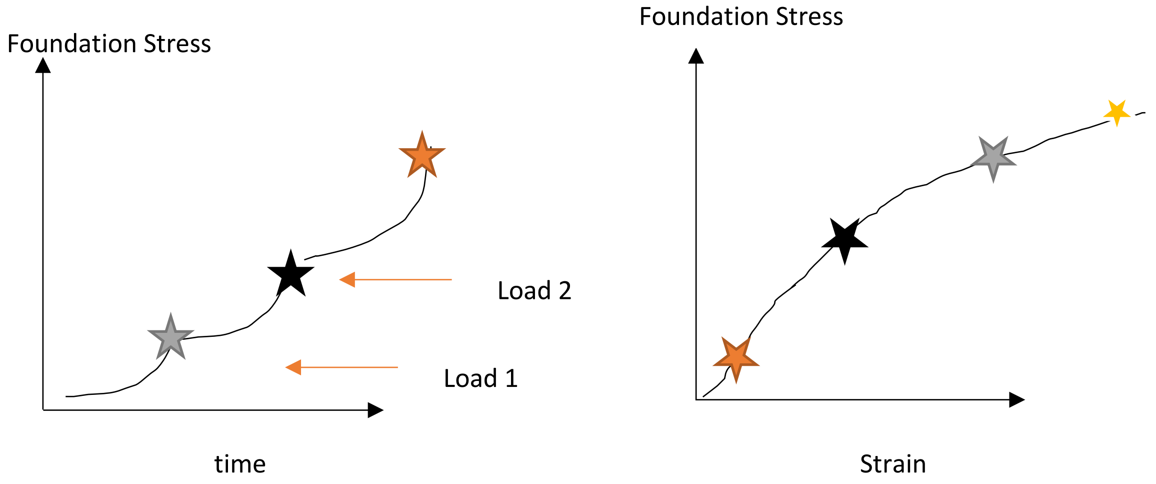


Figure 6-32: Illustration of static and CRS tests

6.4.2 Column Behaviour and Improvement Factors

A comparison of behaviour using improvement factors (I_f) based on foundation stress achieved in reinforced soil/foundation stress achieved in unreinforced are presented in Table 6.10.

It is very difficult to compare the Improvement factors for each test (i.e., static/CRS) as in the case of the CRS the recorded vertical deflection is not true settlement and for the static load tests the range of settlement experienced at each load increment is very large in comparison the CRS tests. Using the results shown in figures 6.36-6.39 an improvement factor was estimated based on foundation stress achieved at a settlement of 5 % (or 25 mm), see Table 6.10. The If factors for each material appear to be comparable for the static and CRS testing which is an interesting finding given the difference between the loading (both in terms on load application and time scale).

Table 6.10: Improvement Factors

Improvement Factors	CRS 60mm*	CRS 100mm*	Static 100mm*
APCr		2.0	2.2
Granite	3.3	1.8	2.0
Container Glass	3.0	1.5	1.5
Window glass		2.0	

**Based on foundation stress, recorded at 25 mm settlement (or 5 % strain), with column/foundation stress achieved without column*

6.4.3 Quasi Force-Displacement (or Stress-strain) Comparison for Static and CRS Testing

The purpose of running these two stypes of tests (CRS and static) was to enable comparisons to be drawn between the two loading methods and to attempt to assess what impact adopting a constant rate of strain approach to laod application has on column performance. It would be misleading to simply plot foundation stress against settlement together for both tests as previously noted the settlement is simply a function of the test apparatus in the CRS. To overcome this snap shots taken at particular values of stress (i.e., each static load step to enable direct comparison) and the recorded settlement at that stress value have been plotted (as described prevously, Figure 6.35).

The results of the static load tests show that column performance, based on measured settlement, reduces as the angle of friction of the column aggregate decreases (i.e., granite – 51 °, APCR – 35°, container glass – 31 °) which aligns with the literature (Serridge, 2005). However, the results of the CRS, based on foundation stress, do not echo this pattern. The column that achieved the greatest foundation stress was formed of APCr and this was followed by granite and window glass (which performed comparably based on the average results) and lastly container glass. This was surprising, APCr did not appear to perform particularly well in the Phase 1 'standard' geotechnical engineering and aggregate performance index tests (Chapter 4.0) yet does appear to perform well when utilised within the model GCs.

The figures below (6.36 to 6.39) show recorded values of foundation stress and settlement for both the stratic and CRS tests for each of the column materials trialled (no colum, granite, APCr and Container Glass).

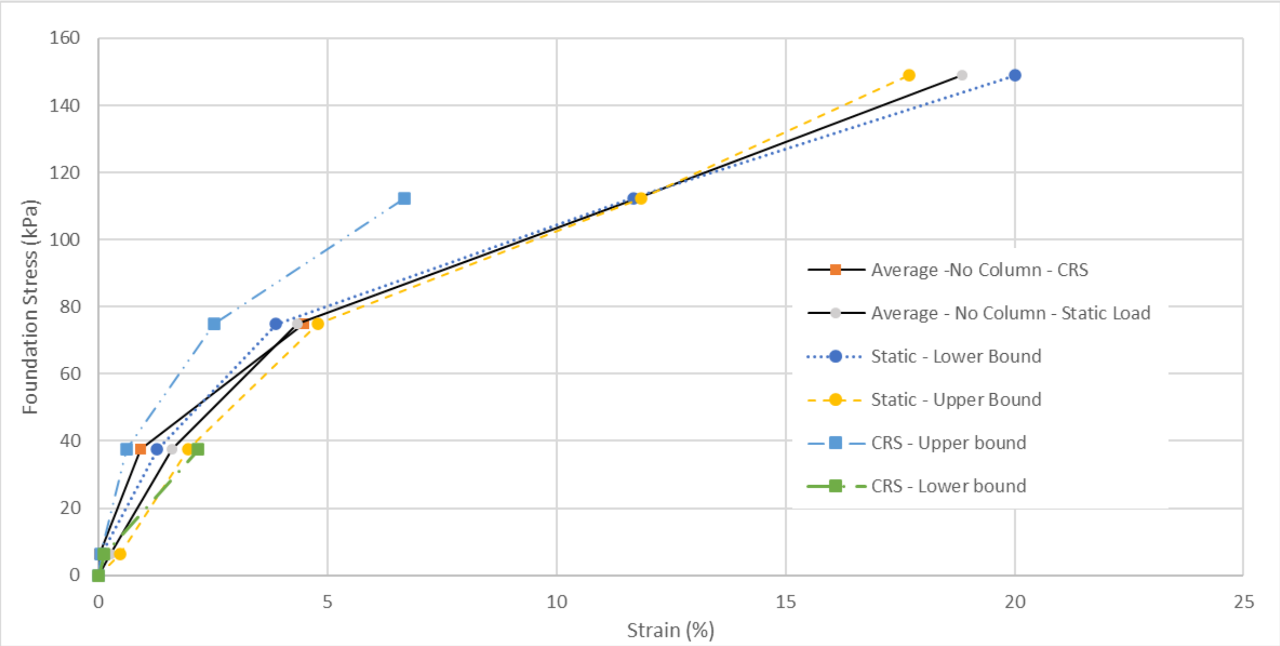


Figure 6-33: Average No column (host soil only) - Stress vs. foundation pressure CRS and static load tests

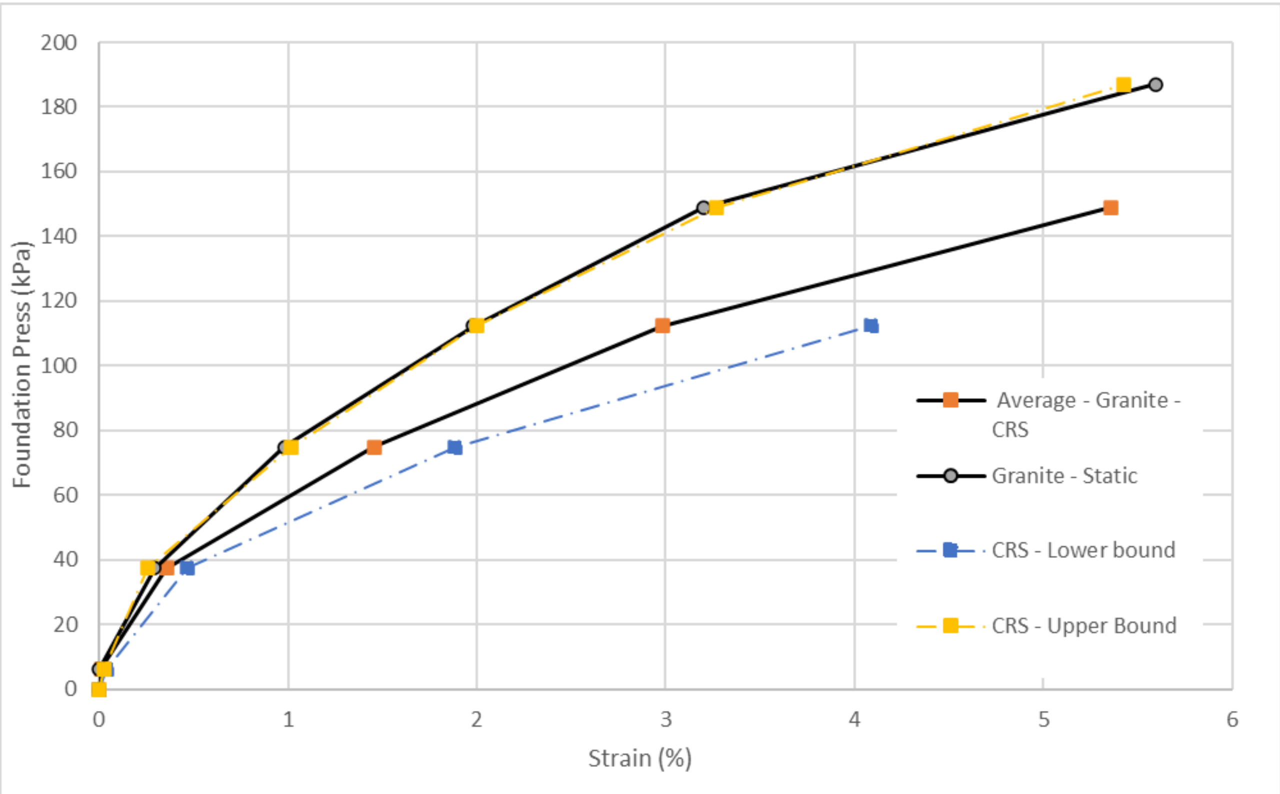


Figure 6-34: Average Stress vs. foundation pressure CRS and static load tests for granite columns (100 mm load plate)

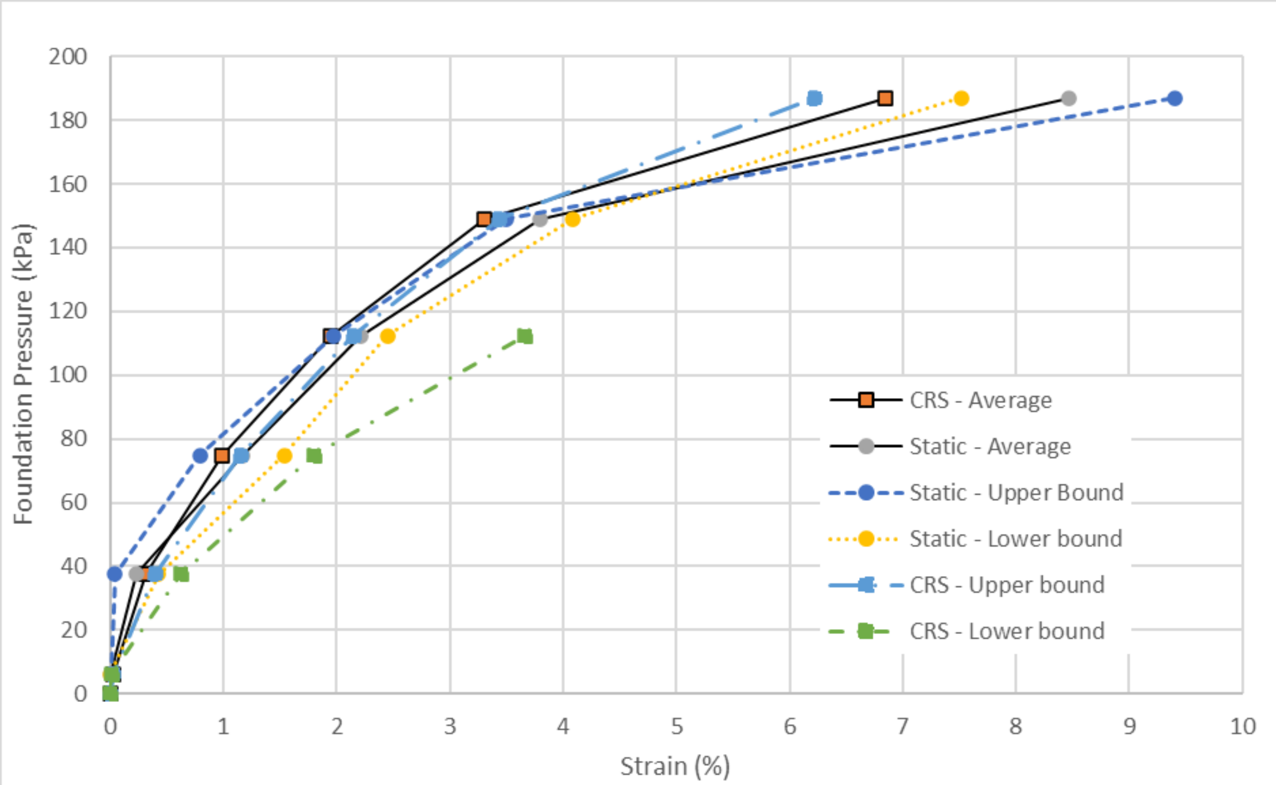


Figure 6-35: Average Stress vs. foundation pressure CRS and static load tests for APCr columns (100 mm load plate)

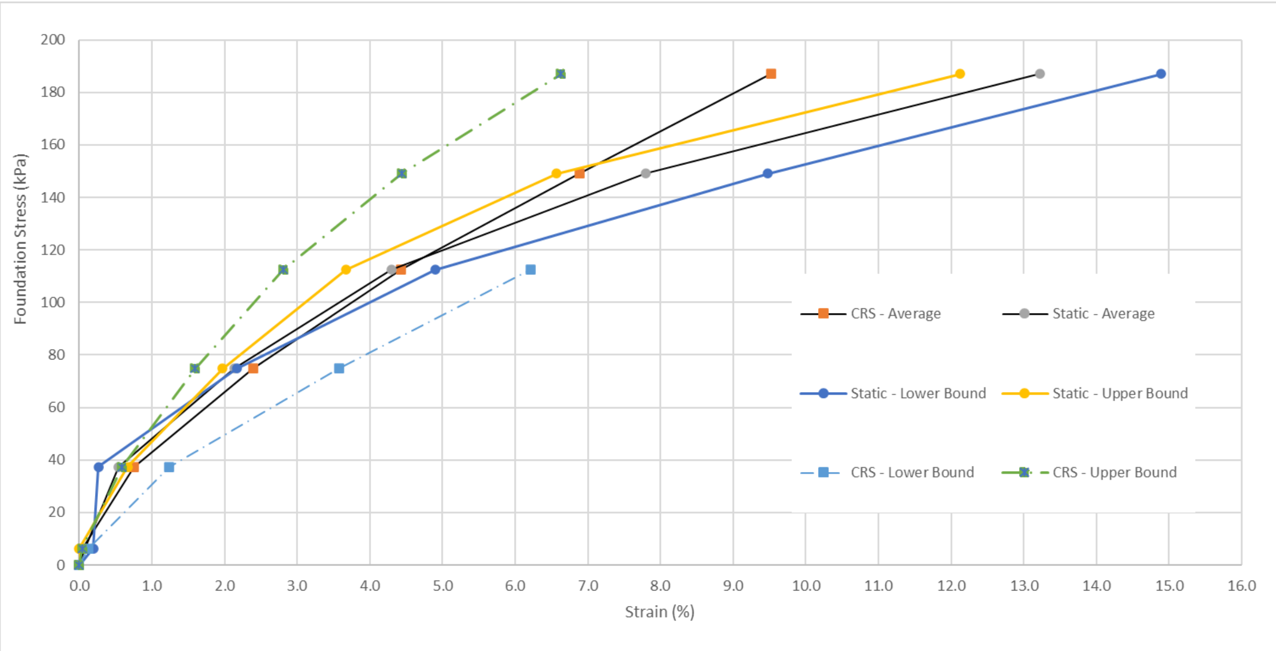


Figure 6-36: Average Stress vs. foundation pressure CRS and static load tests for container glass columns (100 mm load plate)

When considering the host soil alone, Figure 6.38 indicates that the load/settlement performance of the CRS and static load tests appear comparable. It is unfortunate that there are no results for the CRS tests at stresses above 74 kPa as they were not reached during testing (experiment timed out), but the proximity of the two trends does suggest such a comparision could be made (when considering the clay soil alone).

Figures 6.38-6.41 indicate that even with two very different types of loading being applied to the column the response is (broadly) similar. For example, in the case of APCr, which exhibits the greatest harmony between the two test types, at a settlement of 5 mm (1 % strain) the foundation stress is 75 kPa. However, it is apparent that as stress increases this relationship weakens; for example at 187 kPa the settlement measured for the CRS test is 31 mm, 10 mm less than of the static tests. This trend is echoed in the results of granite tests. This may be due to the longer duration of the test enabling the rearrangement of particles to support the load. In addition, the greater test duration allows more time for the clay to drain, leading to a stiffer material surrounding the column and the provision of additional lateral support.

The average of the three CRS granite tests indicates that after a foundation stress of 74 kPa the settlements measured for the static loads test are notably higher. However, during the analysis it was noted that CRS granite column tested on 09.12.20, the upper bound result for the CRS column tests, exhibited stress/settlement behaviour that closely matched that of the static load test. Whilst it is noted that this result cannot be 'cherry picked' to form a conclusion, it is an interesting observation. However, it is unfortunate that there are only the results of one static granite test to use in this comparison, additional data is required to form any firm conclusions. However, based on the spread of data observed in the CRS tests (means and maximum/minimum plots have been applied to Figures 6.38-6.41) it is envisaged that a similar spread could be encountered in the static testing as the host soil, and GC, were constructed in the same method. This suggests that absolute matching of one trend for another is clearly problematic (as evidenced with the mean and granite 09.12.20 in Figure 6.39) but the overall non-linear shapes of the curves are similar in nature and it appears the CRS (for the 100 mm diameter foundation) underestimates settlement for a given load; presumably because the tests are, comparatively, rapid and do not allow the host soil to respond to the load from the columns. This is not thought to be the case with the static tests, where the soil/column appear to form a new equilibrium with applied load over time.

The larger settlements measured at the higher stresses (e.g., 150, 187 kPa) in the static load tests for the granite and APCr could be attributed to particle breakage, although this was not confirmed by the data (Section 6.4.5). As mentioned previously, the longer duration of the static load tests allows additional time for drainage to occur. It is plausible that this additional drainage has led to increased settlements due to volume losses of both air and water.

In the case of the CRS tests as the load is applied more slowly particles have more time to rearrange to accommodate the increasing load which would still lead to settlements but these would potentially be less than settlements caused by particle breakage.

6.4.4 Particle Breakage

During the testing one of the observations noted was the sound of crushing, although no crushing was heard for the granite column. In the case of the static container glass columns for tests 01 and 03 crushing was heard when stresses of 149 and 187 kPa were applied, for test 02 crushing was only heard after the application of 187 kPa. It is interesting to note that it is at these two load steps where the results for the static and CRS tests diverge. Whilst this note of audible crushing cannot be quantified (see Section 6.4.5), especially due to the challenges faced when trying to recover 100 % of column material post-load, it is a key indicator that particle crushing, and rearranging of particles is the cause of additional settlement exhibited by the static load tests.

This finding can only be partially supported by the results of the APCr tests, where crushing was heard at stresses of 187 kPa in the static load tests. However, the sound of the crushing was much gentler than that of the glass and could be described as a fizz. This low volume sound could have been missed during earlier load applications due to background noise in the laboratory, particularly if crushing was minimal as could be inferred from the difference in measured settlements of the CRS and static tests. The crushing of the

APCr could be confirmed on removal of the aggregate post-test, where the particles had visibly crushed, forming a much denser material with finer particles, produced during the test of APCr filing the voids.

The crushing heard during the column load tests and the resulting difference in material structure (i.e., the increased amount fine particles of APCr leading to the creation of a very dense column) was also seen in the aggregate crushing value (ACV) test. The ACV test comprises a load of up to 400 kN being applied in increments of 40 kN over a 10-minute period to a sample 150 mm in diameter) and is designed to indicate the extent of crushing for aggregates by measuring the amount of fine particles (passing a 1.18 mm) produced. The ACV results for APCr and container glass are approximately comparable (ACV values were 29 and 33 for the glass and APCr respectively, see Table 4.5). After the ACV test the glass and APCr had formed a much denser material with additional fine particles produced due to application of load. This effect was far more pronounced for the APCr than the glass. However, when exhumed from the static load test, the APCr particles appeared to be 'fused' together with fine particles of APCr creating a denser material, yet this was not the case with the glass and granite, which did not appear to exhibit the production of fine particles and so did not become as tightly packed. This difference in behaviour suggests a fundamental difference in how load is carried by the materials. For example, the APCr bulges more readily (likely due to the rounded nature of the particles) and crushes more under loading, carrying load by compressing and forming a denser structure whilst the granite may draw load carrying capacity through the contacts between particles, both in terms of friction and vertical transfer of load.

6.4.5 Particle Size Distributions

In figures 6.40 to 6.42 (below) the PSD obtained before and after the static and CRS load tests are compared to determine whether the type of loading has an impact on particle breakage.

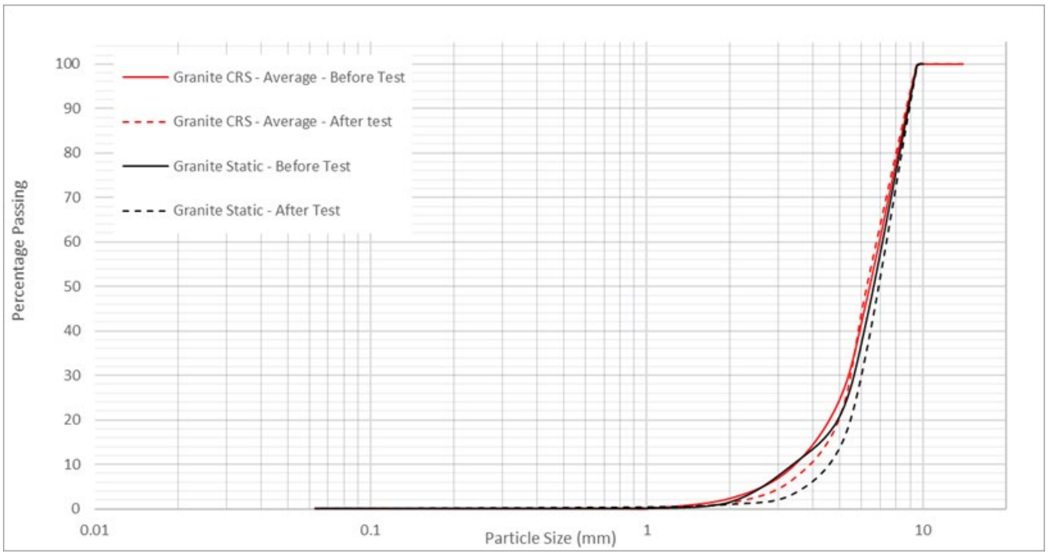


Figure 6-37: Comparison of PSD before and after both CRS and static load tests

The PSD curves shown in Figure 6.42 indicate for particles 6.3 – 1 mm less material passes each sieve, which as discussed previously could be attributed to the fact that it was difficult to recover 100 % of the material due to it being pressed into the sides of the bore. This means that unfortunately this data cannot be used to determine the extent of breakage occurring. It can be seen that there is a greater difference between the before and after PSDs in the static loads, suggesting that more material was lost into the sides of the bore due to increased bulging but the data on bulging does not confirm this (see Table 6.12 later in the chapter).

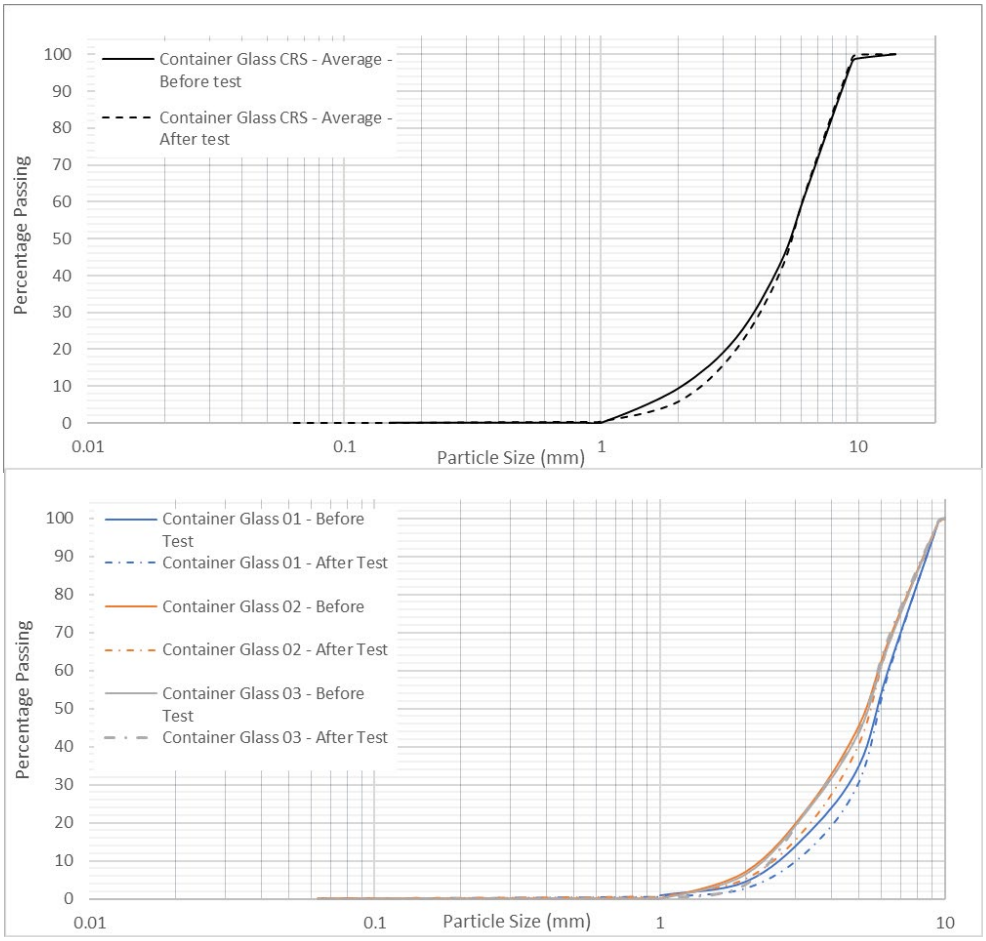


Figure 6-38: Comparison of PSD before and after both CRS (top) and static load tests (bottom)

The results shown in Figure 6.43 do not indicate that additional breakage was caused in either of the types of load test. As mentioned previously, crushing was heard during the static loading tests at stresses above 149 kPa which was not the case for the CRS tests; this could either be due to less breakage occurring during the CRS tests or simply the noise of the loading apparatus rendering the crushing inaudible. Unfortunately, the data available, Figure 6.42, cannot confirm which was the case as it appears that, as for the granite, the inability to recover the full sample of aggregate has affected the results of the PSD.

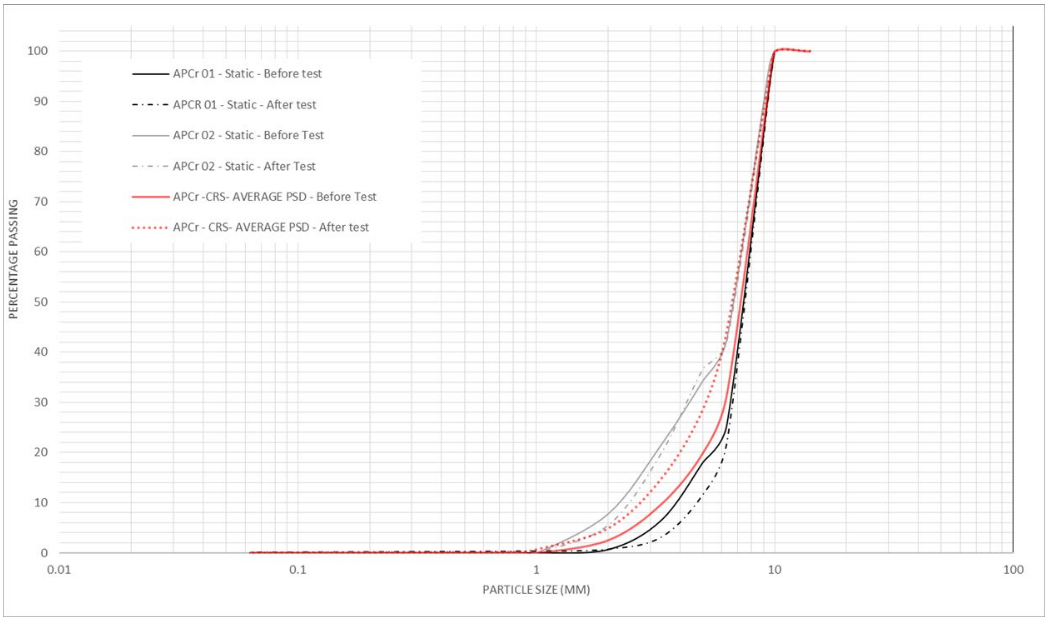


Figure 6-39: Comparison of PSD before and after both CRS and static load tests

In Figure 6.44 it can be seen that during CRS loading crushing of particles does occur. The results of the static load test are less conclusive and the PSD of APCr 01 indicate minimal breakage. This, however, is in contrast with both the crushing heard during the experiment and the visual observations of the aggregate post load. As mentioned previously, the APCr particles appeared to be broken and were 'fused' together with fine material, which is not reflected in the PSDs. It is disappointing that the observations made during the testing cannot be corroborated with the data.

6.4.6 Post-Load test Water Content of Aggregates

Table 6.11: Water content of aggregate post load – Static load tests

Column Aggregate	Test Date	Movement at toe (mm)	Moisture Content of Aggregate Post Test (%)	
			Top 150 mm of Column	Bottom 250 mm of Column
Container glass (1)	24.06.21	0	0.30	1.60
Container glass (2)	08.07.2021	0	0.09	0.23
Container glass (3)	15.07.21	5	0.22	0.72
APCr (1)	06.07.21	0	10.62	12.33
APCr (2)	15.07.2021	0	16.43	15.44
Granite	22.06.21	0	0.91	0.97

The post-load moisture content of the container glass reflects the same pattern as seen previously that the lower half the material is wetter than the top, this most likely is due gravity and the water percolating downwards through the column during the test. The moisture contents measured for the container glass are lower for the static load tests than for the CRS tests and this is likely due to the longer duration of the test (approximately 5 hours for the CRS, 24 hours for the static test) allowing greater time for water to drain to the base of the column, making it less recoverable. What is interesting is that despite this longer duration for water to pool at the base of the column and potentially lead to softening at the base, vertical movement of the column was only detected in one test (container glass 03). This suggests that the carrying capacity of the columns in these tests was not reliant on end bearing.

6.4.7 Column Bulging

In the case of the static tests, for the container glass in particular, the application of load over a longer period than the CRS appears to have provided time for the column materials to rearrange and support the load without causing vertical movement of the column. The increased bulging compared to the CRS tests suggests (Table 6.12), observed for container glass, that the longer period of loading has enabled the lateral strength of the host clay to be mobilised to a greater extent which is another explanation of why no vertical displacement at the column base was detected for the columns tested under static load (toe movements were recorded in 58 % of container glass CRS columns compared to 1/3 in static tests.

The minimal difference between the bulging in observed in APCr static and CRS is interesting as it does not follow the same trend as the container glass. There are no clear explanations for this behaviour, but this could be related to the spherical shape of the particles enabling rearranging to occur more readily.

In the case of granite column bulging appears to have reduced in the static load tests; as mentioned previously, issues with creating a cast of this column may have distorted the results and so further work is needed to confirm whether or not this is true reflection of behaviour under loading.

Table 6.12: Summary of bulging behaviour for static and CRS tests

	CRS - 100 mm plate - Average % Increase in diameter	**Static** - 100 mm plate -Average % Increase in diameter
Granite	26	12
APCr	36	33
Container Glass	14	29

6.5 Summary

The results of the CRS and static load test discussed in this chapter clearly indicate that the AA investigated have potential use within granular columns and would merit further work. This finding is in contrast to the results of the phase 1 testing which would suggest that both types of glass and APCr are not suitable for use within GCs. This is an observation that has been made by many previous researchers and it is imperative that this issue be addressed to avoid the dismissal of materials, which could successfully be utilised, based on their performance during index tests.

It is disappointing that the data collected on particle breakage is somewhat limited, however, a positive outcome is that in the case of both types of the glass the breakage does not appear to be as great as might be expected (when compared to index test results for example) and in the case of the APCr particle crushing does not appear to have detrimentally affected column capacity. This finding opens up the possibility for other materials to be investigated which may ordinarily be dismissed due to concerns over particle breakage.

Another key finding of this work is the similarity in stress-settlement/stress-strain behaviour of the columns in both the CRS and static tests. This is surprising, due to very different method of load application, but positive as many researchers have adopted the CRS method which is relatively quick and requires less supervision but does not closely stimulate the loading environment of columns in practice. It is accepted that the static load method does not exactly simulate the loading of full-scale closer, however, it is arguably a closer approach.

7 Conclusion and Recommendations for Further Work

In this chapter the main findings of the research, including the aggregate index tests and simulated column tests, are presented along with recommendations for future work.

The aim of this research was to determine the feasibility of utilising three types of alternative aggregate (APCr, window and container glass) within granular columns when compared to the performance of a primary aggregate (crushed granite). The motivation behind this work was to contribute to the research initiative that promotes engineering understanding of materials that have potential to act as alternatives to those currently used; in so doing encouraging an increase in the adoption of AA within the field of geotechnical engineering. This motivation was driven by two key issues: Firstly, reducing waste sent to landfill; and secondly reduce reliance of the construction industry on non-renewable, high quality, aggregate sources, saving them for applications that have stringent requirements for the engineering parameters of the material used (such as the materials required for a road base). Repurposing wastes, and reducing the reliance on materials that have greater engineering properties than required (such as primary aggregates in granular columns; as evidenced herein) would help, in some small way, to contribute to the UK Government's stated aim of adopting a circular, and sustainable, economy (HM Government, 2018) and the road to net zero.

To date there have been many studies indicating the potential for AA to be successful adopted within the context of GCs and these materials including waste concrete, spent railway ballast, scrap tyres and incinerator bottom ash (Serridge, 2005; Amini, 2015; Ayothiraman and Soumya, 2015). APCr has not been explored from a geotechnical perspective and despite recommendations for the adoption of waste glass within GCs (Serridge and Sarsby, 2010; Zukri and Nazir, 2018) only one published study exists to date (Kazmi *et al.*, 2022) (and this was for recycled container glass and not crushed window glass). Further research, exploring the potential use of alternative aggregates, such as APCr and waste glass, is an important and necessary pursuit due to the potential for diverting these wastes from landfill and conservation of resources. Increasing the understanding of the behaviour of these materials will hopefully aid their wider adoption within industry.

This investigation was undertaken in two phases: the first considered the performance of the four aggregates in standard aggregate index tests such AIV, ACV, LA, PSD (and PSD changes pre- and post-testing) and compaction. The second investigated column tests using two different sized load plates (60 and 100 mm) along with two different types of loading: static and constant rate of strain. The main conclusions from both phases of the research are summarised below.

7.1 Phase 1: Index tests

The tests performed on the aggregates during phase 1 include: Loose Bulk Density, PSD, Proctor compaction, shear box, ACV, AIV, Flakiness and LA, with the following conclusions:

1. The results of the LA test indicated that only the PA, granite, met the recommended criteria. APCr was marginal according to the ICE Specification for Ground Improvement (1987) but did not meet the value recommended by Serridge and Slocombe (2012) and resulted in the generation of a large proportions of fine sands.
2. The results of the AIV test show that all of the aggregates met the criteria recommended by BRE (2000).
3. In the case of the ACV test, APCr achieved a value of 33 % which exceeded the recommended maximum value of 30 %, and both types of glass were marginal, achieving a value of 29% only just meeting the recommendation.
4. The internal angles of friction of the AA, measured using the shear box apparatus, all fell short of the recommended value of 40 to 45° (Serridge, 2005) and only the PA met this criteria. However, in more recent publications no minimum value is specified (BS EN 14731:, 2005), only that materials with ‘high angles of internal friction’ should be adopted (Serridge and Slocombe, 2012) which is vague and open to interpretation (which on reflection of the findings herein, when comparing Phase 1 and Phase 2 outcomes, this might be a sensible way forward if appropriate testing, such as a bench scale column test, is undertaking before making a decision on the suitability of the material)

5. Particle flakiness is cited as a material quality that is unsuitable for use within GCs (Serridge, 2005; Serridge and Slocombe, 2012) and there is no specific value/quantity of allowable flaky particles. Both types of waste glass include a large percentage of flaky particles (100 % and 94 % respectively); the opposite is true for APCr which contains 0 % flaky particles due to the spherical shape of the particles. The granite utilised within this study featured a moderate amount of flaky particles at 24 %.

With reference to items 1 and 3, materials that are susceptible to particle breakage are not recommended for use within GCs (Serridge, 2005) and the currently accepted way to determine the potential for particle breakage is the Los Angeles Abrasion test (LA) (the AIV and ACV are also methods of determining the potential for breakage but are not longer specifically recommended for use in relation to GCs: Serridge and Slocombe, 2012). The results of the ACV and LA indicate that the AA used within this study are susceptible to particle breakage and so it is unlikely that they would be recommended for use within GCs. However, it is questionable how relevant the LA test is to the application of materials within GCs given that the installation of aggregates primary involves vibration yet the LA test comprises a number of steel balls inside a rotating steel drum (and was developed to investigate abrasion/breakage of aggregates in road surfaces (Mohajerani *et al.*, 2017)).

7.1.1 Summary of Phase 1 Outcomes

Overall, the AA appear to perform poorly in the index tests, whilst the granite met all the recommended criteria. This is an unsurprising finding given that granite is a primary aggregate. If these were the only tests undertaken to assess the viability of the AA the glass would have been considered marginal, and probably rejected, and the APCr would have been rejected as wholly unsuitable. Fortunately, Phase 2 was undertaken with the materials and the performance of the AA was deemed acceptable for consideration/use in these columns.

The difficulty in characterising recycled/secondary aggregates (particularly given the variability in their composition) utilising the existing index tests (e.g., AIV, ACV, LA) alone remains a challenge to the increased adoption of these types of materials. This conclusion has been drawn by many other researchers (Schouenborg, 2005; Chidiroglou, O'Flaherty and Goodwin, 2009; Serridge and Sarsby, 2010; Amini, 2015; Shahverdi, 2020) and is a challenge that needs to be addressed to enable the increased utilisation of AA. It would be understandable if the performance of the aggregates observed in these index tests also reflected the performance encountered when used in a column. However, as Phase 2 outcomes illustrate, these index tests do not appear to represent *in situ* behaviour, therefore the nature of index tests used to assess aggregates need changing or, perhaps more appropriately, small scale laboratory tests that simulate *in situ* behaviour should be adopted.

7.2 Phase 2: Bench-Scale Laboratory Columns Tests

Phase 2 comprised the construction and load testing of bench-scale granular columns. Two types of loading were applied: constant rate of strain and static. The key findings resulting from these tests are summarised below:

1. The potential for breakage of particles during the column installation process and with the application of loading is cited as a reason to reject types of aggregates (BRE, 2000; Serridge, 2005). In this research, particle breakage of the APCr and granite was negligible (even though the APCr which performed particularly poorly in the LA test). Some breakage during installation was observed for both types of waste glass but this was also minimal. During loading particle breakage was noted for each of the aggregates (this includes the granite), however, the success in quantifying the extent of breakage was limited by the challenge in removing the full sample of column aggregate post-load. However, the breakage observed during the index tests, primarily the LA test, test did not accurately predict the extent of breakage that occurred during loading for the AA (this difference in performance was particularly notable for APCr, Figure 6.37). Despite some particle breakage occurring the columns, including those formed using AA, significantly

improved the capacity of the soft host soil and so perhaps particle breakage is not as great a concern as it is cited to be.

2. In the static load tests the performance of the APCr was inferior to the PA, granite, which achieved a settlement reduction of 70 %. However, the APCr achieved a settlement reduction of 55 % and the container glass achieved a settlement reduction of 30 % which is significant.
3. In the case of the columns loaded using CRS and a 100 mm plate the load-settlement curves show that the average behaviour of window glass and APCr actually exceeded those using granite. The container glass performs less well but still achieves supports a load 1.5 times greater than an unreinforced sample of the clay.
4. The columns loaded with a 60 mm plate (CRS only for just container glass and granite) show that the container glass column supports a load 2.7 greater than for the average unreinforced sample which exceeds the performance of the granite columns which achieve an average increase in load capacity of 2.4.
5. The load-settlement performance of the columns, when loaded using static and CRS conditions, followed similar trends (direct comparison is problematic) indicating that the CRS method of loading (commonly adopted for small-scale laboratory GC studies) appears an appropriate method to provide an indication of aggregate behaviour in bench scale columns. This is despite the loading conditions being very different to those loading environments experienced by full-scale columns. Whilst the static loading approach adopted within this research does not exactly simulate the *in situ* loading of full-scale columns, arguably it is a closer representation than the CRS. Validating the CRS method for laboratory studies is a positive finding as these tests required much less time and supervision.

7.2.1 Summary of Phase 1 and Phase 2 Testing

The poor results of the Phase 1 testing suggested that the AA, selected for investigation within this, study would perform poorly within in application of granular columns (especially the APCr). However, the results of Phase 2 show that, contrary to the findings of Phase 1, the AA (container glass, window glass and especially the APCr) do have potential to be used as aggregate within GCs. In particular, the APCr and window glass exceeded the performance of granite (based on the average results). The container glass performance was inferior to granite (based on load-settlement behaviour) but still increased the capacity of the host soil significantly.

The aim of this research was to determine whether APCr and two types of waste glass could be successfully utilised within GCs and it is clear (from the bench-scale testing undertaken herein) that they could be suitable alternative to primary aggregates.

Clay was used as the host soil during this research. A variety of ground types could have been adopted but clay (Kaolin) was selected due the ease of creating uniform samples, enabling comparison of column behaviour to be drawn without the influence of variation in the host soil. GCs in soft cohesive soils (e.g., clays) have been studied extensively in laboratory studies and are widely installed in these types of soils in practice.

7.3 Implications of This Work

It is hoped that the positive findings of this work will contribute to the existing body of research on the use of AA within GCs and encourage the wider adoption of these materials, reducing the consumption of precious resources and reducing waste sent to landfill, thus contributing to the circular economy.

The use of AA currently within the UK is approximately 30 % (Mineral Products Association, 2020) and this rises to around 40 % within the construction of GCs (Balfour Beatty, 2022). These figures show that there is a desire to adopt AA within industry and it is hoped that research, such as this study, will help increase the consumption of AA (whilst simultaneously decreasing the use of PA). This research has shown that even when AA appear to be inferior in terms of quality (as shown in the phase one index tests) they can successfully be applied within GC construction. Findings such as these are encouraging and should inspire the exploration of the use of other alternative materials, increasing the types of aggregates utilised and promoting their wider adoption.

7.4 Recommendations for Future Work

This research has provided an insight into the adoption of three AA for granular columns and has produced some useful findings, however, there is still much further research to be done to facilitate the increased adoption of AA within GCs. Some ideas for further work are listed below.

- The use of a large test cell (i.e., much greater in diameter than 5D) would enable the potential influence of effects of boundary conditions on the performance of the columns to be determined
- It would be of interested to trial a group of columns formed of AA as is known that columns within groups do perform differently to single columns. Without modelling a group of AA columns in the laboratory it is difficult to predict how differently the column group would behave compared to single columns. However, based on previous research, conducted into column groups, it could be estimated that the columns would carry increased loads without bulging due to the additional confining effect of the surrounding columns. It would be interesting to investigate this further
- Varying the area replacement ratio to determine whether a reduced amount of material could be successfully used could potentially lead savings both in terms of financial and environmental impact
- The use of a casing (such as a geotextile) for the AA, including the materials investigated in this research, would be of great interest. The addition of a casing enhances the load-settlement performance of granular columns (Al-Obaidy, Jefferson and Ghataora, 2016) due to the additional confining pressure created by the casing. It would be of interest to see the extent of

improvement in load carrying capacity achieved when using a casing with the aggregates used within this research

- Long-term testing, particularly in the case of the APCr, is important to determine the behaviour of the AA over a greater amount time and to ensure that no degradation of the material occurs
- Full-scale field studies investigating the performance of AA granular columns would enable the results of this research to be validated
- Further research into the potential carbon reductions that could be achieved through the use of alternative aggregates in GCs would be of great interest and support the road to net zero

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Appendix (A)

geotechnics

Review

The Use of Recycled and Secondary Aggregates to Achieve a Circular Economy within Geotechnical Engineering

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Abstract: The construction industry's current dependence on primary aggregates is unsustainable as these are non-renewable resources and the consumption of these materials has a high environmental impact. The global annual production of primary aggregates is estimated to be 50 billion tonnes. In Europe, where 2 billion tonnes of primary aggregates are produced annually, approximately 90% of aggregates are utilised by the construction industry, whilst over 1 Gt of waste are sent to landfill; in the UK, 44% of landfilled waste arises from the construction industry. The drive to adopt a circular economy necessitates changes in resource use (including non-renewable aggregates). Recycling wastes, such as aggregates, could help this situation; whilst this concept is not new, it does not appear to have been widely embraced in geotechnical engineering. The aim of this paper is to highlight the benefits of increasing the use of alternative aggregates as this would enable the reserves of primary aggregates to be better maintained and less material would be landfilled—a win-win situation and a contributing step towards developing a truly circular economy.

Keywords: recycled aggregates; secondary aggregates; alternative aggregates; circular economy; waste management; geotechnical applications

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1. Introduction

It is well recognised that the overuse, or mismanagement, of a resource can lead to its depletion or exhaustion, which is unsustainable [1,2]; this situation is compounded by simultaneously sending wastes to landfill sites that could be recycled and reused in place of non-renewable resources, such as primary aggregate (PA). PA has been described as the most valuable non-fuel mineral commodity, and without it, life as we know it would be difficult to sustain [3]. The global consumption of PA combined with the extent of the landfilling of wastes that could act as aggregates is a case in point.

An estimated 50 billion tonnes of primary aggregate are currently produced annually worldwide, and this is predicted to increase 5% year after year [4]. The EU alone is estimated to consume 2 Gt of PA [5] whilst simultaneously sending over 1 Gt of waste (total weight, not just that which could be used as aggregates) to landfill sites annually [6].

The construction industry is a major consumer of aggregates and a contributor to landfilling (estimated to account for 35% of waste sent to a landfill globally, 44% in the UK [7]. Similarly, in the USA, approximately 1.4 billion tonnes of PA are consumed annually (58% of which are utilised within the road construction industry, and 90% of these aggregates are virgin materials) [8,9]. Millions of tonnes of CDW are generated by the construction industry globally each year; for example, an estimated 170 M tonnes are produced in the USA along with 860 M tonnes across the EU [10,11]. Therefore, successful management of CDW is critical to enable vast amounts of waste to be diverted from a landfill and precious resources to be conserved.

In order to enhance the utilisation of recycled and/or secondary aggregates (RA and SA) as alternatives to PA, it is imperative that industry practices are modified significantly. This may require a substantial reduction in the demand for PA or a step change in how recycled and secondary aggregates are both viewed and produced within the industry (both in quantities and range of techniques used to maximise uptake of available wastes, i.e., not just crushing materials but using additional processes to valorise previously unsuitable materials, such as accelerated carbonation to produce lightweight aggregates from ashes: [12–14].

In recent history, despite growing awareness of the need to act in a sustainable way, there has been some resistance to greater utilisation of alternative aggregates (AAs) over PA due to (but not limited to) concern regarding the availability of materials and potential issues with engineering performance [15–19]. There has been a vast amount of research into alternative aggregates for concrete, including materials such as municipal sludge ash, animal bones, human hair [20–26], but in comparison to uses within concrete, such studies are more limited for geotechnical applications. That said, there has been a considerable research effort to try to improve the understanding of the engineering performance of these alternative materials within geotechnical applications; however, in many cases, this is still poorly understood, and additional research is fundamental. This paper reflects the government's stated need to embrace a circular economy and hence develop better resource/waste management practices and focuses on the need to reduce the reliance on PA for AAs. AAs have been considered, illustrating both potential and current gaps in understanding engineering performance.

The term 'aggregate' is used herein to describe potential materials for geotechnical applications, as this term is commonly used within the existing literature, although it is not used without reservation. The term aggregate seems evocative of 'coarse-grained' materials (used in railway and highway applications, where engineering properties must be stringent), which are not necessarily required for many geotechnical applications. Perhaps a better term would be 'particles', as the materials under review provide the solids for the placed soils (many of which form three-phase materials), although as noted above aggregate is used.

2. The Ethos of the Circular Economy as Motivation for the Use of Alternative Aggregates

The preferred solution to waste management is to prevent waste generation in the first instance [27,28]. If that is impossible, the next goal is to minimise the waste generated and promote recycling, minimising landfilling/burning. These are the concepts underpinning the circular economy (CE) [20], shown in Figure 1. The concept of CE was first raised by Pearce and Turner (1990) and has since been gaining traction. Whilst there is not yet a clear definition of what a CE entails, it can broadly be described as a shift from a linear model of 'take-make-dispose' to an industrial economy that is restorative by intention, i.e., reducing the reliance of economic activity on finite resources and designing waste out of the system [29]. It has been argued that the CE concept has clearer objectives than the precursor ('sustainable development') and has had more resonance with the industry due to the view that CE offers operationalisation benefits [30–33]. For example, it was suggested that up to £130 million is accruable to the UK economy by reducing just 5% of its construction waste sent to landfill [34,35]; thus, there is not only an evironmental incentive to reduce waste and preserve resources but also a financial one.

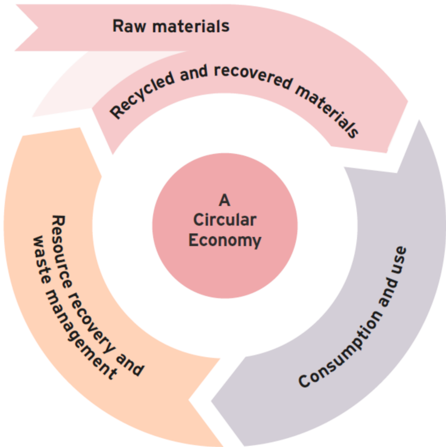

Figure 1. The circular economy for sustainable development. The win-win potential of circular economy [1].

This paper intends to draw on the concept of CE as motivation for the widespread use of RA/SA and to promote a change in attitudes towards the adoption of non-virgin materials rather than focusing on the concept of CE itself. For further information on the principals underpinning CE, the authors refer the reader to the following sources: [29–32].

Achieving a CE is currently a political goal in many countries worldwide, including the UK, China, USA, Japan and the European Union [1,28,36]. Clearly, the construction industry must embrace change as part of wider societal and economic changes if this is to be achieved. However, recognising the need for change does not necessarily prompt change, and markets may require government intervention to stimulate action. For example, there have been calls for the use of alternative aggregates (AAs) in construction for decades [33,37,38]. Since the introduction of the landfill tax and aggregate levy in the UK, 1996 and 2002, respectively [39,40], there has been an increase in the amount of AAs consumed such that the UK now achieves the highest rate of recycled aggregates within the EU (Figure 2 shows an increase in the total AAs consumed from approximately 16% in 1996 to 29% by 2016 [41]. This is a step in the right direction but more needs to be carried out to reduce the reliance of the construction industry on PA [42].

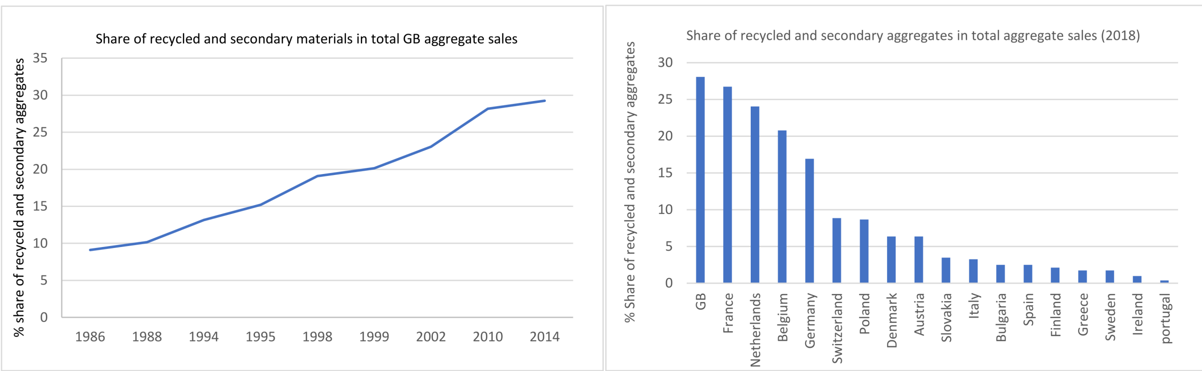

Figure 2. (**Left**) Share of recycled and secondary materials in GB aggregate sales and (**Right**) share of recycled and secondary materials in total aggregate sales (in 2018) (Mineral Products Association, 2019).

Germany, China, Japan and the EU have been amongst the first to enact the principles of a CE within Law [43,44]. Whilst the introduction of governmental regulations and legislation clearly has a positive effect on the reduction in waste generation and the adoption of AAs; a pilot CE study in a city in China reduced municipal waste by 17% over a 5-year period [43], whilst in Japan, 96% of CDW waste is recycled [45]. There remains the need for a greater societal change in attitudes towards the use of AAs [43]. Education, information, and encouragement to promote this change in attitude are required [43]. Typically, recycling has only taken place, by businesses at least, where there is an economic incentive. Attitudes need to be widened to ensure that recycling/re-use happens because it is socially desirable, not just financially desirable [46].

2.1. Primary, Recycled and Secondary Aggregates

PA can be defined as materials extracted directly from the earth, such as limestone, dolomite, sandstone, granite and terraced sands and gravels, that require little further processing. Within the UK, it is estimated that 300 Mt of aggregates and minerals were produced in 2014, and the construction industry consumes roughly 90% of these products [41]. Applications include: embankments, formation of road bases and subbase layers, placement of fills for infrastructure and construction purposes, the establishment of foundations (including stone columns) as well as being incorporated into concretes [17,47–49]. However, various studies [49–53] have shown that RA (defined herein as aggregates recycled from waste with minimal processing, such as crushing of demolition waste) and SA (defined herein as aggregates manufactured from waste via various processes, such as sintering or chemically stabilising them) can successfully be adopted in many of these applications if the engineering properties of these AAs are known.

2.1.1. Primary Aggregates

It seems an abstract question to ask if we can run out of rock; there certainly seems a large quantity on Earth, so why concern ourselves with alternatives? The Mineral Products Association (MPA) (2020) [54] states that, based on the 10-year average, the quantity of aggregates extracted and used each year exceeds the new reserves that have been granted planning permission, thus resulting in a long-term trend of reserve depletion. This is an issue that most urgently affects sand and gravel as there is only an estimated 8 years' supply remaining in England [54]. However, crushed rock reserves are also of concern as only 75% of (compared to 63% of sand and gravel) reserves were replenished between 2009 and 2018 [54]. It is estimated that in the UK alone, over 5 billion tonnes of aggregates will be required (even when factoring in more efficient construction methods), and clearly, shortfalls are likely to occur in the near future unless alternative sources can be found [41,54].

Availability is not the only factor when assessing the viability of a resource, as there are clearly societal, environmental and economic considerations. The development of land has led to sand and gravel resources becoming inaccessible (leaving the accessible resources nearing exhaustion) and increasing the costs, both financial and energy, associated with extraction [16,39,41].

Since aggregate sources are located where Mother Nature placed them, and this does not always coincide with where it is required, there are large regions around the world where sources of PA are non-existent. This varying distribution of PA leads to additional pressure being placed upon areas of resource [55–58]. An example of this is the UK, where there is an imbalance in the areas of aggregate supply and demand in the: 70% of hard the rock sources lie within the south-west and East Midlands regions (of England and Wales) [38] (Figure 3) whereas the area of greatest development, i.e., the greatest demand for material, is located in the south-east [16,58–61]. This region has accounted for one-third of the total construction activity in the UK for at least the past twenty years [15,61]. In 2004, only 40% of aggregates could be sourced in the south-east; thus, materials were transported over 100 miles [62].

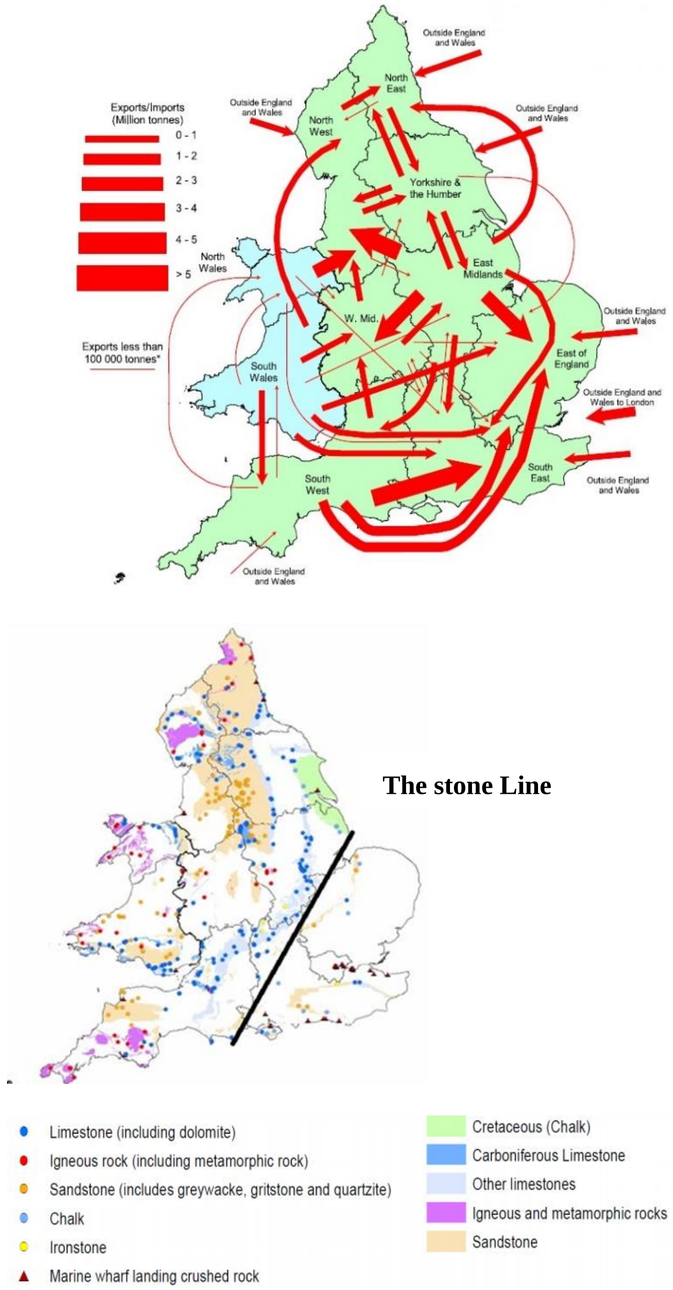

Figure 3. (**Top**) Hard rock reserves in the UK (Reproduced with kind permission from the British Geological Survey (for supporting information see: [48]. (**Bottom**) Crushed rock inter-regional flows within England and Wales (2014) [17].

The cost of haulage is a major component of aggregate cost, both financial and environmental [27]. It is estimated that road transport dominates (90%), leading to increased carbon emissions [63,64], which further negatively impact the environment. Thomas et al. (2009) [62] suggested that carbon emissions (related to the use of aggregates) were reduced by one-third with the production (and utilisation) of RA via in situ recycling (i.e., crushing of demolition waste) when compared to trafficking in PA.

2.1.2. Recycled and Secondary Aggregates

The adoption of alternative aggregates diverts waste from landfill sites and can lead to large financial savings, especially since in the UK where RA and SA are not subject to the aggregate levy and in the case of hazardous waste streams where landfill gate fees are typically higher [63]. Furthermore, many of these waste streams are generated in, or comparatively near, urban settings (i.e., near the centres of demand for aggregates); hence, there is the potential for reduced transport distances (when compared to quarries).

Treating previously unsuitable wastes to produce SA offers benefits; for example, chemical treatment of hazardous materials can remove the potential to contaminate, producing aggregates from wastes with particles that would otherwise be too small to be workable (i.e., silt and clay-sized particle sizes), although these processes can be energy-intensive (in the case of sintering, etc.), so some techniques are not without cost. Examples of SA include lightweight products such as Lytag (sintering of pulverised fuel ash, PFA), carbonated ashes and cemented or thermally treated wastes [12–14,64].

2.1.3. End-of-Waste Status

The introduction of the revised Directive on Waste, Directive 2008/98/EC, intended to simplify the existing legislation on waste within Europe to promote the diversion of waste from landfills and to encourage lifecycle thinking, shifting the perception of waste from something to be disposed to a potentially valuable resource [31]. One of the important aspects of the revision was to clarify the definition of waste and the distinction between recovery and disposal [65,66], and the introduction of 'End-of-Waste' (EoW) criteria aimed to achieve this. In order for a waste to achieve EoW status, the following criteria must be met:

- the substance or object is commonly used for specific purposes;
- there is an existing market or demand for the substance or object;
- the use is lawful (substance or object fulfils the technical requirements for the specific purposes and meets the existing legislation and standards applicable to products);
- the use will not lead to overall adverse environmental or human health impacts.

There is some variation in EoW regulations across the EU. For some materials, such as for inert aggregates, each member state has their own quality criteria [66,67]. In the UK, the 'Aggregates from inert waste' quality protocol, set by the Environment Agency, indicates how compliance with EoW status can be demonstrated and gives guidance on matters such as storage and transportation of materials [68].

Another aim of the revised Directive is the promotion of reducing natural resource use [66,67]. This aspect is explicitly met by the diversion of wastes from landfills via their utilisation as aggregates (and has commercial benefits due to the simultaneous avoidance of the landfill and aggregate tax). However, there are more benefits to be gained during the production of the recycled/secondary aggregates, such as the sequestering of carbon dioxide during the accelerated carbonation of certain waste streams. In addition, research is currently being carried out into the extraction of valuable metals from waste streams prior to their treatment and subsequent production into aggregates [68,69], further preserving natural resources and adding to the commercial viability.

Compliance with EoW criteria requires the material to 'fulfil the technical requirements for the specific purpose', meaning that meeting the EoW criteria does not give widespread approval for the use of the waste material, and the application must be suitable, e.g., certain secondary aggregates may not meet leaching limits but would so if they were utilised in a bound capacity. In addition, so long as the EoW criteria are complied with, there is no limit to the number of times an RA/PA can be re-purposed. At the 'end of life' for one application, the material can simply be re-assessed against the EoW criteria and utilised, either in the same capacity or another more suitable application, enabling a truly circular economy. However, from a geotechnical viewpoint, it is clear that more research is required to understand the long-term performance of these materials (especially if exposed to cyclic loading after placement; or repeated cycles of removal and replacement with the recycling of these materials on a number of occasions over the life of the RA) to determine if/how the engineering parameters of these materials change.

2.2. Perceived Barriers to the Use of Recycled and Secondary Aggregates

Possible barriers to the widespread use of AAs have been identified by [25,68–71] and include: lack of confidence and/or perceived risk with the product (perception that they are inferior to PA, or there are issues with consistency: [49,55,72,73] a lack of suitable specifications and testing protocols (reliability and quality control issues for certain applications); and certification of the produced product. In addition, there may be a lack of awareness of AA products [69]), supply–demand and other market issues and waste management licensing regulations/environmental issues.

The construction industry has been characterised as being conservative [73–76]; thus, preference might be given to known materials (which they have experience of using), such as PA [69,76,77], over alternatives. Consequently, the desire to adopt AAs can be low whilst natural resources are still available (especially if lower in price than more sustainable alternatives), the benefits of AAs are poorly promoted, and there is uncertainty over appropriate testing to assess the properties of these materials [77]. Therefore, it is suggested that more research into material behaviour is required to help address both the negative perceptions of AAs but also raise awareness of these materials. Examples of research in this field are presented in the subsequent section.

2.2.1. Financial Aspects of AAs

It should be noted that the focus of this paper is on the geotechnical engineering performance of these materials and not on the economic or statuary considerations related to the use of these materials. This is not to say that these factors are considered of secondary importance to the engineering properties but a reflection of the authors' area of expertise.

PA still represents an attractive option, even when compared to AA, due to the inherent low cost of the materials. This is perhaps a major reason that the uptake of AAs by contractors has not been more widespread. The introduction of the aggregate Levy in the UK (similar taxes apply in other European countries, such as Denmark and Sweden [78,79]) was designed to encourage the adoption of AAs, and the data shown in Figure 2 do indicate that the adoption of AAs is increasing (the UK currently has the greatest usage), but there is still scope for improvement. In order to consider AAs as a viable option, the whole cost of the material, including the transport costs (both financial and environmental) and other environmental aspects of recycling or reusing wastes (e.g., diverting waste from landfill), must be taken into account.

2.2.2. Appropriateness of Specification Testing When Assessing Suitability of an Aggregate for Geotechnical Applications

The revision of the European standards to incorporate the use of RA and SA can be viewed as a positive step, but the applicability of these tests to engineering applications have been called into question [43]. Traditionally, specifications for aggregates might be generalised as focusing upon aspects of crushing resistance, abrasion, shattering, etc. Clearly, these provide an indication of how the aggregate might perform during handling/placement, which is an important facet but does not necessarily describe the short and long-term behaviour of the placed fill (once the placed material has formed an equilibrium with the surrounding environment). Arguably, it is the behaviour, in the short and long term, of the placed material structure [48,79,80] rather than that of the individual particles, i.e., it is the response of the placed material to changing environmental and loading conditions, that is the key concern.

Key parameters used to describe the behaviour of soils include: the angle of internal shearing resistance, apparent cohesion, coefficient of compressibility and coefficient of consolidation and are all based on the interactions between the two/three phases within the soil structure and not individual particle characteristics. To reiterate (for granular soils), it is the macro-parameters such as particle size distribution, particle shape, relative density, degree of saturation, etc., that are key to behaviour [48,69,81]. Tests, such as the fragmentation and abrasion (the Los Angeles value and micro Deval values), indicate the resistance of the individual particles [78], but they do not represent the overall interaction between the two/three phases encountered within soils (In addition Schouenborg, Aurstad and Pétursson (2004) [76] note that the LA test only utilises a certain size fraction of the aggregate sample (between 10–14 mm) and so cannot assess the performance of the entire grading which is particularly important for heterogeneous materials, which AAs often are). Clearly, the shattering of particles with placement, resulting in reducing the internal angle of shearing resistance, is a concern that cannot be dismissed as it can potentially influence the strength of placed material. AAs such as crushed masonry and (burnt) colliery spoil have been identified as potentially experiencing particle degradation with dynamic loading, although cases of PA shattering have also been identified.

Schouenborg, Aurstad and Pétursson (2004) [76] demonstrated that many aggregates, including a PA, are sensitive to traditional test methods, such as mechanical sieving; the abrasive process of mechanical sieving can lead to unrepresentative particle size distribution and can influence other properties, such as aggregate impact value: a measure of resistance to sudden impact, see Figure 4 [79], and yet, demolition waste and colliery spoil have successfully been used in geotechnical applications previously [82,83]. This highlights the importance of understanding the material properties and selecting appropriate engineering applications together with suitable construction methods.

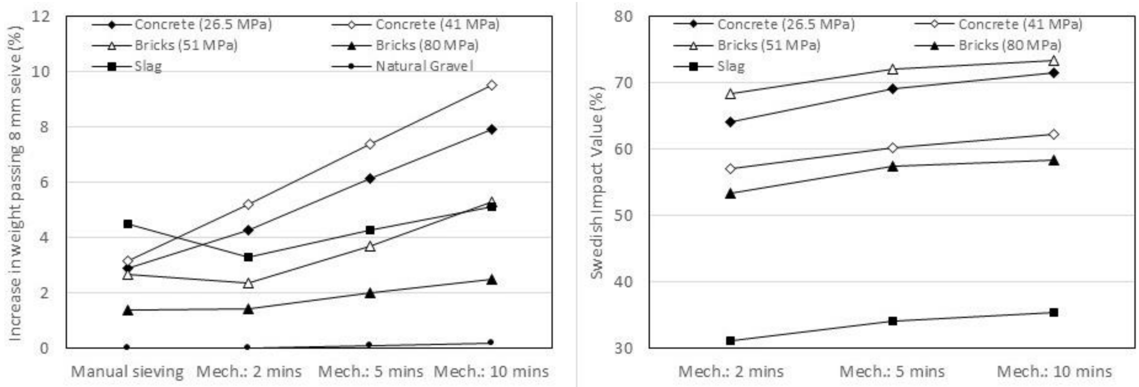

Figure 4. (**Left**) Change in the amount of material passing the 8mm sieve due to different sieving procedures and (**Right**) change in Swedish Impact Values for the AAs post-sieving [78].

Whilst particle testing provides a basis for a comparison of the durability of the materials, it does not give any insight into the interaction between the particles when they are confined. Thus, it is the contention of the authors that this is the key consideration when assessing the suitability of AAs for geotechnical applications and that perhaps specified characteristic tests should be modified to reflect this.

2.2.3. Los Angeles Coefficient

The method for determining an aggregate's resistance to fragmentation, as detailed in BS EN 1097-2:2010 [84], also known as the Los Angeles test, involves placing a sample (which passes the 14 mm sieve and is retained on the 10 mm sieve) into a drum with 11 400–445 g steel balls. The drum is then rotated 500 times, and the mass retained on a 1.6 mm sieve is recorded (m) [79].

$$LA\ Coefficient = \frac{5000 - m}{50} \tag{1}$$

This method may be suitable to replicate the experience of aggregates in the wearing course of highways, but it does not necessarily reflect that of confined aggregates. This could mean that some AAs, which may look poor when assessed using LA values, will perform well in certain applications (see Figure 5). An example of this is crushed concrete.

Whilst the AAs appear to have high LA and low UCS values (see Figure 5), these AAs have been a real success story [85–87].

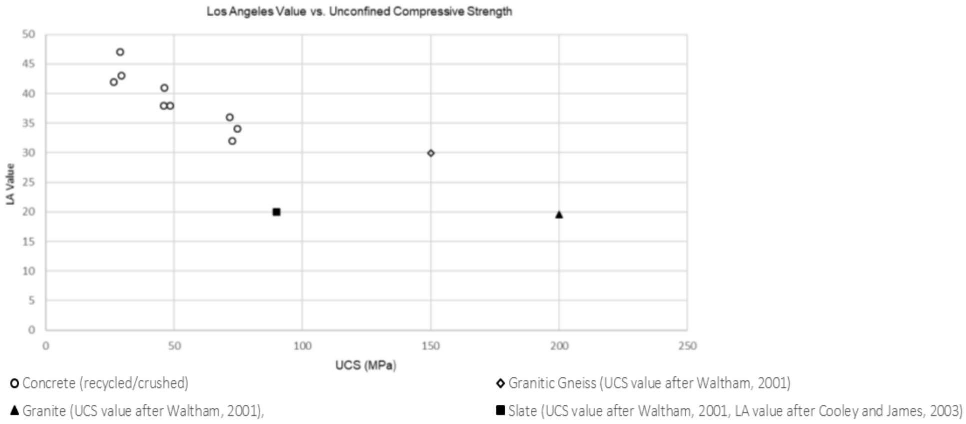

Figure 5. Los Angles vs. Compressive strength [79].

Koohmishi (2019) [88] notes that the main shortcoming of the LA test is that the extent of inter-particle abrasion is overshadowed by the force exerted by the steel balls on individual particles, potentially causing misleading results. In addition, Cooley and James (2003) [89] suggest that the nature of the test, i.e., steel balls dropped a distance up to the drum diameter, results in the aggregate particles being subject to large impact loads, which for some crystalline materials (such as granites), results in yielding artificially high loss values. Other lower-quality materials, such as slate, may actually reflect lower loss values as the structure enables impact loads to be better absorbed [89], again presenting a slightly skewed indication of their performance in engineering applications.

The LA values for natural gravel (see Table 1) and the two types of crushed brick, shown in Table 3, are comparable. Within *the Manual of Contract Documents for Highways Works, Volume 1* [90], the upper limit of the Los Angeles coefficient is 50 for applications such as gabion filling, general fill, drainage layers, starter layer, and relaxes to 50 for capping layers [90], suggesting that despite concerns over friability, recycled crushed bricks may have many potential uses.

Table 1. Published values of LA coefficients [53].

Aggregate	LA Coefficient
Natural Gravel	36
Flint Gravel	22
Quartzite Gravel	19
Latite Basalt	15
Limestone	20–43
Natural Granite	27
Dolerite	12–16
Quartz Diorite	22
Gritstone	18

3. Examples of the Utilisation of Wastes as Alternative Aggregates in Geotechnical Engineering and an Alternative that Potentially Could Be Used

There are undoubtedly specific geotechnical applications where high-quality PA is fundamental and stringent requirements necessitate aggregates with exacting engineering properties (e.g., railway ballast). However, it is argued that there are undoubtedly lower-grade applications where AAs might be perfectly acceptable (for example, for bulk fills or drainage). This is reflected in the reported usage of AAs within the literature, where applications include hard standing, bulk fills and capping layers for haul roads [62,91–93]. However, there are calls for the use of AAs in higher grade applications, such as is in ground improvement [18,50,94] (8). For this to become a reality, a greater understanding of the behaviour of AAs is required to enable the appropriate end-use of the AAs to be selected [75]. Examples of research into this field are given below.

When determining if an AAs is suitable for use in geotechnical applications, there are a number of aspects to consider, summarised as [69,82,94]: availability, both in tonnage and location; existing precedent of usage in engineering applications; the chemico-physical properties of the material, the variability of these properties and the potential for these to change with time (i.e., through creep and leaching) and the cost of the material. Nowak and Gilbert (2015) [94] state that selected materials must not have the potential to cause harm to the environment and must be stable; for example, they should not be biodegradable, miscible, contain significant inter-particle weaknesses or flaws, be flammable or prone to degradation when encountering buried concrete or substances such as fuel.

This paper draws on the results of previous studies which have shown successful use of wastes as alternative aggregates. The materials discussed within only represent an overview of some of the most common types of AAs currently being produced/utilised. There are many more materials, including plastics, various industrial wastes, such as colliery spoil [48,63,95–101], and mineral wastes [18,38,51–55], which may have geotechnical applications and merit further research.

3.1. Scrap Tyres

The disposal of waste tyres is a global problem, with an estimated 1000 million tyres being discarded each year, and 50% of these are disposed of with no further treatment [57]. Regulations preventing the landfilling of tyres within the EU and further restrictions on the disposal of tires elsewhere around the world (2006 under the European Landfill Directive 1999/31/EC; [49,102]) has necessitated alternative uses for this waste. Tyres are fabricated using vulcanised rubber, which means that the material cannot be melted and re-set, so recycling/reusing is the only viable alternative to disposal. Research into the usage of tyre bales, tyre strips and rubber crumb in geotechnical applications [103–107] indicates that these AAs are lightweight, free draining, durable and offer good thermal insulation [108–111]. These properties make the material particularly suitable for applications, such as fill behind retaining structures (horizontal earth pressures acting on retaining structures were found to reduce by 50-60% by utilising a rubber shred–sand backfill [110] and highway embankments, particularly over weak or highly compressible soils [111]. The low cost of the material is a possible added benefit [112,113].

Utilising rubber strips (without soil infill) is not advised due to an incident where an embankment experienced auto combustion [108,114]. Mixing the tyres with soil appears to have reduced this risk and potentially improves the performance of the material [105,110,115]. The data shown in Table 2 reflects the increase in shear strength achieved through tyre–sand mixtures in comparison to pure tyre shreds. The optimal mix, in terms of shear strength (friction angle) and deformability (void ratio), has been found to comprise a 30/70% tyre-chip-to-sand ratio [112,116,117], where tyre content exceeds 30% the mixture behaves more as a tyre chip mass rather than a reinforced soil [117,118]. Research by Anbazhagan, Manohar and Rohit (2017) [116] show that the optimal size particle size (in terms of friction angle) is between 6 and 12 mm.

Table 2. Properties of tyre chips and tyre–sand mixes.

Material	Tyre Particle Size (mm)	Average Unit Weight (Mg/m^3)	Specific Gravity	φ′ (°)	Cohesion (kPa)	Reference
Tyre shreds	25–305	0.64	1.24	21	7.7	125
Tyre shreds	25–305	0.62	1.27	26	4.3	125
Tyre shreds	25–305	0.62	1.14	19	11.5	125
Tyre shreds	25–305	0.63	1.23	25	8.6	125
Tyre shreds	0.85–12.5	-	-	27	7	108
Tyre-Sand mix (20% tyre)	5.6–8	1.580	-	40–41	-	121
Tyre-Sand mix (30% Tyre)	9.5–12	1.550	-	40–41	-	121
Tyre shreds	10 × 10 × 20	0.65	1.08	28	-	115
Tyre-Sand mix (10% tyre)	10 × 10 × 20	1.460	2.25	51	-	115
Tyre-Sand mix (20% tyre)	10 × 10 × 20	1.400	1.94	52	-	115
Tyre-Sand mix (30% tyre)	10 × 10 × 20	1.320	1.82	56	-	115
Tyre-Sand mix (40% tyre)	10 × 10 × 20	1.230	1.71	51	-	115
Tyre-Sand mix (50% tyre)	10 × 10 × 20	1.040	1.53	44	-	115

In addition to acting as lightweight AAs, rubber crumb has also been used as a soil improver in clayey, silty, organic (e.g., peat) and expansive soils [107,111,118,119]. Additionally, the ability of sand-tyre mixes to retain high levels of permeability even under large vertical loads makes the material particularly suitable as drainage media [118,120].

3.2. Construction and Demolition Waste

Construction and Demolition Waste (CDW) describes several materials including, crushed brick, gypsum board, plastics, timber and crushed concrete (which typically accounts for much of this type of waste. The utilisation of CDW materials varies depending on both the waste and the location. For example, in Japan, 99% of concrete waste is recycled [43], and technologies have been developed to improve the qualities of the waste material, enabling it to be used in higher grade applications [64]. The recycling of CDW is also relatively high within the UK; in 2016, 91% of non-hazardous CDW was recovered [52,120–123]. However, in other parts of the world, such as Australia, where only approximately 40% is recycled (usually within low-grade applications), the situation is less positive [121].

Concerns over the stability (disintegration) and lack of understanding of engineering properties of RA and SA, such as crushed concrete, crushed brick and gypsum, have typically resulted in lower recycling rates.

Crushed concrete aggregate (CCA), produced by crushing material recovered during demolition of structures, has been considered inferior with qualities such as lower strength and stiffness when compared to other primary materials. It is the case that CCA could be considered as poorer quality due to the stiff core particles being surrounded by weak layers of mortar [124]. However, the extensive research that has been carried out into the properties of CCA reflects that AAs can exhibit strength and stiffness equivalent or better than higher grade materials when utilised in applications where it is well-compacted [124].

Large quantities of gypsum plasterboard are produced around the world, particularly in Europe, the USA and Japan, which leads to the creation of 15 million tonnes of waste annually, which is disposed of in landfill sites [123]. Research carried out by Ahmed et al. (2015) [125] indicated that waste gypsum (mixed with a solidification agent) could potentially be successfully utilised within earthworks projects, providing a much-needed alternative to landfilling the material. Similarly, another study, which focused on brick dust, showed that the waste could be chemically treated to form a useful SA [126]. Going further, the Bermondsey Dive Under project, undertaken in the UK [53], illustrates that significant quantities of crushed masonry can be successfully adopted as AAs. This study is considered in more detail below as an example of how AAs can be used effectively in place of PA.

Bermondsey Dive Under

Crushed masonry has not typically been used as structural fill material due to concerns over the friability and long-term performance [52]. However, redevelopment work (the Bermondsey Dive Under: BDU) has successfully utilised this material. The BDU project comprised the demolition of 900 m of masonry viaduct together with the construction of 900 m of new structures, including five new bridges, reinforced earth structures and 100 m of retaining walls. In order to avoid the requirement for large amounts of demolition material, i.e., crushed masonry, to be removed from the site and to be replaced with imported fill, research was carried out into the performance of the crushed brick prior to the start of the project [52]. In this case, the financial gains were very clear as the cost of transporting and potentially sending to landfill, such a large amount of material would have had a huge impact on the project budget. The project team identified a number of barriers to the utilisation of recycled materials [52], which reflect general concerns cited previously, i.e., risk-averse (design, construction and client) teams or organisations; overly onerous 'standard' specifications; bespoke nature of projects; resistance to change, lack of motivation; weak leadership and lack of published data that would allow those with aspirational ideas to recycle masonry, to assess more confidently the basis of their designs and manage risk.

The successful use of crushed brick at the site was attributed to three main factors [52]: early introduction of the concept during project planning; motivation of the whole project team (client, engineering, project management, environmental and commercial) and the creation of a business case for the research study; and the design and build team's (the main contractor in this case) willingness to take on and develop the proposals. These key factors illustrate a shift in approach, similar to the ethos of the circular economy, which is potentially achievable for all construction projects. Whilst the project had to incur the initial cost of planning and carrying out the laboratory testing for the waste masonry, this was likely to have been relatively small in comparison to the cost of removing all the waste material from the site. In addition, the project generated useful data that others are able to benefit from in the future (see Table 3).

Ellis et al. (2016) [52] highlighted one of the barriers to the adoption of crushed masonry as a lack of published data. This is a statement echoed by others [50,68,124]. Ellis et al. (2016) [52] went further to cite the need for academia and industry to forge better-working links in this area to disseminate research findings and promote greater uptake of alternative aggregates. Perhaps this is one explanation for the relatively low uptake of alternative aggregate sources despite various studies [49,125–131] proving their likely suitability; the authors concur with this call for better collaboration.

Table 3. Summary of brick properties compared to the requirements for fill material specifications.

Characteristics	Crushed Brick [130]	Crushed Brick [90]	Red Brick [52]	Yellow Brick [52]	Limits for Fill Types [90]		
					1A	6I and 6J	6N and 6P
Gravel %	54	53	92	93	95–100% < 125 mm	100% < 125 mm, 85–100% < 75 mm, 25–100% < 14 mm,	100% < 75 mm
Sand %	40	38	8	7		15–100% < 2 mm, 9–100% < 0.6 mm	
Fine %	6	9			<15%	<15%	<15%
Particle density (Mg/m^3)	-	-	2.32	2.37	-	-	-
Angle of friction φ'	48	51–57	47.5	32	specified by design (<36%)		
Cohesion (kPa)	-	-	15 (0 at low stresses)	49 (0 at low stresses)	specified by design		
LA (%)	36	33–35	37	34	No limits for type 1A, 6L and 6J fills (for 1C general fill and 6P structural fill <50, for 6F1/6F2 capping <60 and 6N structural fill <40)		
Maximum dry density (kg/m^3)	1973	1900–2200	1500	1470	specified by design		
Optimum water content	11.25	12–13.5	9	14.5	specified by design		

3.3. *Recycled Crushed Glass*

The production of glass is energy-intensive, requiring large amounts of non-renewable resources and energy [132,133]; hence, there is a ready market for (certain) recycled glass (Table 4). In the USA, the largest element of municipal solid waste is food and beverage bottles.

Waste container glass cullet (WGC) (crushed glass created from waste glass bottles) is recycled within the glass manufacturing industry (i.e., to create new containers) where possible. This type of closed-loop recycling of WGA is the most energy-efficient use of material, potentially enabling reductions of 215–250 kg CO_2/tonne and energy savings of greater than 1.5 GJ/tonnes to be achieved [134]. However, due to the prevalence of comingled glass collection schemes and the difficult task of separating all WGC back into colour streams (which is required for use in the container glass manufacture due to the differing chemical compositions of each glass colour [75]), significant quantities of waste glass are sent to landfill. Within the EU, this is estimated at 30%, but this figure is much larger in other parts of the world, see Table 4 [134]. The disposal of WGC in landfills is largely due to the lack of knowledge on the engineering properties of the material and the inclusion of contaminating material (non-recyclable glass) [127,129,131,132]. This is extremely wasteful, especially since studies have shown that recycled glass exhibits geotechnical properties similar to PA, such as sands and gravels [126,129], and research shows that recycled glass has some favourable characteristics, particularly when in finer fractions, such as an LA coefficient comparable to natural granite (see Tables 5 and 6). In addition, effective angles of friction should be similar to sands and gravels. Evidence suggests the smaller the particle size (in the coarse-grained size range), the less susceptible to crushing the glass becomes [135]. However, the potential for particle crushing to occur should be considered when deciding on the application of the material, as clearly, this could lead to changes in behaviour [136].

Table 4. Container glass waste: reuse, recycle and landfill figures around the world [136].

Country	Reuse	Recycle	Landfill
USA	0	28	72
Australia	0	37	63
Japan	0	14	86
China	50		50
EU	5	64	31

WGC has numerous possible applications, including drainage material, filter media or drainage blankets and load-bearing material in road pavements and asphalt aggregate projects [126,129,131,136]. Furthermore, waste glass is an inert material and non-biodegradable, thus staying in landfills for up to 1 million years [137], which is an obvious disadvantage in landfills but an advantage for AAs in geotechnical applications. Case studies of WCG being successfully used as pipe bedding material in Australia have been published, and many states in America and New Zealand have adopted the WGC-blends for use in road construction [138].

The varying chemical composition of waste glass, debris content and the ability to sort it by colour are limiting factors in the recycling rates of this material [128,132]. The recycling of flat glass (e.g., windows from buildings or vehicles) is substantially lower than cullet, which is largely due to the stricter quality requirements [138–141]. This is disappointing as it is possible to directly reuse windows (e.g., from buildings), but due to constraints that include: additional time and labour costs for glass removal, ease of damage during transportation and difficulties in matching supply with demand, much of this is landfilled [134,142,143].

Table 5. Market share of glass production in the EU (data from 2007) [143].

Type of Glass	Total Produced (Mt)	Waste Glass Generated (Mt)	Total Recycled (Mt)
Container/packaging	21	17	8
Flat (e.g., windows)	9.5	5.1	2.9
Domestic (e.g., tableware)	1.5	0.8	0.5
Misc. (insulation wood, optical, filament fibres)	5.4	2.9	n/a

Table 6. Aggregate characteristics of recycled glass.

Characteristic	Particle Size (Coarse $\leq$ 19 mm, Medium $\leq$ 9.5 mm, Fine $\leq$ 4.75 mm)				
	Coarse [134]	Medium [13,134]	Medium	Fine [133]	Fine [134]
Specific Gravity	2.5	2.5	2.48–2.49	2.48	2.48
Flakiness %	94.7	85.4	-	-	-
Modified Proctor Values			-		
Maximum Density kN/m^3	-	19.5	17.9	17.4	17.5
Optimum water content %	-	8.8	-	10.5	10
CBR (%)	-	31–32	-	-	42–46
LA (%)	27.7	25.4	-	25	24.8

Table 6. *Cont.*

Characteristic	Particle Size (Coarse ≤ 19 mm, Medium ≤ 9.5 mm, Fine ≤ 4.75 mm)				
	Coarse [134]	Medium [13,134]	Medium	Fine [133]	Fine [134]
φ' (from Direct Shear Test) (°)	-	-	-	-	-
φ' (σ_n = 30–120 kPa)	-	52–53	-	-	-
φ'(σ_n = 60–240 kPa)	-	50–51	-	-	-
φ'(σ_n=120–480 kPa)	-	-	-	-	-
φ' (from CD Triaxial Test) (°)	-	-	47.5	37	-
φ' (σ'_c = 30–120 kPa)	-	42	-	-	40
φ'(σ'_c = 60–240 kPa)	-	41	-	-	38
φ'(σ'_c = 120–480 kPa)	-	41	-	-	35

3.4. Incinerated Waste By-Products

Unlike demolition waste, scrap tyres and waste glass, which can form RA with relatively little processing, there are wastes that, with processing, could produce AAs (i.e., SA). For example, deriving energy from waste (EfW) results in the formation of new wastes that could be processed into AAs. These wastes (e.g., municipal solid waste incinerator bottom and fly ashes: MSWIBA and MSWIFA, and air pollution control residue, APCr) can be hazardous and might require disposal in controlled landfill sites unless remediation ensures the End-of-Waste (EoW) status is achieved.

3.4.1. MSWIBA and MSWIFA

The volume of municipal waste incinerated for EfW is reduced by up to 80–90%, yet EfW plants still produce considerable amounts of ash that are currently either disposed of in landfills (MSWIFA is usually considered hazardous) or reused in relatively low-grade civil engineering applications (MSWIBA is considered potentially suitable for use) [144–151].

Coarser fractions of the ash are typically utilised as bulk fill for road building or drainage material. The finer fractions are more problematic due to variability of material and the potential for leaching (tends to be higher than for the coarser fractions), although it has been used in landfill liners [150]. For this reason, the process of sintering has been adopted to treat MSWIFA [148,151], producing lightweight aggregates.

Lightweight aggregates, defined as materials with a maximum loose bulk density of 1.2 Mg/m^3, BS EN 13055:2016 [152], can present additional challenges (hence they are not suitable for all applications), but they do offer potential benefits (with comparatively low levels of loading on subsoils, etc.) [36,153,154].

Testing on lightweight aggregates formed from sintering PFA (produced as a dry material) indicated that upon contact with water, there was an initial period of rapid absorption that continued for over six to twelve months [155]. This behaviour can be attributed to the range and degree of porosity of the material produced as a result of the sintering process [156]. Gunning et al. (2011) [13] also reported water absorption of 21%, 30% and 29% for sintered PFA, expanded clay and APCr, respectively.

The response to water absorption above potentially calls into question the validity of the standard 24-h period of soaking in water (recommended for lightweight aggregates) under atmospheric pressure before determining the water absorption, freeze-thaw response, bearing capacity, etc., tests and for determination of thermal properties in moist or wet conditions [16,33,68].

3.4.2. Sewage Sludge Ash (SSA)

Each year millions of tonnes (approximately 1.6 Mt in the UK, 8.9 Mt within the EU [157–159] and 78 Mt- which accounts for 70% of industrial waste produced in Japan [43]) of sewage sludge waste are produced as a by-product of the wastewater treatment process sludge each year. Sewage Sludge Ash (SSA) is a by-product of the firing of sewage sludge, and whilst SSA is not classed as hazardous waste and is not subject to higher rates of taxation, alternatives to disposal in landfill are being sought due to the increasingly limited space available (as of 2016 the remaining landfill capacity the UK was 554,751,000 m^3 [121]).

Research has shown that when SSA is sintered at temperatures 1040–1060 °C, a lightweight aggregate is formed. SSA has been used in brick and tile production, an aggregate for concrete and a material used within road bases and embankments [158]). The resulting aggregate is lightweight compared to PA, so the material lends itself to situations where a reduction in imposed load is important. The following benefits of SSA as SA (Table 7, compared to Lytag) were suggested by Cheeseman and Virdi (2005) [159] and Smol et al. (2015) [160]: cost of disposal of SSA in landfills is increasing; there is a shortage of readily available aggregates in some regions of the UK; incinerators tend to be located in urban areas, close to areas of high construction activity; and the process of sintering immobilises hazardous heavy metals and destroys pathogens making them safer to use.

Table 7. Properties of SSA (with 1% clay) compared to commercially available lightweight aggregate, Lytag [159].

Property	SSA (Sintered at 1060 °C, 1% clay)	Lytag
Mean Density (Mg/ m^3)	1.35	1.43
Compressive Strength (MPa)	7.5	7.0
Water Absorption (%)	8	13

3.4.3. APCr

Sintering of ash requires significant energy input. However, an ambient-temperature (or a low temperature when compared to sintering) treatment using the process of accelerated carbonation (ACT) has been developed. ACT carbonates the waste by reacting carbon dioxide with calcium or magnesium [13,14,160–162] and produces AAs from wastes previously considered unsuitable (in their natural state are considered unusable as aggregates due to small particle size, little or no apparent cohesion, and potentially hazardous nature). This process consumes more quantities of carbon dioxide than is released during production [162], hence contributing to carbon sequestration/utilisation efforts whilst also consuming less energy than traditional methods [13,14]. The aggregates produced are lightweight (rounded) particles in the sand and gravel range (see Table 8) and have achieved End-of-Waste status. In the UK (at least), this SA is predominantly used in the manufacture of lightweight construction blocks [13].

Table 8. Properties of carbonated wastes.

Material (Particle Size Range 4–16 mm)	Mean Bulk Density (Mg/m^3)	Mean Compressive Strength (Individual Pellets) (MPa)	Water Absorption (%)	LA (%)
Carbonated APCr (164)	1.025	0.26	18.8	39
Carbonated MSWI/APCR (+sand and cement) (13)(165)	0.900	0.20	29	-

There is currently very little geotechnical information available on these materials, and the authors argue that they have the potential to act as AAs in geotechnical engineering applications and thus merit research in this area.

4. Conclusions

Despite numerous studies highlighting the potential of RA/SA, the construction industry, and by extension geotechnical engineering, still relies heavily on PA for applications that could adopt non-virgin materials. With sources of PA under pressure and landfilling being politically undesirable, now is the time to change the way in which resources are consumed, especially if the stated desire to adopt a circular economy is to be realised.

AAs have been produced from waste streams (such as scrap tyres, ashes, waste glass and demolition rubble) and have successfully been utilised in geotechnical solutions. Despite this, there are several barriers preventing the widespread use of waste materials, including the lack of confidence and perceived risk with product quality, lack of suitable specifications, financial concerns and a lack of awareness. Case studies, such as the BDU project, offer inspiration to others to utilise materials that initially may not seem suitable. In addition, various laboratory studies have been undertaken to further understand the engineering properties of AAs and consider their use in (somewhat limited) geotechnical engineering applications.

It is clear, however, that there is still much to do with regards to AAs before the barriers (or a number of influencing financial constraints is probably beyond the remit of such studies) preventing a much greater utilisation in geotechnical applications are addressed. This includes furthering the understanding of engineering behaviour in more geotechnical contexts (other than road base, etc.), assessing new materials as they become available (such as AAs from APCr) and addressing the required criteria for use (standards) in these applications. If this can be achieved, then greater uptake of AAs may be facilitated, improving the use of non-renewable resources, limiting the amount of waste sent to landfills and moving towards the desired circular economy.

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Abbreviations

AAs	alternative aggregates (comprising recycled and/or secondary aggregates)
ACT	accelerated carbonation treatment
APCr	air pollution control residue
BDU	Bermondsey Dive Under
CCA	crushed concrete aggregate
CDW	Construction and Demolition Waste
CE	circular economy
EfW	energy from waste
EoW	End of Waste
LA	Los Angeles coefficient
MSWIBA	municipal solid waste incinerator bottom ash
MSWIFA	municipal solid waste incinerator fly ash
PA	primary aggregate
PFA	pulverised fuel ash
RA	recycled aggregate
SA	secondary aggregate
SSA	sewage sludge ash
WGC	waste glass cullet

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