GROUND IMPROVEMENT USING PET BOTTLE WASTE AS A POTENTIAL REINFORCEMENT MATERIAL FOR GRANULAR COLUMNS: AN EXPERIMENTAL APPROACH

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Declaration

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Dedication

To the supreme Lord Shri Krishna

Thank you for being so merciful to me and for always guiding me towards the ultimate knowledge

(*For one who sees Me everywhere and sees everything in Me, I am never lost, nor is he ever lost to Me* – Bhagavad Gita 6.30)

In memory of my late brother Ravi Sobhee

It was your wish to see me as an educated, independent and successful woman. Thank you for having big dreams for me; my entire educational journey is dedicated to you.

To my parents, Mr Ramraj Krishnadeo Sobhee and Mrs Savitri Sobhee

You are the best parents one can ask for. All that I have achieved so far is all because of your hard work, dedication and sacrifices. I am who I am simply because of your unquestionable upbringing. There are no words to say thank you for all that you have done for me.

To my dear husband Ashvind Beetul

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To my darling son Rishabh Beetul

You have taught me how to be patient. This was the most highly needed quality on this journey. Although you are so young, you have no idea how you contributed hugely for Mama to achieve her goals. Thank you sweetheart, Mama loves you 'more than infinity' – as you always say it.

Hare Krishna Hare Krishna Krishna Krishna Hare Hare

Hare Rama Hare Rama Rama Rama Hare Hare

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Abstract

Out of the many ground improvement methods aimed at ameliorating the weak engineering properties of certain soils, granular columns are often preferred due to their cost effectiveness and environment friendliness. Despite their high usage in other countries, this technology remains rather unpopular in South Africa. Therefore, this study was undertaken to extend the associated knowledge of the granular column within the local context. In line with the need to develop environmentally sustainable construction technologies, the study incorporated Polyethylene Terephthalate (PET) bottle waste as a reinforcement material for these columns.

Several laboratory experiments were conducted to improve the load carrying capacity and settlement characteristics of a local fine silt. A wet silt bed (prepared at optimum moisture content or liquid limit) was created within a steel cylindrical tank. An ordinary granular column (OGC) or reinforced granular column (RGC) was then installed within the tank and a compressive vertical load was applied to the prepared sample up to a settlement of 50 mm. Reinforcement for the columns was used in different forms and arrangements. The stress-settlement characteristics were electronically captured and subsequently analysed. Post-testing, the deformation of the column was physically modelled by vacuuming out the column material to create an empty opening. A prepared wet mix of plaster of Paris and sand was then poured into the empty hole until it was filled to the top. Once set, the casted column was removed from the tank and its circumference was measured at different intervals up along the length of the column. This process was repeated after each test and these measurements were later used for determining the respective maximum bulging.

The study confirmed that the inclusion of granular columns generally improved load carrying capacity, as well as reduced settlement in weak soils such as fine silts. Also, certain conditions of reinforcing of the columns further improved their performances. From the results, it appeared that reinforcing a column with a concentration of 0.1 % of randomly mixed fibres, and installed in a base soil at liquid limit, produced the largest percentage improvement of 244 % in load carrying capacity. Furthermore, the diameter of maximum bulging was reasonably low and was measured as 144 mm, compared to 150 mm for an OGC which was tested under similar conditions. The outcome of this study considerably extended the understanding of the reinforcement of granular columns using PET bottle waste. Since the concept of reinforcing granular columns with waste is new, several areas were identified for future research to further increase knowledge pertaining to this ground improvement method.

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Notations

Symbols

A	Area of foundation
A_c	Cross-sectional area of column
A_e	Effective area of a single column
A_s	Area of the unit cell
a	Equivalent radius $D_e/2$ given by $\sqrt{A/\pi}$
a_s or A_r	Area replacement ratio
α	Group interaction factor
B	Foundation width
β	Failure surface inclination
cm	Centimetre(s)
c_{avg}	Composite cohesion on the shear surface
C_c	Coefficient of curvature
c_p	Soil cohesion at base or point of granular column
C_u	Coefficient of uniformity
c_u	Undrained shear strength of the soil or undrained cohesion
c_s	Side cohesion of granular column in clay
CBR	California Bearing Ratio
d	Average diameter of the granular column
1D or 2D	One-dimensional or two-dimensional
D	Drained test
D_B	Maximum bulging diameter
D_c	Constrained modulus of the column
D_c or D	Diameter of column
D_e	Effective diameter of a granular column
D_f	Foundation or footing size
D_s	Constrained modulus of the soil
D_t	Diameter of test sample
E	Drained modulus of deformation

E_c	Elastic modulus of the column
E_s	Elastic modulus of the soil
g, kg or Mg	Gram(s), kilogram(s) or Megagrams
G_s	Specific gravity
H	Height of test sample
h	Thickness of the layer of soil
k	A factor of $\frac{4}{\pi}$ and $\frac{2}{\pi\sqrt{3}}$ for square and triangular column layout, respectively
kPa or MPa	Kilopascals or megapascals
K_c	Lateral coefficient of deformation of column
K_s	Coefficient of lateral earth pressure of soil
L_c	Length of column
l	Litre(s)
m	Metre(s)
min	Minute(s)
mm	Millimetre(s)
mol	Mole
$m'_{v,s}$	Coefficient of volume compressibility of the treated soil
$m_{v,s}$	Coefficient of volume compressibility of the natural soil
n	Stress concentration ratio
n_{3D}	Stress concentration ratio considering column lateral deformation
N_p	A bearing capacity factor
N or kN	Newtons or kilonewtons
p	Footing stress
p_o	Foundation pressure
p_c	Column pressure
p_s	Pressure in unreinforced soil
P, F, W, V	Reinforcement content in empirical equation for flakes, fibres, Betatex and Fibertex, respectively
q_{ult} or $q_{ult,c}$	Ultimate bearing capacity of a single column
$q_{ult,s}$	Ultimate bearing capacity of surrounding soil
R_e	Radius of the unit cell or effective radius

r_o	Radius of column
S or s	Spacing between vertical drains or granular columns
S	Stress in empirical equations
S_c	Immediate settlement of column
s_{c1}	Settlement of untreated soil within depth of treatment
s_{c2}	Settlement of untreated soil below stone columns
S_{imp}	Vertical applied stress at a settlement of 50 mm for an improved soil
S_N	Suitability number for backfill material
S_n	Settlement of a natural foundation
S_o	Settlement of the unimproved ground
S_s	Immediate settlement of soil
S_t	Settlement of the composite ground
S_{unimp}	Vertical applied stress at a settlement of 50 mm for an unimproved soil
u	Pore pressure
U	Undrained test
δ	Settlement of foundation
γ_c	Saturated unit weight of the cohesive soil
°C	Degrees Celsius
ε_z	Vertical strain at a depth of z
μ	Stress reduction factor
μ_s	Ratio of stress in column to the average stress over the unit cell area
σ	Increase in vertical stress at any depth z below the footing
σ_1 and σ_3	Principal stresses
$\Delta\sigma_c$	Vertical stress on the column
$\Delta\sigma_s$	Vertical stress on the surrounding soil
$\Delta\sigma_{cx}$, $\Delta\sigma_{cy}$, $\Delta\sigma_{cy}$	Stresses on the column in the *x*, *y*, *z* directions, respectively
$\Delta\sigma_{sx}$, $\Delta\sigma_{sy}$, $\Delta\sigma_{sz}$	Stresses on the soil in the *x*, *y*, *z* directions respectively
σ_o	Horizontal earth pressure at rest in total rest
σ_l	Reduced limit pressure
σ_{ro}	Total in situ lateral stress
σ_r	Lateral stress from the surrounding soil
$\Delta\sigma$	$p_cK_c - p_sK_s$

$\Delta\sigma_z$	Average vertical stress applied on the composite foundation
σ'_{vc}	Ultimate vertical effective stress
φ_{avg}	Composite angle of internal friction
ϕ_c or φ'_c	Angle of internal friction of column material
φ_s	Angle of internal friction of the granular soil
Ψ	Failure plane angle in the soil
Ψ_p	Passive failure plane angle within the column ($\Psi_p = 45 + \frac{\phi_c}{2}$)

Abbreviations

AGIS	Agricultural Geo-Referenced Information System
ASTM	ASTM International which was previously known as American Society for Testing and Materials
BS	British standards
CSIR	Council for Scientific and Industrial Research
F	Fibres
GV	Fibertex geotextile
GW	Betatex geotextile
HDPE	High density polyethylene
LDPE	Low density Polyethylene
L	Layering
LL	Liquid limit
M1	Optimum moisture content (OMC)
M2	Liquid limit (LL)
MDD	Maximum dry density
NEM: WA	National Environmental Management: Waste Act
NHBRC	National Home Builders Registration Council
NWMS	National Waste Management Strategy
OMC	Optimum moisture content
OGC	Ordinary granular column
P	Flakes

PET	Polyethylene Terephthalate
PETCO	PET Plastic Recycling South Africa NPC
PETRA	PET Resin Association
POLYCO	Polyolefin Responsibility Organisation NPC
PP	Polypropylene
PS or PS-E	Polystyrene or expanded polystyrene
PSPC	Polystyrene Packaging Council
PVC	Unplasticised Polyvinyl Chloride
PVD	Prefabricated vertical drain
R	Random mixing
RDP	Reconstruction and Development Programme
RGC	Reinforced granular column
RSD_r	Repeatability relative standard deviation
S	Sand
SA	South Africa
SAPRO	South African Plastics Recycling Organisation
SAVA	Southern African Vinyls Association
SPT N	Standard Penetration Test N value
SRR	Settlement reduction ratio
UK	United Kingdom
US	United States of America
UV	Ultraviolet

Chapter

1

Introduction

1.1 Introduction

Previously, soils having weak engineering properties were categorized as economically and technically unfeasible, thereby urging engineers to either replace the in-situ soil with an engineered fill or to opt for an alternate location for the project (Raju & Valluri, 2008). Over the years, the continuing evolution in engineering research and application has manifested into a series of ground improvement technologies whereby each one is mostly suited to address the challenges faced when constructing on these 'problematic' soil types. Although there are many techniques for treating soft soils, the following is a list of those which share some common benefits: dynamic compaction, chemical stabilisation, granular columns, preloading, vertical drains, indirect and direct ground freezing, grouting and reinforcing inclusions such as geosynthetics.

Out of the many methods of improving weak soils, granular column is one such technology which has established itself as highly versatile and cost effective in improving soft soils (Isaac & Madhavan, 2009). Granular columns, also known as stone or sand columns, are installed by using either the vibratory or the ramming technique (Som & Das, 2006). Compared to the vibrated columns which use a vibratory probe during the installation procedures either by replacing or displacing the in-situ soil, rammed columns use a rammer which is repeatedly dropped in a pre-bored opening in the ground through a certain height. This ramming compacts the replacement material in stages to form the stiff granular column which is levelled with the ground. Although both types of installation are effective, rammed columns appear to be more advantageous, especially in developing countries, since they require less sophisticated instrumentation and specialised skills when compared with the vibrated ones (Smadi, 2016). As such, advanced technology is not required to install rammed columns, which makes them more economical than vibro ones.

Generally, the most popular advantages of soil improvement are densification, drainage, mitigation of liquefaction, settlement control, reduced permeability and enhanced bearing capacity. The aim is to improve the engineering properties of the land for construction in an economical, fast and environmentally friendly way.

1.2 Background to the study

Granular column is an old ground improvement technique (Al-Obaidy, 2017; VGNL, 2011; Hughes & Withers, 1974; Arman et al., 2009). According to Al-Obaidy (2017), these types of columns have been used in Iraq since probably the 2^{nd} or 3^{rd} century B.C. This was revealed through archaeological works of Hatra, in Northern Iraq. Evidence of granular columns was obtained in the forms of holes which were filled with pieces of rock and covered with lime. It was further discovered that rock discs were present within the columns whereby their diameters were identical to those of the columns. They were positioned at different depths down the column. Another documented and successful example of this application was in the construction of one of the seven wonders of the world, the Taj Mahal, which was completed in A.D. 1653 (VGNL, 2011). Hand-dug pits were created and filled with stones to form a strong foundation system for the still-standing structure.

Figure 1.1: The Taj Mahal in India

Following the Indians, the French military engineers also successfully employed the technique in 1830 to provide support for heavy foundations of ironworks carried on soft estuarine deposits (Hughes & Withers, 1974). Stakes were driven in the ground and eventually removed to form holes which were then backfilled with crushed stones. These 0.2 m diameter granular columns were installed at depths of 2 m and they each had a load carrying capacity of 10 kN. Despite their use in ancient times, the method was long forgotten until they were rediscovered in the 1930s as a by-product of vibro-flotation (a method typically used for densifying granular soils through vibration generated from a vibrator upon insertion into the ground). By the 1960s, the technique regained its popularity in the form of vibro-columns (granular columns which are installed using the vibratory approach) which were installed in cohesive soils (Hughes & Withers, 1974; Arman et al., 2009; Ambily & Gandhi, 2007). Today, granular column has achieved wide reputation in improving the grounds of some construction projects in United States, Europe and Asia (McKelvey et al., 2004).

1.3 Problem statement and research motivation

1.3.1 Overview of the research motivation

This section provides the problem statement and motivates the need for this study. At the onset of this research project, the problems related to the proposed technique were assessed, whereby 2 primary research motivations were identified to justify the need for investigating the granular column technology. Figure 1.2 outlines these and the following subsections elaborate on each one of them.

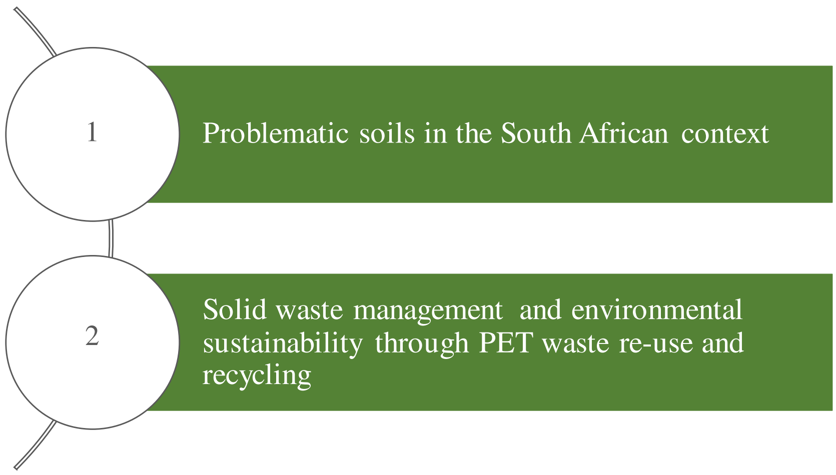


Figure 1.2: Two primary motivations for this research

1.3.2 Problematic soils in the South African context

Problematic soils are those which tend to increase the complexity of engineering design and subsequently the erection of structures. These soils exhibit their behaviour based on their chemical composition or on the associated changes in environmental conditions. Out of these, Diop et al. (2011) reported expansive soils, collapsible soils, soft clays, and dispersive soils as the more problematic ones in the South African construction industry. Table 1.1 provides brief descriptions of these soils with some common methods of treatment.

Table 1.1: Description of some common types of problematic soils in South Africa

Type of problematic soil	Description	Common methods of treatment
Expansive soils	These are soils which expand in the presence of water and shrink when they dry out. The swelling and drying cause a continuous change in volume which may result in cracking of buildings and surrounding structures, including pipe lines and underground infrastructure.	– Replacement of expansive soil by a non-expansive material – Soil stabilization using lime or calcium oxide – Drainage controls including cut-off walls, sand columns
Collapsible soils	Despite the loose particle arrangements, these soils can sustain high pressures, without any volume change, under unsaturated conditions. Upon saturation, they undergo sudden and large deformations. As a result, considerable structural damage may occur.	– Prewetting prior to dynamic compaction – Stone columns – Chemical stabilization
Soft clays	Soft clays are usually highly compressible and have low shear strengths. The particles are very fine and hence they have low permeability. Consequently, consolidation takes longer and thus influences the rate of settlement. Common issues related to construction on soft clays are slope instability and structural failures.	– Soil replacement – Chemical stabilisation – Granular columns
Dispersive soils	Dispersive soils are soils which contain high percentages of exchangeable sodium ions. Although they may appear to be stable clays, the particles are easily dislodged when exposed to water, especially flowing water. Dispersive soils are highly susceptible to erosion, thereby causing failures in structures such as roads, embankments, dams and buildings.	– Soil stabilisation using lime is highly popular – Stone columns

In South Africa, the Council for Geoscience incorporates 13 geotechnical parameters in their 1:50 000 scale regional geotechnical mapping programme. Out of these, 5 are the problematic soils referred to as active, collapsing/settling, dispersive, acidic and erodible soils (Diop et al., 2011). The National Home Builders Registration Council (NHBRC) of South Africa, however, classifies problematic soils in 3 categories (expansive, collapsible and compressible) in its Home Building Manual (2014). In a report written by Diop et al. (2011) for the Council of Geoscience, it is emphasised that expansive soils occupy 35 % of the soil coverage of South

Africa. On the other hand, a study conducted by the Agricultural Geo-Referenced Information System (AGIS, 2011) on the South African soil coverage revealed an approximation of 50 % of soils of poor properties, which complicates the design and execution of civil engineering projects.

Problem soils have long been a challenge in the building sector in South Africa. Although little attention was given to the associated problems with these types of soils, the population growth and rapid development in the country has necessitated the need for larger areas of land, owing to the higher infrastructure demand. Engineers are thus faced with the challenge of satisfying the infrastructural requirements through improvement of the previously so-called "unfeasible development land". As such, ground improvement methods have gained huge popularity. In 1985, the South African Institution of Civil Engineers organised a local conference with the focus on problem soils. A second conference was held in 2008, by the University of Pretoria, to address similar local ground issues.

Despite the many existing techniques of treating problematic soils on the global market, granular column was considered as one of the most relevant and beneficial technique to be researched on since it satisfies the design needs in low capacity structures, while integrating an environmentally sustainable criterion by using naturally existing materials for the column. An extensive amount of research has been conducted on the use of granular columns as a means of improving soils of poor geotechnical properties and the results have regularly shown the benefits of ground improvement through this technology (Al-Obaidy, 2017; Sobhee-Beetul, 2012; Ambily & Gandhi, 2007; McCabe et al., 2007; McKelvey et al., 2004; Hu, 1995).

Even though, the granular column technique is gradually drawing the attention of some designers (Van Der Westhuizen & Parrock, 2010; Franki, 2018), the high reluctance in its application, especially in South Africa, is unavoidably noted. It is believed that a deficiency in adequate design information, research findings and specialised skills pertaining to local ground conditions, may possibly be the reason behind the minimal use. In 2012, Sobhee-Beetul studied the possibility of using granular columns to improve a South African soil of poor geotechnical properties, to accommodate for desired ground conditions such as increase in load carrying capacity and reduction in settlement. Positive results from the work encouraged further research in this line to extend the current knowledge in this area of ground improvement while also creating awareness about the efficiency of the technique. In fact, Sobhee-Beetul (2012) recommended that an in-depth analysis of the failure mechanism of the columns needed to be undertaken. Hence, this study investigated the use of rammed granular columns to improve a weak soil in South Africa, through laboratory experiments. Load carrying capacity, settlement reduction and the deformation characteristics of the columns were the primary factors to be explored. But, the columns were modified compared to the ones used by Sobhee-Beetul (2012) which generated additional research interests; the soil reinforcement theory was applied when these columns were internally reinforced.

1.3.3 Solid waste management and environmental sustainability through PET waste re-use and recycling

Besides the engineering nature of this work, an environmental aspect was also considered whereby a contribution to the plastic waste management was envisioned. Used PET plastic bottles (in the forms of flakes, fibres and geotextiles) were included in the granular columns to further improve their strength and settlement reduction characteristics. The different forms of PET were anticipated to behave as soil reinforcement materials. According to Sobhee (2010), HDPE shopping plastic bags can potentially reinforce soils efficiently. Sobhee recommended that thicker plastics needed to be studied for reinforcing soils. Besides, significant amount of work had been conducted in this area globally whereby different types of plastics had been used as soil reinforcement materials (discussed later in chapter 2); as such, the proposed modified technique of granular column was justified.

At present, research and technologies are generally migrating towards "greener" approaches, following the devastating effect which development in general has brought to the world environment. Similar to the rest of the world, South Africa is also enforcing laws to maximise the use of resources. In the Waste Act 2008, the government emphasised on the ***reduction, re-use, recycling and recovery*** of waste (NEM: WA 2008, 2009). It is specified that waste must be treated and disposed of in such a manner that health or the environment are not endangered while also avoiding any nuisance through noise, odour or visual effects. Since then, efforts are being made by the different industries into managing waste, in general, in a sounder manner. Nevertheless, the problem is not restricted to waste disposal only but has eventually created a concern of where to dispose of since most landfills appear to be nearing their end life according to the Council for Scientific and Industrial Research (CSIR, 2011). CSIR further added that finding suitable land for landfill sites is becoming complicated. The quest for housing land further increases the complexity of the problem. Hence, CSIR (2011) proposed, as part of their solution, to divert some recyclables from the landfill, thereby extending the life span of current landfills in the country. The waste hierarchy, as shown in Figure 1.3, indicates the preferences for solid waste management, with prevention being the most desirable and disposal the least preferred. By incorporating waste PET bottles into the granular columns, the amount of these wastes destined for landfills was anticipated to reduce.

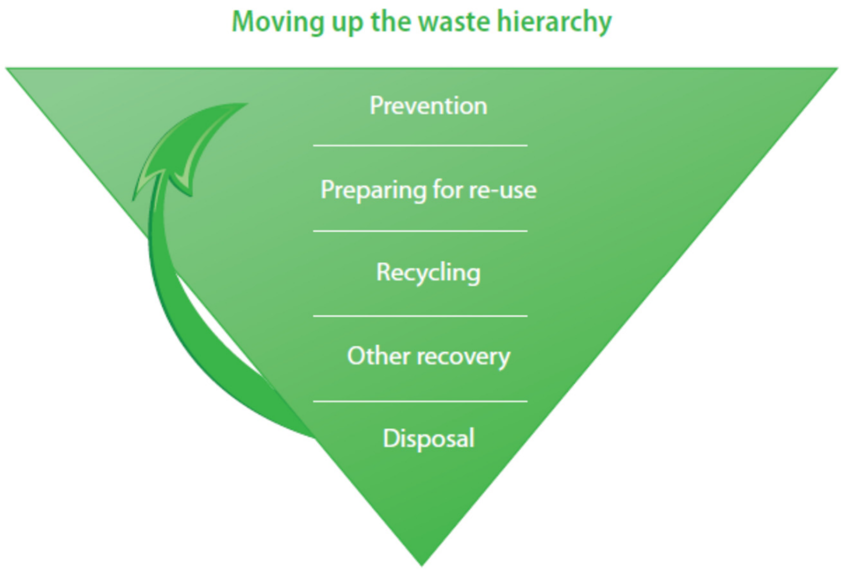


Figure 1.3: Waste management hierarchy (European Union, 2010)

Plastics SA, a representative of all sectors of the South African Plastics Industry, released their 2017/2018 annual report regarding plastic recycling in South Africa (Plastics SA, 2019). According to this report, 313 780 tons of plastics were mechanically recycled in South Africa; this corresponded to a recycling rate of 43.7 % for 2017. Compared to the 1 492 000 tons of the virgin material which was converted in 2017, the margins for recyclates seemed low. Nevertheless, a drop of 5.2 % was recorded in terms of the recyclables which were disposed of in landfills. Furthermore, the tonnages of plastics which were recycled in 2017 saved 214 220 tons of carbon dioxide, while also reducing the volume of space which would have been used to store them in landfills. In fact, this was quantified as a saving in landfill space which was equivalent to filling 714 Olympic size swimming pools. Plastics SA has set itself recycling targets to achieve “Zero plastics to landfill by 2030”. Polymer associations such as PETCO, POLYCO, PSPC, SAVA and SAPRO have willingly taken the commitment to work towards this vision. Seven key areas were initially identified (shown in Figure 1.4) to achieve this goal such that the objectives of the plastic industry were in line with the Government's National Development Plan.

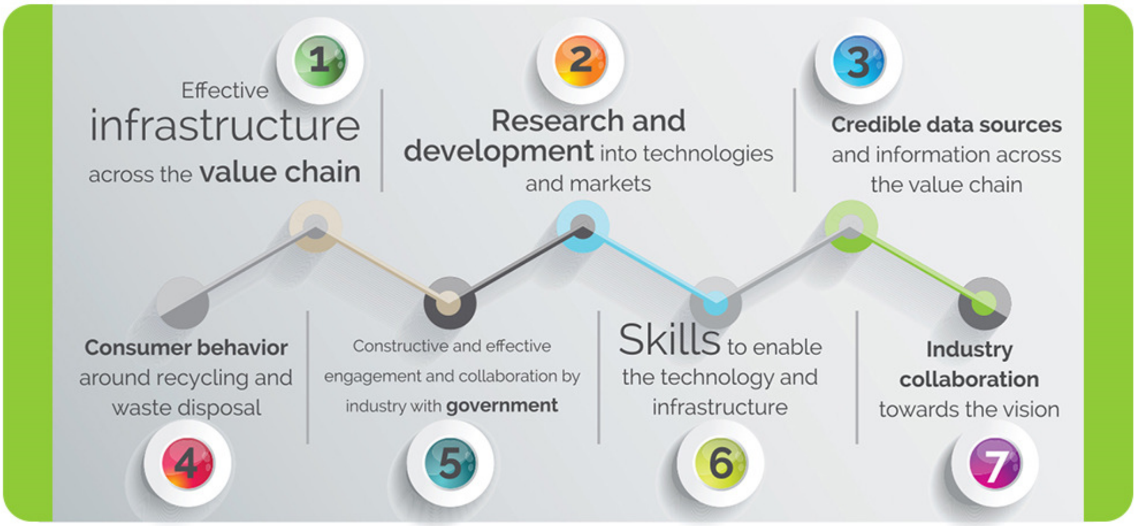


Figure 1.4: 7 Key areas to achieve "Zero plastics to landfill by 2030 (Plastics SA, 2016a)

PETCO, the company which was incorporated in 2004 and which represents the polyethylene terephthalate (PET) plastic industry, self-regulate PET recycling in South Africa (PETCO, 2014). This was necessary to improve the management and recycling of large volumes of post-consumer products made from PET. In the 2017/2018 annual report by Plastics SA, PET was reported as the fourth most used plastic material in the country, with PE-LD/PE-LLD being the foremost one (Plastics SA, 2019). Figure 1.5 shows some typically used PET bottles locally.

Figure 1.5: Typical PET bottles used locally

PETCO (2018a) claimed that the recycling of post-consumer beverage PET has grown over the years to reach a recycling rate of 65 % in 2017. Nevertheless, there still remains a significantly large volume of this plastic which is being dumped in landfills. The destiny of these bottles is believed to play a notable role in the construction industry, more precisely in ground improvement. Therefore, this study proposed the inclusion of the used PET bottles into the granular soil to be utilised for constructing the columns and was termed as reinforced granular columns. The latter were expected to have better load bearing strengths after reinforcing. Consequently, this technology was expected to contribute to minimising PET waste bottles to the landfills by satisfying key area number 2 from Figure 1.4, thus justifying the importance of this research. In addition, the inclusion of PET was anticipated to reduce the volume of sand used, thereby possibly contributing to the minimisation of the use of a natural resource while simultaneously reducing the carbon dioxide emissions produced during sand mining.

1.4 Aim and objectives of investigation

The main aim of this study was to investigate the possibility of enhancing the load carrying capacity and settlement reduction of a local fine soil by improving it through the installation of reinforced granular columns, in a laboratory environment. Waste PET plastic bottles were used in different forms to reinforce these columns. To understand the effect of each of these on the performance of the improved ground, several objectives were established. These included the following:

a) Identify and gather the different potential reinforcement forms of materials which are generated from waste PET bottles during the recycling process.
b) Determine the effect of each type of PET reinforcement on the stress-settlement relationship with a variation of the following factors: arrangement of the reinforcements within the columns, amount of reinforcement used and the moisture content of the base soil.
c) Quantify the extent of improvement achieved in terms of the load carrying capacity, stress concentration ratio (n) and the settlement reduction ratio (SRR).
d) Establish the maximum bulging diameter, and its corresponding position, by physically modelling each column post-testing.

e) Develop empirical equations to relate the effect of reinforcement content on the maximum vertical applied stress and the largest bulging diameter, for a maximum settlement of 50 mm.
f) Evaluate the influence of the reinforcement content on the quantity of sand used to form the columns.

1.5 Key research questions

Previous studies have highlighted the associated benefits of improving soils, of relatively poor mechanical qualities, with granular columns (e.g. by Hughes & Withers, 1974; Ambily & Gandhi, 2007; Krishna & Madhav, 2009; Sobhee-Beetul, 2012). Unrelatedly, the positive effect of reinforcing sands with plastic waste strips has been noted (Benson & Khire, 1993; Choudhary, Jha & Gill, 2010; Sobhee 2010). While the methods of granular columns and soil reinforcement are rather independent, this research proposed a combination of both to produce a new technology – reinforced granular columns. The following are key research questions addressed in this study:

a) To what extent is the improvement in load carrying capacity and settlement reduction affected by the degree of saturation of the base soil?
b) What are the different types of materials, derived from PET waste, which exist on the South African market and which ones can potentially reinforce the granular columns?
c) How is the performance of the improved ground related to the type, quantity and arrangement of the reinforcement used?
d) What approaches can be adopted to physically model the deformation shape of each column post-testing?
e) Under what testing conditions do the columns undergo the highest lateral deformations?

1.6 Major hypothesis in this study

In this study, the hypothesis was thus: '*The reinforcing of granular columns with waste PET further improves the load carrying capacity and settlement reduction of fine soils having poor geotechnical properties, compared to when ordinary granular columns are used.*' More precisely, the following were supposed:

a) Inclusion of any unreinforced granular column in a wet fine soil raises the load bearing stresses while simultaneously reducing settlement.
b) Reinforcing the columns by the proposed plastic material enhances the strength and settlement properties of the treated soft soil.
c) Columns installed in wetter base soils produce lower load bearing stresses although the percentage improvement herein is higher than that in base soils of lower level of saturation.
d) For the different types of reinforcement in this study, there is an optimum weight of PET which produces the maximum improvement in load carrying capacity.

1.7 Scope of work

A review of the existing literature and an intensive laboratory testing programme formed the primary methods of investigation. Several bench-scale experiments were conducted in a 300 mm diameter steel tank. The main findings included the stress-settlement characteristics and the deformations of the vertically loaded reinforced granular columns, for a maximum allowable settlement of 50 mm (based on Eurocode 7 for normal structures). To establish the effect of different parameters, selected variables were studied namely: moisture content of the base soil, type of reinforcement, quantity of reinforcement and arrangement of reinforcement. However, the sand used to form the columns and the silt utilized for the host soil were kept constant throughout the investigation; the column diameter was also kept fixed in all the tests. In addition, only reinforcements, which were derived from PET plastic, were used.

1.8 Research limitations

Since the main aim was to identify the diverse types of products from PET waste recycling, and to investigate whether they could possibly be used as reinforcements for granular columns to further improve their performances, the principle focus was on generating the stress-settlement and deformation characteristics of the columns when different testing conditions were used. Consequently, limitations had to be imposed to achieve the intended aims. The following are the most relevant ones which were identified:

- Group study of columns: From the literature review, several researchers had highlighted that the behaviour of a singular column was equivalent to that of any one column within a group of them, while some had claimed that a difference in performance existed and these may not be ignored. Nevertheless, the study investigated a single column based on the unit cell concept which dictated the validation of several of the existing design approach.

- Pilot scale tests: Large scale tests are highly beneficial in understanding the actual behaviour of the columns in the field. It further simplifies an understanding of the scale effect of the small models which have been used in the laboratory experiments. However, as many related parameters and concepts were yet to be investigated, pilot scale tests were omitted since it would have been economically unfeasible to execute them.
- Effect of column length: A review of the existing literature had highlighted the effect of column length on the performance of granular columns. In the present work, the length of the columns was kept constant at 400 mm since the study was building on to the research work by Sobhee-Beetul (2012) who utilised a similar column length.
- Drainage benefits: Granular columns are widely used for their drainage capabilities. But, in this research, it was not considered since the purpose was to rather identify which reinforcement material performed better with regards to improvement in load carrying capacity and settlement reduction.

1.9 Ethical considerations

Before any laboratory undertakings, an ethics approval was required from the University of Cape Town regarding the type of work associated with the research. An ethics form was submitted to the faculty office whereby detailed clarifications were stated regarding any ethical content. Since the scientific work was purely experimental and was conducted in a university laboratory, whereby the well-being of any individual involved was not compromised, the research was granted an approval by the ethics committee within the faculty.

1.10 Outline of the study

This thesis presents six chapters, a list of references and an appendix section. A description of each chapter is as follows:

Chapter 1 introduces the research topic, which is followed by some background information. The problems of concern are stated, which provides the motivation for undertaking this investigation. Furthermore, the aims and objectives of the investigation, and subsequently other important components are presented. These include the key research questions, the major hypothesis, and the scope and limitations of the work.

Chapter 2 reviews the literature related to the topic. The purpose of this chapter was to familiarise the author with the theories, applications and studies related to the topic. This exercise assisted in identifying key gaps in the existing information, following which a research approach was devised to achieve the aims and objectives of the investigation.

Chapter 3 describes in detail the design and fabrication of the equipment, the materials used, the preparation of the samples and the execution of the experiments. Information pertaining to the load-settlement behaviour, in addition to the measured deformations of the tested columns, formed the data obtained from the series of laboratory tests.

Chapter 4 involves the processing of the raw test data to obtain the different stress-settlement curves, as well as to determine the maximum column deformation achieved for each test.

Chapter 5 presents the analysis and the discussions of the results obtained in Chapter 4. From the processed data, several relationships were established to understand the effect of the variables on the performance of the columns. Additionally, the effect of the variables on the maximum deformations was also established.

Chapter 6 summarises the key findings from this fundamental research. A list of recommendations for further research is also provided, whereby their relevancy is targeted at both local and international research.

Chapter

2

A review of literature covering main concepts of the study

2.1 Introduction

Existing knowledge relating to granular columns and soil reinforcement is in abundance. However, very little work has been done with regards to reinforced granular columns. Therefore, an intensive review of the available resources was undertaken independently to understand the basic principles of each method, while also identifying the key factors which influence their behaviour. Figure 2.1 is a summary of the main sections covered in chapter 2. The chapter is broken down into sections and sub-sections. The firsts section is an overview to enable the reader to follow the ideas presented herein while, the second section elaborates on some common ground improvement techniques which share certain similar benefits of granular columns. A summary of these methods is then presented to demonstrate the advantages of these columns, following which a section is written about the relevant theory and current knowledge; significant amount of information is presented which covers aspects such as applications, theories and research progress.

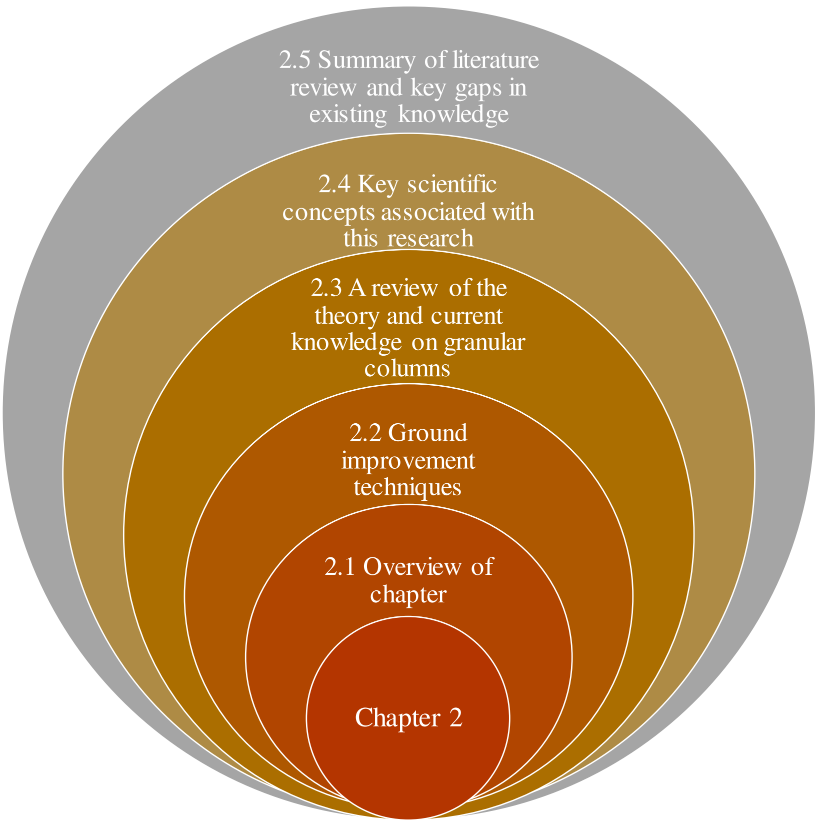


Figure 2.1: Overview of the literature review

Since a substantial component of the research is anticipated to assist with plastic waste management, the subsequent section emphasises on the appropriate legislation responsible for waste re-use and recycling. A description of the types of locally available plastic is further specified, which precedes a sub-section featuring soil reinforcement using plastic waste. The chapter ends with a summary of the gathered information, from which some key gaps in the existing knowledge is identified.

2.2 Ground improvement techniques

2.2.1 Introduction

The continuous expansion of the different ground improvement techniques has generated a series of methods capable of modifying the soil properties to suit the diverse geotechnical constraints in construction projects. This considerable fast progress has encouraged the sub-categorization of the techniques mainly under the following: mechanical, hydraulic, chemical, inclusions and reclamation. Each technology normally serves a main purpose although secondary, or multiple benefits may exist. In this section, emphasis is specifically drawn on the most common techniques which exhibits improvement in bearing capacity or in reducing settlement. The methods are briefly described in the following sub-sections and a comparison of their characteristics is subsequently drawn to highlight the suitability of each of them.

2.2.2 Some selected methods

2.2.2.1 Preloading

The concept of preloading involves the application of a surcharge to the ground over a long period of time prior to construction. In this application, surcharging may be temporary although in some cases it is permanent since it forms part of the engineering design for the project. Normally, the applied load is higher than the one which is anticipated to be exerted on the ground post construction. Preloading, which is mostly effective on normal to lightly over consolidated silts, clays and organic deposits, may be employed before or after construction (Bowles, 1997). Common applications of this technique involve road embankments, bridge abutments, warehouses and storage tanks.

Preloading is essentially used for two main reasons: increase in the bearing capacity and reduction in the compressibility (that is settlement) of a weak ground. Upon application of the temporary load on a soil of low permeability, the excess pore water pressure increases. Gradually, the pore pressure dissipates, thus increasing the effective stress. The weak ground therefore consolidates to form a stiff stratum of relatively low to negligible compressibility, once the surcharge is removed. However, any presence of excess pore pressure, post the removal of the load, must be catered for to avoid undesired settlement under the final structure. Preloading can be achieved by any of these methods: embankment loading, preloading through final structure, lowering of water table, inundation or preponding, vacuum preloading and jacking. Figure 2.2 shows preloading through the placement of a temporary embankment.

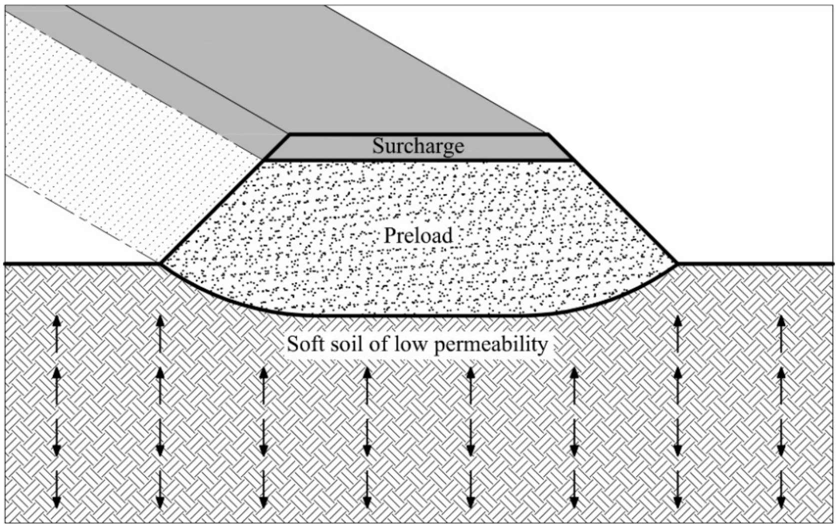


Figure 2.2: Preloading through an embankment

This technique is often associated with cost effectiveness in terms of execution and monitoring. Also, it produces grounds of uniformly improved properties and reserves the possible re-use of the fill material for other purposes on the same project. However, transportation of this fill material to the actual construction site can be a costly procedure. In addition, it takes a long time (several months) to achieve results, thereby causing delays in construction which in turn raises the project cost.

2.2.2.2 Vertical drains

The consolidation of fine grained cohesive soils is generally very slow since the packed particle arrangement increase the degree of saturation while reducing the permeability. Therefore, the rate of settlement for these types of soil is relatively low since excess pore water pressure dissipates over a long period of time. Under these circumstances, vertical drains are installed to promote rapid drainage, thereby accelerating the rate of settlement. This rapid drainage is explained by the shortening of the drainage paths created within the clay. Typical vertical drains are: prefabricated vertical drains (PVD), sandwick drains and band drains.

Vertical drains are mainly installed by a mandrel method, that is, the drains are placed in a mandrel and driven into the ground. Another method of installation involves pushing them into pre-bored holes. These draining elements are normally installed in either a square or a triangular pattern whereby columns are placed at a spacing of S, with the effective diameter of each drain being 1.13S and 1.05S, respectively (Han, 2015; Bell, 2004). During the design process, the spacing of the drains is critically analysed since it is the governing design factor in reducing drainage path. Vertical drains are often used with preloading to speed up consolidation through the fast drainage as shown in Figure 2.3.

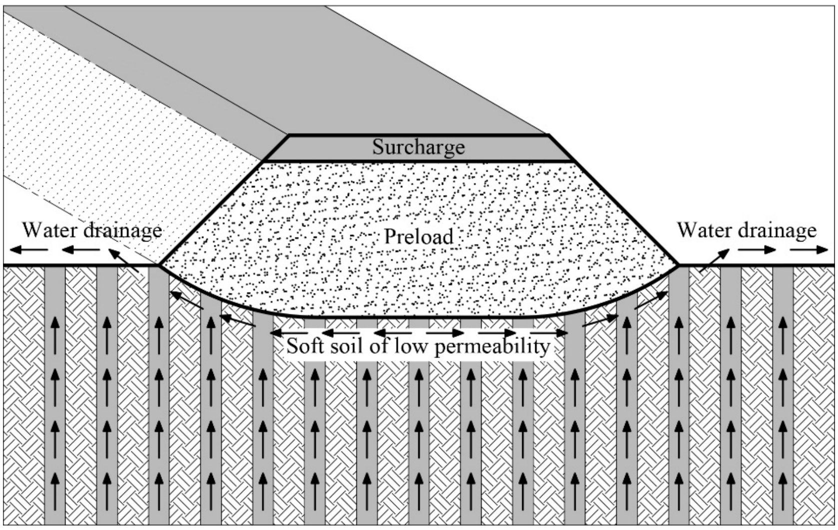


Figure 2.3: Vertical drains used with preloading to accelerate consolidation

Prefabricated drains can be a fairly economical process; it also fastens the process of bringing a land to service, with regards to geotechnical engineering constraints. Nevertheless, installation under certain conditions can be quite costly. In addition, improper installation can cause necking thereby reducing the drainage efficiency.

2.2.2.3 Ground freezing

Ground freezing involves freezing any in situ liquid water within the soil to obtain a solid stratum, thereby increasing its bearing capacity and reducing the permeability to nearly zero. This method mostly adapts to finer soils such as silts and clays, although it can be used in any type of soil or rock. However, the quantity of water present in the soil dictates the effectiveness of the technique in terms of the strength and permeability achieved. For saturated soils, an impermeable stratum is usually formed. In cases where water content is not sufficient to fill up all the pores, extra water is added. An intensive site investigation is usually recommended for the most efficient design.

Two ways of freezing the ground are via the indirect or the direct method. The indirect method uses a secondary coolant (calcium chloride – brine) to remove heat through the ground driven pipes while the direct method extracts heat by means of a primary refrigerant (liquid nitrogen). The system of heat extraction in the direct method increases the speed of freezing thereby making it more efficient than the indirect method. Ground freezing is mainly applied in tunnel excavation, earth support, groundwater cut-off, soil stabilisation and temporary underpinning of bordering structures during permanent underpinning (Jessberger et al., 2003). Figure 2.4 is a simple illustration of the process of ground freezing.

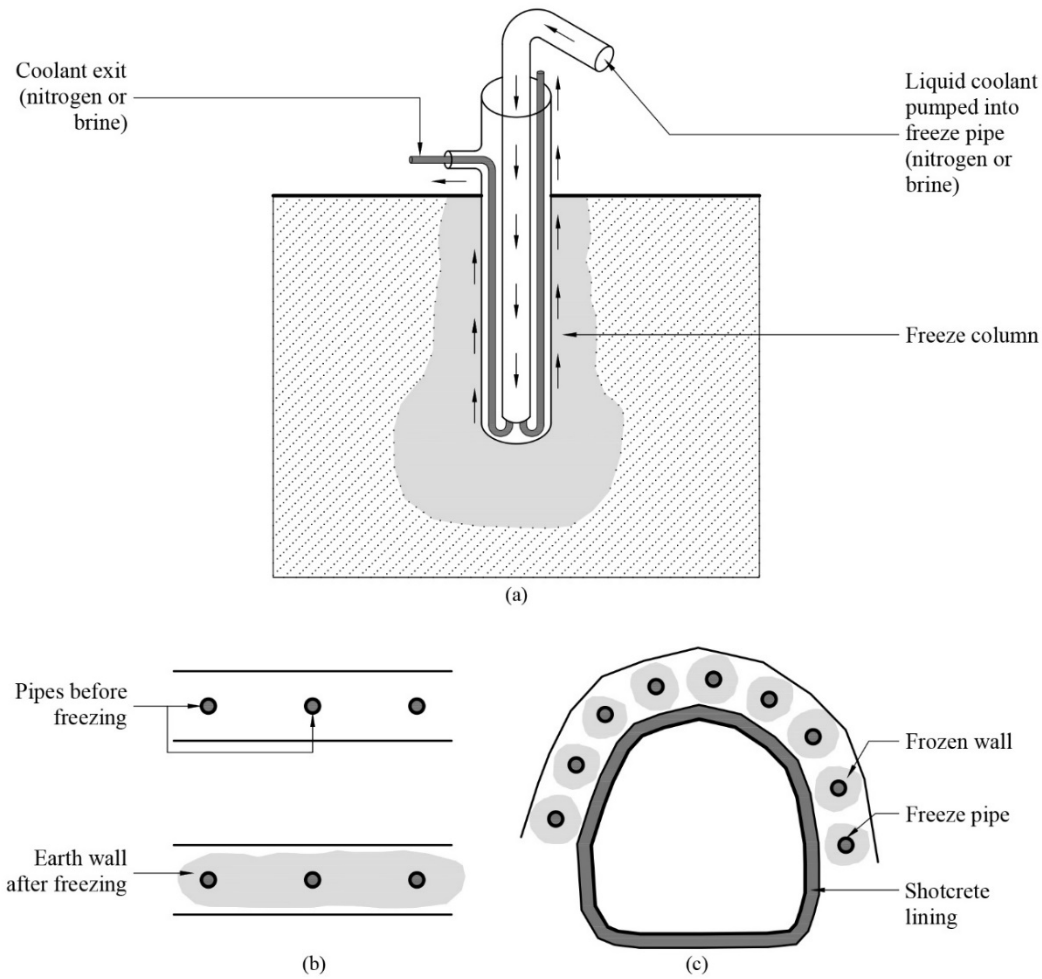


Figure 2.4: (a) Illustration of the process of ground freezing, (b) frozen earth wall, and (c) application of ground freezing in tunnel excavation

Ground freezing requires no extraneous materials and the cooling plant can be used on different projects. Furthermore, it can act as a temporary support during excavation of very wet soils. Having undergone freezing, a ground is capable of reversing to its original state. Nonetheless, ground freezing certainly have some drawbacks. In regions of high water flow at high temperatures, freezing may be impossible. Also, there is the possibility of frost heaves occurring. On the other hand, melting of the frozen water can also promote an increase in permeability as a result of ice lenses build ups forming enlargement of fine fissures. In terms of speed, the application of the indirect method is slow although it can be less expensive than the direct method.

2.2.2.4 Chemical grouting

Grouting, a popular process for many years, is generally used to fill voids in soils or rock fissures with a fluid substance such that the grouted soil of higher strength is more stable while simultaneously being less permeable. Applicable for most soil and rock types, this technique is more prominent in the following areas: settlement reduction, mitigation of liquefaction, repairs around tunnels or beneath dams, void filling, underpinning and control of ground movement during tunnelling. Table 2.1 briefly describes a few of the approaches to grouting, out of the numerous existing ones.

The flexibility of each approach encourages their use since voids, fissures or fractures may be filled to produce a better performing ground. Also, the numerous techniques allow their suitability for different types of soils, even to existing structures. Nevertheless, the process, which requires large working space, can sometimes be expensive. Despite their ability to produce high strength soil-grout matrices, failure in determining the accurate strength of the grout and its injection pressure may produce inefficient results. Another drawback of the technique is its negative impact on the environment. Grouting is not typically ideal in this regard since it makes use of cement which is manufactured from large quantities of raw materials. Cement production additionally results in high release of solid waste materials and significant levels of carbon dioxide emissions.

Table 2.1: Different types of grouting

Type of grouting	Brief description	Most common application
Compensation	• Intentional fracturing of the ground through the injection of a cement grout at high pressure • Fractures form interlinked veins of grout which behave as a soil reinforcement • An increase in amount of fractures often cause heaving of the overlying soils and structures	• Control or reversal of settlement of existing structures
Compaction	• Injection of a relatively stiff grout in the ground to form a bulbous cement mass • Displacement and densification of the surrounding soil produced by the bulbs	• Increase in bearing capacity • Reduction in settlement • Mitigation towards liquefaction • Pre-treatment of sinkholes • Stabilisation of karstic formations
Permeation	• Introduction of a fluid grout to fill all existing voids in the ground, thereby forming a solidified mass of low permeability	• Reduction in settlement • Increase in bearing capacity and stiffness
Jet	• Radial jetting of a cementitious grout slurry which mixes with the in situ coarse material to form a large diameter grout column	• Formation of cut-off walls for ground water control • Grout columns for structure support • Underpinning • Decrease in permeability of the ground

2.2.2.5 Inclusions

Soil reinforcement through inclusions dates back to ancient times where roots of trees have always acted as a means of erosion control along slopes. However, it is not until 1966 when Henry Vidal, a French architect and engineer, developed the modern form of soil reinforcement, whereby the reinforcing material were normally long and thin metallic strips of high strengths and low extensible properties (Vidal, 1966).

Today, the market has expanded to produce a broad spectrum of reinforcing materials, among which geosynthetics (polymeric synthetic materials) have reported very high usage and efficiency. Their design extends beyond soil reinforcement to accommodate for soil erosion control, separation, drainage as well as filtration; a probable reason for their popularity. Resistant to biological and chemical degradation, these materials are many including (refer to examples in Figure 2.5) geotextile, geomembranes, geocells, geocontainers and geogrids, whereby each product is best suited for a specific function. They have the added advantages of being easy to install, having light weight and thinness properties which thus requires less working space, and possessing published standards and design methods. When used as a soil reinforcement material, they are known to increase the bearing capacity while concurrently decreasing the settlement.

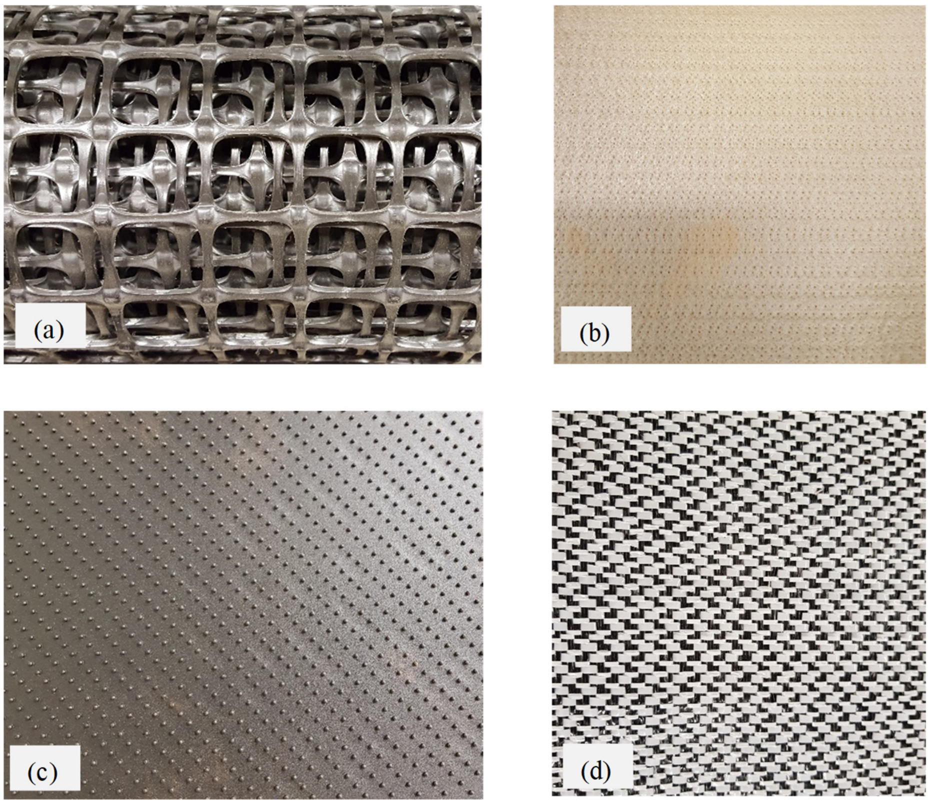


Figure 2.5: (a) Geogrids, (b) Non-woven geotextile, (c) Geomembrane and (d) Woven geotextile

Beside geosynthetics, other materials continue to be of interest for reinforcing soils possessing weak mechanical properties. Rigorous investigations have been underway over the past decade to verify the possibility of using scrap tyres, plastic waste, palm fibres, carpet waste and bamboo, amongst many other materials, for this purpose (Benson & Khire, 1993; Zornberg et al., 2004; Marandi et al., 2008; Miraftab & Lickfold, 2008 and Mustapha, 2008). Under specific conditions, these materials have positively responded to reinforcing sands, silty sands or clayey soils, triggering an improvement in shear strength, unconfined compressive strength, California Bearing Ratio and load bearing capacity.

The above-mentioned inclusions can most evidently improve the mechanical properties of a soil (especially sands or silty sands) although the degree of improvement is dependent on the type of material used to reinforce the soil. Geosynthetics, as opposed to the other cited materials, are more likely responsive to higher stresses. But, they remain less environmentally friendly due to the energy consumption in the manufacturing process as opposed to using existing natural products, or waste materials which are destined to landfills. Furthermore, the use of waste material for soil reinforcement is an inexpensive option, compared to geosynthetics, where lower bearing stresses are desired since they are cheaply and readily available.

2.2.2.6 Dynamic compaction

The principle of repeatedly dropping a heavy weight, through a specific height, at regular spaced intervals is referred to as dynamic compaction. The technique is applicable to all soil types, with the exception of soft silts, peat and clays, and it is often employed when a denser ground (lower air voids content) is required. The treated ground is resistant to high bearing capacities and undergoes negligible settlement, if any. Vibration, being the principal mode of compaction, depends on characteristics such as soil layer thickness, dropping weight, falling height and the type of soil to produce the required degree of compaction. When the vibratory waves propel through the ground, the particles rearrange themselves such that a denser soil layer is formed. Figure 2.6 shows the dynamic compaction process.

Dynamic compaction is widely used since it is fast and economical for sites of areas between 5000 to 10 000 m^2 (Bowles, 1997). It also avoids excavation and replacement with a fill material when deep treatment becomes mandatory. Adding to its advantages, the method further allows for presence of rubble, boulders and rocks. Nonetheless, the high energy impact through the falling weights can cause possible damage to surrounding structures, besides the level of noise pollution and the flying of debris during pounding.

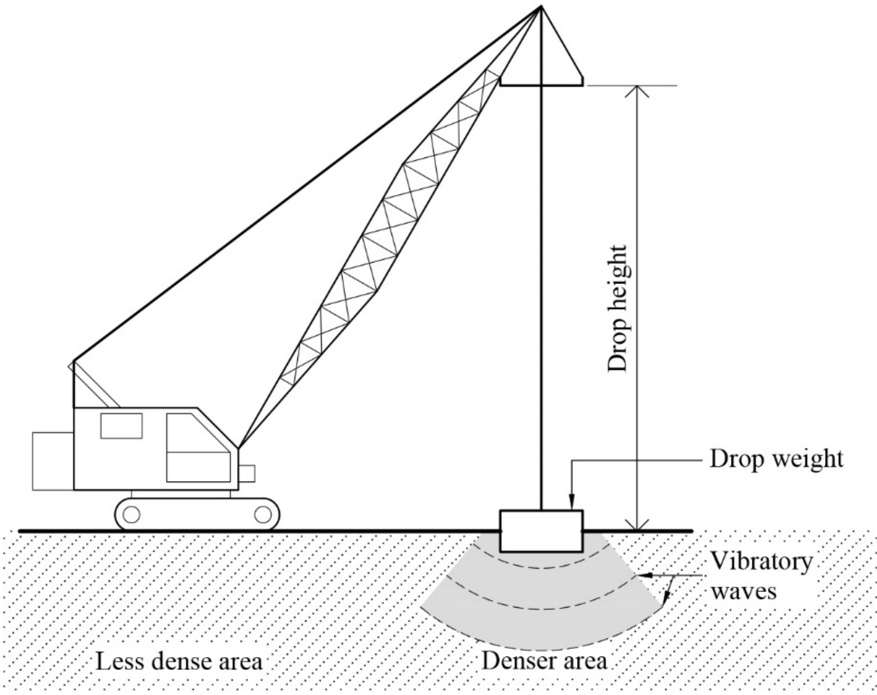


Figure 2.6: Dynamic compaction through the dropping of a weight

2.2.2.7 Granular columns

Granular columns are vertical columns, formed from a compacted coarse material (stone or sand), which are usually installed in very fine sands, silts or clays. Their main purposes are to increase bearing capacity, reduce settlement, mitigate liquefaction and improve drainage. In comparison with piles which are rigid columns, these ones are classified as flexible whereby the column material does not form a single unit and hence they have lower loading capacities and stiffness. Compared to piles which develop their strength through friction at either the toe or on the side of the column, granular columns inhibit their strength characteristics through the lateral stresses exerted onto them from the surrounding weak soil.

Granular columns are generally installed by a process which includes vibration or ramming (Som & Das, 2006). With regards to the vibration method, two approaches exist namely the displacement and the replacement methods. Generally, a vibratory probe penetrates the ground to be treated, while compacting the soil in its immediate surrounding. Thereafter, the opening, which is created during retraction of the probe is backfilled with a new material. However, installation of vibro replacement columns involve air or water jetting to create the opening in the ground. The column material is then fed to the equipment, through the top or the bottom. Figure 2.7 demonstrates the installation of vibro displacement columns.

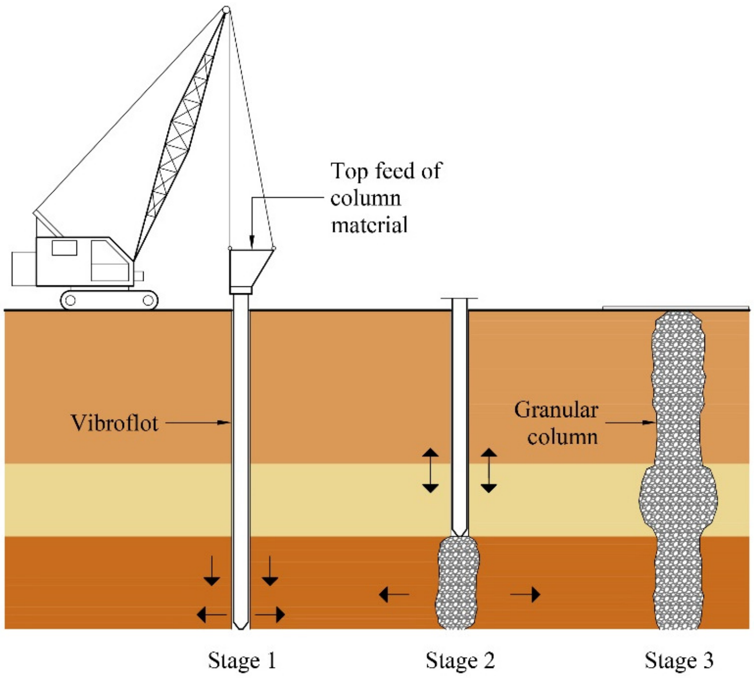


Figure 2.7: An example of a vibro installation of granular columns known as vibro displacement

Contrastively, in the ramming technique, a pre-bored hole is made which is supported by the inclusion of a hollow metal pipe. The pipe is then filled in layers, each compacted through a dropping weight. Before compaction, the pipe is retracted by a predetermined height and the stages are repeated until ground level is reached. Figure 2.8 illustrate a typical installation procedure of granular columns, by the ramming technique.

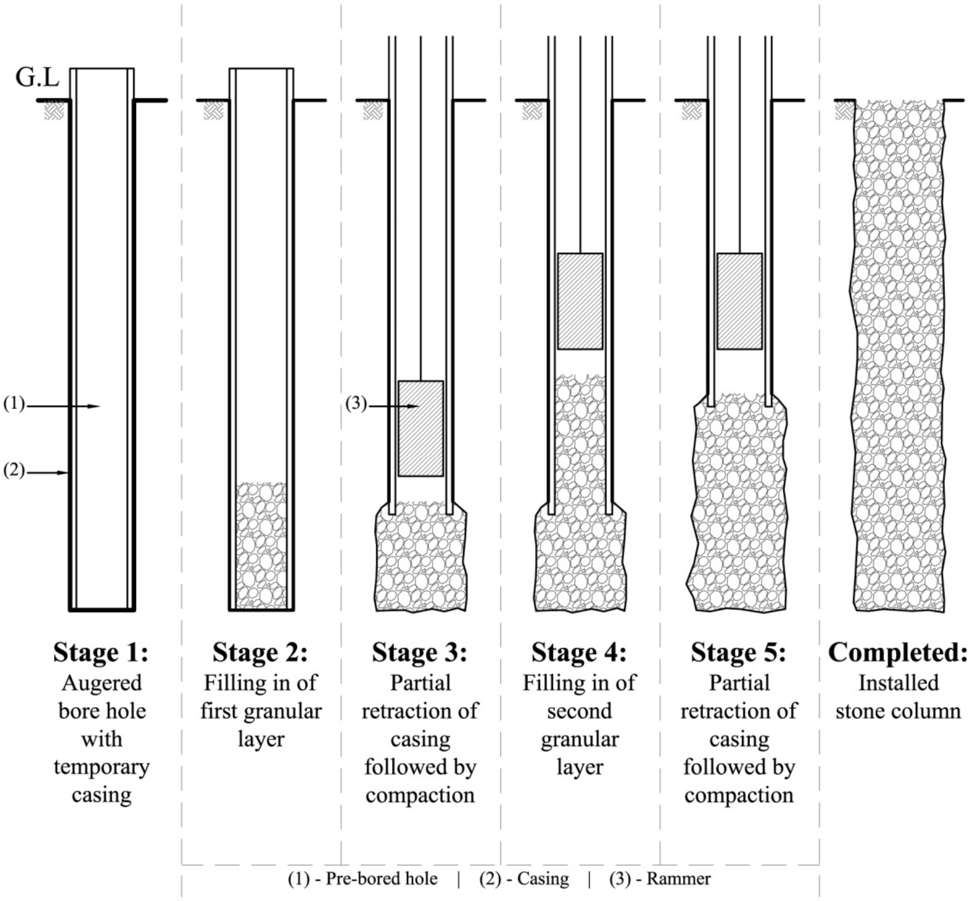


Figure 2.8: Installation of granular columns through the ramming method (Sobhee-Beetul, 2012)

Granular columns have often demonstrated some additional benefits as opposed to the other ground improvement technologies which can be applied under similar problematic soil conditions. For instance, ramming, compared to vibration, is a rapid method which is cheap and requires no special skills. They also generally serve a few functions simultaneously. Besides their engineering capabilities, they generate the most natural and ecologically neutral foundation system since they make use of natural materials which are widely available. Nevertheless, the drawback of the technique is its reserved application in low capacity structures, such as oil tanks, embankments and low-rise buildings; in addition to the high amount of dumping material produced with vibro-replacement methods (Shivashankar et al., 2011). Also, the process of installing the columns through vibration, can be relatively costly.

2.2.3 Summary of properties of the different techniques

This section summarises the information relating to the necessity (Figure 2.9) and benefits (Table 2.2) of some of the most popular ground improvement techniques which have been discussed in section 2.2. Figure 2.9 indicates the approach to be adopted in cases when construction needs to occur on soils with poor engineering properties.

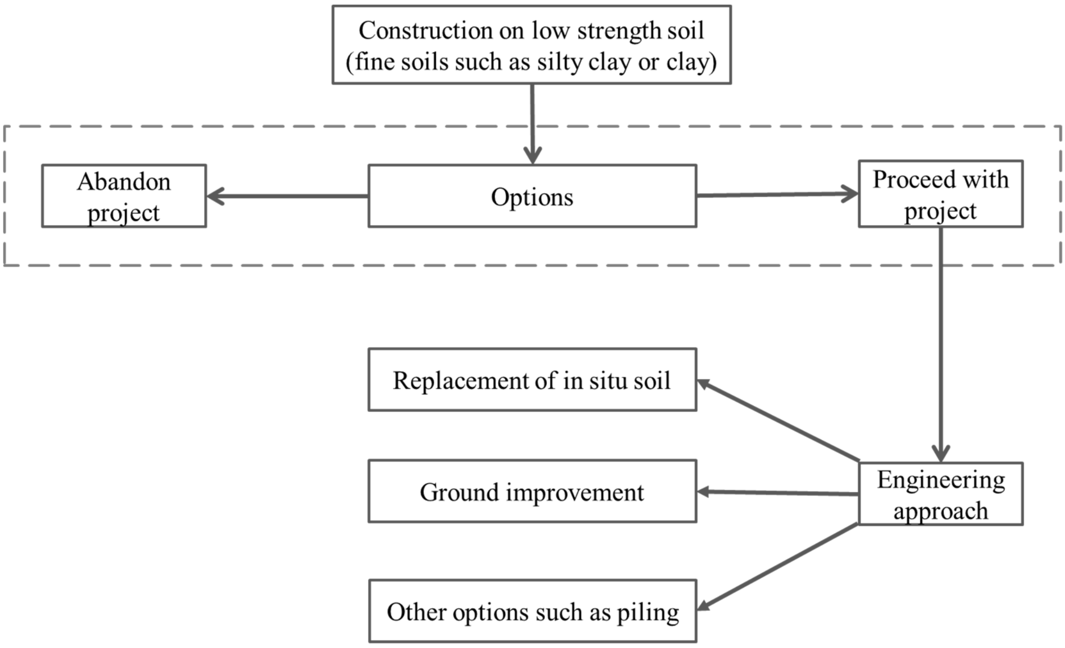


Figure 2.9: Approach to constructing on a soil with poor engineering properties

In section 2.2, a few of the ground enhancement methods were identified and described. This identification was based on the similarity in advantages which each technique provides. Table 2.2 summarises these benefits, where, a shaded cell represents the occurrence of the given engineering aim to be attained through the relevant technology. Based on the current existence of problematic soils locally, as well as the need for further development, especially in the context of providing low cost housing to the underprivileged, it is necessary that engineering approaches are adopted to bring lands, which were previously considered unfeasible, to use. To achieve this, it is necessary to satisfy the associated engineering constraints while incorporating environmental and economic aspects within the applied technologies. From Table 2.2, granular columns are evidently the most beneficial option (with the highest score of 7) provided that their aim is to improve weak grounds for supporting the foundation of low capacity structures such as low-rise buildings, storage tanks, and embankments.

Table 2.2: Summary of the benefits of the different ground improvement techniques

Replacement of in situ soil	Most soils									
Ground improvement	Most soils									
↓	Soil suitability	Densification	Drainage	Liquefaction	Settlement control	Environment	Economical	Fast process	Reduced permeability	Total score
Dynamic compaction	Most soils except soft silts, peat and clays	1		1	1		1	1		5
Granular column (Vibro)	Fine sands, silts or clays	1	1	1	1			1		5
Granular column (Rammed)	Fine sands, silts or clays	1	1	1	1	1	1	1		7
Preloading	Silts, clays and organic deposits	1	1		1		1			4
Vertical drains	Clays		1		1	1				3
Indirect ground freezing	Mostly silts and clays				1		1		1	3
Direct ground freezing	Mostly silts and clays				1			1	1	3
Grouting	Most soils and rocks	1		1	1				1	4
Geosynthetics	Sands, silty sands or clayey soils		1		1		1	1	1	5

2.3 A review on the theory and current knowledge on granular columns

2.3.1 Applications

This sub-section elaborates on the types of soil which can benefit from the granular column technology. The different types of granular columns, and the materials used to form them, are discussed. A brief description of the loading scenarios which improved grounds (by granular columns) may experience is subsequently given, in addition to certain typical applications of the technique. Information related to the cost effectiveness and the environmental friendliness of the method, is lastly presented in this subsection.

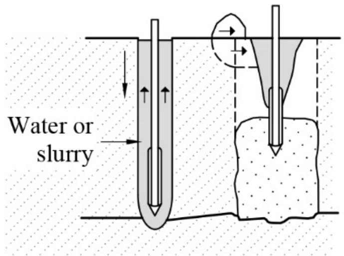
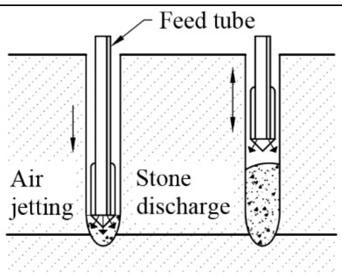
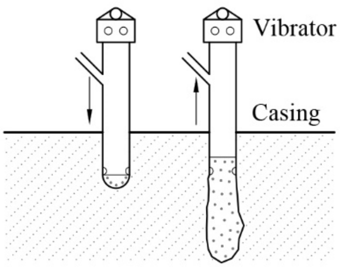
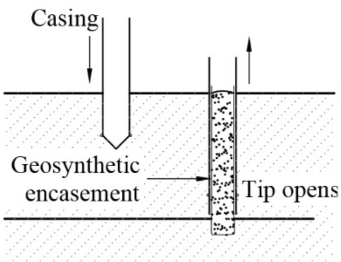
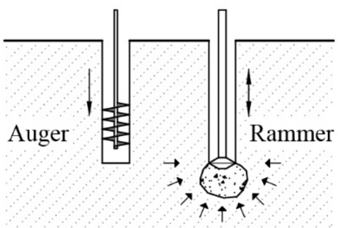
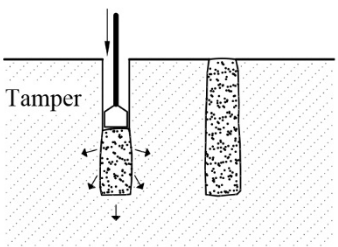
2.3.1.1 Soil suitability for improvement by this technology

The use of granular columns is typically common in the ground improvement stage of weak strata comprising of soils such as soft clays, peat, cohesive deposits and silts (Hughes & Withers, 1974; Barksdale & Bachus, 1983; Goughnour, 1983; Van Impe, 1983; Priebe, 1995; Murugesan & Rajagopal, 2008). Although granular columns are used for several purposes, they are often selected for two main functions, namely: reinforcement and drainage, in such problematic soils (Shivashankar et al., 2011). The aim is to improve the load bearing strength while reducing any volume change effect in the weak soil such that excessive deformations and stability problems may be avoided. In most engineering soils, particle arrangement within the soil structure is often the determining factor with regards to strength and compressibility. Hence, a densification approach is normally useful. For granular soils such as fine sands and silts, vibration methods, for example vibro-compaction and vibro-stone columns, are usually preferred to densify the ground since their particles can be easily packed into closer configurations. Contrastively, the density of cohesive soils (clays) is improved through consolidation. Clays usually have low void ratios since their particles are smaller than 0.002 mm. Additionally, the capillary attraction forces between the particles reduce drainage capability by retaining water in the clay for longer periods of time, when compared to granular and coarser materials. Under the application of a direct load on the weak ground, excess pore pressures are dissipated, and a consolidated state is attained. Through this process, the maximum volume change occurs thereby reducing the possibilities of settlement related complications. Nevertheless, differential settlement and cracking remain potential challenges, especially when treating expansive clays. Despite the effectiveness of the loading approach for consolidation, it is time consuming and therefore deep treatments (which also provide better engineering solutions), such as granular columns or piling, are normally preferred. The following section provides detail of the different types of granular columns used for such treatments.

2.3.1.2 Types of granular columns and popular materials used for them

Granular column is the general term used for describing a compacted column, made from coarse materials, which is installed in weak grounds to improve their physical engineering properties. As mentioned in Chapter 1, this technique is relatively old although continuous research and advances are being made in that area of ground improvement. Consequently, several installation techniques have been proposed over the years. The development of each is based on the economical, engineering or time constraints of the project. In practice, each approach is mostly suited for specific conditions. Han (2015) provides a clear description of the different types of granular columns existing on the market, and their suitability for different types of soils. Table 2.3, which summarises the relevant information, highlights sands and stones (or aggregates) as the principal materials which are used to form granular columns. Their use is normally dependent on the surrounding soil conditions. For instance, stones are generally not preferred for very soft clays since the lateral confinement provided by the weak ground is significantly low. As a result, bulging of the columns are high and stones tend to migrate within the fine soil; hence reducing the shear strength of the granular column. From Table 2.3, it is also observed that sand compaction column is the only method which treats foundations to larger depths; 70 m according to Han (2015). In comparison, the column lengths, installed through the other technologies, are typically about 10 to 15 m.

Table 2.3: Types of granular columns (Pictures sourced from Han 2015)

Technology	Schematic illustration	Column material	Undrained shear strength (USS)
Vibro-replacement		– Granular fill such as sands and stones – Depth: 10 to 15 m	– Cohesive soils, fine sands, silt, clays – USS: > 15 kPa
Vibro-displacement		– Stones or aggregates – Depth: 10 to 15 m	– Granular soils, fine sands, silts, insensitive cohesive soils – USS: between 15 to 60 kPa
Sand compaction column		– Granular fill; mostly sands but sometimes aggregate – Depth: up to 70 m	– Cohesive soils – Granular soils such as fine silts – USS: unknown
Encased granular column		– Granular fill such as stones and sands – Depth: 5 to 10 m	– Very soft soils and organic soils – USS: between 5 and 15 kPa
Rammed aggregate column		– Granular fill such as stones or sands – Depth: within 10 m	– Soft to stiff clays, loose silt and sand to dense sand, uncontrolled fill – USS > 15 kPa
Dynamic replacement		– Granular fill – Depth: up to 8 m	– Saturated cohesive soils and soft organic soils – USS = unknown

2.3.1.3 Loading scenarios with granular columns

When treating a ground with granular columns, several loading conditions may be exerted onto the improved soil. According to Hughes & Withers (1974), there are 3 distinct types of such loadings namely: point loads, strip footings and widespread loads. Small pad footing is an example of point loading - a direct load is applied onto the column which experiences equal and all around restraint laterally. In contrast, since strip footings are supported by a line of columns, horizontal movement is restricted in the direction of the alignment of the columns. Usually, the effective improvement area provided by each reinforcing member overlaps with that of the adjacent ones on each side; hence the restriction in the path of motion. With regards to widespread loading, lateral restraint is similar to that in point loads. However, under large loads, the host soil settles which cause an increase in its lateral resistance. Figure 2.10 illustrates the 3 different types of loading conditions discussed in this section.

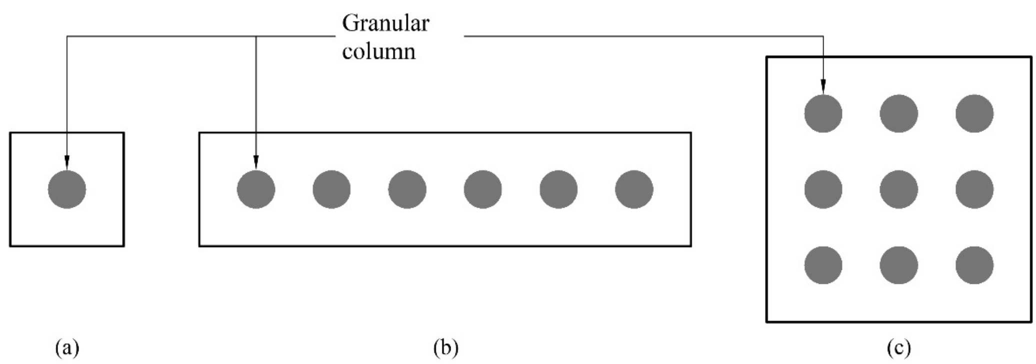


Figure 2.10: Three common types of loading conditions on ground improved by granular columns (a) point load, (b) strip footing and (c) widespread loads

2.3.1.4 Typical applications

Granular columns are popular for providing an adequate foundation for low capacity structures such as oil tanks, embankments and low-rise buildings (Shivashankar et al., 2011). Figure 2.11 shows a few structures which can be supported on ground improved by these columns.

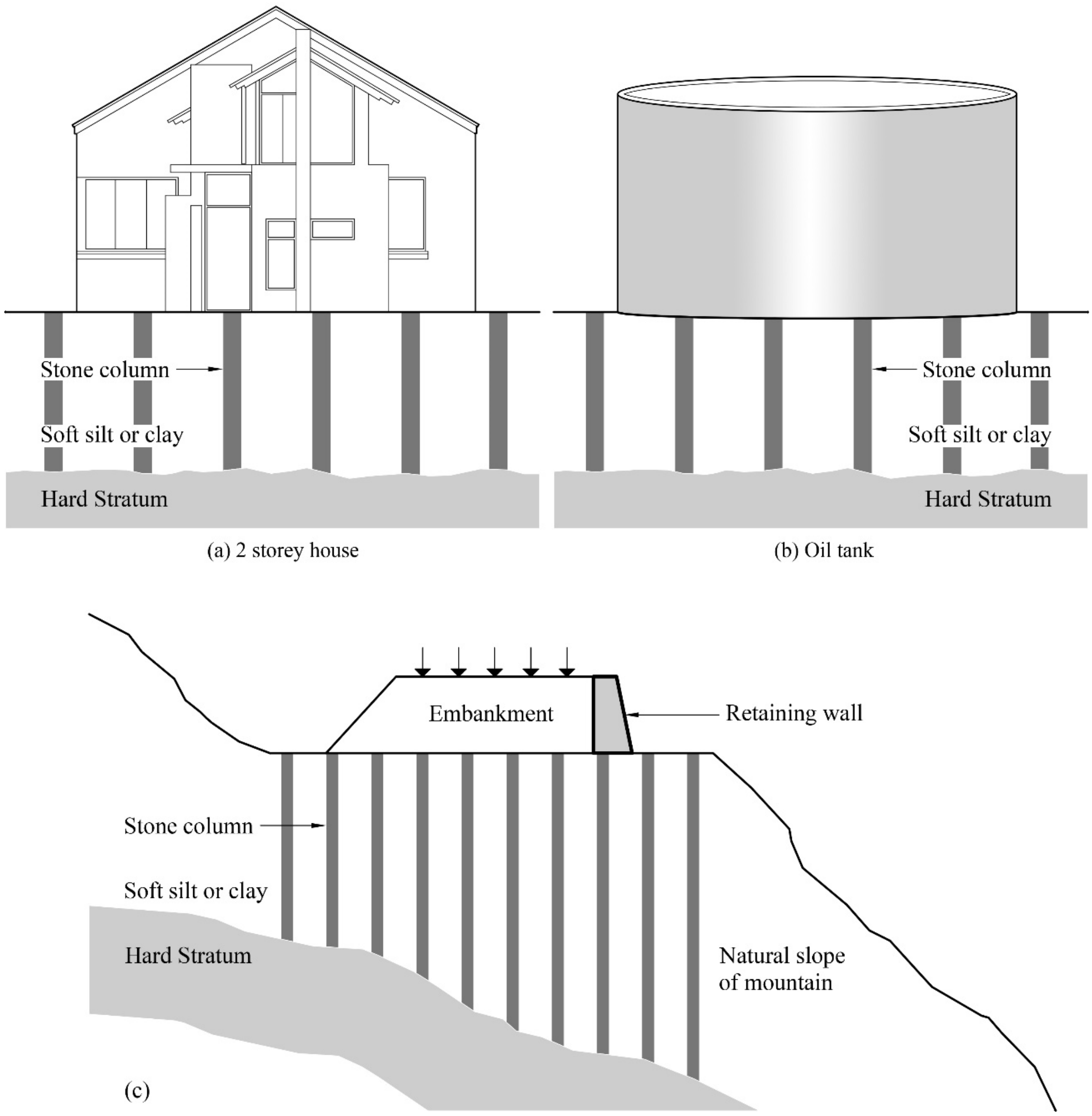


Figure 2.11: Typical examples of low capacity structures supported on granular columns (Adapted from Sobhee-Beetul, 2012)

In a report written by Barksdale & Bachus (1983), the popularity of the technique in the U.S, Canada and Europe were specifically pointed out, where the latter was the predominant user at that time. According to them, 21 granular column (referred to as stone columns) projects were successfully executed in the US before 1982. Some of the applications include:

- Embankment support for highways, interchanges and bridges
- Structures: buildings of up to seven-storeys, sewage treatment plants, warehouses, ship building facilities and parking garages

- Tanks: 5-million-gallon water tank
- Railroad and wharf structure

Datye & Madhav (1988) also stated that granular columns may be used for supporting grounds in similar projects as mentioned by Barksdale & Bachus (1983). They further add that this technique is also applicable in projects where soft grounds are required to support large water pipelines or box abutments of major bridges.

Today, granular columns are far more popular and their widespread application across the globe is particularly observed. A very common application is in the construction and support of embankments in road projects. Saroglou, Antoniou & Pateras (2008) presented a case study whereby a 600 m wide and 3 m high embankment, as part of a road construction in Greece, was built on a ground improved by granular columns. The in-situ soil comprised of soft clays of low plasticity. Silty to clayey sands of medium density with gravel was also observed within the soil strata. Beside the soft nature of the soil, the alignment of the highway road was close to the sea (less than 400 m). Also, the area was situated in a low land environment prior to this construction. The main concern in this project was the anticipated degree of settlement due to the soft soil, as well as drainage issues. Several methods were considered to improve the ground. However, calculations relating to conventional method showed high settlements, which would thus prolong the time of construction. As a result of the analysis, stone columns were chosen as the most appropriate technique to address the issues of concern. It was proposed that these must be columns installed within depths of 14 m. Parametric analyses were performed, and it was revealed that the settlement was reduced from 150 to 70 mm, for different replacement ratios. Additionally, the time of construction had reduced from 16 to 4 months.

Though it has proven its effectiveness in some projects, the use of granular columns is minimal in South Africa. One such example is in the improvement of poor ground conditions for the construction of the Midfield Terminal at the O.R Tambo International Airport in Johannesburg. From the geotechnical investigation conducted, it was revealed that clayey gravel, soft organic clays and a seepage area constituted the main challenges on this site. Granular columns, in the form of stone columns, were therefore proposed to reduce the high settlements by encouraging a faster drainage rate. Besides the engineering success of the technique, time constraint was concurrently addressed.

Granular columns are also vastly beneficial in mitigating liquefaction in highly susceptible soils. The Adapazari city in Turkey is located in a region with deep alluvial deposit and high seismic activity. According to Arman et al. (2009), the area has undergone severe earthquakes in the past which has resulted in thousands of deaths and injuries, as well as the destruction and heavy structural damage of buildings. Post the harsh earth movements, built structures are found to be severely settled or tilted. Hence, the authors suggested stone columns as a mean of strengthening the ground conditions such that future damages resulting from liquefaction, may be avoided. In the study, the investigators proposed that a modified dry bottom feed method be used. A structurally damaged five-storey reinforced concrete building was selected to investigate the effectiveness of this ground improvement technique, under the given conditions. Stone columns of length 7 m and mean diameter 47 cm were installed. Numerical modelling and analysis showed an average improvement of 35 % in terms of the horizontal and vertical displacement, when using the modified stone columns. Another example is that provided by Mahoney & Kupec (2014) who also recommended the use of stone columns to improve grounds prone to seismic activity in Wainoni, New Zealand. These columns were intended to mitigate the risk of liquefaction. A new screw displacement stone column was developed for this project to reduce any impact on residents around the site while keeping vibration and noise levels low. Also, the supermarket had to be operational during the construction activities. To verify the suitability of the new technique, trials were done before any construction. Through the results obtained, it was confirmed that the method was effective in improving the silty sandy subsoil conditions. Consequently, 600 mm diameter granular columns were installed in triangular arrangements of 1.85 m spacing and replacement ratio of 10 %.

2.3.1.5 Cost effectiveness of granular columns

For any project, cost management is usually one of the most significant factors that is of direct interest to the developers. Therefore, the concerned construction professionals are obliged to adopt solutions which are typically beneficial to the client's budget. From the architectural to the engineering discipline, each aspect of design is influenced by the allocated funds, which urge these consultants to produce cost-cutting implementations using the most profitable products in their respective domain. For construction engineers, the granular column technique is one such product on the market which may be applicable in ameliorating certain ground conditions, whereby the soil is fine or soft and cohesive. Previous researchers, engineers and geotechnical firms have claimed the efficiency of this ground improvement method as part of value engineering measures (Balfour Beatty Ground Engineering, 2018; Barksdale & Bachus, 1983; Franki, 2018; Pulko & Majes, 2006; Shivashankar et al., 2011).

While the popularity of granular columns is gradually increasing in present times, with respect to the associated relatively low cost, it must be noted that Barksdale & Bachus (1983) had long claimed the competency of the technique in reducing construction fees. Some of the applications which they utilised for describing the proclaimed benefit included the installation of granular columns to support embankments, abutments and bridge foundations located on marginal soils. Several decades after, Balfour Beatty Ground Engineering, one of the leading specialist geotechnical contractors in the UK, still maintain that vibro column is generally the most cost-effective solution when treating poor grounds (Balfour Beatty Ground Engineering, 2018). In contrast to piling, the company highlights a distinct pricing difference - vibro columns may possibly be cheaper by 50 to 70 %. Furthermore, they confirm that the construction of foundations and floor slabs are faster and cheaper on grounds improved by vibro columns as opposed to those treated by piling. Franki, a Keller company in South Africa, also points out the economical aspect of stone columns, more specifically the vibro replacement ones (Franki, 2018). It is therefore understood that, even today, granular column remains a low cost ground improvement method, depending on the soil conditions.

2.3.1.6 Environmental benefits of granular columns

Above and beyond the engineering and cost benefits associated with granular columns, they are also popular as an environmentally friendly ground improvement method (McKelvey et al., 2004; Etezad, Hanna & Ayadat, 2015 and Madun et al., 2018). In fact, Sobhee-Beetul (2012) and Zukri & Nazir (2018) stated that granular column improved ground is possibly the most natural and ecologically neutral foundation system since the column material typically exists in nature, or a relatively low amount of energy is used to obtain the material in the required state. The comparison was made with other ground improvement techniques or even structural solutions such as bored cast-in-situ piling under certain circumstances (Chawla, Raju & Krishna, 2010).

Balfour Beatty Ground Engineering proposes several ground improvement solutions which are environmentally sustainable. According to them, most of their techniques contain no cement, concrete or steel, thus reducing the carbon footprint drastically. For example, this company confirmed that their vibro systems generate only 5 to 10 % of carbon dioxide in comparison to piling systems. Besides, no spoil is produced and the material to be used for the column may be sourced locally, thereby reducing carbon emissions associated with transportation. Recycled aggregates can also be used in vibro stone columns, which enhance the environmental characteristics associated with the technology. The use of recycled aggregate reduces the carbon dioxide emission by 30 % compared to newly quarried stones.

Chawla et al. (2010) described the environmental benefits associated with dry vibro stone columns in contrast with bored cast-in-situ piles, for the foundation of a power plant in New Delhi (India), and concluded that the granular column technique utilised less fuel. Apart from the minimal production of waste soil during installation, the site may also be easily reused for future purposes due to the presence of natural materials. By making a few assumptions, they further performed individual calculations on the embodied carbon dioxide based on the different materials and processes employed for installing both bored cast-in-situ piles and dry vibro stone columns. The outcome indicated that vibro stone columns produced significantly lower greenhouse gas emissions. While the mass of embodied carbon dioxide for the piles was 1250 tons, and that for the stone column was only 161 tons. Therefore, this confirmed that the stone columns produced 87 % less carbon dioxide emissions than the piles for this project.

For the many reasons provided in this section, granular columns have certainly gained popularity in terms of environmental sustainability. In the current age, eco-friendly approaches are encouraged in all possible ways. Since the construction industry largely contribute to the carbon dioxide emissions, it is essential to explore and apply 'Greener' technologies.

2.3.2 Theory of granular columns

2.3.2.1 Theoretical analysis based on the unit cell concept

The Unit Cell Concept

When analysing granular columns, the idealized unit cell concept is normally adopted. In principle, this concept determines the extent to which improvement from one column is achieved. Figure 2.12 describes a typical unit cell for a single granular column.

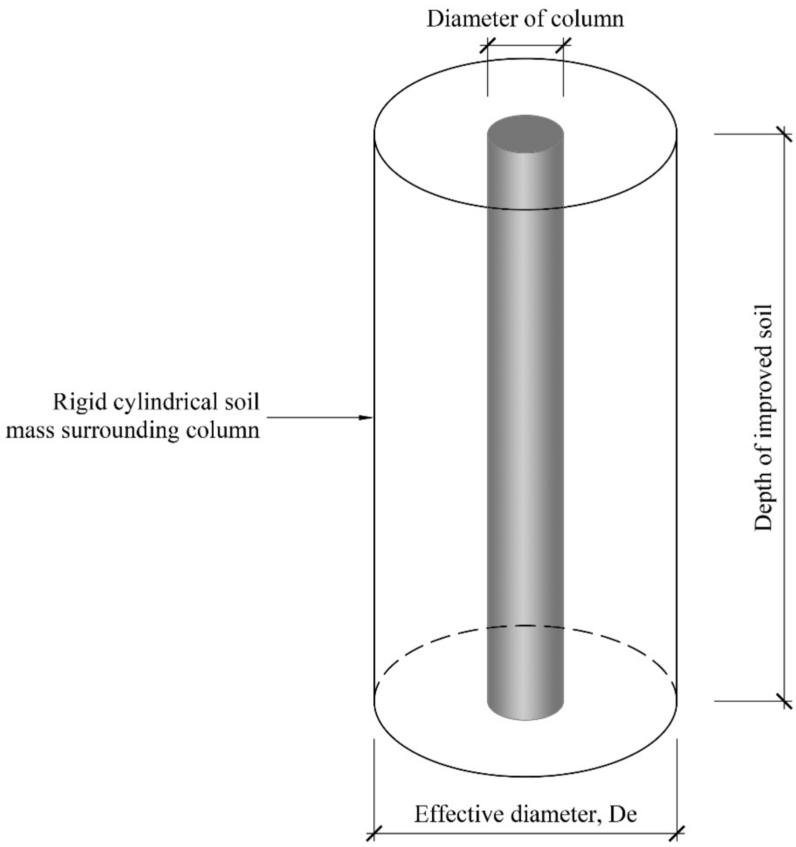


Figure 2.12: Unit cell showing the effective diameter, D_e (Adapted from Goughnour, 1983)

Han (2015) analyses the unit cell concept with an equal strain and equal stress approach. He distinguished between these two conditions by highlighting the importance of the type of loading, that is, the presence of equal strain under rigid loading and equal stress under flexible loading. When a soft foundation is reinforced with a granular column, the latter carries a higher stress than the soil under an equal strain. This is explained in terms of the difference in modulus between the column and the surrounding soil. Han (2015), as well as several other researchers have defined the ratio of the stress on the column ($\Delta\sigma_c$) to that on the soil ($\Delta\sigma_s$) as the stress concentration ratio (n). According to Han (2015), many researchers have assumed a one-dimensional (1D) unit cell, which therefore eliminate any lateral deformation of the column. However, Castro & Sagaseta (2011) and Jiang et al. (2013) revealed that a unit cell undergoing lateral deformation will affect the degree of settlement in conjunction with the rate of consolidation of the composite foundation. They provided the following relationship for a 1D unit cell under an equal vertical strain condition:

$$\varepsilon_z = \frac{\Delta\sigma_c}{D_c} = \frac{\Delta\sigma_s}{D_s} \qquad \text{(Equation 2.1)}$$

where ε_z = vertical strain at a depth of z

$\Delta\sigma_c$ = vertical stress on the column

$\Delta\sigma_s$ = vertical stress on the soil

D_c = constrained modulus of the column

D_s = constrained modulus of the soil

Nevertheless, if a column is allowed to deform laterally (3D deformation) under equal vertical strain condition, then the vertical strain at a depth of z will be given by the following equation:

$$\varepsilon_z = \frac{\Delta\sigma_{cz} - v(\Delta\sigma_{cx} + \Delta\sigma_{cy})}{E_c} = \frac{\Delta\sigma_{sz} - v(\Delta\sigma_{sx} + \Delta\sigma_{sy})}{E_s} \qquad \text{(Equation 2.2)}$$

$$n_{3D} \neq \frac{E_c}{E_s}$$

where: ε_z = vertical strain at a depth z

$\Delta\sigma_{cx}, \Delta\sigma_{cy}, \Delta\sigma_{cy}$ = stresses on the column in the x, y, z directions, respectively

$\Delta\sigma_{sx}, \Delta\sigma_{sy}, \Delta\sigma_{sz}$ = stresses on the soil in the x, y, z directions respectively

E_c = elastic modulus of the column

E_s = elastic modulus of the soil

n_{3D} = stress concentration ratio considering column lateral deformation

Basic granular column theory

The theoretical behaviour and design procedures of granular columns have been explained by many researches with various approaches and assumptions (e.g. Hughes & Withers, 1974; Barksdale & Bachus, 1983; Priebe, 1995). In 1970, Greenwood explained that the degree of improvement achieved, when reinforcing a soft soil with granular columns, is largely dependent on the passive pressure exerted by the surrounding soil onto the column, the column diameter and the degree of compaction within the column. During loading, granular columns carry most part of the load, thereby bulging into the base clay. The outward lateral stress exerted by this bulging behaviour are resisted by the passive pressure of the clay. Thus, a similar situation as that in a triaxial chamber is generated. Hughes & Withers (1974) shared a similar opinion and thus by applying equations from Gibson & Anderson (1961) and records from a few field tests by Wroth & Hughes (1973), they proposed the following formula to calculate the maximum vertical stress that the column can carry during lateral bulging:

$$\sigma'_{vc} = \frac{1+sin\varphi'_c}{1-sin\varphi'_c}\,(\sigma_{ro} + 4c_u - u) \qquad \text{(Equation 2.3)}$$

where: σ'_{vc} = ultimate vertical effective stress,

φ'_c = angle of internal friction of column material,

σ_{ro}= total in situ lateral stress,

c_u = undrained cohesion, and

u = pore pressure

Kruger *et al* (1980) applied the same equation, coupled with an assumption about the bulging behaviour to be similar to a pressure meter test, to check the column stability by applying limit equilibrium of the column as follows:

$$\sigma'_{vc} = \frac{1+\sin\varphi'_c}{1-\sin\varphi'_c}\,(\sigma_l - \sigma_o) \qquad \text{(Equation 2.4)}$$

where: σ_o = horizontal earth pressure at rest in total rest, and

σ_l = reduced limit pressure

2.3.2.2 Principal engineering design calculations

Existing design methods

Najjar (2013) presented a State-of-the-Art review on granular column reinforced clay systems. This review provides detail of the development of work based on this technique, over the past 40 years. It is evident that intensive amount of work has been conducted in this field of research. Besides the practical difficulties faced by engineers and contractors when installing these columns, it also appears that several gaps exist in the available design procedures. Table 2.4 is a compilation of the different design approaches which exist. The table contains information only for methods which calculate bearing capacity and settlement since these are the primary aims of this study.

Table 2.4: Summary of the existing design methods for granular columns

	Single column		Group of columns	
Design method	Bearing capacity / Vertical stress	Settlement	Bearing capacity/ Vertical stress	Settlement
Hughes & Withers (1974)	✓			
Bauman & Bauer (1974)		✓		
Brauns (1978)	✓			
Mitchell & Katti (1981)	✓			
Barksdale & Bachus (1983)	✓			
Priebe (1995)	✓	✓	Only for strip footings	Only for strip footings
Watts et al. (2000)	✓			
Ambily & Gandhi (2007)	✓		✓	

From table 2.4, it is apparent that the theory behind granular columns have been studied for several decades. Researchers have worked on both singular and group of columns. Equations provided from the early stages of theoretical development have been modified and used on several occasions; each time, respective justifications have been provided using different scientific understanding. Despite the depth of knowledge achieved with regards to the behaviour of these columns, it remains impossible to compute bearing capacity and settlement (for both single and group of columns), using exclusively any one approach.

While the previous section has given the basic equation proposed by Hughes & Withers (1974), which afterwards became the foundation of theoretical advancement of granular columns, this section presents the most impactful equations which were progressively derived to execute these engineering calculations.

Bearing capacity and stress concentration ratio

While granular columns usually serve several purposes in ground improvement, bearing capacity is often the primary reason for this application. In 1978, Brauns assumed an axisymmetric model and proposed a simplified method to determine the ultimate bearing capacity of a singular column that is installed in a saturated soft soil. In this method, undrained conditions were considered as well as a passive shear failure from the granular column to the surrounding soil. For this study, a smooth interface was assumed between the column and the soil. Any circumferential stress was assumed non-existent while the self-weight for the column and the soil was ignored. The following Figure illustrates the failure mode of an individual column.

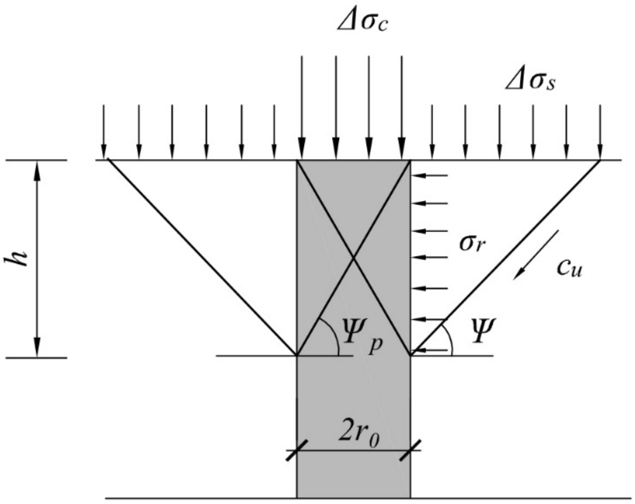


Figure 2.13: Bearing capacity failure of a singular column (Han, 2015)

When applying the equilibrium of forces, the following formula is obtained:

$$\sigma_r = \left(\Delta\sigma_s + \frac{2c_u}{sin2\Psi}\right)\left(1 + \frac{tan\Psi_p}{tan\Psi}\right) \qquad \text{(Equation 2.5)}$$

where: σ_r = lateral stress from the surrounding soil

$\Delta\sigma_s$ = vertical stress on the soil

c_u = undrained shear strength of the soil

Ψ_p = passive failure plane angle within the column ($\Psi_p = 45 + \frac{\phi_c}{2}$), where ϕ_c is the friction angle of the column

Ψ = failure plane angle in the soil

Han (2015), as well as Mitchell & Katti (1981), proposed that the ultimate bearing capacity (q_{ult}) of a single column can be approximated using the undrained shear strength of the surrounding soft cohesive soil.

$$q_{ult} = N_p c_u \qquad \text{(Equation 2.6)}$$

where: N_p = a bearing capacity factor which these authors recommended as 20 and 25 respectively.

In the proximity of these values, Barksdale & Bachus (1983) reported values between 18 and 22 for a similar equation, while Bergado, Alfaro & Chai (1991) proposed even lower values of 15 to 18. Han (2015) compiled a list of references whereby the N_p value recommended by each author was stated as follows: 25.2 by Hughes & Withers (1974), 15.8 to 18.8 by Mokashi, Paliwal & Bapaye (1976), 20.8 by Brauns (1978), 20 by Mori (1979), 25 by Broms (1979), 14 to 24 by Han (1992), and 12.2 to 15.2 by Guo & Qian (1990). From this list, it appears that the suggested numbers range from 12 to 25. Although Han (2015) recommended a value of 20 for stone columns, he explained that lower values are normally used for columns with a low friction angle (sand compaction columns), while higher values are generally considered for columns of high friction angle (rammed aggregate column). Datye (1982) was more specific about the aptness of the numbers with regards to the installation technique. He specified that a number between 25 to 30 was more appropriate for vibro replacement columns. For uncased rammed stone columns, he suggested a value of 40, while a range of 45 to 50 was considered suitable for cased columns.

Under loading conditions, Han (2015) explains that granular columns mobilize their strengths at a similar strain level as the surrounding soil. He thus proposed that the ultimate bearing capacity of these column reinforced soil be calculated using the following equation:

$$q_{ult} = q_{ult,c} a_s + q_{ult,s}(1 - a_s) \qquad \text{(Equation 2.7)}$$

where $q_{ult,s}$ = ultimate bearing capacity of surrounding soil which is estimated as $5c_u$ according to Barksdale (1987).

From the equations provided in this section, it is evident that the vertical stress on the soil affects the laterally generated stress within the surrounding soil. Since the lateral stress is the main generator of strength in granular columns, the stress in the column is simultaneously affected. The ratio between these two stresses is commonly referred to as the stress concentration ratio, n. This ratio has been used by several authors in the past, although the method of measuring the stresses required for the calculations may differ to some extent (Bachus, 1989; Priebe, 1995; Fattah, Shlash & Al-Waily, 2011; Sobhee-Beetul 2012). Additionally, the stress determination approach is often unclear. This fraction, which is subsequently given, basically represents the stress in the column to that in the surrounding base soil, for any given settlement.

$$n = \frac{\Delta\sigma_c}{\Delta\sigma_s} \qquad \text{(Equation 2.8)}$$

where: $\Delta\sigma_c$ = vertical stress on the column

$\Delta\sigma_s$ = vertical stress on the surrounding soil

In general, all design methods incorporate this ratio, in addition to the area replacement ratio and the stiffness of the subgrade soils (Griffith, 1991). Barksdale & Bachus (1983), Griffith (1991) and Bergado et al. (1996) explained the effect on stress distribution within a granular column reinforced ground, under the application of a heavy load. These authors claim that the higher stiffness of the column material, compared to that of the base soil, cause a redistribution of the applied stress when loaded, whereby a higher concentration of stress is experienced by the column. This behaviour is normally explained though the assumption of equal deflection. In other words, the strain in both materials are assumed to be approximately the same. As such, the laws of equilibrium indicate a much higher stress within the column. Hence, the concept of stress concentration ratio is introduced.

So far, most of the theories on composite foundations with granular columns, assumes rigid loading and equal strain conditions (Han, 2015). Han (2015) elaborates on the design calculations of such foundations, under rigid loading. Assuming a composite foundation as shown in Figure 2.14 with the given stress distribution, the following equation can be obtained when applying the equilibrium of forces.

$$\Delta\sigma_z A_e = \Delta\sigma_s (A_e - A_c) + \Delta\sigma_c A_c \qquad \text{(Equation 2.9)}$$

where: $\Delta\sigma_z$ = average vertical stress applied on the composite foundation

A_e = effective area of a single column

A_c = cross-sectional area of column

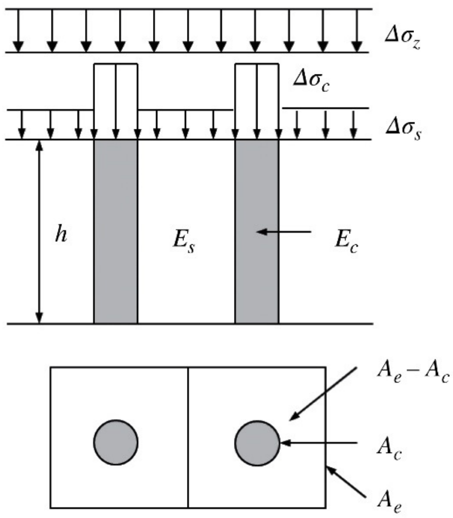


Figure 2.14: Stress distribution on a granular column improved foundation

If equation 2.9 is divided by A_e, the following is achieved:

$$\Delta\sigma_z = \Delta\sigma_s(1 - a_s) + \Delta\sigma_c a_s \qquad \text{(Equation 2.10)}$$

Based on the definition of the stress concentration ratio, this can be substituted in the equation to yield:

$$\Delta\sigma_z = [1 + (n - 1)a_s]\Delta\sigma_s \qquad \text{(Equation 2.11)}$$

Since the stress on the soil ($\Delta\sigma_s$) is equal to $\mu\Delta\sigma_z$, it can be substituted in equation 2.11 to obtain the stress reduction factor (μ) in terms of the stress concentration ratio (n) and the area replacement ratio (a_s).

$$\mu = \frac{1}{1+(n-1)a_s} \qquad \text{(Equation 2.12)}$$

In the literature, n has been found to vary mostly between 1 and 5 (Barksdale & Bachus, 1983; Vautrain, 1977; Goughnour, 1983; Fattah et al., 2011 and Sobhee-Beetul 2012). However, Aboshi et al. (1979) and Bergado, Huat & Kalvade (1987) recorded stress concentration ratios as high as 9.

Settlement and settlement reduction ratio

In 1974, Bauman and Bauer proposed a quantitative method of estimating both the immediate and the consolidation settlement of stone column reinforced cohesive soils. The authors adopted the unit cell concept when analysing the testing system which was loaded simultaneously on the soft clay and the column. In comparison to Zahmatkesh & Choobbasti (2010), they also used an effective diameter (D_e) as 1.13S and 1.05S for square and equilateral triangular installation patterns, respectively, while the unit cell area for the respective patterns were determined as S^2 and $0.866S^2$. In the calculation of the immediate settlement, it was assumed that the foundation pressure of the unit cell was shared by both the pressure in the unreinforced soil as well as that in the compacted column. The following equation was generated to support this assumption.

$$p_o A = p_c A_c + p_s A_s \qquad \text{(Equation 2.13)}$$

where: p_o = foundation pressure

A = area of foundation

p_c = column pressure

p_s = pressure in unreinforced soil

A_s = area of the unit cell

As the system was loaded, an increase in lateral pressure of the base soil and column was induced. Using the lateral earth pressure coefficient and the lateral coefficient of deformation of the column, the immediate settlement was thus proposed to be obtained through the subsequent equation.

$$S_c = \frac{2\Delta\sigma}{E_c} L_c \ln\frac{a}{r_o} \qquad \text{(Equation 2.14)}$$

$$S_s = \frac{L_c}{E_s} p_s \qquad \text{(Equation 2.15)}$$

where: S_c = immediate settlement of column

$\Delta\sigma = p_c K_c - p_s K_s$

K_c = lateral coefficient of deformation of column

K_s = coefficient of lateral earth pressure of soil

L_c = length of column

a = equivalent radius $D_e/2$ given by $\sqrt{A/\pi}$

r_o = radius of column

S_s = immediate settlement of soil

E_s = modulus of elasticity of soil

E_c = modulus of elasticity of column

From these assumptions, the stress concentration factor n, which is the ratio of the stress in the column to that in the surrounding clay, and the consolidation settlement were also calculated as follows.

$$n = \frac{p_c}{p_s} = \frac{1+2\frac{E_s}{E_c}K_s \ln\frac{a}{r_o}}{2\frac{E_s}{E_c}K_c \ln\frac{a}{r_o}} \qquad \text{(Equation 2.16)}$$

$$S_2 = \sum_0^H \frac{\Delta\sigma}{E}\Delta z \qquad \text{(Equation 2.17)}$$

where: σ = increase in vertical stress at any depth z below the footing

E = drained modulus of deformation

Generally, the immediate settlement is considered small for areas which are significantly large compared to the thickness of the compressible soil layer. Mitchell & Katti (1981), as mentioned by Som & Das (2006), proposed the following modified equation to determine the total settlement of a composite ground which has been treated by granular columns, as shown in Figure 2.15.

$$\delta = SRRs_{c1} + s_{c2} \qquad \text{(Equation 2.18)}$$

where: SRR = settlement reduction ratio for stone column treatment,

δ = settlement of foundation,

s_{c1} = settlement of untreated soil within depth of treatment (layer 1),

s_{c2} = settlement of untreated soil below stone columns (layer 2),

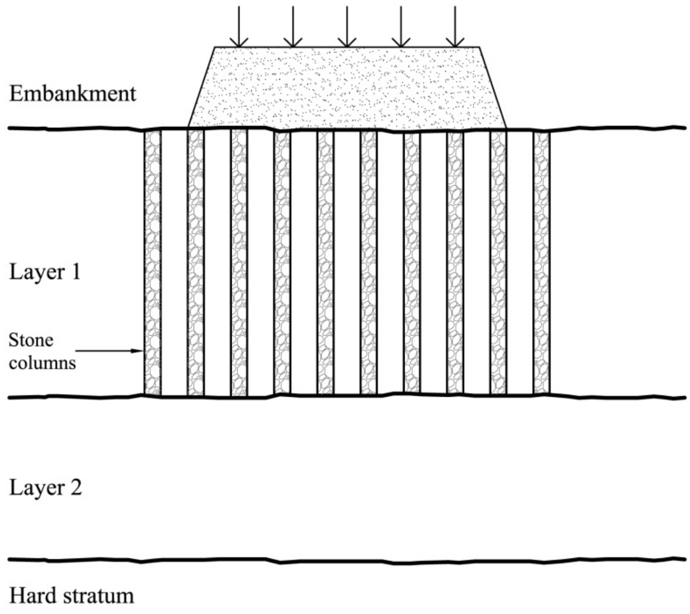


Figure 2.15: Settlement of embankment on a ground improved by granular columns (Sobhee-Beetul, 2012)

According to Barksdale & Bachus (1983), granular columns experience an increase in shear strength as a result of the overburden load. Simultaneously, there is a reduction in settlement in the base soil. Hence, granular columns are popular for settlement reduction. In literature, the computation of the settlement has often been associated with a ratio of settlements which is commonly referred to as the settlement reduction ratio (SRR) or settlement reduction factor. Bergado, Alfaro & Chai (1991) defined this ratio, β, using the following equation:

$$\beta = \frac{S_t}{S_o} \qquad \text{(Equation 2.19)}$$

where: S_t = settlement of the composite ground

S_o = settlement of the unimproved ground

Bergado, Alfaro & Chai (1991) compiled the SRR values from different authors, who individually used the finite element method, the granular wall method, the Priebe method or the equilibrium method. From their gathering, it is noted that the SRR ratio mostly varied between 0 and 0.8. Similar results have been obtained by Shivashankat et al. (2011) and Sobhee-Beetul (2012), although the latter's maximum SRR value attained was 0.65.

In 2010, Han summarised 3 main methods, proposed by different authors, for determining the settlement of foundations improved with granular columns. These are the stress reduction method by Aboshi et al. (1979), the improvement factor method by Priebe (1995) and the elastic-plastic method by both Pulko & Majes (2005) and Castro & Sagaseta (2009). Out of these 3 approaches, Aboshi et al. (1979) directly makes use of the settlement reduction ratio in a simple equation for calculating the settlement.

Aboshi et al. (1979) suggested the following equation to compute the settlement of a composite foundation, S', where, the soil compressibility has a high impact factor.

$$S' = m'_{v,s}\Delta\sigma_s h = m'_{v,s}\mu\Delta\sigma_z h \qquad \text{(Equation 2.20)}$$

where: $m'_{v,s}$ = coefficient of volume compressibility of the soil after installing the column

μ = stress reduction factor

$\Delta\sigma_z$ = average vertical stress applied on the composite foundation

$\Delta\sigma_s$ = vertical stress on the soil which is given by $\mu\Delta\sigma_z$

h = thickness of the layer of soil

The authors further stated that the settlement ratio of the composite foundation to the natural foundation may be defined by the given relationship:

$$\frac{S'}{S_n} = \frac{m'_{v,s}}{m_{v,s}}\mu \qquad \text{(Equation 2.21)}$$

where: S_n = settlement of a natural foundation

$m_{v,s}$ = coefficient of volume compressibility of the natural soil

Since the change in the coefficient of volume compressibility, both prior to and post column installation, is negligible for soft soils, $m_{v,s}$ is assumed to be the same as $m'_{v,s}$ and therefore simplifies the previous equation such that the settlement ratio, SRR, is equal to the stress reduction factor, μ:

$$SRR = \frac{S'}{S_n} = \mu = \frac{1}{1+(n-1)a_s} \qquad \text{(Equation 2.22)}$$

where: n = stress concentration ratio

a_s = area replacement ratio (ratio of the column cross-section area to the effective area)

From this section, it is evident that the approaches to calculating the bearing capacity and settlement of grounds improved by granular columns vary considerably. Nevertheless, the stress concentration ratio (n) and the settlement reduction ratio (SRR) remain of key importance in several design methods.

2.3.2.3 Factors affecting behaviour of granular columns

Arrangement and geometry

Granular columns are typically installed in a square, rectangular or triangular layout. Irrespective of the pattern of installation, the unit cell concept remains applicable in both situations. Han (2015) additionally adds that these columns may also be placed radially. He further elaborates that rectangular and triangular arrangements are normally used for most foundations, while the radial one targets circular foundations such as that required for supporting tanks. Greenwood (1970) claimed that column spacing affects the settlement improvement ratio. According to him, most practical problems for triangular grids can use a spacing to diameter ratio of 2.5 to 4. Nevertheless, the ratio must not be as minimal as 2 or lower for feasibility purposes. Som & Das (2006) added that column spacing is also dependent on the desired bearing capacity, in conjunction with the time allocated for consolidation arising from radial drainage through the columns. Therefore, a typical design must accommodate these factors. Figure 2.16 show the tributary areas for each layout. Zahmatkesh & Choobbassti (2010) suggested ratios of the effective diameter (2 times R_e) of the column to its spacing as 1.05 and 1.13, for triangular and square patterns respectively, where, R_e is the radius of the unit cell. Similar values have been proposed and used by other researches like Goughnour (1983) and Najjar (2013). From Figure 2.16, it is evident that a relationship is present between the effective diameter of columns and their spacing.

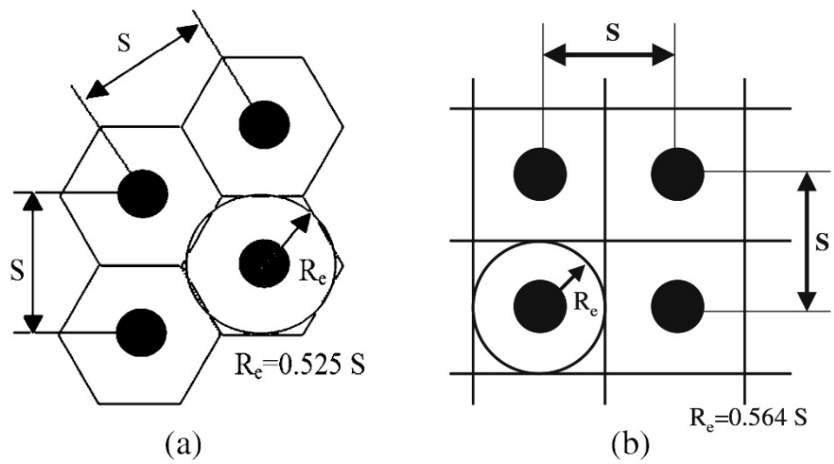


Figure 2.16: (a) Triangular and (b) Square pattern (Zahmatkesh & Choobbasti, 2010)

Over the years, the equation provided by Hughes et al. (1974) to calculate the maximum vertical stress, has been manipulated in several studies where findings extended the understanding of the effect of various parameters on stone column performance. From literature, it is noted that the following parameters largely influence the behaviour of stone columns: area of foundation, area of unit cell, length of column, diameter of column, modulus of elasticity of both column and base soil material and shear strength parameters of the column. Out of these, the most common factor is possibly the dimensions of the columns, that is, the diameter and the length. In general, these two and the spacing of columns have often been recorded as being inter-dependent. While different researchers have prescribed specific equations for determining the column geometries, Priebe (1995) suggested that it is more appropriate to determine column spacing and diameter alongside. Therefore, he proposed that the following relationship be followed in granular column design.

$$\frac{A_c}{A_s} = k\left(\frac{S}{D_c}\right)^2 \qquad \text{(Equation 2.23)}$$

where: $\frac{A_c}{A_s}$ = area replacement ratio,

A_s = plan area of unit cell attributed to a column,

A_c = cross-sectional area of the column,

k = a factor of $\frac{4}{\pi}$ and $\frac{2}{\pi\sqrt{3}}$ for square and triangular column layout, respectively,

S = spacing of column, and

D_c = diameter of column.

According to Bowles (1997), the resistance which is developed from granular columns is principally from the perimeter shear, rather than from the end-bearing. Hence, he claims that a minimum length (L_c) is necessary and he subsequently defines it using the following equation, which evidently shows that this dimension is reliant on the diameter:

$$L_c \geq \frac{P - A_c(9c_p)}{\pi d c_s} \qquad \text{(Equation 2.24)}$$

where: P = total load applied to granular column,

A_c= cross-sectional area of granular column,

d = average diameter of the granular column,

c_s = side cohesion of granular column in clay, and

c_p = soil cohesion at base or point of granular column.

In a separate study, Mitra & Chattopadhyay (1999) stated that the ratio of the length to the diameter of the column must be at least 4.5 for the development of the full limiting axial stress within the member. In comparison, Sobhee-Beetul (2012) used 3 different column geometries, in a laboratory study, by varying the column diameter while keeping the length constant. These variations produced respective length to diameter ratios of 4, 5.7 and 8. The generated findings confirmed the adequacy of the column dimensions, for the compression tests.

Engineering parameters

Besides the arrangement pattern and the column geometry, several other factors affect the behaviour of granular columns. Hughes et al. (1975), as cited by Som & Das (2006), pointed out some of the engineering properties which influence the soil-column behaviour. They are as follows:

- shear strength of the weak soil in an undrained state,
- lateral stresses within the host soil,
- radial stress-strain properties of the in-situ soil,
- dimensions of the columns,
- stress-strain relationships of the column material,
- and friction angle of the column material.

2.3.2.4 Behaviour of groups of columns

In most applications, granular columns are practically used within groups as opposed to their single installation. Small groups of columns are generally used to support pads or strip footing, while larger groups aim at treating grounds to support widespread loads such as those from embankments and storage tanks (McCabe, Nimmons & Egan, 2009; Al-Obaidy, 2017). Barksdale & Bachus (1983) explain that, within such groups of columns, the ultimate load capacity of any single column is slightly higher than an isolated single column. They explain this difference in terms of the confinement generated by the exterior columns in the group. As such, the stiffness of the bordering columns is enhanced, thereby producing an increase in the ultimate load capacity of each column. Killeen & McCabe (2014), as cited by Al-Obaidy (2017), shared similar views and further noted that this effect is more remarkable in smaller column groups. They additionally claimed that the vertical stress under small foundations is drastically affected with depth, when compared to that beneath largely loaded areas.

To analyse the performance of granular columns in groups, Balaam, Poulos & Brown (1978) equated the performance of any single column, in a group of them, to that of another, and thus applied the unit cell concept. This perception has since been adopted in many studies (Goughnour, 1983; Priebe, 1995; Alamgir et al., 1996; Abhijit & Das, 2000; Ambily & Gandhi; 2007; Ghanti & Khashliwal, 2008). Barksdale & Bachus (1983) considered an extension of the unit cell for analysing an infinitely large group of columns. A uniform load was applied over the foundation and the concept of unit cell was applied to each interior column. Consequently, lateral deformations across the boundaries of each cell was assumed to be zero since the load and geometry was symmetrical. This further resulted in the shear stress, on the external walls of the cell, to be zero. Therefore, following these explanations, any applied load on a cell is expected to be internally contained. Subsequently, load carrying capacity and settlement calculations are conducted similar to that adopted in the unit cell concept, as described in previous sections, provided that the surrounding soil is soft and cohesive.

In case of large group of columns improving firmer and stronger cohesive soils, with undrained strengths greater than 30 to 40 kN/m^2, Barksdale & Bachus (1983) proposed a method for determining the ultimate bearing capacity whereby both the angle of internal friction of the surrounding soil and the cohesion in the column were ignored. Further assumptions considered loading through a rigid foundation while presuming that the mobilization strength of both the column and the cohesive soil had been achieved. The proposed method additionally incorporated the effect of the following: shape and size of the foundation, the internal friction angle of the column, the composite shear strength of the improved soil, the shear strength and overburden pressure applied to the soil around the foundation and the compressibility of the unimproved ground. Based on these, Barksdale & Bachus (1983) subsequently analysed a cohesive soil which was improved with granular columns and covered with an infinitely long rigid concrete footing. They assumed that the soil under the foundation failed along a straight rupture surface, thereby forming a triangular block as shown in Figure 2.17.

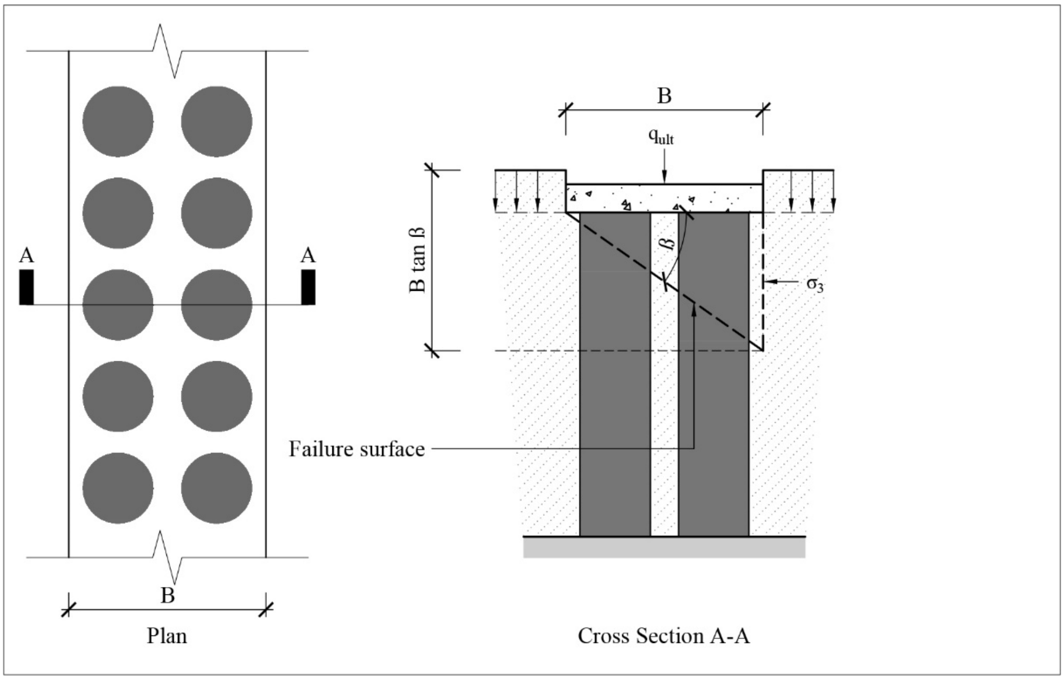


Figure 2.17: The assumed rupture surface during failure of a soil supporting an infinitely long foundation on several granular columns (adapted from Barksdale & Bachus, 1983)

Following this assumption, the average shear resistance would be developed on the failure surface within the improved soil. This would thus influence the ultimate stress (q_{ult}) of the composite ground, in addition to the lateral and ultimate motion resistance of the block (σ_3). By applying the laws of equilibrium, and regarding the ultimate vertical and lateral stresses (σ_1 and σ_3) as the principal stresses, the following equation was derived and the method of determining the different parameters was presented in Table 2.5:

$$q_{ult} = \sigma_3 tan^2\beta + 2c_{avg} tan\beta \qquad \text{(Equation 2.25)}$$

Table 2.5: Description of the different parameters in equation 2.25 and their method of determination

Parameter	Equation	Description	Equation No.
σ_3	$\sigma_3 = \frac{\gamma_c B tan\beta}{2} + 2c$	• γ_c is the saturated unit weight of the cohesive soil • B is the foundation width • β is the failure surface inclination • c is the undrained shear strength within the unreinforced cohesive soil	Equation 2.26
β	$\beta = 45 + \frac{\varphi_{avg}}{2}$	• φ_{avg} is the composite angle of internal friction	Equation 2.27
φ_{avg}	$\varphi_{avg} = tan^{-1}(\mu_s a_s tan\varphi_s)$	• μ_s is the ratio of stress in column to the average stress over the unit cell area • a_s is the area replacement ratio • φ_s is the angle of internal friction of the granular soil	Equation 2.28
c_{avg}	$c_{avg} = (1 - a_s)c$	• c_{avg} is the composite cohesion on the shear surface	Equation 2.29

2.3.2.5 Typical failure mechanisms of granular columns

The design of granular columns provides the necessary information to verify their adequacy to safely sustain the desired loading conditions. Besides these calculations, an understanding of the mechanism of failure of these columns remain critical. Granular columns mainly derive their strength from the lateral support provided by the surrounding soil. When a vertical stress of σ_o is applied to a column, which has higher stiffness and shear strength properties, the stress within the column is also affected. Figure 2.18 shows the effect of lateral support on column stress.

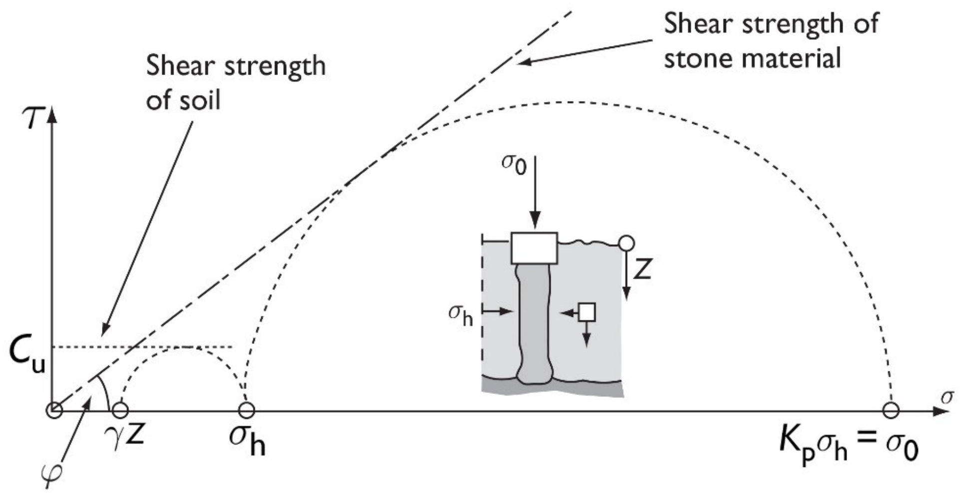


Figure 2.18: Effect of lateral support on the stress in the column (Brauns, 1978)

Sondermann & Wehr (2004) explained that, under such loading conditions, an interaction is generated between the soil and a column. Therefore, to mobilise the lateral support, it is necessary for a horizontal deformation to occur. However, under the ultimate vertical load, the support provided by the surrounding soil to the column diminishes, thereby producing high deformation rates. Under these circumstances, foundations supported on granular columns attain their serviceability limit state, following which failure occurs. Brauns (1978), as cited by Sondermann & Wehr (2004), described the column failure as either bulging or punching, beyond the serviceable state; while Barksdale & Bachus (1983) extended this understanding by suggesting different types of failures, for both singular and group of columns. These are described as follows and illustrated in Figures 2.19 to 2.21.

Single granular column

Granular columns are normally installed as either end bearing (column resting on a hard stratum which is beneath a soft soil) or as floating (bottom end of column embedded in the soft soil) elements, although end bearing ones are more commonly used (Barksdale & Bachus, 1983). Figure 2.19 illustrates both conditions under which only the column area is loaded. These suggested modes of failure apply to homogeneous soft soils.

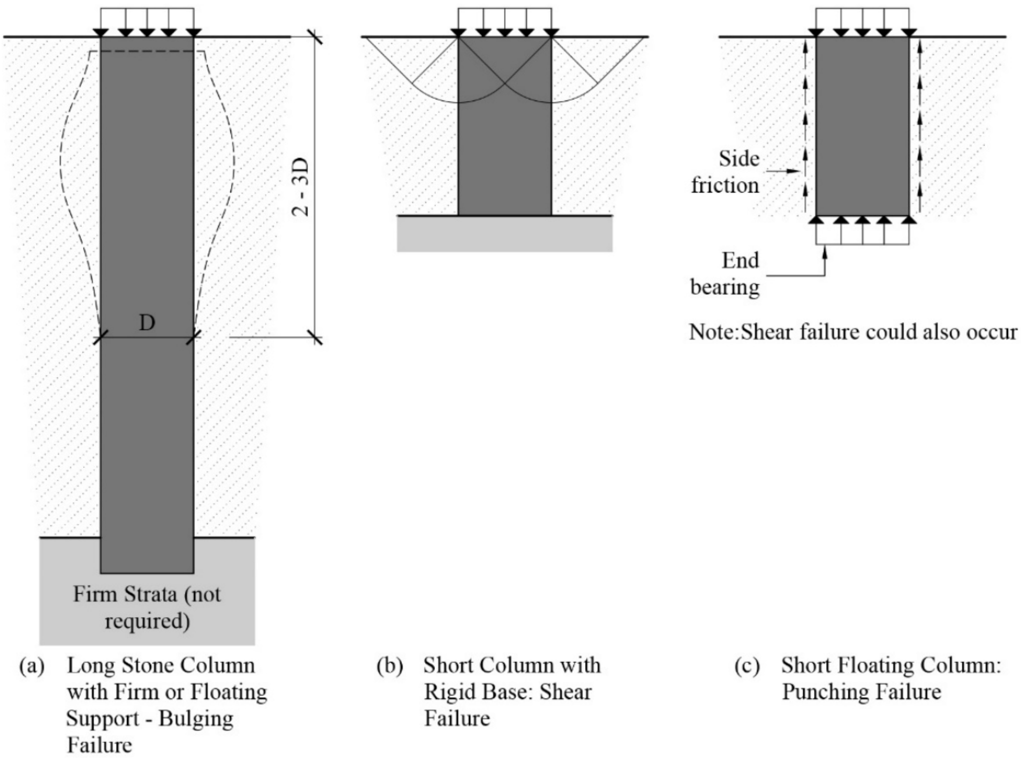


Figure 2.19: Failure mechanism of single columns in soft homogeneous soils (adapted from Barksdale & Bachus, 1983)

For both end bearing and floating columns, failure occurs in bulging provided that the length is greater than 3D (3 times the diameter of the column). For much shorter columns which rest on a firm support, they experience a general or local bearing capacity (punching of a relatively rigid column in the soft soil) failure at the ground surface. Contrastively, floating columns may initially fail in end bearing, which is subsequently followed by a bulging failure. This is generally observed for columns which are shorter than 2 to 3 times the diameter. Barksdale & Bachus (1983) further explained that the application of the load on the columns makes an apparent influence on the extent of bulging, as well as on the ultimate load capacity. For instance, if a column is loaded through a rigid foundation, which is larger than the column area, both the vertical and the lateral stresses in the weak soil are increased, which eventually results in an improved load bearing capacity.

Hughes & Withers (1974) reported that single columns, with length to diameter ratios of less than 4, fail in end bearing before bulging failure occurs. The authors explained that failure in end bearing occurs due to a disequilibrium of vertical forces acting on the column only; this is attained when the vertically applied load surpasses the ultimate base bearing pressure and the shear resisting forces along the sides of the column. It is further pointed out that the column only provides strength within a certain depth (the bulging zone), beyond which it's base only function as an end bearing pile, hence, partially supports the vertical stresses. Therefore, Hughes & Withers (1974) recommended a critical length of 4.1D (4.1 times the diameter of the column) to achieve end bearing and bulging failure concurrently. Van Impe et al. (1997) shared similar views regarding this combined failure and explained it in terms of the lateral confining stresses, in addition to the shear stresses which the column undergoes.

For non-homogeneous cohesive soils, the modes of failure differ. Barksdale & Bachus (1983) applied knowledge from field observations, model tests and finite element analysis. Accordingly, zones of very soft cohesive soils (deep or shallow depth), undergo substantial amount of bulging, which ultimately affects the strength and settlement behaviours of both single or group of columns. An example is the presence of a very soft layer (1 to 3 m thick) at the surface which influence both of these engineering characteristics. Figure 2.20 shows how the presence of a layer of very soft soil affects the failure of singular columns.

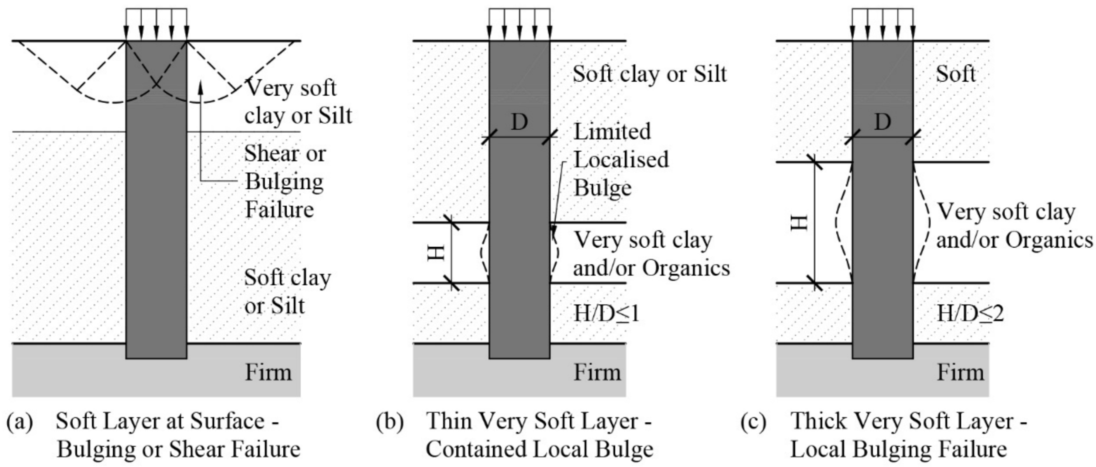


Figure 2.20: Failure mechanism of a single column in a non-homogeneous soil (adapted from Barksdale & Bachus, 1983)

Group of columns

Studies relating to the failure of groups of columns, especially in the field, is limited (Barksdale & Bachus, 1983; Hu, 1995). However, through the few available studies, it is evident that a column in a group has a slightly higher ultimate load capacity than an isolated single column due to the confinement provided by the columns in the group. Hu (1995) had reported that, within such small groups, bulging is more visible in the outer columns.

For wide flexible loading conditions such as in embankments, Vautrain (1977), as cited by Barksdale & Bachus (1983), claimed that the settlement of the underlying compressible soil is equal to that of the granular column. Upon construction of the embankment, the weak soil undergoes lateral outward movement, a phenomenon which is referred to as spreading. Spreading reduces the lateral support, thereby increasing the extent of both bulging and settlement. Figure 2.21 demonstrates the failure of columns, in both large and small groups, whereby the type of failure may possibly be through both bulging or local bearing.

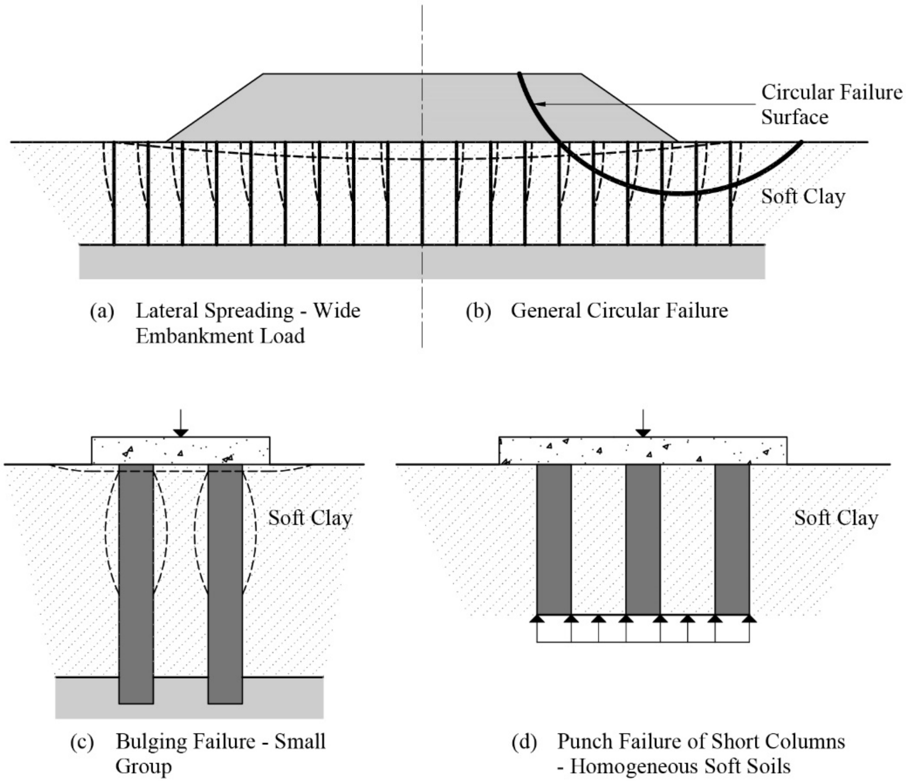


Figure 2.21: Failure mechanism of large and small column groups in homogeneous soils (adapted from Barksdale & Bachus, 1983)

2.3.3 Overview of experimental research progress and recent advances on granular columns

Different areas of study have been explored in the past such as laboratory investigations, full-scale testing and numerical modelling, all with positive outcome regarding the application of granular columns (McKelvey et al., 2004; Ambily & Gandhi, 2007; Murugesan & Rajagopal, 2008; Sobhee-Beetul & Kalumba, 2012; Al-Waily, 2012). However, in recent years, it is observed that research on the technology is gradually being shifted to a more advanced level by the introduction of a reinforcement material in the columns to improve their strength through an enhanced stiffness. Despite the added reinforcement, the main parameters which influence the overall performance of these columns remain partly the same as for non-reinforced stone columns. Among these, column diameter, column length, column spacing, column material, area replacement ratio and strength of the base soil appear to be the most researched aspects of granular columns. Nevertheless, the limitation in experimental evidence and types of reinforcement material necessitates the need for further research. Subsections 2.3.3.1 and 2.3.3.2 present the general outcome of selected laboratory and field works, conducted on both ordinary granular columns (OGC) and reinforced granular columns (RGC). For both types of columns, laboratory works are first presented, followed by the field ones. Irrespective of the mode of investigations, the studies are presented in chronological order (older ones first) with respect to their year of publication. This allows for a better understanding of the approaches which have been adopted over time, and their respective main outcomes. Key gaps in existing knowledge are presented and discussed at the end of Chapter 2.

2.3.3.1 Ordinary granular columns (OGC)

Laboratory works

In 1974, Hughes and Withers conducted a series of laboratory model experiments on single columns made of sand and installed in clay (kaolin). The clay was initially consolidated in one dimension to the desired pressure and kept under a constant stress. The column was then installed in the clay. For all tests, the length of the column was kept constant at 150 mm while the diameters ranged between 12.5 and 38 mm. A stress-controlled test was run under drained conditions on each prepared sample such that the load was applied only on the column. The displacement in the clay and the sand was obtained through radiographing of lead shot markers which were introduced within both materials. General observations from the experiments showed that the rate of settlement can be increased by a factor between 4 and 6 (as shown in Figure 2.22). Hughes & Withers (1974) further concluded that the lateral support, provided by the surrounding soil, around the bulging zone primarily governs the ultimate strength of a single column. In terms of failure, end bearing (prior to bulging) was found to be prevalent if the length to diameter ratio of the column was less than 4. To minimise the effect of bulging on the column strength, Rao & Bhandari (1977) investigated single and group of granular columns which were skirted at the top, and they recorded an improvement of 50 % in the load carrying capacity. Later in 1982, Madhav performed bench scale model tests on reinforced granular columns. He also concluded that there was an improvement in the behaviour of the column when a skirt was provided around it. He further proposed the following to enhance the column performance: use of horizontal sheets or grids to internally reinforce the column, addition of a rigid concrete plug in the upper portion of the column and increase of the footing size in relation to the column.

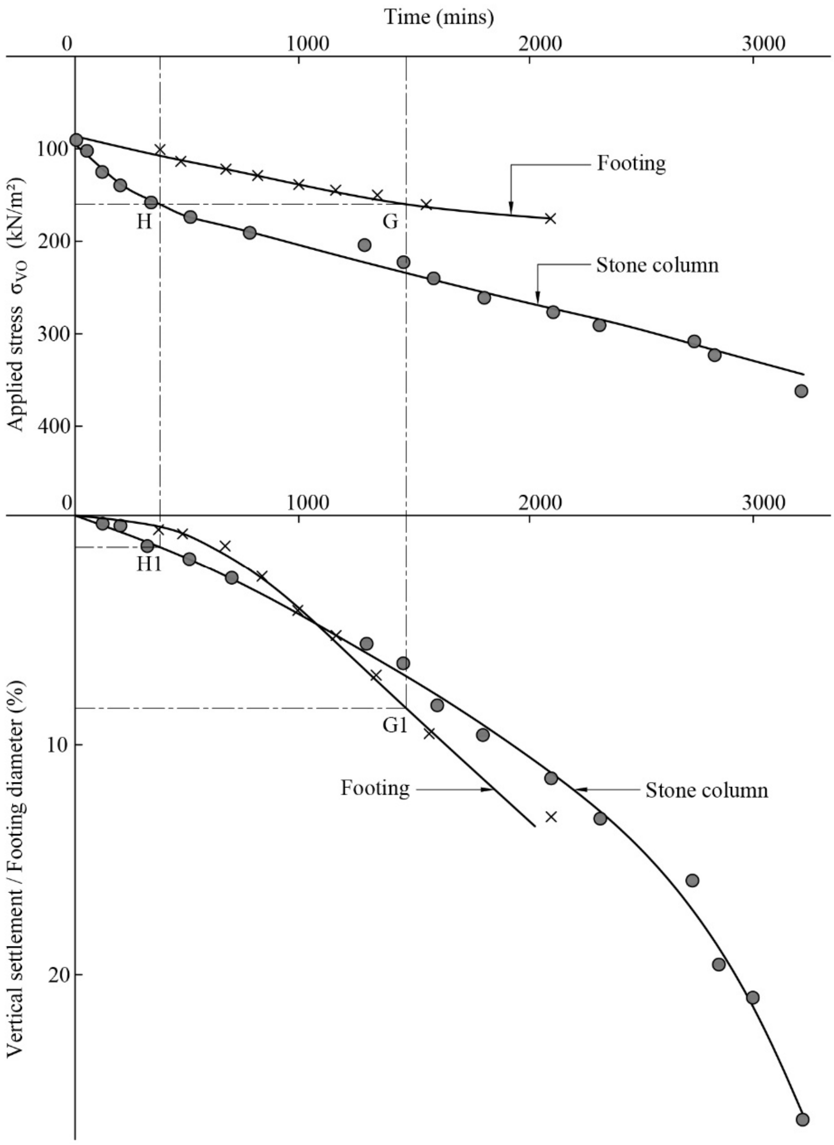


Figure 2.22: Upper part of the figure represents the stress-time relationship, and the lower part represents the vertical settlement as a percentage of the footing diameter against time (adapted from Hughes & Withers, 1974)

An intensive research on the physical modelling of foundations comprising of granular columns was carried out by Hu (1995). Laboratory tests were conducted on group of columns, installed in a clay, to study their failure mechanisms when loaded through a rigid circular footing. Parameters such as the area replacement ratio (10, 24 and 30 %), the column length (100 to 170 mm) and the column installation techniques (replacement or displacement) were investigated to understand their effect on the performance of these reinforcing elements. It was subsequently revealed that the behaviour of a column within a group was dissimilar to that of a single isolated column. In addition, columns became shorter as they were loaded and therefore expanded laterally to form a bulge, an occurrence which was dependent on the lateral confinement provided by the neighbouring columns. During loading, the concentration of stress within the column was increased such that it was higher than in the clay. This phenomenon was measured in terms of the stress concentration ratio and it was found to vary between 0.5 and 5. In terms of the load bearing capacity, higher area replacement ratios (generally over 25 %) were required to achieve a high level of improvement. Figure 2.23 shows the effect of area ratio on the stress concentration (n) against the footing stress (p), which is normalised by the initial undrained shear strength (c_u).

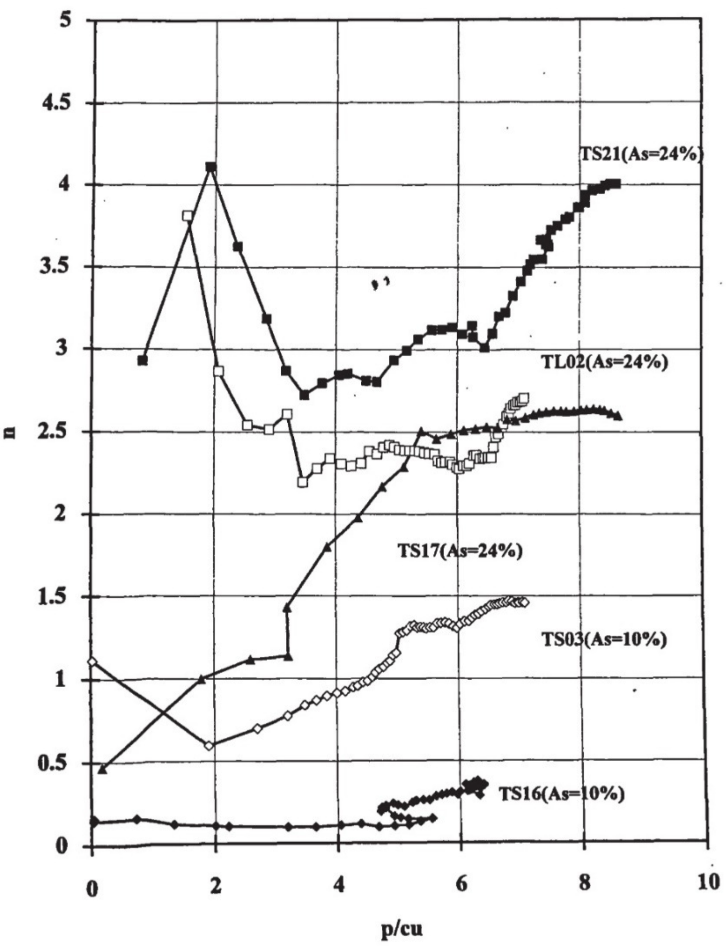


Figure 2.23: Effect of area ratio on the relationship between stress concentration ratio and the footing stress which is normalised by the initial undrained shear strength (Hu, 1995)

Experimental works were also conducted by Shroff & Patel (2003). They tested the behaviour of full length and floating columns which were constructed in kaolinite clay. The authors modified the columns by reducing their diameters at lower depths. They further substituted the stones in those depths by sand in some of the tests and termed them composite columns. The results obtained confirmed that the critical column length was equivalent to 4.25 times its diameter. Additionally, the reduction in diameter and sand replacement at lower depths resulted in relatively similar behaviour of the columns when compared to full length ones. However, a 30 % saving of aggregates was achieved. Therefore, these composite columns were proposed as an economically sustainable alternative to conventional stone columns.

The failure mechanisms of granular columns were examined by McKelvey et al. (2004), through a series of laboratory tests, installed within a consolidated clay bed. Two types of materials were used for the bed, whereby one was transparent and had properties similar to clay, while the second material was speswhite kaolin, which was prepared as a slurry (water content of 1.35 times the liquid limit). The use of the transparent material enabled visual inspection of the columns under the effect of the load. For the kaolin tests, 4 sand columns (diameter of 25 mm) were installed in a square arrangement within the wet clay bed, under a 90 x 90 mm model pad footing. The depths extended to 150 and 250 mm such that the respective length to diameter ratios were 6 and 10. Displacement controlled tests were performed at a rate of 0.0064 mm/min, up to a maximum vertical movement of 40 mm of the footing into the clay. From the observations, it was evident that bulging was more common in long columns while punching typically occurred in shorter ones. In general, the load carrying capacity of the clay bed was also improved with the inclusion of the columns. However, the authors reported that columns which were longer than about 6 times their diameter did not provide additional increase in the load carrying capacity. In fact, the estimated load at failure was 6 % lower in longer columns compared to the shorter ones. Figure 2.24 represents the variation in stress concentration ratio with increasing applied load on the foundation, where TS-11 and TS-13 are the longer columns and TS-14 is the short one.

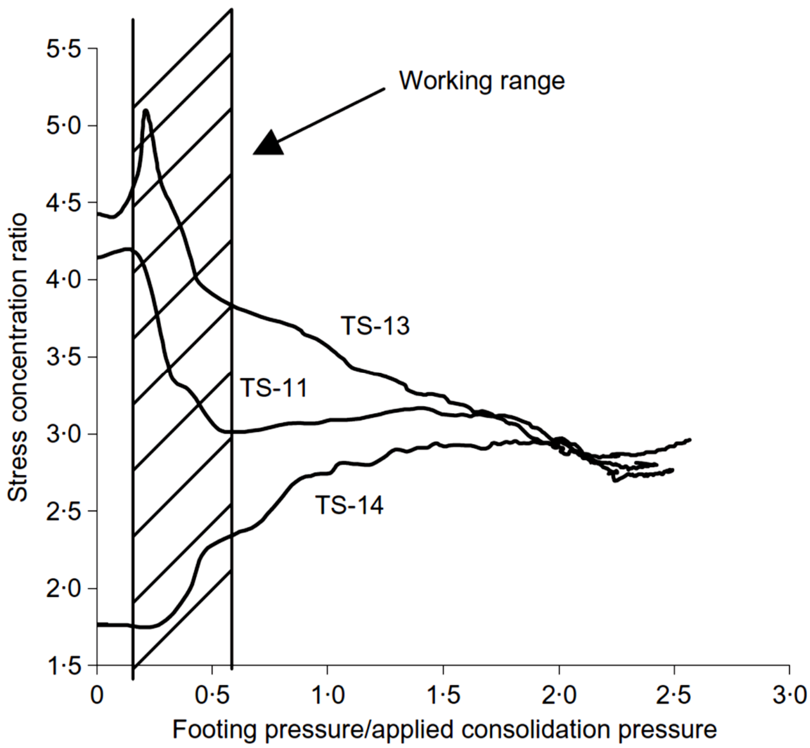


Figure 2.24: Effect of foundation load on the stress concentration ratio for kaolin tests (McKelvey et al., 2004)

Ambily & Gandhi (2007) also studied singular and groups of 7 columns. Different cylindrical testing tanks were used whereby the height was kept constant at 500 mm while their diameters ranged between 210 and 835 mm. A local clay was used to create a bed (at 3 different moisture contents – 25, 30 and 35 % corresponding to the respective shear strengths of 7, 14 and 30 kPa) in the tank into which the columns were installed. The diameter and height of the columns made from crushed aggregates of sizes ranging between 2 and 10 mm, were kept fixed at 100 and 450 mm, respectively, for all the tests. Each column was installed in 9 equal layers whereby each one of them was compacted using a 2 kg steel tamper. The tamper was dropped 10 times, through a drop height of 100 mm. Once the column was formed, tests with single columns were immediately conducted, with only the column area being loaded. For the column groups, the authors adopted a similar approach to that of Maurya, Sharma & Naresh (2005) who rather performed field tests. A 30 mm thick sand blanket was placed above the entire specimen surface which was subsequently loaded. For both types of column arrangements, the load was applied at a rate of 0.0625 mm/min. Figure 2.25 shows a typical test set-up for both single and groups of columns.

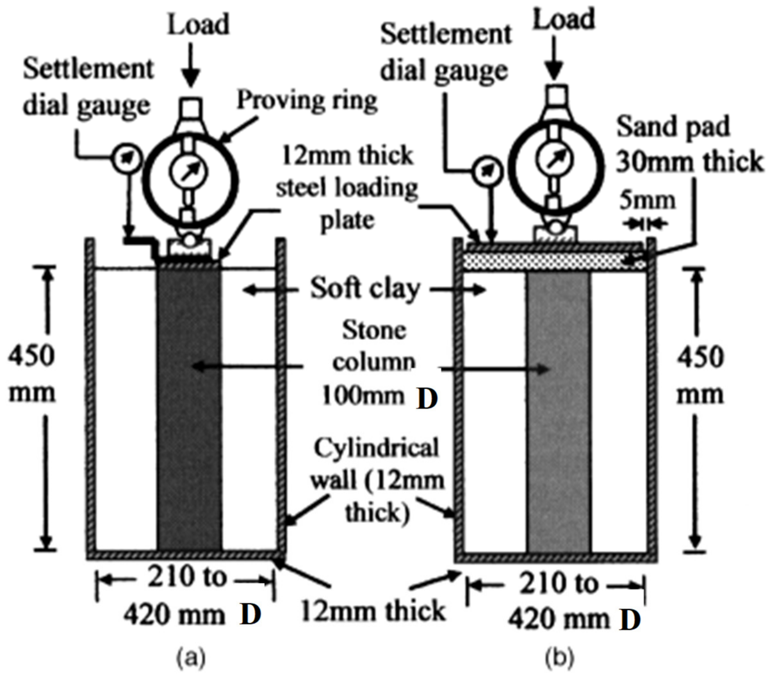


Figure 2.25: Typical test set-up for (a) single column and (b) group of columns (adapted from Ambily & Gandhi, 2007)

From the load-settlement characteristics and the column deformations, it was deduced that failure was predominantly through bulging when only the column area was loaded. The maximum bulging was recorded at a depth of approximately 0.5 times the column diameter. Figure 2.26 is an example of a typical comparison of the deformation of the columns when they were loaded alone and when the entire surface area was loaded. The results further showed that the axial capacity of the column decreased as the spacing was augmented. In contrast, larger spacings reduced the settlement up to a spacing to diameter ratio of 3.

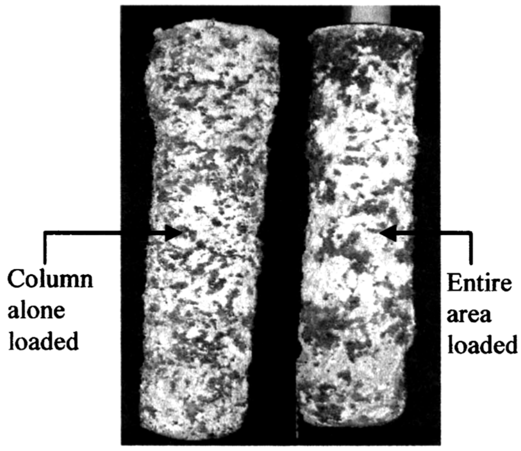


Figure 2.26: Typical column deformation post testing (Ambily & Gandhi, 2007)

Andreou et al. (2008) performed triaxial tests in the laboratory on a cylindrical clay (Speswhite kaolin) specimen, improved with a single column which was constructed from either sand or gravel. The study specifically investigated the effects of the following: drainage conditions, particle size of the column material, confining pressure of the soil, and the deformation rate. The clay sample was prepared at a moisture content equivalent to 1.4 times its liquid limit and subsequently filled in a 100 mm diameter tank of a height of 200 mm. The column diameter was kept fixed at 20 mm for all tests, with the height being similar to that of the testing tank. Tests were conducted under both drained and undrained conditions, at a shearing rate of 0.003 and 0.3 mm/min, respectively. Positive results were obtained in terms of strength gain post treatment. It was further deduced that, at high confining pressures (200 kPa), sand columns produced higher bearing capacities than gravel ones. Nevertheless, the bearing capacity was relatively similar for both of them at low confining pressures (20 and 100 kPa).

Isaac & Madhavan (2009) also performed laboratory research on the behaviour of granular columns. However, they focussed on the effect of the type of column material. In their study, they used quarry dust, sea sand, river sand, gravel and stones to form single and groups of columns (3 and 7) in a clay base. The effect of spacing of the columns was also studied. All columns installed in these tests were of diameter 50 mm and height 250 mm. For single columns, the testing tank was 210 mm in diameter compared to 520 mm for the one used for the group tests. Compression tests were performed on the test specimen at a displacement rate of 0.048 mm/min. The load-settlement response demonstrated that stones were generally more effective than any of the other materials. In addition, variation in spacing indicated that shorter spacing produced better load deformation characteristics.

Pivarc (2011) compared the settlement achieved through numerical, analytical and laboratory models. For the tests, 600 mm high cylindrical test boxes were used with variable inner diameters, ranging from 125 to 253 mm. The box was filled with a wet clayey sand on moisture content of 16 %. Singular columns of 60 mm diameter and lengths of 300, 420 and 540 mm were individually installed using gravel with particles ranging between 2 and 5 mm (as shown in Figure 2.27).

Figure 2.27: A prepared test specimen (Pivarc, 2011)

A rigid 10 mm thick steel plate was placed on the composite sample such that the whole specimen area was covered. Subsequently, loading was vertically applied through a compactor at a velocity of 5 mm/min. Table 2.6 summarises the column settlement at 2 different pressures, for differ spacing to diameter ratios.

Table 2.6: Summary of the settlement achieved for the different s/d ratios

Spacing to diameter ratio *(s/d)*	**Diameter of loading plate (m)**	**Settlement for 50 kPa (m)**	**Settlement for 100 kPa (m)**
2	0.115	0.0021	0.00582
3	0.181	0.00314	0.00945
4	0.243	0.00492	0.012

Shivashankar et al. (2011) undertook experimental studies on 90 mm diameter (D) single granular columns, installed in layered soils which consisted of clay and silt (properties given in Table 2.7. The tests were performed in a 780 mm high cylindrical tank of diameter 237 mm. The area replacement ratio was 15 % while the column spacing was equivalent to 2.5D, with the assumption that the columns were installed in an equilateral triangular pattern.

Table 2.7: Properties of the soft clay and silt used in the investigation (adapted from Shivashankar et al., 2011)

Property	**Soft clay**	**Silty soil**
Soils		
Specific gravity	2.62	2.6
Liquid limit (%)	68	47
Plastic limit (%)	32	34
Soil beds		
Moisture content (%)	45	40
Dry unit weight (kN/m^3)	12.7	12.8
UCC strength (kPa)	19	38

Within the testing tank, the bottom layer was a silt while the top part was a clay. Four thicknesses (1D, 2D, 3D and 4D) of the clay layer were investigated. The authors adopted a relatively identical testing procedure to that followed by Ambily & Gandhi (2007)., whereby tests were conducted such that the vertical load was either exerted on to the entire test specimen, or only on the column surface. Figure 2.28 represents how the top weak layer affects the settlement reduction ratio, with increasing load intensity when only the column is loaded. The study further concluded that, in layered soils, a stronger bottom layer is not necessary for a weak layer thickness greater than 2D. Also, in terms of the limiting axial stress, it is identical to that of the granular columns in a soft clayey ground, when the top layer thickness is equal to 4D.

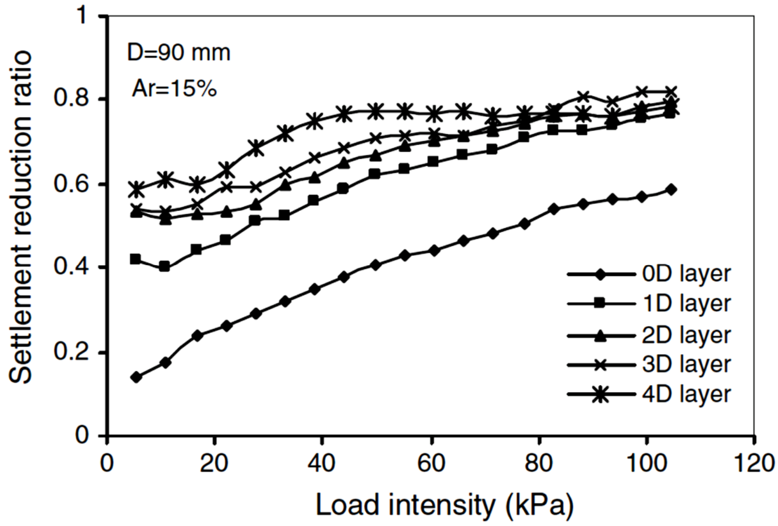


Figure 2.28: Effect of clay layer thickness on the settlement reduction ratio for column only loaded tests (Shivashankar et al, 2011)

Experimental laboratory research by Al-Waily (2012) aimed at establishing the relationship between the bearing improvement ratio (ratio of the bearing capacity of the treated soil to that of the untreated soil, $q_{treated}/q_{untreated}$) and the area replacement ratio. The variables in this study were the column diameter (20, 30, 50 and 60 mm with respective area replacement ratios of 0.042, 0.099, 0.333 and 0.563) and the undrained shear strength (11, 16 and 22 kPa) of the soil needing treatment. A 240 x 240 mm square testing tank, 265 mm high, was used. The weak soil used, collected from a site in the North of Babylon in Iraq, consisted of 13 % sand, 35 % silt and 52 % clay and was, therefore, classified as an inorganic sandy silty clay. To improve the performance of this material, 1 to 2 mm granite stone chips were used to construct the granular columns. Stress controlled loading tests were performed on the prepared specimen, with a loading increment of 20 N until a maximum settlement of 40 mm was reached. From the results, the bearing improvement ratio was found to increase slightly as the load was increased, until a plateau was reached towards then end of the test. The highest bearing improvement ratios were recorded in a soil with shear strength equal to 16 kPa. These values were 1.16, 1.29, 1.64 and 2.29 with corresponding area replacement ratios of 0.042, 0.099, 0.333 and 0.563.

In 2012, Sobhee-Beetul investigated the possible use of granular columns in South Africa. A local clay, obtained from a construction site, was used for the laboratory tests to mimic typical ground conditions. The clay was tested at 3 different moisture contents (optimum moisture content, liquid limit and 1.2 times the liquid limit), while the column materials (crushed aggregate, uniformly graded sand, well graded sand) and the column diameters (50, 70 and 100 mm) were also varied. The experimental procedure followed by the author was almost comparable with that of Ambily & Gandhi (2007), although the level of compaction of both the column material and the clay differed. Displacement controlled tests were performed at a rate of 1.2 mm/min, up to a maximum settlement of 50 mm. Crushed aggregate was found to be the best performer for 100 mm diameter columns, and at all degrees of wetness of the clay. In fact, at 1.2 times the liquid limit, the improvement achieved in the vertically applied stress was roughly 6 times that of the unimproved clay. In tests with clays at this same moisture content, the stress concentration ratios were found to be 3.29, 2.29 and 4.57 for the columns made of the well graded sand, uniformly graded sand and crushed aggregate, respectively. In terms of the settlement reduction ratios, they appeared to vary between 0.05 and 0.65. Column deformations were additionally observed in selected tests. It was revealed that the general maximum bulging occurred within the top third of the column height, with the largest values of 133 % being recorded for a 70 mm column made from the uniformly graded sand.

Field works

Munfakh, Sarkar & Caslelli (1983) researched the effectiveness of vibro replacement granular columns. Full scale tests were conducted on an embankment, resting on a very soft cohesive soil, which was used to model a proposed wharf structure. Through the installation of monitoring instruments, the performances of the unimproved and improved ground were recorded. The use of granular columns in these tests showed an amelioration of 50 % in load carrying capacity while reducing the settlements by 40 %. Similar settlement observations (20 to 49 %) were obtained by Bergado, Rantucci & Widdodo (1984) when full scale loading tests were conducted on an embankment which was constructed on a soft clay foundation, improved by granular columns. The ultimate bearing capacity, post the column treatment, was found to be 3 to 4 times higher than that of the unimproved ground. Wood et al. (1996) also reported such reductions (44 %) in the settlement of a strip footing which was supported on a heterogeneous fill reinforced by granular columns. The ground conditions on the field were of granular nature in the top 5 m layer, whereby ash and soft silty clay were the predominant materials. The strip foundations were 9 m by 750 mm, with thicknesses of 250, 500 and 750 mm.

Saha & Das (2000) investigated the group interaction effect of granular columns installed in a weak and highly compressible soft soil, to support oil storage tanks. The tank diameters and heights of the tanks varied between 30 to 55 m and 10 to 13 m, while that of the granular columns corresponded to a range of 0.8 to 0.85 m and 14 to 16 m. Twelve full scale load tests were conducted on two different sites. Afterwards, the hydrotest load-settlement data were used for analysis purposes. A comparison was then drawn between the actual group settlements and the different analytical settlement predictions designated for individual 'cylindrical-unit'. The aim was to identify the behavioural effect of neighbouring columns on the single ones. The group interaction factor (α) was eventually proposed in terms of the area ratio (A_r), using the raw data and a regression analysis:

$$\alpha = 2.2346 A_r^{0.2987} \qquad \text{(Equation 2.26)}$$

In 2003, Raju from Keller Malaysia reported on the different ground improvement techniques which have been employed by Keller to support railway embankments across the world. Some case histories from Germany were presented whereby vibro replacement stone columns were used. The introduction of the high-speed (250 kmph) railway system in Germany necessitated an upgrade to their existing railway network. For the Hamburg-Berlin high-speed route (Wittenberge Section), a 6 km stretch of rigid pavement was supported on stone columns. These columns were installed in a triangular arrangement, having a 4-row layout with a horizontal spacing of 2 m and vertical spacing of 1.25 m. While the diameter of the columns varied between 0.6 and 0.8 m, the depths extended to a maximum of 7 m. On another railway line project in Germany (Hannover – Berlin high-speed line, Schonhausen Embankment section), stone columns were installed in a rectangular arrangement on a grid spacing of 1.85 m by 2015 m centre to centre. The columns, which extended to depths of up to 12 m, aimed at supporting the new extension backfill which was inter-connected to the old embankment.

Maurya, Sharma & Naresh (2005) completed several vertical footing load tests on single and groups of rammed granular columns in the field. These columns were adapted as a ground improvement measure to support 2 stretches of embankment (each of 800 m in length, with crest width of 8 m, side slopes of 1:3 and height varying between 3.5 and 4 m) in a thermal power plant project, along the east coast of India. From the subsurface investigation, it was found that the soil underlying the embankment comprised of 4 to 10 m of soft to very soft marine clay, covered by a thin layer of clay with sand, with the SPT ‘N’ value varying between 17 and 29. Ground water was established as being relatively close to the ground level. Initially, a stability analysis of the embankment was performed, using a model before and after treatment, from which the factor of safety of the unimproved ground was determined as 0.536. Trial tests were then conducted within 20 m distance of the proposed embankment position. Columns of 900 mm in diameter were constructed in a triangular pattern, as shown in Figure 2.29, using well graded stone aggregates of particle size ranging between 20 and 63 mm.

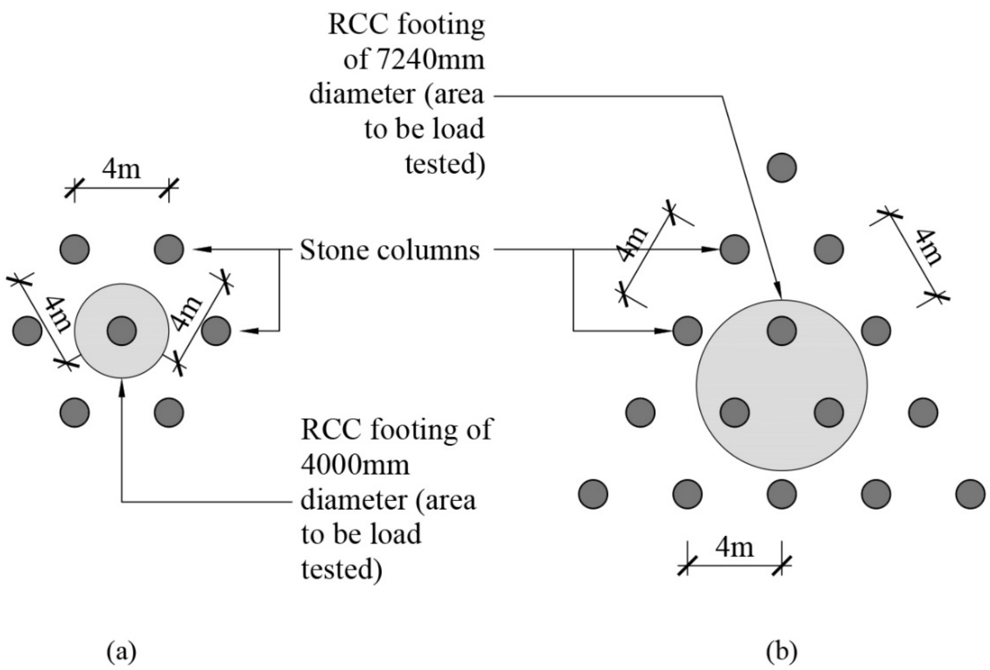


Figure 2.29: Triangular column arrangement for (a) single column and (b) group of columns (adapted from Maurya, Sharma & Naresh, 2005)

A 300 mm thick medium to coarse sand blanket was placed over each test area and the single or group of columns were loaded (arrangement shown in Figure 2.30) in stages to maximum respective surcharge loads of 1885 and 6100 kN, through a reinforced concrete footing larger than the column area. The load-settlement relationships obtained from the tests were presented and the authors subsequently used the settlements to determine the factor of safety, which was found to be equal to 1.5. Although the ultimate load capacity was found to be better in group of columns when compared to single ones, settlements were recorded as being slightly higher.

For instance, in one of the comparisons, the ultimate load for a single column was found to be 800 kN, with a settlement of 23 mm. For similar tests in a group of columns, an ultimate load and settlement of 3450 kN and 34 mm were achieved, respectively. This clearly showed a 40 % higher ultimate load per column in the group.

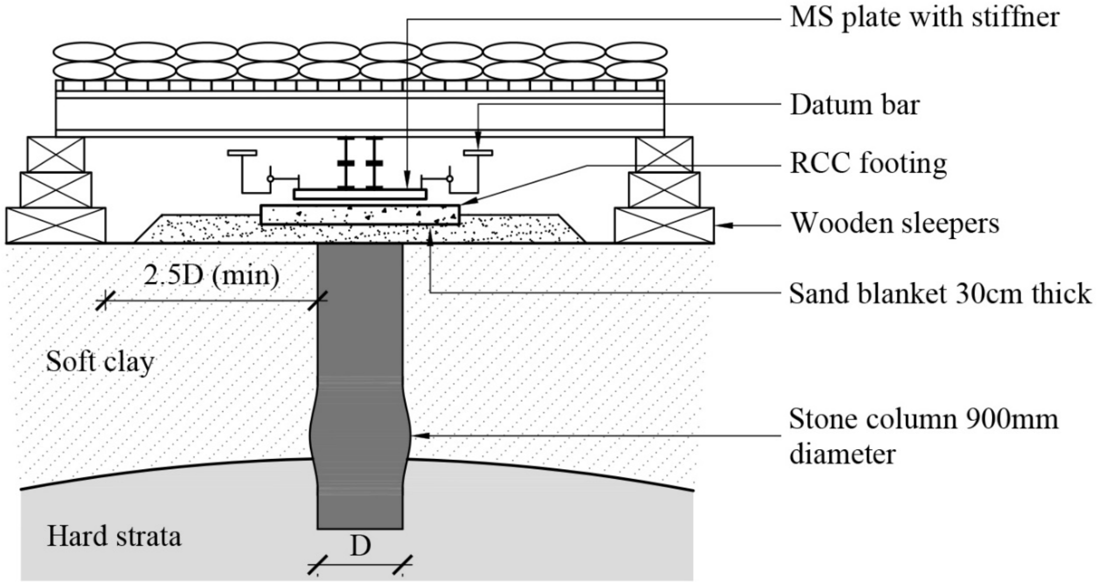


Figure 2.30: Typical test set-up for a single column (adapted from Maurya, Sharma & Naresh, 2005)

From the literature presented on ordinary granular columns in this section, it was observed that the techniques, parameters and results varied from one study to the other. Since the method of investigation in this research was anticipated to be through laboratory tests, details from the most recent bench scale studies were compiled and presented in Table 2.8 to gain an understanding of the different variables and scale involved, as well as, the type of tests undertaken.

Table 2.8: Test characteristics of the most recent research presented in this section on ordinary granular columns

Authors	**Base soil**	**Test sample size (mm)** (Height=H and Diameter= D_t)	**Column material**	**Column dimensions (mm)** (Length=L_c and Diameter=D_c)	**Area replacement ratio (%)**	**Column reinforcement**	**Foundation or footing size (mm)** (Diameter= D_f)	**Tests conducted** *Single (S)* *Group (G)*	**Test speed (mm/min)** *Undrained (U)* *Drained (D)*
McKelvey et al. (2004)	Kaolin	D_t=413 H=500	• Sand	D_c=25 L_c=150 or 250	23 - 40	-	90 x 90	Vertical compressive loading *(G)*	0.0064
Ambily & Gandhi (2007)	Clay	D_t=210-835 H=500	• Crushed aggregates (2mm-10mm)	D_c=100 L_c=450	1.4 – 22.7	-	D_f=100 or D_f=200-825	Vertical compressive loading *(S and G)*	0.0625 *(D)*
Andreou et al. (2008)	Kaolin	D_t=100 H=200	• Sand • Gravel	D_c=20 L_c=200	4	-	-	Triaxial *(S)*	0.003 (D) 0.3 (U)
Isaac & Madhavan (2009)	Clay	D_t=210 H=270 or D_t=520 H=270	• Sea sand • Quarry dust • Gravel (2-10mm) • Crushed aggregate (2-10mm)	D_c=50 L_c=250	-	-	-	Vertical compressive loading *(S and G)*	0.048
Shivashankar et al. (2011)	Clay overlying stronger silt	D_t=237 H=720	• Crushed aggregates (2mm-10mm)	D_c=90 L_c=540	15	-	D_f=90 or 237	Vertical compressive loading *(S)*	0.0625
Al-Waily (2012)	Clay	240 x 240 x 265	• Granite stone chips (1-12mm)	D_c=20, 30, 50, 60 L_c=260	4.2, 9.9, 33.3 and 56.3	-	D_f=100mm	Vertical compressive loading *(S)*	Loading controlled - 20N increments *(U)*
Sobhee-Beetul (2012)	Clay	H=450 Length=1000 Width=150	• Crushed aggregate (2-10 mm) • Well graded and uniformly graded sands	D_c=50, 70 and 100 L_c=400	-	-	D_f=2 times D_c (100, 140 and 200)		

2.3.3.2 Reinforced granular columns (RGC)

Although conventional granular columns already display a general good performance, extensive work is gradually being undertaken to assess the possibility of further improving the behaviour of these columns. More precisely, several authors have explored the effect of reinforcing these columns, both internally and externally. This subsection presents a selected series of literature, both on laboratory and field investigations, in this area of research.

Laboratory works

Al-Refeai (1992) tested granular columns (made from sand), which were randomly reinforced with fibrillated polypropylene fibres, to improve a clay of low plasticity and having the following properties: liquid limit of 40 %, plastic limit of 25 %, maximum dry density of 16.8 kN/m^3 and optimum moisture content of 18.3 %. The fibres, which were used in 25 mm and 50 mm bundles, had an equivalent diameter of 0.4 mm, tensile strength of 360 MPa and a specific gravity of 0.9. The reinforced columns were installed in a silty clay, whereby the randomly mixed fibres in the sand were placed in different layer thicknesses within the column, starting from the top, until the full column was reinforced. The following situations were investigated: pure sand column, reinforced sand layer of thickness equal to the column diameter (D), reinforced sand layer of thickness equal to twice the column diameter (2D), and lastly the column was made of only fibre reinforced sand. The degree of reinforcement added to the column was reported in terms of a percentage which did not exceed 1.2 % by weight of the column material. The outcome of the triaxial tests performed on these columns indicated that a fibre content of 0.2 % by weight improved the resistance properties of the sand effectively without compromising the density and permeability of the sand. Additionally, the thickness of the reinforced sand layer significantly influenced both the load carrying capacity and the settlement of the composite soil. This was quantified in terms of the stress-strain characteristics whereby the deviator stress was found to increase as the depth of reinforcement was augmented, for a maximum axial strain of 10 %. In fact, the deviator stress for a fully reinforced column was approximately twice that for an unreinforced column, under similar strain conditions.

Rao, Kumar & Bindumadhava (1992) also performed model tests on reinforced granular (stone aggregates and sand) columns installed in a soft clay (liquid limit of 68 % and plastic limit of 37 %). Two approaches of reinforcement were employed: 50 mm PVC tubes for peripheral restrainment and perforated metal discs as layered reinforcement. The results achieved showed an improvement of approximately 2.5 times in bearing capacity, when PVC tubes of length up to twice the column diameter (2D) was used, compared to conventional granular columns. Similar amelioration was achieved with the metallic discs when spaced at intervals of 1D, up to a length of 5D.

Rao & Nayak (1995) also researched the use of tubes and explained it as a means of circumferential reinforcement, a similar approach to that used by Rao, Kumar & Bindumadhava (1992). Sand columns with respective diameter and length of 100 mm and 250 mm were installed in a 300 mm diameter by 400 mm high cylindrical testing tank, which was filled with a soft compressible clay. The number of layers (1, 2 and 3 layers) of the geogrid (Netlon mesh) was varied in the study to understand their effect on the column performance. Cyclic loading tests were also conducted to understand the effect of repeated loading. This experimental study concluded that the strength and stiffness of these reinforced columns increased significantly when more Netlon mesh layers were used. Additionally, repeated loading appeared to increase the stiffness of the columns, irrespective of the number of geogrid layers, although a constant was achieved after 2 to 3 cycles of loading.

Sharma, Phanikumar & Nagendra (2004) used geogrid for reinforcing granular columns which were installed in a clayey silt. However, the geogrids were placed as horizontal layers within the column which was made from crushed stones. The principal variables in this study were the number of geogrid layers and their spacing. This study concluded that the reinforced columns produced an additional increase in load-carrying capacity when compared to an ordinary column, with an increase in the number of geogrid and shorter spacing causing an improvement in the behaviour of the columns. For instance, the stress of a composite ground with a reinforced column (5 layers at a spacing of 10 mm) was 258 % higher than that of an unreinforced clay, to achieve a settlement of 3 mm. For similar conditions, only 80 % increase was obtained with an ordinary granular column. Besides the gain in strength, the diameter of the bulge also decreased when the reinforced column (5 layers at a spacing of 10 mm) was used in comparison to ordinary ones. The respective bulge in diameter in each of these columns were 1.04D and 1.27D. For the same reinforced column configuration, the length of the bulge also decreased when reinforcing the column with geogrids.

In the laboratory investigation undertaken by Malarvizhi & Ilamparuthi (2004), both ordinary and reinforced single granular columns were investigated; the latter was encased using geogrids. Marine clay of high plasticity, and liquid limit and plasticity index of 55 % and 37 %, respectively, was used as the soft clay bed which was prepared in a 280 mm high tank of diameter 300 mm. Granite stone chips, of particle size ranging between 5 and 10 mm, were used to form the columns while 3 types of geogrids (different stiffness) were used to encase the columns. Loading on the column was done through a rigid circular plate of diameter 2.3 times the column diameter, and the load-settlement behaviour was recorded. The results confirmed the importance of encasing through the enhancement achieved in terms of load carrying capacity, whereby the stiffest reinforcement produced higher ultimate load capacity. In general, the ultimate bearing capacity of ordinary and reinforced columns was found to be twice and thrice that of the untreated base soil, respectively.

In comparison with Malarvizhi & Ilamparuthi (2004), Murugesan & Rajagopal (2008) also found that geosynthetic encasement increased the load capacity of the columns. They claimed that this improvement is 3 to 5 times higher than ordinary columns, although the stiffness of the geosynthetic played a major role in this observation. They further noted that small diameters of geosynthetic encased columns produced better improvement. Similar views, regarding load capacity of reinforced columns, are shared by Afshar & Ghazavi (2014). Their study also concluded that lateral bulging, during bulging failure mechanism, decreases by the inclusion of the geotextiles. Additionally, an increase in strength of the reinforcement material further contributed to the reduction in bulging.

Ayadat, Hanna & Hamitouche (2008) studied the effect of internally reinforcing sand columns using horizontal wire meshes made of plastic, steel and aluminium materials (properties are given in the following table). These were placed horizontally in the upper part of the column, as shown in Figure 2.31.

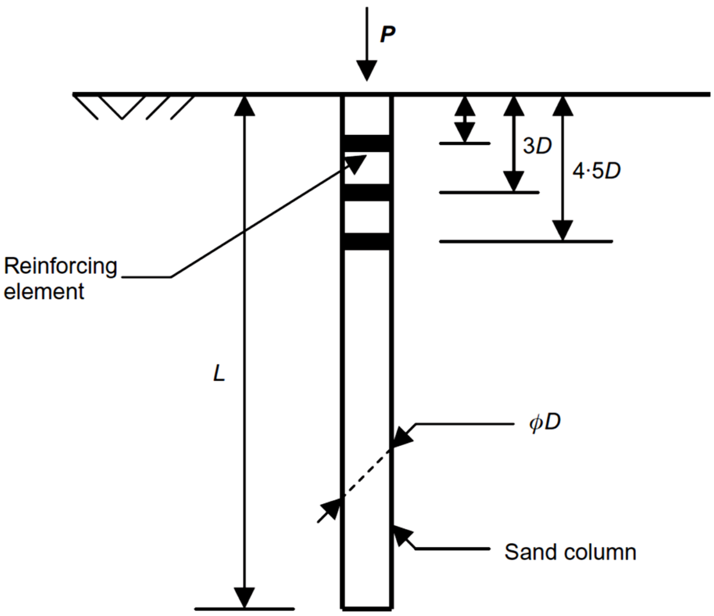


Figure 2.31: Horizontal meshes placed within sand column to act as reinforcing elements (Ayadat, Hanna & Hamitouche, 2008)

The base material used was kaolin (normally consolidated) while the sand in the column was coarse and uniformly graded. Results from this study confirmed that the load carrying capacity of the clay was improved through the installation of these reinforced columns. An increase in the number of meshes additionally raised this performance. For instance, when 3 meshes were used to reinforce the columns, an increase in load carrying capacity of up to 75 % was achieved compared to 54 % and 38 %, respectively, for double and single mesh. Furthermore, aluminium meshes product appeared to be the most effective. Table 2.9 summarises the testing configuration and both the computed and the laboratory results are presented with regards to the ultimate load carrying capacities.

Table 2.9: Laboratory results of the ultimate carrying capacity for each test (adapted from Ayadat, Hanna & Hamitouche, 2008)

Type of reinforcement	Diameter of column (mm)	Strength (kN/m)	Number	Ultimate carrying capacity (N)
Biaxial geogrid	60	7.68	2	-
			3	-
			5	-
Nylon Meshes	23	90	Single at 3D	106
			Double	127
			Triple	143
Steel meshes	23	200	Single at 3D	111
			Double	138
			Triple	152
Aluminium meshes	23	160	Single at 3D	129
			Double	145
			Triple	179

Similar to Ayadat, Hanna & Hamitouche (2008), Wu & Hong (2008) also explored the internal reinforcing of granular columns made from sand of internal friction angle of 36.8°. Horizontal geotextile sheets were inserted within the sand column at different intervals, as shown in Figure 2.32. Triaxial compression tests were conducted on cylindrical specimens, which were 140 mm high and 70 mm in diameter, to verify the analytical procedure proposed. Four layers of reinforcement were placed at equal spacing. Results from this study showed that a stiffer geotextile lowered the axial strain of the sand column. Moreover, shorter spacing between the geotextiles produced an enhanced stiffness of the column. Figure 2.32 shows a prepared specimen before and after the triaxial test (at 26 % axial strain)

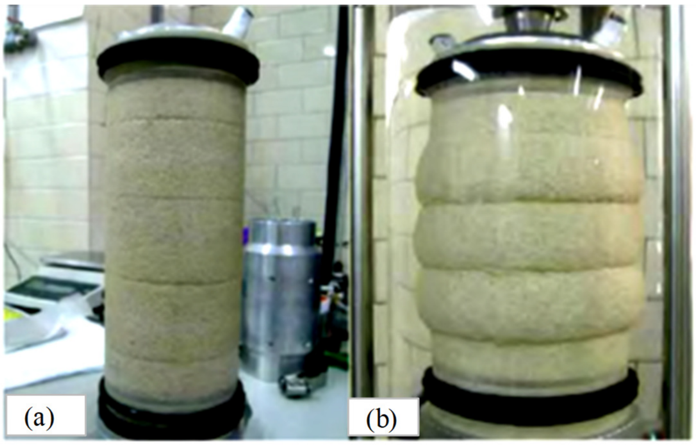


Figure 2.32: Sand column reinforced with 4 layers of geotextile (a) before testing and (b) after testing at an axial strain of 26 % (Wu & Hong, 2008)

Besides the most common materials used for granular columns (stone and sand), Tallapragada, Golait & Zade (2011) proposed the use of stone dust and lime, in addition to encasing the column with a geosynthetic. They believed that the mixing of sand or stone dust with a small quantity of lime induces some bonding between the particles which improves the column performance without affecting its permeability. Black cotton soil (95.4 % silt and clay) was used as the base material. This soil had a liquid limit of 67 % and plasticity index of 46 %. It was prepared at a water content of 25 % and left for 24 hours before being used in the tests which were performed at a rate of 0.02 mm/hr. The results from this test indicated a reduction in settlement from 11.9 to 41.5 %, when the geosynthetic encasement was used in shorter and smaller columns and in longer and larger columns, respectively. The corresponding increase in load carrying capacity, due to the encasement, for these types of columns were from 21.6 to 45.0 %.

In 2014, Ali investigated both floating and end-bearing columns. He further experimented with partial and full encasement of the columns by geotextiles. However, he reported that partially encased columns, both floating and end-bearing, were not beneficial. Since the increase in load carrying capacity of the improved ground was directly proportional to the encasement length, floating columns (despite being fully encased), did not perform well. He concluded that fully encased end-bearing columns were the best performers.

Chen et al. (2015) also researched geosynthetic encased granular columns. They used a physical model (scale 1:25) in the laboratory to observe the failure mechanism of these columns in a road embankment application. Bending failure was reported as being the main failure mode for such loading conditions. The columns bended due to sliding of both the embankment and the foundation soil. The unbalanced lateral stresses further contributed to this failure. Figure 2.33 illustrates this behaviour.

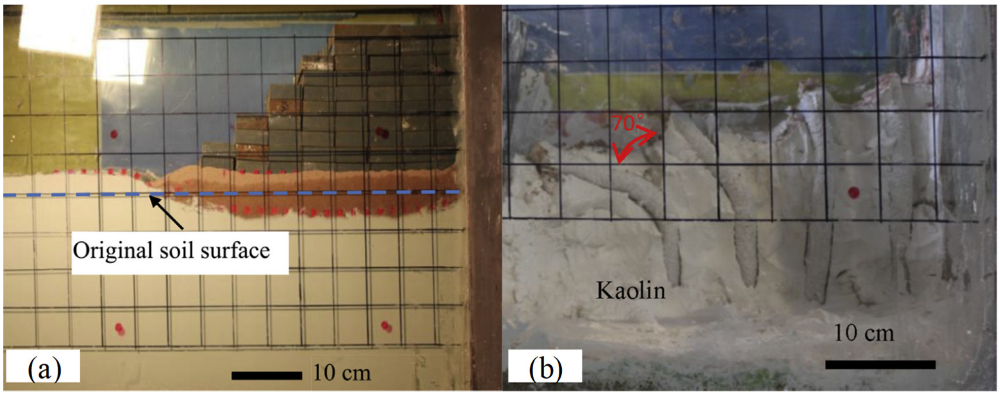


Figure 2.33: (a) Soil deformation before the last loading stage and (b) bending of the geosynthetic encased column after testing (adapted from Chen et al., 2015)

Han (2015) explained that geosynthetic encasement increases the stiffness of columns when compared with granular columns. However, geosynthetic-encased columns are more expensive and slower to install as compared with granular columns without geosynthetic. Therefore, the application and allowable time for implementing the method directly affects the choice of the technology adopted.

Al-Obaily (2017) examined the stress-settlement characteristics and the failure mechanism of a footing-type foundation which was supported by either one of these conditions: untreated soil, soil treated with an ordinary granular column, and soil treated with an encased granular column. The untreated soil was described as being an artificial loess deposit which was to be subjected to inundation. Conventional geotechnical laboratory work was undertaken, in addition to using the electrical resistivity tomography method. An analytical solution was also presented using a MATLAB script such that the load carrying capacity of the reinforced foundation could be determined. The script was also used to validate the experimental results. Columns (made of crushed stones of particle size 1 to 3 mm) of 40 mm diameter and 360 mm long were installed in a soil bed cell (diameter of 349 mm and height of 360 mm) to form a scaled-down model. Loading of the test specimen was performed in doubled increments, whereby each increment was kept constant until a fixed settlement was reached. A typical experimental set-up is shown in Figure 2.34. From the laboratory model, it was found that the geotextile encased columns significantly raised the ultimate bearing capacity, in both the presence and the absence of water. However, when water was added, a higher improvement was recorded which corresponded to 49 and 77 %, for added water of 2.73l and 6.15l, respectively, as opposed to 14 % and 20 % improvements under similar conditions but without the encasement. The introduction of the geotextile also impacted the settlement characteristics. For inundation situations of 2.73l and 6.15l being added to the soil, the settlement improvement factor increased from 216 to 285%, correspondingly. With regards to bulging, the high degree saturations increased the amount of bulging. However, for columns which were encased, bulging was restricted since the geotextile acted as a confining barrier.

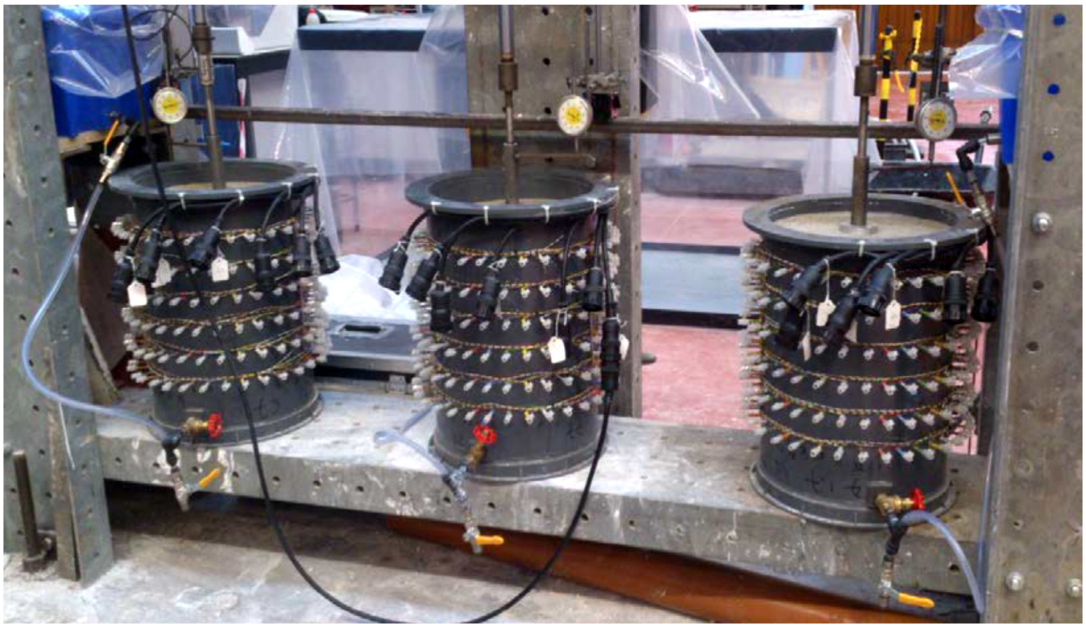


Figure 2.34: Experimental set-up of 3 cells being loaded at a time while one resistivity channel is connected to the acquisition system, water is provided from a source and the dial gauges are set (Al-Obaily, 2017)

Field works

Tandel, Solanki and Desai (2014) carried out field load tests on single sand columns, reinforced with a geotextile, such that only the column area was loaded. The geotextile was used in the form of a tube to support the column. The tube was made by bonding the section with epoxy-resin. The results concluded that an improvement of 20 to 85 % in load carrying capacity was achieved when reinforcing sand columns, while the settlement reduced by 20 to 54 %. The column diameter and the stiffness of the reinforcement principally influenced these improvements. For example, when the reinforcement stiffness changed from 121.9 to 450 kN/m, an increase of about 27 to 44 % was observed in terms of the stress in the column, at a 50 mm settlement. The outcome further confirmed that the optimum reinforcement length was about 4D.

Alexiew, Moormann & Jud (2009) reported about the foundation solution which was required for building a steel plant in the lowlands at the Brazilian seashore, near Sepetiba. The total area of the stockyard was 800 x 600 m which was situated on soft soil layers, between 15 to 20 m thick of clay, which was underlain by sands and rock; the ground water table was located just below the surface. The different activities which were anticipated to occur on this site, including the use of heavy equipment similar to those found in open mining, implied that a deep foundation and/or a soft soil improvement method was necessary for these ground conditions. This was specifically needed to address issues such as high settlement, low bearing capacity, general deformability or ductility, construction and maintenance costs and construction time. A combination of 2 methods was proposed: (1) reinforced granular columns in the form of geotextile encased sand columns, and (2) high strength geosynthetic to be placed horizontally over the entire area. The aim of the columns was to enhance the bearing capacity while reducing the compressibility of the ground; the additional use of the horizontal geosynthetic was to increase the overall stability while also reducing the lateral pressures and horizontal displacements beneath the sensitive runways. In this project, preliminary survey and measurements confirmed that the proposed solution was effective.

Almeida et al. (2014) undertook a separate study based on a test area which was a small section of the large stockyard which was reported by Alexiew, Moormann & Jud (2009). The aim of the investigation was to assess the performance of the geotextile-encased column with regards to surface settlement, radial deformation of the geotextile encasement, surface vertical stresses and excess pore water pressure. A 5.35 m high trail embankment was constructed, in 4 stages and in over 65 days, on the improved foundation which resulted in a total applied stress of approximately 150 kPa. The results revealed that differential settlements augmented with increase in the height of the embankment and when the consolidation progressed. Measurement from the inclinometers also indicated that the maximum horizontal displacement occurred on the middle of the soft clay. Additionally, the horizontal displacement continuously increased as the excess pore pressure was dissipated. In terms of the vertical stresses, the stress concentration (which resulted from soil arching) showed that the stress which was transmitted to the column was as high as 2.3 times higher than the vertical stress on the surrounding soil. Overall, the test embankment displayed a satisfactory performance whereby the maximum horizontal and vertical displacements were 0.15 and 0.5 m, respectively; when the final load was applied, pore pressure and settlement were stabilised at around 6 months.

From the literature review presented on reinforced granular columns, the test characteristics of the most recent ones have been presented in Table 2.10. This assisted in observing the trends followed by different researchers, which was believed to be important when designing the testing approach in this research.

Table 2.10: Testing information from the most recent studies which have been presented in this sub-section on reinforced granular columns

Authors	**Base soil**	**Test sample size (mm)** (Height of H and Diameter of D)	**Column material**	**Column dimensions (mm)** (Length=L_c and Diameter=D_c)	**Area replacement ratio (%)**	**Column reinforcement**	**Foundation size (mm)** (Diameter= D_f, Length-L_f and Width=W_f)	**Tests conducted** *Single (S)* *Group (G)*	**Test speed (mm/min)** *Undrained (U)* *Drained (D)*
Malarvizhi & Ilamparuthi (2004)	Marine clay	D=300 H=280	Granite chips (5-10mm)	D_c=30	10	Geogrid (3 types)	D_f=2.3Dc	Vertical compressive loading *(S)*	Time controlled – Hourly loading increments
Murugesan & Rajagopal (2008)	Clay from lake beds	D=210 H=500	Granite chips (2-10mm)	D_c=50, 75 and 100 L_c=500	23.8, 35.7 and 47.6	• Geotextile • Geogrid	D_f=50, 75 or 100 (D_f=D_c)	Vertical compressive loading *(S)*	1.2 *(U)*
Ayadat, Hanna & Hamitouche (2008)	Kaolin	D=390 H=520	Sand (1.18-2.36mm)	D_c=23 L_c=470	5.9	• Steel mesh • Nylon mesh • Aluminium mesh	D_f=390	Static loading with surcharge of 100 kN/m^2 on clay surface *(S)*	-
Hong & Wu (2008)	-	D=70 H=140	Sand	D_c=70 Lc=140	-	Geotextile	-	Triaxial compression test *(S)*	-
Afshar & Ghazavi (2014)	Clay	1200 x 1200 x 900 H=900	Crushed aggregates (2mm-10mm)	D_c=60, 80 and 100	9, 16 and 25	Geotextile	D_f=200	Vertical compressive loading *(S)*	1 *(U)*
Ali (2014)	Kaolin	D=400 H=700 and D=400 H=500	Stone chips (2-4.75 mm)	D_c=50 L_c=450	25	Geotextile	D_f=100	Vertical compressive loading *(S)*	1 (*U*)
Chen et al. (2015)	Kaolin	1200 x 400 x 800 H=800	Sand (2-4 mm)	D_c=32 L_c=400	-	Geotextile	L_f=1200 W_f=400	Static loading with surcharge of 21 kPa over the top surface *(G)*	*Consolidated (U)*
Al-Obaily (2017)	Artificial loess containing silt and clay	D=347 H=360	Angular crushed stones (1-3 mm)	D_c=40 L_c=360	33	Geotextile	D_f=347	Vertical compressive loading *(S)*	Loaded in increments after a fixed settlement was reached *(U)*

2.4 Key scientific concepts associated with this research

Since PET waste was proposed as a reinforcement material for the columns, it was necessary to understand the legislative frameworks around waste management. The science behind the manufacturing of PET bottles is also presented, in addition to the recycling process and its significance in the South African context. The section ends with a compilation of previous geotechnical related studies which have made use of plastic waste as a soil reinforcement material.

2.4.1 Understanding plastics and the legislation

2.4.1.1 Legislation

In the constitution of the Republic of South Africa, 1996, the Bill of Rights describes the right to an environment in Chapter 2 (Section 24) as follows:

"24. Environment. -Everyone has the right-

a) to an environment that is not harmful to their health or well-being; and

b) to have the environment protected, for the benefit of present and future generations, through reasonable legislative and other measures that-

(i) prevent pollution and ecological degradation;

(ii) promote conservation; and

(iii) secure ecologically sustainable development and use of natural resources while promoting justifiable economic and social development."

This right essentially underpins the environmental policy and law, especially the framework established by the National Environmental Management Act, 1998 (Act No. 107 of 1998) (National Waste Management Strategy - NWMS, 2011). Thus, the National Environmental Management: Waste Act 2008 – Act No 59 (hereafter referred to as the Waste Act) modifies the law to accommodate for a more systematic and hierarchal approach to integrated waste management. The objectives of this Waste Act are outlined as follows:

"(a) to protect health, well-being and the environment by providing reasonable measures for-

(i) minimising the consumption of natural resources;

(ii) avoiding and minimising the generation of waste;

(iii) ***reducing, re-using, recycling and recovering waste****;*

(iv) treating and safely disposing of waste as a last resort;

(v) preventing pollution and ecological degradation;

(vi) securing ecologically sustainable development while promoting justifiable economic and social development;

(vii) promoting and ensuring the effective delivery of waste services;

(viii) remediating land where contamination presents, or may present, a significant risk of harm to health or the environment; and

(ix) achieving integrated waste management reporting and planning;

(b) to ensure that people are aware of the impact of waste on their health, well-being and the environment;

(c) to provide for compliance with the measures set out in paragraph (a); and

(d) generally, to give effect to section 24 of the Constitution in order to secure an environment that is not harmful to health and well-being."

From the list of objectives, the importance of ***reducing, re-using, recycling and recovering of waste*** is obvious. It is further stated that ecologically sustainable development must be secured while promoting justifiable economic and social development. Hence, any effort towards the reduction, re-use and recovery of waste is in line with the fulfilment of the objectives of the Waste Act in South Africa.

2.4.1.2 Types of plastic

Plastics have become such a significant contribution to man's daily activities that the daunting effects of this synthetic material, on health and the environment, are often not given considerable attention. Over the years, sophisticated technological processes applied in plastic manufacturing have resulted in a wide variety of plastics available on the market for different applications such as packaging, spare parts, technological gadgets and medical equipment among many other products. However, the type of plastic used for these productions vary since each of them is most appropriately suited for certain fabrications. Therefore, recycling of plastics is more difficult since it cannot be done when they are all mixed together. As a result, it has now become popular globally to label the plastic material by a number, known as the identification code, to ease the recycling processes. This code further facilitates consumer choices for a more environmentally healthy product and safer food packaging. Table 2.11

describes the types of plastics available on the market and their most common uses are given. In this research, PET (identification code 1) was used in different forms, such as flakes from waste bottles, fibres from waste bottles and geotextiles manufactured from either waste or virgin materials, to reinforce the granular columns.

Table 2.11: Plastic identification code (Plastics SA, 2015)

THE PLASTIC IDENTIFICATION CODE				
Symbol	**Type of Plastic**	**Properties**	**Common Uses**	**Recycled into:**
1 PET	**PET** Polyethylene Terephthalate	Clear, tough, solvent resistant, barrier to gas and moisture, softens at 80°	Soft drink and water bottles, salad domes, biscuit trays, salad dressing containers	Pillow and sleeping bag filling, clothing, soft drink bottles, carpeting, building insulation
2 HDPE	**HDPE** High Density Polyethylene	Hard to semi-flexible, resistant to chemicals and moisture, waxy surface, opaque, softens at 75°C, easily coloured, processed and formed	Shopping bags, freezer bags, milk bottles, ice cream containers, juice bottles, shampoo, chemical and detergent bottles, buckets, rigid agricultural pipe, crates	Recycling bins, compost bins, buckets, detergent containers, posts, fencing, pipes, plastic timber
3 PVC	**PVC** Unplasticised Polyvinyl Chloride PVC-U Plasticised Polyvinyl Chloride PVC-P	Strong, tough, can be clear, can be solvent welded, softens at 80°C Flexible, clear, elastic, can be solvent welded	Cosmetic containers, electrical conduit, plumbing pipes and fittings, blister packs, wall cladding, roof sheeting, bottles Garden hose, shoe soles, cable sheathing, blood bags and tubing	Flooring, film and sheets, cables, speed bumps, packaging, binders, mud flaps and mats, new gumboots and shoes
4 LDPE	**LDPE** Low density Polyethylene	Soft, flexible, waxy surface, translucent, softens at 70°C, scratches easily	Cling wrap, garbage bags, squeeze bottles, irrigation tubing, mulch film, refuse bags	Bin liners, pallet Sheets
5 PP	**PP** Polypropylene	Hard but still flexible, waxy surface, softens at 140°C, translucent, withstands solvents, versatile	Bottles and ice cream tubs, potato chip bags, straws, microwave dishes, kettles, garden furniture, lunch boxes, packaging tape	Pegs, bins, pipes, pallet sheets, oil funnels, car battery cases, trays
PS 6 PS-E	**PS** Polystyrene **PS-E** Expanded polystyrene	Clear, glassy, rigid, opaque, semi-tough, softens at 95°C. Affected by fat, acids and solvents, but resistant to alkalis, salt solutions. Low water absorption, when not pigmented is clear, is odour and taste free. Special types of PS are available for special applications.	CD cases, plastic cutlery, imitation glassware, low cost brittle toys, video cases\ Foamed polystyrene cups, takeaway clamshells, foamed meat trays, protective packaging and building and food insulation	Coat hangers, coasters, white ware components, stationery trays and accessories, picture frames, seed trays, building products
7 OTHER PACKAGING	**OTHER PACKAGING** In packaging, it could be multi-layer materials e.g. PE+PP.	Includes all resins and multi-materials (e.g. laminates). Properties dependent on plastic or combination of plastics.	Automotive and appliance components, computers, electronics, cooler bottles, packaging	Plastic timber, sleepers – looks like wood, used for beach walkways, benches etc.

2.4.1.3 A basic understanding of PET bottles

Properties

Polyethylene terephthalate is a thermoplastic polyester which is semicrystalline in nature (Leychik & Weil, 2004; Bergeret, Ferry & Ienny, 2009; Kint & Muñoz-Guerra, 1999). PET possesses several properties such as high strength, durability, low gas permeability, chemical and thermal stability, in addition to their ease of processing and handling (Awaja & Pavel, 2005; Kint & Muñoz-Guerra, 1999). The wide range of beneficial characteristics of PET often makes it a product of choice, whereby typical applications include the following: fibres, sheets and films, food and beverage packaging, electronics, automotive parts, houseware, lighting products, power tools, sports goods, photographic applications, X-ray sheets, textiles and in the manufacture of construction materials such as geotextiles (Awaja & Pavel, 2005; Kint & Muñoz-Guerra, 1999; Sinha, Patel & Patel, 2010; PETCO, 2018b; PETRA, 2018). Table 2.12 summarises some of the intrinsic properties of PET polymers; the ranges shown in some of the values indicate a variation of these properties which is dependent on the crystallinity and the degree of polymerisation.

Table 2.12: Typical intrinsic properties of PET (adapted from Awaja & Pavel, 2005)

Property	Value
Average molecular weight	30, 000 – 80, 000 g/mol
Density	1.41 g/cm^3
Melting temperature	255 – 265 °C
Glass transition temperature	69 – 115 °C
Young's modulus	1700 MPa
Water absorption (24h)	0.5 %

Manufacturing process

PET is a saturated polyester which is a product of the chemical reaction between ethylene glycol and either terephthalic acid or dimethyl terephthalate. The polymerisation reaction produces a long chain of repeated units of the basic ethylene terephthalate monomer group, as shown in Figure 2.35. Normally, the duration of this polymerisation reaction determines the length of the chain and consequently the strength of the bottle. Longer chains are stronger and more expensive to achieve since they possess a high molecular weight. For instance, Mitchell

(1990) claimed that a typical beverage bottle is achieved through a polyester chain length of 130 monomer units. To modify the moulding behaviour and properties of the product, part of the terephthalic acid is often replaced by another dibasic acid, through a process referred to as copolymerisation.

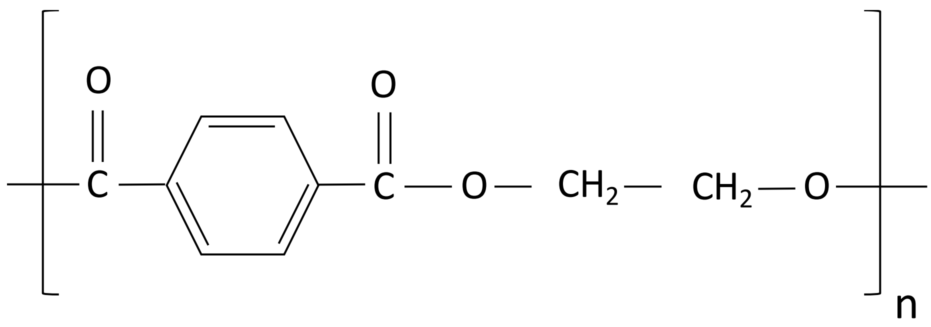


Figure 2.35: Basic ethylene terephthalate monomer

In general, PET beverage bottles are manufactured in 2 main stages: (1) pre-form manufacture and (2) bottle manufacture. In the first stage, PET granules are plasticised in an injection moulder at approximately 270 to 280°C, following which the material is injected into a multi-cavity mould, at very high pressures, to form the pre-form shape. By circulating chilled water around the mould, the molten PET is rapidly cooled and clear amorphous PET pre-forms are yielded. These preforms are then used in stage 2 which is known as stretch-blow moulding. Basically, a preform is subjected to heat and placed inside a mould which is of shape and dimensions identical to that of the desired bottle. A stretch rod is pushed down the preform to allow for axial stretching through the pressure of air blown within, until the material covers the inner surface of the mould. The temperature is then lowered to obtain the bottle. Figure 2.36 illustrates the two stages which are involved in the manufacturing process of PET bottles.

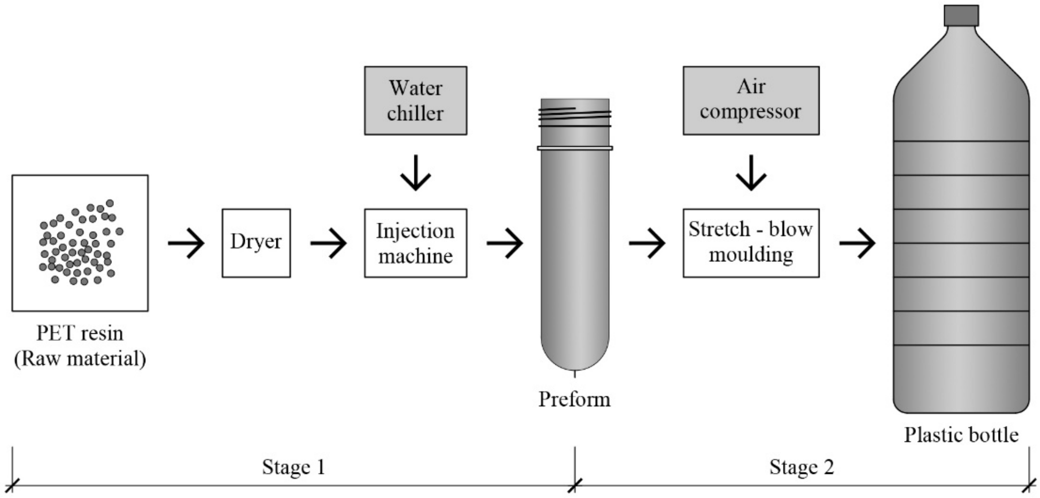


Figure 2.36: Stages in the manufacturing process of PET bottles

Mitchell (1990) explained that the thickness of PET is altered from a thickness of 3.5 to about 0.3 mm during these processes, besides attaining a biaxial orientation. Therefore, this change in alignment of the polymer chains affect the physical properties of PET such that the tensile strength and stiffness of PET are increased, by approximately 50 % in the longitudinal direction and 200 to 300 % in the circumferential direction.

Degradation

Generally, the degradation of commonly used plastics neither occur naturally nor to a high degree, when disposed of in the environment (Yamada-Onodera et al., 2001; Zheng, Yanful & Bassi, 2005; Bonhomme et al. 2003). Andrady (2011), as cited by Webb et al. (2013), described 4 mechanisms by which plastics undergo degradation when released in nature. These are: photodegradation, thermooxidative degradation, hydrolytic degradation and biodegradation by microorganisms. Degradation of plastic is typically initiated by photodegradation which is followed by the other degradation stages, in the order which is mentioned previously (Shah et al. 2008; Andrady, 2011). Basically, when exposed to ultraviolet light from the sun, energy is acquired in the form of heating which enables the incorporation of atmospheric oxygen into the polymer. Consequently, the plastic becomes brittle and thus breaks into smaller pieces. Breaking of the polymer continuously occurs until the molecular weight of the polymer is drastically reduced such that they may be metabolised by microorganisms. During metabolization, carbon which is present in the polymer is either converted to carbon dioxide or incorporated into biomolecules. Although the sequence of degradation appears to be clear and simple, the process is significantly slow. According to Müller, Kleeberg & Deckwer (2001), 50 years or more is spent on this process to achieve complete degradation of plastics. Shah et al. (2008), in fact, claimed that the stability and durability of plastics have improved so much over time that the material is mostly considered as being resistant to many environmental factors. Mueller (2006) further added that plastics are normally resistant to microbial attack since no new enzyme has yet been developed to fully degrade such synthetic polymers.

With regards to PET plastics, Hermanova et al. (2015), Webb et al. (2013) and Chiu & Cheng (1999) explained that, besides the highly beneficial properties of PET, this material also tends to be highly resistant to degradation. Zheng et al. (2005) shared similar views regarding the degradation of polymers, in particular those with pure carbon backbones. However, they explained that the presence of heteroatoms in the backbone, such as in polyesters (another name for PET), increase the possibility for the material to degrade. Nevertheless, there exists a secondary factor which also impacts on this occurrence. Polymers which are aromatic in nature are more likely to resist degradation. Webb et al. (2013) suggested PET as being a typical example of such a polymer, whereby breaking of the ester bonds in the polymeric chain are practically impossible; hence making PET non-degradable under normal conditions.

Disposal

Disposal of PET are generally of 3 main forms namely: landfilling, incineration and recycling (Zhang et al., 2004; Saha & Ghoshal, 2005). According to Chiu & Cheng (1999), post-consumer PET bottles are not a direct threat to the environment. However, due to their high resistance to degradation under normal atmospheric conditions, as well as their consequent fraction in the solid waste stream, it becomes necessary to consider the methods of disposal. This section briefly describes the above-mentioned disposal methods.

When PET is disposed of in landfills, they generally occupy large spaces which could have been used more productively, although they are normally shredded into small pieces. The slow degradability of plastic implies the unavailability of that land for longer periods of time. The anaerobic conditions in landfills, due to the limited amount of oxygen, further slows the degradation rates (Webb et al., 2013).

Incineration, which is the second means of disposal, not only reduces the occupancy of plastic in landfills, but also allows some energy recovery in the form of heat if the proper technique is applied (Sinha, Patel & Patel, 2010). Nevertheless, the technique is not often desirable due to the production of certain harmful compounds (Zhang et al., 2004). In South Africa, commercial energy recovery from plastic waste incineration is not practiced. However, a few trial plants are operating mostly on experimental basis; incineration is usually favoured for plastics which are considered difficult to recycle (Plastics SA, 2016b).

The third most popular method of disposal of PET is through recycling, which may be performed using 2 approaches: chemical and mechanical (Awaja & Pavel, 2005). Awaja & Pavel (2005) explained that recycling through chemical methods involve chemolysis between the plastic and a compound, such that depolymerisation (by hydrolysis, methanolysis, glycolysis or aminolysis) occurs and monomer units are obtained to manufacture new products. However, the process is generally expensive, thus, mechanical recycling is commonly preferred. The section which follows provides a brief description of the mechanical recycling of PET and its significance within the local context.

2.4.1.4 PET bottle recycling and the associated facts and figures for South Africa

Although PET has several benefits to humans, it appears that an immense amount of resources is necessary in the process. Seymour (1989) reported that the basic materials used in the manufacture of plastics are derived from oil, coal and natural gas while still expelling harmful gases like carbon dioxide, besides the high use of energy. Franklin Associates (2011) claimed that 2440 kg of atmospheric carbon dioxide emissions is generated, from cradle to gate, in the production of 1000 kg of PET resin. However, Williams et al. (2012) stated that this environmental cost may be decreased by making use of p-xylene in the production of terephthalic acid. This is environmentally beneficial since p-xylene may be produced from renewable sources like cellulose and hemicellulose biomass. Nevertheless, PET remains a material which is produced from relatively high amounts of energy and non-renewable resources. Therefore, it is critical to appreciate both the economic and environmental value of the material.

Since recycling is preferable over landfilling and incinerating, this section provides an understanding of the mechanical recycling of PET and its significance within the South African context.

The recycling of PET has been practised for approximately 4 decades, with Saint Jude Polymers (USA based company) being the first to initiate a recycling process for PET bottles (Forrest, 2016). Although the recyclates were only used for making plastic strapping and paint brush bristles in the first year, pellets were made from the recycled PET post that initial stage, such that their application was noted in the non-food packaging industry. One common product made from these recycled materials was carpet fibres. Forrest (2016) explained that over the previous 25 years, a dramatic increase had been observed in plastic recycling, whereby economic, environmental, societal and legislative factors had been the principal influences. He further highlighted the need to conserve natural resources and stated some important legislations which required new technologies and manufacturing processes for the recycling and re-using of waste plastic in general. These legislations included the Landfill Directive (1999/31/EC) and the Packaging and Packaging Waste Directive (94/62/EC) in the European Union.

In the early 1990s, significant interest in the research and application of the recycling of post-consumer PET emerged as a result of this costly polymer being used in the food packaging industry (Komolprasert & Bailey, 2008). Besides the high cost, these PET waste, which are disposed of in the landfills, occupy large volumes since their production rate on the market is high but they are non-biodegradable.

According to Komolprasert & Bailey (2008), the bottom of 2L PET bottles were previously made up of an HDPE base cup. However, to improve the quality of recyclates and the recycling rates of PET, HDPE is now omitted from the bottles to avoid mixing of both types of plastics. The recycling process of PET is generally desirable to the industry since they can be recovered using similar processing techniques as when they are in their virgin states, although separation and decontamination remain highly imperative.

PET bottles recycling involves several processes from collection of the waste materials to sorting, cleaning and grinding, which in turn are used for the generation of plastic pellets (through processes including washing, drying and heating). These pellets are ultimately utilized for manufacturing several products such as jackets, pillow fibre and geosynthetics. The following figure summarises the stages in the recycling of PET bottles.

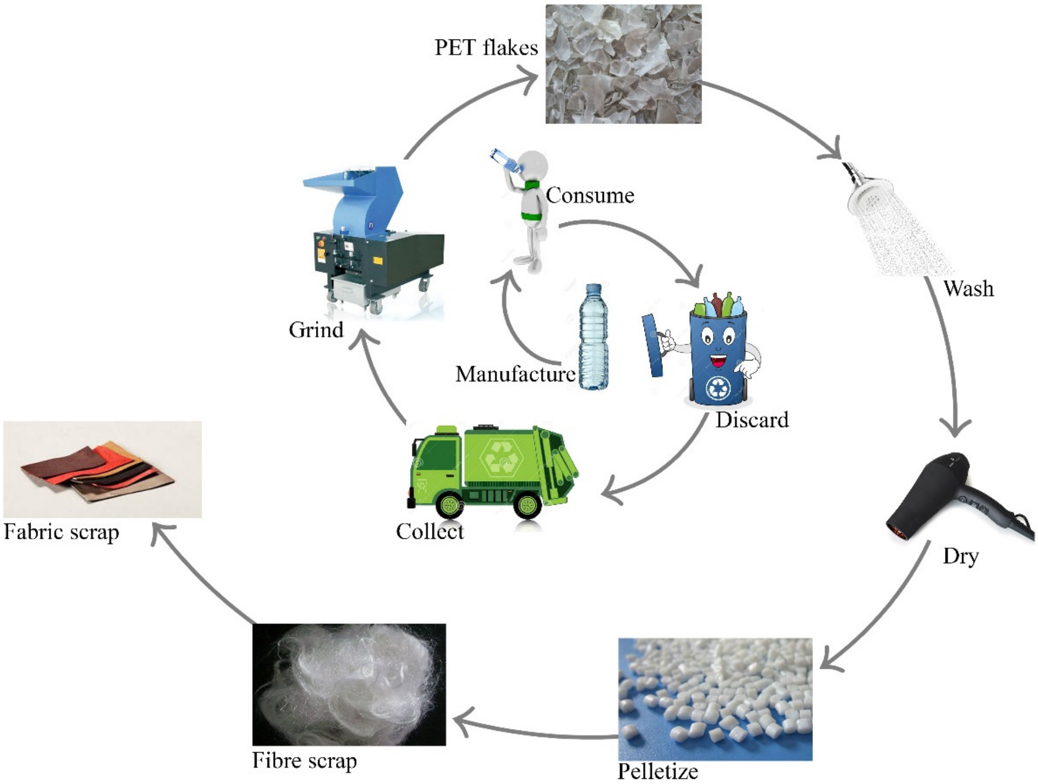


Figure 2.37: Stages in the recycling process

In South Africa, the responsibility to self-regulate post-consumer PET recycling is managed by PETCO, a company which was incorporated in 2004 to represent the South African PET plastic industry. Through financing from a voluntary recycling levy paid by the industry when purchasing PET resin, as well as grants from brand owners and resin producers, PETCO invests in the collection and recycling of post-consumer PET bottles in South Africa. In so doing, the environmental objective of recycling PET is met while simultaneously contributing to the local economic and social development. In a media release in May 2018, PETCO claimed that 93 235 tonnes of used PET bottles were collected in 2017, which was equivalent to 5.9 million PET bottles being collected across the country per day (PETCO, 2018a). Out of these, 2.15 billion bottles were recycled which corresponded to a post-consumer recycling rate of 65 % for that year. Besides the re-use of resources, this act additionally generated 64 000 income opportunities in the form of waste pickers, collectors and recyclers. Landfills also benefitted from such high recycling rates; 578 000 m^3 of landfill space was spared from storing these waste bottles. From an environmental point of view, a saving of 139 000 tonnes of carbon was attained in the process. In fact, since 2004, a saving in carbon which exceeds 900 000 tonnes was reported in this media release. This was achievable through the recycling of 609 306 tonnes of PET bottles, for which contracted recyclers paid R2.3 billion to collectors for baled bottles. Consequently, a cut-back of 4 million m^3 in landfill space was accomplished, thus allowing storage capacity for other waste materials. Figure 2.38 highlights some important and relevant facts with regards to the recycling of PET bottles.

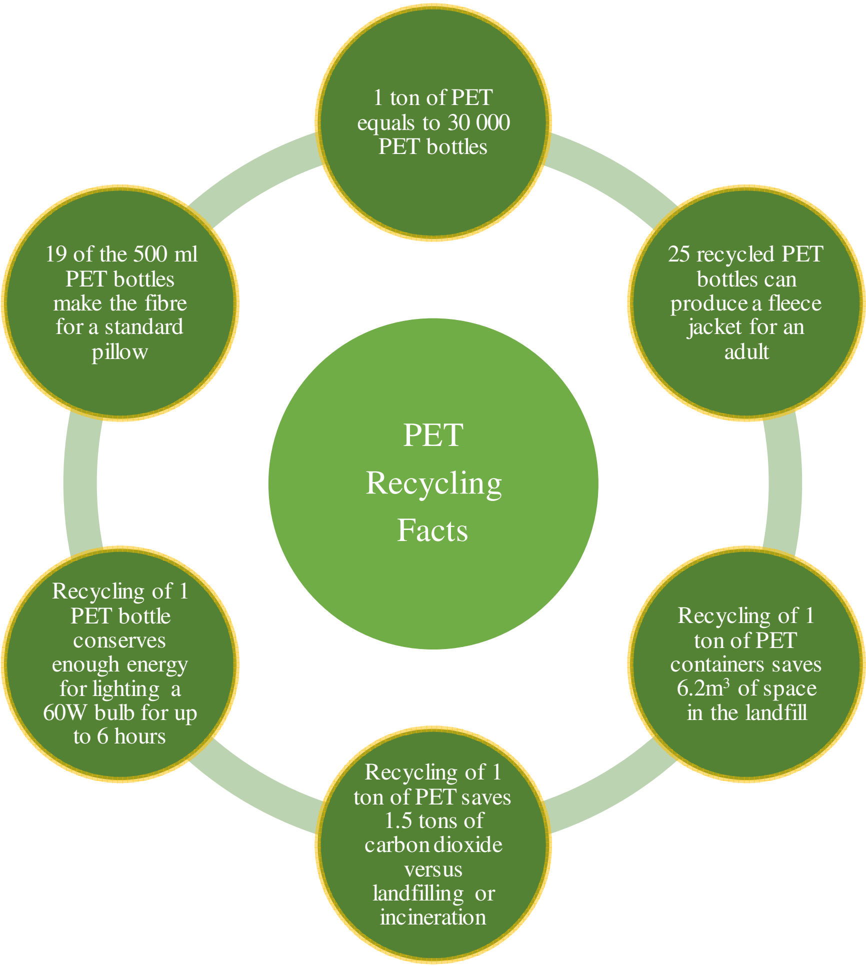


Figure 2.38: Some facts about PET recycling in South Africa (Information sourced from PETCO, 2018b)

PET bottles on the market are normally available in different colours. However, PETCO (2018c) has established some guidelines, with regards to the design of PET plastic packaging in South Africa, to facilitate recycling of waste generated from these sources. Concurrently, this approach is also expected to aid in sustainable production and consumption. One of the design requirements explicitly focusses on the colour of the PET packaging. According to PETCO (2018c), the economic value of coloured plastics is much lower than that of non-pigmented ones. As such, they encouraged the use of alternatives, such as sleeves, to incorporate any visual effect of colour. Direct printing on the PET material is also not advisable since it may interfere with the automatic operation of the sorting equipment.

2.4.2 Soil reinforcement with plastic waste

2.4.2.1 Theory of soil reinforcement

Soil reinforcement dates back to ancient times as a basic principle existing in nature through the activities of animals, birds and plants (Purushothama, 2005). However, in the mid-1960s, French architect and engineer Henri Vidal invented a modern form of soil reinforcement which he termed reinforced earth (Saran, 2010). This technique basically combines soil with some inclusions to form a composite mass which can be easily assembled as a homogeneous material having higher resistance and lower deformability than the unreinforced soil (Schlosser & Bastick, 1991). Schlosser & Delage (1987) pointed out that the frictional generation between the soil and the reinforcing member defines the basic mechanism of the technology. Figure 2.39 shows the main components of a reinforced earth wall.

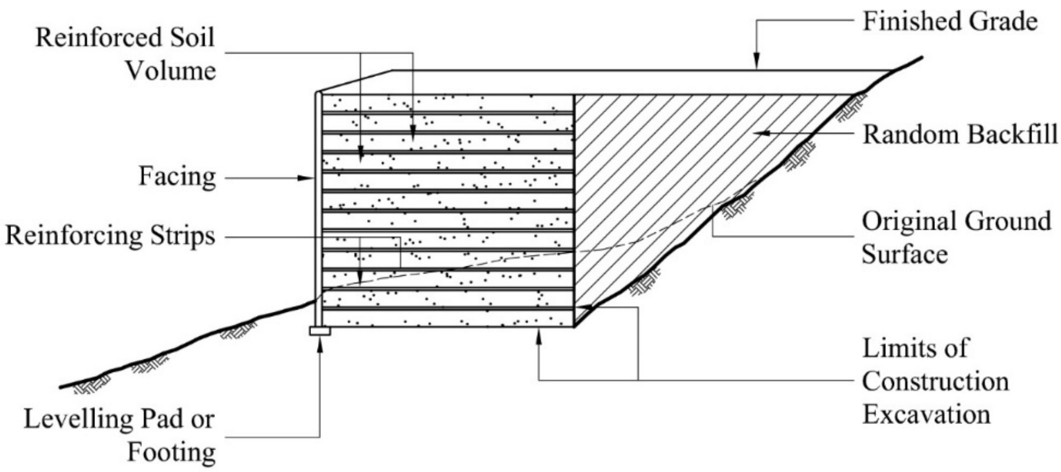


Figure 2.39: Principle elements of reinforced wall (adapted from Christopher et al., 1989)

According to Jones (1988), reinforced soils are most probably dominated by the presence of a compressive stress field, especially in non-cohesive soils. Therefore, in contrast with reinforced concrete where the reinforcement carry the tensile forces, these members in soil tend to carry the anisotropic reduction or suppression of one normal strain rate. This mechanism has been explained by Vidal (1966) whereby individual soil particles are tied together by the reinforcement thereby yielding a form of pseudo-cohesion. Saran (2010) explains the basic concept of soil reinforcement through the Rankine state or stress theory. When a two-dimensional non-cohesive soil block is subjected to biaxial stresses, it undergoes uniform compression. However, if one of the stresses was to increase, the block of soil will critically deform until failure is reached. If the soil was to be reinforced, frictional forces would have been generated along the soil-reinforcement interface which in turn would have created some tensile stresses in the reinforcing member while simultaneously producing a compression in the soil block to maintain equilibrium. This behaviour sustains as long as no slippage of the reinforcement occurs. From the additional lateral earth pressure which holds equilibrium, the Mohr's circle is shifted to the right and thus away from the failure envelope.

Reinforced soil performance is governed by several factors such as (Jones, 1988):

- reinforcement - size, form, strength, stiffness;
- soil - particle size, mineral content, index properties;
- soil state – density, overburden, state of stress, degree of saturation; and
- construction – structure geometry, degree of compaction, construction method, durability.

The type of reinforcement used is not limited to any specific material although Vidal's reinforced earth made use of metal exclusively (Christopher et al., 1989). Purushothama (2005) highlighted some materials which can be used for reinforcing soil from an engineering perspective. These are found more commonly in the form of strips, grids, anchors and sheet material, chain, planks, rope, vegetation, steel, concrete, glass, fibre, wood, rubber, aluminium and thermoplastics. Bonaparte & Schmertmann (1987) noted that reinforcing members can be classified as either extensible or inextensible. The description of these two types of reinforcement was later extended by Bonaparte, Holtz & Giroud (1987) who defined each one of them as follows:

- Inextensible reinforcement: "*reinforcement used in such a way that the tensile strain in the reinforcement is significantly less than the horizontal extension required to develop and active plastic state in the soil. An "absolutely" inextensible reinforcement is so stiff that equilibrium is achieved at virtually zero horizontal extension (K_o conditions prevail).*"
- Extensible reinforcement: "*reinforcement used in such a way that the tensile strain in the reinforcement is equal to or larger than the horizontal extension required to develop*

an active plastic state in the soil. An "absolutely" extensible reinforcement has such a low modulus that virtually no tensile forces are introduced to the soil mass at the strain required to develop an active plastic state (K_a conditions theoretically prevail)."

Figure 2.40 provides an understanding of the degree of extension of a few materials, with steel being the less extensible compared to nylon which exhibits a contrasting behavior. Soil reinforcement can be beneficial when appropriately applied. Some of the benefits are as follows (Mirafi, 2010):

- improved structural capability through a better shear resistance;
- minimal land acquisition as a result of the steepness of walls;
- lower time for construction;
- enhanced soil properties resulting in the soil to be used as a structural component.

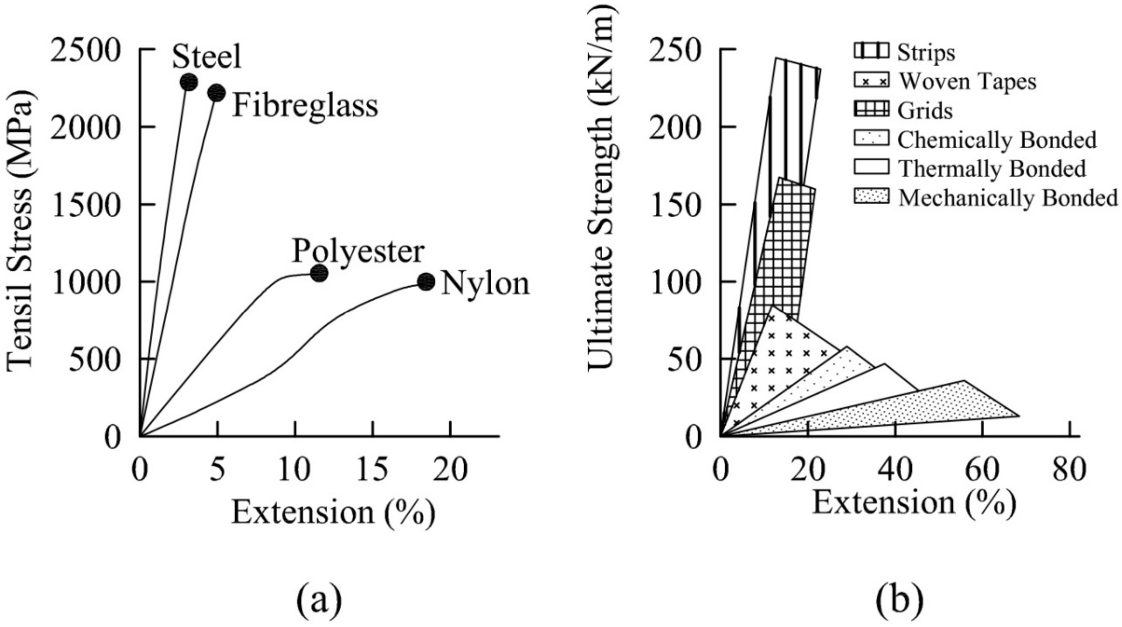


Figure 2.40: Stress – strain relationship of (a) Different material fibers (Schlosser & Delage, 1987) and (b) Geosynthetic products (John, 1987) (adapted from Pokharel, 1995)

2.4.2.2 Recent studies on plastic waste as a soil reinforcement material

Although soil reinforcement through inclusions is an ancient technique, it is not until the last 3 to 4 decades that it gained its popularity (Zornberg et al., 2004). Today, ongoing research are undertaking different types of investigations to reveal potential soil reinforcement materials. Out of the large variety of materials which have been studied and applied, geosynthetics are possibly the most common soil reinforcement material used in present time.

Apart from geosynthetics, waste products such as tyres and plastics have progressively become common in the civil engineering research field (Benson & Khire, 1993; Edil & Bosscher, 1994; Zornberg et al., 2004 and Bhattarai et al., 2013). The need to use waste as an alternative product in engineering has mainly arisen due to the environmental concerns about the high energy use and carbon emissions associated with the manufacturing of new products and their disposal after single use, in addition to the high project costs associated with the use of virgin materials. Moreover, valuable resources (originating from petroleum) which are used to manufacture these products are then dumped in landfills, without any further usage. Besides the need to preserve the currently available resources, constraints are also faced with regards to the storage capacity of landfills (CSIR, 2011). Therefore, placement of an improved solid waste management system across the globe, which caters for the elevated consumer use of such products on the market, contributes to the minimisation of dumped waste while making efficient use of the prevailing resources. Although such strategies have been implemented in several countries, there is still a need to further improve solid waste management, irrespective of the type of waste.

Out of the generated wastes, plastic appear to have gained the interest of geotechnical engineering investigators, who believe that this material made from a non-renewable resource can be utilized in the construction industry. Table 2.13 provides a short compilation of selected studies, within the geotechnical research area, whereby results have repeatedly indicated plastic as a potential soil reinforcement material.

From Table 2.13, it is noted that the type of plastic used varied from one study to another, owing to the wide range available on the market. Nevertheless, PET bottles and HDPE shopping bags appear to be rather popular in this area of research. They have typically been used in the form of strips of different lengths and widths, and hence varying the aspect ratios of the strips.

In general, plastic reinforcement have been more commonly employed to improve the properties of sand (Benson & Khire, 1993; Consoli et al., 2002; Choudhary et al., 2010; Sobhee-Beetul & Kalumba, 2011; Chebet, Kalumba & Avutia, 2012; Dave & Thaker, 2017). However, their use in clays and silts, normally of inorganic nature, have also been explored

(Dutta & Sarda, 2007; Sivakumar et al., 2010; Neopaney, Wangchuk & Tenzin, 2012; Bhattarai et al., 2013; Laskar & Pal, 2013; Das et al., 2017). Some of the reasons for using plastics as reinforcement were mainly to achieve the following in the soil: (1) increase in the bearing capacity of the soil, (2) improvement in the shear strength, and (3) reduction of settlement. These were independently achieved in each study by introducing different variables, the most common of which was the element size.

For the majority of the studies, the reinforcement was used in the form of strips which were either cut manually or by means of a machine. The strips were found to differ in size such that the width varied from 1.25 to 18 mm, while the length ranged between 10 and 45 mm. Besides the strip dimensions, the content (percentage by weight) of the plastic was also considered as an important variable, whereby the minimum concentration used was 0.1 % (Sobhee-Beetul & Kalumba, 2011; Chebet, Kalumba & Avutia, 2012) while the maximum reached 4 % (Benson & Khire, 1993; Dutta & Sarda, 2007; Choudhary et al.; 2010), although most of the studies did not utilise more than 1 %.

In a remarkable number of cases, when the plastic was introduced to the soil, it was randomly mixed to prepare the sample for testing. However, Dave & Thaker (2017) did not use plastic strips, but rather used the collected waste PET bottles to produce a new geogrid whose performance was then compared with a conventional geogrid. In this study, random mixing was not applicable due to the size of the geogrid. Therefore, they placed the reinforcement in 4 different layers within the soil.

With regards to the testing approaches, a distinct variation is evident. While many investigators opted to perform the CBR test (Benson & Khire, 1993; Dutta & Sarda, 2007; Choudhary et al., 2010), others rather undertook tests such as the direct shear test (Benson & Khire, 1993; Sobhee-Beetul & Kalumba, 2011), the triaxial test (Sivakumar et al., 2010), unconfined compression tests (Consoli et al., 2002), consolidation tests (Sivakumar et al., 2010; Laskar & Pal, 2013) and plate load tests (Dave & Thaker, 2017). In fact, the differing choices indicate the intensiveness of work conducted in this area of research.

The results obtained continuously demonstrated the effectiveness of plastic as a soil reinforcement material. While a gain in shear strength and bearing capacity was generally achieved in the studies, other beneficial influences were also noted, such as a decrease in the settlement ratio, a delay in the failure of the reinforced material, an increase in the coefficient of consolidation, and an enhancement in the CBR value. The most common optimum length and concentration of the plastic strips used was reported as being 30 mm and less than or equal to 1 % (0.1, 0.25, 0.5 and 1 %), respectively.

Table 2.13: Characteristics of selected studies on soil reinforcement using plastic waste

Authors	Base soil type	Plastic waste material	Variables	Tests conducted	Outcome
Benson & Khire (1993)	Uniformly graded medium sand	HDPE strips from waste milk jugs	• Strip shape (rectangular-plain, rectangular-punched and kinked) • Strip content in % (0, 1.0, 2.0, 3.0 and 4.0)	• CBR tests • Direct shear tests	• Increase in CBR (maximum factor of 5), secant modulus, resilient modulus (35%) and shear strength of sand
Consoli et al. (2002)	Uniformly graded fine sand	PET fiber from recycling waste plastic bottles	• Plastic content (up to 0.9%) • Length of plastic (up to 36 mm) • Cement content (up to 7%)	• Unconfined compression tests • Splitting tensile tests • Saturated drained triaxial compression tests	• Improvement of peak and ultimate strength of both cemented and uncemented sand • Reduction in brittleness of cemented sand • No significant change in original stiffness of reinforced sand
Dutta & Sarda (2007)	• Stone dust (predominantly sand) • Fly ash (predominantly silt and clay • Kaolinite clay	HDPE strips from plastic waste collected from a dumpsite	• Strip content in % (0, 0.25, 0.5, 1.0, 2.0 and 4.0) • Strip length in mm (12, 24 and 36)	• CBR tests	• Increase in CBR and secant modulus • Optimum plastic concentration of 2% • Reinforced stone dust more effective than reinforced fly ash
Choudhary et al. (2010)	Poorly graded sand	HDPE strips from plastic waste collected from a dumpsite	• Strip content in % (0, 0.25, 0.5, 1.0, 2.0 and 4.0) • Strip length in mm (12, 24 and 36)	• CBR tests	• Increase in CBR and secant modulus • Optimum plastic concentration and aspect ratio are 4% and 3 respectively • Enhanced reinforcement with increase in strip content and length
Sivakumar et al. (2010)	Silty clay	PET strips from waste plastic water bottles	• Strip content in % (0, 0.5, 0.75 and 1.0) • Confining pressure in kPa (50 and 100)	• Triaxial compression tests • One-dimensional consolidation tests	• Improvement in strength of soil • Generation of a constitutive model agrees satisfactorily with experimental results
Sobhee-Beetul & Kalumba (2011)	Two sand types tested independently	HDPE perforated strips of waste plastic grocery bags	• Strip length in mm (15, 30 and 45) • Perforation diameter in mm (0, 1 and 2) • Strip concentration in % (0, 0.1, 0.2 and 0.3)	• Small direct shear tests	• Improved shear strength • Optimum length, concentration and perforation diameter of 30 mm, 0.1 % and 2 mm respectively

Table 2.13: Characteristics of selected studies on soil reinforcement using plastic waste (Continued)

Authors	Base soil type	Plastic waste material	Variables	Tests conducted	Outcome
Chebet, Kalumba & Avutia (2012)	Medium dense sand	HDPE strips of waste plastic grocery bags	• Strip width in mm (6, 12 and 18) • Strip length in mm (15, 30 and 45) • Strip concentration in % (0, 0.1, 0.2 and 0.3)	• Bearing capacity using a compression machine	• Enhanced bearing capacity of sand • Decrease in settlement ratio • Failure of the composite delayed
Neopaney et al. (2012)	Inorganic silt of high plasticity or organic clay of high plasticity	HDPE strips of waste shopping plastic bags	• Strip length in mm (10, 20, 30 and 40) • Strip concentration in % (0, 0.25, 0.5 and 1)	• CBR tests	• Improved CBR • Optimum results at respective aspect ratio and plastic content of 3 and 0.5 %
Bhattarai et al. (2013)	Inorganic silt of high plasticity or organic clay of medium plasticity	Plastic waste strips including shopping bags	• Strip length in mm (10, 20, 30 and 40) • Strip concentration in % (0, 0.25, 0.5 and 1)	• CBR tests	• Enhanced strength of soil • Optimum length and concentration of strip as 30 mm and 0.5 % respectively
Laskar & Pal (2013)	Silty sand with clay	Waste plastic bottle strips	• Strip width in mm (1.25, 2.5 and 5) • Plastic concentration in % (0, 0.25, 0.5 and 1.0)	• Consolidation tests	• Increase coefficient of consolidation with higher plastic contents • 90 % of total compression occurred within 96s when the reinforced soil (plastic of aspect ratio 8 and content 1 %) was loaded at 800 kN/m^2
Dave & Thaker (2017)	Sand	PET bottle wastes which are used to manufacture geogrids	• 4 layers of geogrids placed horizontally in a test pit of size 2100 x 1050 x 900 mm • 2 types of geogrids used: conventional geogrid and the one derived from the bottles	• Plate load tests	• Ultimate bearing capacity of 254, 310 and 292 kPa for unreinforced soil, soil reinforced with the conventional geogrid and soil reinforced with the geogrid which was produced from PET bottle waste • Geogrid made from the PET bottle waste generated a 60 % cost saving when used compared to the conventional one.
Das et al. (2017)	Inorganic silt of low plasticity	Polythene bag wastes	• Plastic fibre content (0.25 and 0.5 %) • Aspect ratio of the fibres (0, 1, 2, 3 and 4)	• Vertical compression loading	• Improved loading strength of soil post reinforcement • Optimum aspect ratio of 3 both fibre concentration of 0.25 and 0.5 %

2.5 Summary of the literature review, key gaps in existing knowledge and new contributions

2.5.1 Granular columns

From the literature review which covered both aspects related to granular columns and soil reinforcement using plastics, it is deduced that the granular column technique can be more beneficial compared to other ground improvement methods, provided that it is used to improve grounds for supporting the foundation of low capacity structures such as low-rise buildings, storage tanks and embankments. The aims of ground improvement through this technology are mainly to achieve the following: increase in bearing capacity, settlement reduction, liquefaction mitigation and enhanced drainage. From literature, different types of granular columns have been identified although the method of installation is typically of 2 modes: vibration or ramming. A secondary important aspect of their installation is the displacement or the replacement of materials to form the columns.

Granular columns, which are made of coarse materials like stone and sand, are normally used to improve weak soil strata comprising of soft clays, peat, cohesive deposits and silts. This chapter presented a compilation of experimental work (laboratory and field studies) relating to granular columns. From the review, it appears that both single columns and groups of columns have been researched. Irrespective of the number of columns, the unit cell concept has generally been adopted whereby any one column is assumed to have an effective tributary area with regards to the improvement of the surrounding soil.

Two types of granular columns were identified: ordinary granular columns (OGC) and reinforced granular columns (RGC). While OGC refers to conventional granular columns, RGC are actually a result of progressive research in this area of study. Basically, a reinforcement material is introduced in the column such that a composite mass is achieved, with the aim of improving the load carrying capacities and settlement reduction properties of the columns.

From the intense laboratory studies, it was evident that there was no standard procedure and equipment for the testing of granular columns, irrespective of the type of column being installed (OGC or RGC). Tests performed in the laboratory were mainly conducted in fabricated tanks, mostly circular, with diameters ranging between 100 and 835 mm, while their heights varied between 200 and 720 mm. Out of these large dimension ranges, a diameter and height of approximately 300 and 500 mm, respectively, were more common. On the other hand, column diameters were generally much smaller (between 12.5 and 100 mm) than the tank, whereby 100 mm diameter columns were more frequently used in the experiments. The heights of columns which have been investigated were generally found to be between 150 and 540 mm. However, the numerous studies reviewed indicated that tests were more often conducted on columns with heights of approximately 250 or 500 mm. These dimensions of the tanks and columns have consequently produced area replacement ratios between 10 and 30 %. With regards to the base soil, clay has been predominantly used, especially in the forms of kaolin and marine clay. These were often prepared by mixing with water at the desired moisture content, which was as high as 1.4 times the liquid limit in certain studies. Usually, a much coarser material (granite stone chips, crushed aggregates or sand) was used to produce the columns.

For laboratory research, the size of the particles of the column material typically ranged between 1 and 10 mm. For reinforced granular columns, the reinforcement was introduced internally or externally. Internal reinforcement was either done by randomly mixing (fibres such as polypropylene at a concentration of 0.2 %) the reinforcement or by horizontally placing them in layers (perforated metal discs, geogrids, geotextile sheets and meshes made from plastic, steel or aluminium) at different intervals along the columns. In contrast, columns which were externally reinforced were basically encased within a material such as PVC tubes, geogrids or geotextiles. Prepared samples were generally subjected to vertical loading tests, through rigid loading plates of diameters equivalent to 1 to 2.3 times the column diameter, which were most often displacement controlled. Test rates between 0.003 and 0.0625 mm/min were adopted for drained test conditions. However, in cases where undrained conditions were required, the selected test speeds were much higher and ranged from 1 to 1.2 mm/min. In general, tests were run up to a maximum settlement of 50 mm.

Comparison of the test results obtained from the different column materials demonstrated better load deformation characteristics with stones. In fact, in one of the studies, aggregates were partially replaced by sand, which resulted in a 30 % saving of the coarser material. Consequently, sand replacement was proposed as a more economical option for that particular study. From these laboratory studies, it was revealed that the inclusion of any type of column produced an improvement in the settlement reduction and in the load carrying capacity, although the lateral support provided by the surrounding soil governed the ultimate strength of the column. Stress concentration ratios (n) generally varied from 0.5 to 5 in most studies, while the settlement reduction ratios (SRR) ranged between 0 and 0.8. Individual column within groups were also generally stronger, in terms of the load carrying capacity, than single ones due to the confinement provided by surrounding columns within the group. When columns were loaded, they were vertically compressed and, therefore, deformed laterally. The bulging was measured, and the failure mechanism of the columns were recorded. One of the studies concluded that failure occurred in bulging for longer columns while punching was more prominent in shorter columns. The maximum bulge was often measured in terms of the diameter of the column and the height along the column. Often, results showed that the maximum deformation occurred within the top third of the column. However, in one of the investigations, the findings confirmed the maximum bulging (133 %) to occur at a depth of 0.5 times the column diameter.

Reinforcing of the columns have continuously shown an amelioration in their load carrying capacities. However, encasement has particularly demonstrated much higher improvements with regards to the load carrying capacity, with increases between 3 to 5 times that of OGC. Bulging of columns was also reduced when reinforced. In the case of encasement, stiffer ones provided further reduction in bulging. Also, when fully encased, better performance was achieved with end-bearing columns compared to floating ones.

Since the method of investigation was primarily based on laboratory experiments in this research, studies with a similar approach were mostly reviewed. However, selected field works were briefly presented in the literature review to understand how the columns performed under full scale conditions. In these studies, column diameters and heights were found to be in the range of 0.8 to 0.9 m and 14 to 16 m, respectively. The columns were typically OGC and were mostly installed in very soft cohesive soils and soft marine clays, with their particle sizes varying from 20 to 63 mm. Trials were often conducted before the actual execution of ground improvement using granular columns. Plate load tests were commonly performed, although hydrotest was also used. The results demonstrated a general improvement with the inclusion of columns. For example, an increase of up to 2.5 times the original bearing capacity of the ground was noted, after the treatment of the base soil. Settlement reduction was a further benefit, and the recorded enhancement varied between 20 and 49 %.

2.5.2 Soil reinforcement using plastic waste

The constitution of South Africa has highlighted the right for an individual to have access to an environment which is not harmful to their health or well-being. As such, the importance of necessary measures is emphasised so as to provide protection to the environment. The National Environmental Management Waste Act put forward a series of objectives to achieve these conditions. Out of this list, the need for reducing, re-using, recycling and recovering of waste has been clearly specified. It is further stated that ecologically sustainable development must be encouraged, while promoting economic and social development. In general, the minimisation of waste disposal to landfill is ideally preferred. With regards to PET, this signifies a saving in landfill space since the volume of PET disposed is typically high, if re-using and recycling is not practiced. To intensify the problem, PET is not easily degradable, especially in landfills, since there is a lack of UV light and also due to the low content of oxygen in such environments. In South Africa, the rate of recycling of post-consumer PET bottles was calculated as 65 % in 2017. Nevertheless, some bottles are still being disposed of in landfills. Besides, the government is also encouraging new technologies for dealing with such waste so as to achieve practically no plastic waste to landfills by the year 2030.

Soil reinforcement was identified as an ancient technique, where roots of plants were traditionally the main form of reinforcing material. However, new materials (metals and synthetic polymers) have been developed over time, and successfully applied as reinforcements in ground engineering. Further research has explored the possibility of using waste materials for this purpose. Very often, these materials have demonstrated good performance with regards to enhancing the engineering properties of weak soils to increase the bearing capacity, improve the shear strength and reduce the settlement of the soil.

Through the review of selected literature (on plastic wastes), it was revealed that plastic waste was frequently employed in the form of polyethylene terephthalate (PET) or high density polyethylene (HDPE). Although PET and HDPE were mostly used in the form of bottles and shopping bags, respectively, milk jug waste has also been studied in the reinforcement of soils. Generally, these plastics were incorporated in sands. However, their inclusion in inorganic silts and clays have also been noted. Despite their popularity in the form of rectangular strips, fibres have also been utilised, in addition to the waste PET bottles which were converted to geogrids. The length of the plastic strips or fibres varied between 10 and 45 mm, while the width ranged from 1.25 to 18 mm. Besides the size, their content by weight was also important. Most of the studies utilised concentrations lower than 1 %, although a range between 0.1 and 4 % was noted. The relevant studies presented in the literature review were typically inclined towards laboratory testing, whereby tests like the CBR, direct shear, triaxial, unconfined compression, consolidation and the plate load were conducted. The key findings from the tests revealed gains in shear strength and bearing capacity of the soils. Furthermore, failure of reinforced materials was found to be delayed. In general, the most common optimum length of the plastic strips was 30 mm, while the concentration was less than or equal to 1 %.

2.5.3 Environmental benefits of the proposed technology

The technology proposed through this research is a combination of the granular column technique with soil reinforcement using PET waste bottles. Conventional granular columns are already well-known for their relatively low level of harm to the environment, when compared to other construction methods which may be used to achieve the same results. This is primarily because they use stones or sands which are natural materials. Although, mining of these materials is fairly energy intensive, the method is still less damaging to the environment compared to technologies which use cement, a man-made product. Besides, the energy required in the manufacturing of cements, the high carbon dioxide emissions are the greatest concern; this is significantly low compared to mining of the materials required for granular columns.

In parallel, the re-use or recycling of waste PET bottles also have a high impact on the level of carbon dioxide emissions. PET is manufactured from oil, coal and natural gas which are non-renewable resources. Because of their resistance to degradation, dumping of PET waste implies that the material will need to be stored in the landfill space for many years, possibly decades, before they are degraded. Consequently, a valuable resource is lost in the form of waste, while the potential for re-using or renewing is nowadays high. When new materials are produced from these wastes, carbon dioxide emissions still occur. However, the amount is relatively low compared to processing raw materials and manufacturing the new products. Therefore, by making use of these waste in the construction industry, a 'greener' soil reinforcement material may be generated.

2.5.4 Key gaps in current literature and new contributions

2.5.4.1 Key gaps

Ordinary granular columns have intensely been researched and applied in the field for few decades, across the globe, although the technology is not so popular in South Africa. However, reinforced granular column is a relatively new concept. From the literature, it is evident that encased columns (external reinforcement) are becoming more popular, especially when geosynthetics are used. Such columns have also demonstrated significant improvements in load carrying capacity and settlement reduction. However, internal reinforcement appears to be a recently emerging method. In fact, limited studies have been found in connection with this approach whereby all of them were laboratory based.

While researchers across the globe have used different types of materials for internally reinforcing the granular columns, none of them has investigated the use of waste for this purpose, especially in the form of plastics which are highly abundant, and a nuisance, in many cities worldwide, including South African cities. This research was, therefore, undertaken to fill in this gap in existing literature pertaining to reinforced granular columns.

2.5.4.2 Contributions

From the previous sections, it is evident that there is a need to further study the internal reinforcement of granular columns since limited work had been covered in this area of research. In parallel, it is also necessary to explore new technologies of re-using plastic wastes in South Africa. Therefore, this study proposed the incorporation of waste PET bottle, as a reinforcement material, within the column to produce a plastic waste RGC. PET bottle wastes were selected based on their abundance in the local context.

Due to the originality of the concept, it was necessary to firstly identify the forms in which the waste was available. The recycling cycle (as shown in Figure 2.37) served as a starting guide to identify the possible available forms on the market. Based on this, 3 forms of waste PET bottle were chosen namely: PET flakes, PET fibres and a geotextile made from PET bottle waste. A second type of geotextile, which was produced from virgin PET, was also investigated for comparison purposes of the performance of the RGC. Thus, a total of 4 types of reinforcement was tested.

Besides the materials, laboratory testing was necessary to understand the behaviour of the columns when reinforced with each material. It was important to identify whether these forms of PET may potentially act as reinforcements in RGC before venturing into large scale testing which are typically costly. Therefore, a testing tank was designed and manufactured such that it operated as a unit cell. A local fine silt was used as the base material and it was tested at 2 different moisture contents (optimum moisture content and liquid limit) to observe the column behaviours at different levels of wetness. Single sand columns were installed (both OGC and

RGC) within the wet silt. For RGC, the reinforcements were arranged in 2 forms: random mixing or layering. A displacement controlled vertical loading was applied to the test specimen, up to a maximum settlement of 50 mm. The load-settlement relationships obtained enabled comparison of the performance of each material under the varying conditions. The load-carrying capacity and the settlement were the primary focus of the analysis of results. Additionally, physical modelling of the column deformation was achieved post-testing through a casting process by means of the plaster of Paris. This enabled the determination of the following in the individual tests: failure modes of the columns, maximum bulging, the length span over which largest bulging was prominent and the position corresponding to the length span.

Chapter

3

Research methodology

3.1 Introduction

In the past, several testing approaches have been used in the investigations of the performance of granular columns, in both bench and pilot scale studies. Although the techniques vary distinctively, a more simplified methodology was used in this research which was largely derived from Sobhee-Beetul (2012) and Ambily & Gandhi (2007). The intention behind the testing design was to create a field scenario of a singular column, as closely as possible, and to record observations of stresses under similar settlements for each test while introducing some variables. However, creating a precise laboratory model which satisfied a typical field condition was practically impossible. The addition of the polymeric reinforcement materials, within the granular columns, increased the complexity of establishing a bench mark behaviour for results comparison. Hence, the unit cell concept was adopted as the most simplified approach to obtain basic information regarding stresses and settlement. This allowed for easy analyses of the results which were influenced by the varying characteristics of each experiment.

While the main importance of this study was to look at the possibility of using different types of PET waste as a reinforcement material in these columns, the basic principles and factors behind the behaviour of granular columns had to be inevitably considered. In fact, these constituted the basis of this research, whereby laboratory tests aided in understanding the effect of introducing the reinforcing polymers on both the load-settlement relationship and the column deformation, as a result of loading. Hence, several variables were introduced to look at the subsequent responses. Throughout this research, the same type of base soil and granular column material were utilised to eliminate any possible sources of error arising from the irregularities within the characteristics of the material. However, since the aim was to study the effect of the different quantities and arrangement of the reinforcement within the columns, it was necessary to vary the type of arrangement and the quantities of the PET to understand their effects on the stress-settlement behaviour of the improved ground during testing. As the unit cell concept is rather popular in laboratory studies, the dimension of the testing tank was kept constant to avoid additional variables. Although, the primary focus was centred around the reinforcement, the moisture content of the surrounding material was considered important since it was in direct contact with the composite column mass. Hence, two conditions of moisture content were selected to understand the performance of the columns when installed in fine soils of different degrees of wetness.

This chapter presents the research approach followed to accomplish the aims of this study. A detail of the testing programme, equipment used as well as the experimental procedure used are presented in the following sections.

3.2 Fabrication of testing equipment and accessories

3.2.1 Testing tank supported on trolley

The approaches and equipment used for testing granular columns, under laboratory conditions, have varied from author to author. Hence, for this study, a bespoke circular steel tank was designed and ultimately fabricated in the departmental workshop at the university. The laboratory testing tank used was designed in accordance with the diameters to be investigated, while simultaneously incorporating the theory behind the unit cell concept. The selected dimensions of the container were based on the information obtained from the literature review, as well as on the material availability on the local market. From previous studies, 100 mm diameter columns were more often researched (Ambily & Gandhi, 2007; Murugesan & Rajagopal, 2008; Afshar & Ghazavi, 2014) while the most common column lengths were either around 250 mm (McKelvey et al., 2004; Isaac & Madhavan, 2009; Al-Waily, 2012) or 500 mm (Murugesan & Rajagopal, 2008; Ayadat, Hanna & Hamitouche, 2008; Ali, 2014). Since the general installation technique of the column involves creating a hole in the soil to form the intended diameter of the column, such information from the existing literature, coupled with the available metal pipe sizes on the market, dictated the choice of column diameter to be used. With regards to the length of the columns, Mitra & Chattopadhyay (1999) stated that the length to diameter ratio of the column must be about 4.5 to develop the full limiting axial stress on the column. Ambily & Gandhi (2007) shared a similar opinion and employed a column length of 450 mm in their study, whereby the diameter of the column was 100 mm. However, Hughes & Withers (1974) claimed that this ratio can be even as low as 4. Sobhee-Beetul (2012) reported that a column length of 400 mm was long enough to allow for lateral deformations, up to a maximum settlement of 50 mm, whereby the maximum bulging predominantly occurred within the top third of the column length. Therefore, to satisfy similar conditions as in the previous research works, while also considering workable conditions and availability of resources, the diameter and length of the tank were preferred to be 300 mm and 450 mm respectively. Thereafter, the columns were chosen to be 100 mm in diameter (the diameter was slightly higher than 100 mm and the reason for this is explained later in section 5.5.3), with a length of 400 mm. These parameters were maintained as constants when preparing the test samples for each experiment. From the literature review, it is evident that the general ratio used for the effective diameter of the column to its spacing is 1.05 and 1.13, for triangular and square arrangements, respectively. The ratio used for the design of the tank was, however, 3. This was chosen to allow for enough space around the column for any secondary effects resulting from the deformation of columns. Besides, the dimensions were also in agreement with Greenwood (1970) who claimed that the column spacing to diameter ratio must be between 2.5 and 4 for feasibility purposes. Figure 3.1 illustrates the tank made for the purpose of this work.

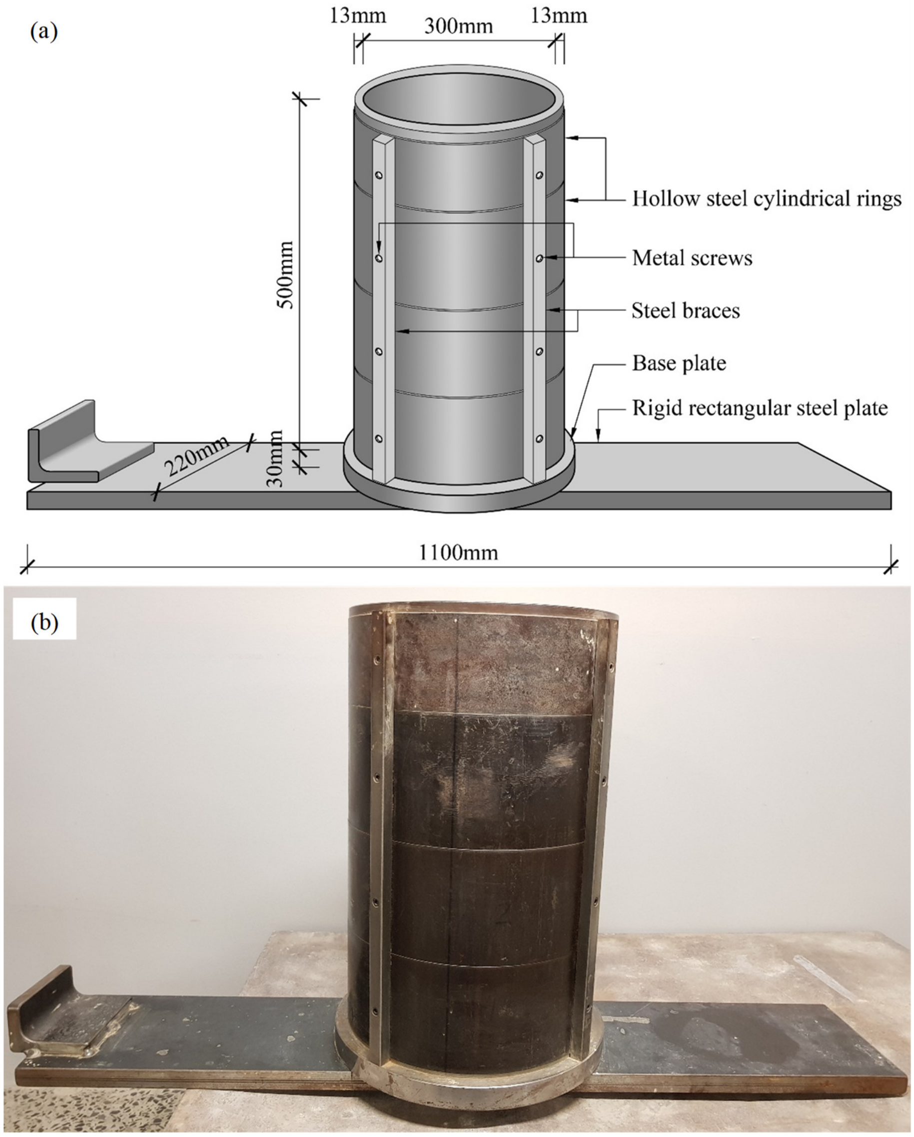


Figure 3.1: Representation of the fabricated tank (a) schematic, (b) pictorial

The tank was made from 4 hollow steel cylindrical rings which were stacked on top of each other by means of an interlocking system. A base plate was then fixed at the bottom of the rings to form a one side open-ended cylindrical vessel. This base was fixed on a rigid horizontal rectangular steel plate to accommodate the tank on the trolley. O-ring grooves were used for this purpose to prevent any leakage of water from the test bed. To restrict any horizontal or vertical movements associated with the rings, 4 vertical steel braces were tightly fitted to them by means of metal screws. The secondary aim behind the design was to create an equipment which could be re-used for any further related investigation. In future, shorter columns could be installed by reducing the number of rings, or contrastively augmenting the number of rings would allow for the execution of taller columns. The concept of re-using formed part of the design since this research emphasised strongly on the re-use of existing products. Practically, it was also beneficial since a small storage space was required post disassembling.

Alongside the container, a trolley was also fabricated such that the heavy load being transferred from the tank could easily be supported, both under stationary or moving conditions. This was necessary since the prepared sample for each experiment was relatively heavy. Additionally, it was important to keep the test sample in an undisturbed state, before and after the tests were run. Hence, an equipment was needed for lifting and placing the container on the testing machine. Figure 3.2 shows the trolley supporting the tank.

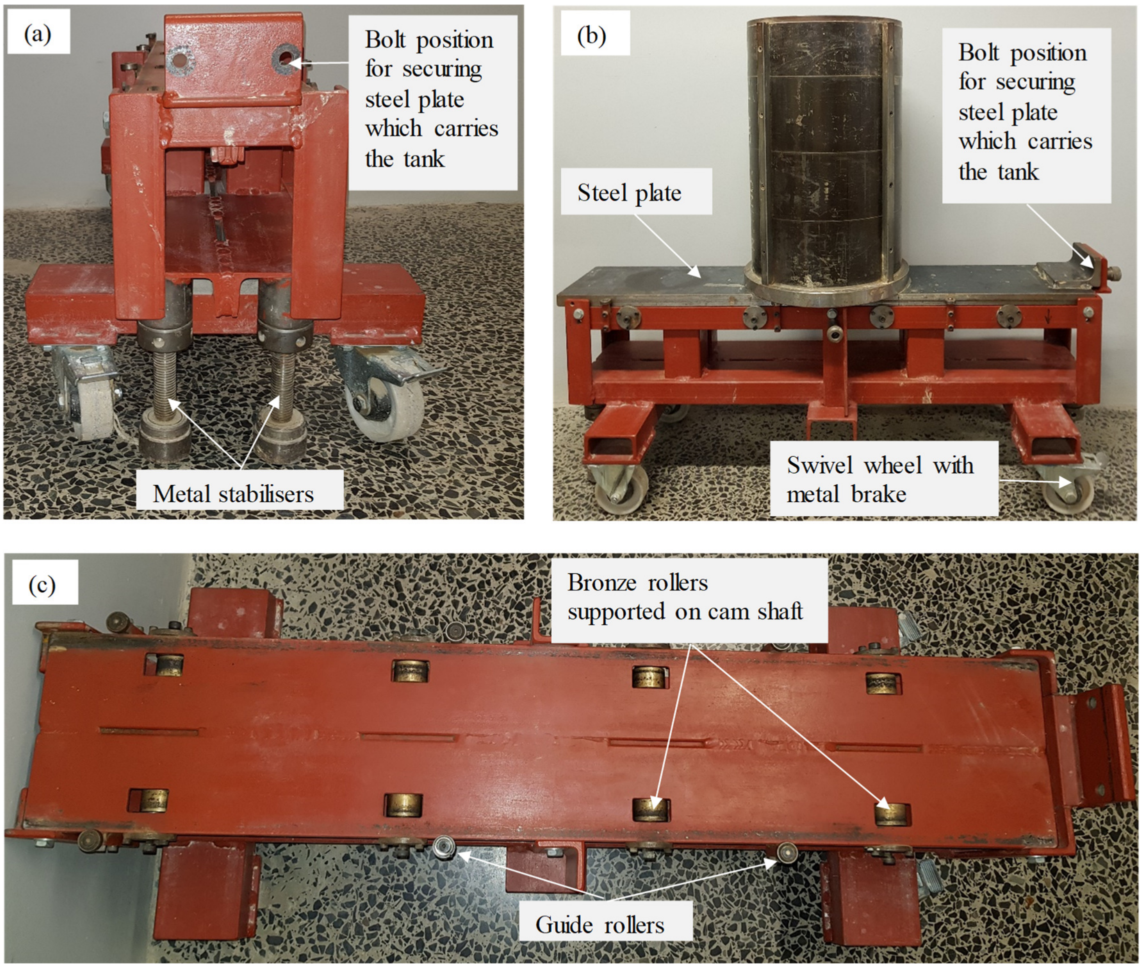


Figure 3.2: (a) Front view of trolley, (b)Testing tank supported on trolley, and (c) Plan view of trolley

The trolley, which was made from mild steel, was mounted on 4 swivel wheels made from nylon. This material was chosen since it has high strength and durability properties, and therefore, was suitable for the required working conditions. Two of these wheels included metal brakes to restrict movement when necessary. A pair of metal stabilisers was further fitted to the equipment to significantly reduce any vibration effect which occurred during compaction when preparing the test sample. On the top horizontal bar, apertures were machined to accommodate 8 bearings whereby these were pushed into bronze rollers and placed on a cam shaft. These enabled raising or lowering of the sample box when transferred to or from the testing machine. To maintain the tank position, 6 guide rollers (3 on each side of the trolley) were fitted along both lengths of the trolley. An extra opening was also created on one of the shorter sides of the trolley such that a securing bolt was used to secure the circular vessel onto it, thus limiting any displacement during the preparatory stages.

3.2.2 Accessories required for test specimen preparation

Alongside the testing tank, several other tools and machines were required to prepare the test samples and to run the experiments. Some of these were also fabricated in the same workshop while others were either purchased from local companies or were already part of the existing facilities within the laboratory. The function of each of them is described in this section, with an individual illustration presented in Figure 3.3.

- Mechanical pan mixer

  This pan mixer, model P25, was manufactured by Pan Mixers South Africa. It had a capacity of 25 litres and was capable of operating at a rpm of 1400/49. The primary use of this equipment was to mix the base soil with water to obtain a homogeneous wet mix of desired moisture content.

- Wooden compaction board

  A 27 mm thick wooden board, made from Balau (a hardwood timber) and sealed with a clear varnish to prevent water migration from the wet soil to the wood, was cut to a diameter of 295 mm such that it was slightly smaller than that of the tank in order for it to just fit horizontally, while almost covering the soil sample. Heavy duty plastic handles were screwed onto it to ease its placing and removal. The purpose of the board was to act as a medium for energy transfer from the drop weight to the base soil, during compaction.

- Hand compactors

  Two types of hand compactors were used in the preparation of the test samples; each weighed 2.3 kg. The first compactor was used for compacting the base material while the second one was used for a similar function, but rather on the column material. The metal weight was fitted on a rod such that moving of the weight along it allowed a smooth flow, when dropped, while maintaining negligible friction between both. Although the weights of both compactors were the same, their diameters differed; the one used in the making of the column had a smaller diameter so as to fit easily into the 100 mm diameter steel pipe.

- Collar

  The collar referred to a short and rigid hollow steel cylinder which was centrally welded to a circular metal frame. The frame, which was designed such that it just fits on top of the tank prior to the column installation in the compacted clay, was secured to the tank by means of screws. This collar ensured that the column was installed in an upright position. Additionally, the screws resisted any movement which could possibly arise from the compaction process of the column material.

- Hollow steel pipe

  A 100 mm diameter rigid hollow steel pipe, with sharp edge at the bottom end, was used to create the opening in the weak soil layer for column installation. This open-ended cylinder was also made in the workshop, with 4 rigid metal handles attached to its top so as to assist with any necessary movement, without causing significant disturbance to the surrounding material.

- Helical auger

  A sharp metal helical auger was fabricated in the university workshop to enable the cutting and removal of clay within the steel pipe manually. The diameter was 95 mm, 5 mm shorter than the hollow steel pipe, to allow for easy retraction of the auger after each cut.

- Loading plate

  A 25 mm thick rigid circular steel loading plate, of diameter (200 mm) twice that of the column, was placed on top of the column to uniformly distribute the compressive load applied to the prepared sample during testing. The loading plate was anticipated to act as a small footing supporting a point load whereby the column installed in the base silt would be singular. The diameter was selected such that an area larger than the diameter of the column was loaded for better strength performance of the latter. However, it was kept smaller than the tank to minimise friction between the loading plate and the surface of the tank. In addition, the smaller diameter of the loading plate, compared to that of the testing tank, allowed for its easy removal post testing.

- Spirit level:

  An engineer's spirit level was utilised to ensure that the loading plate was levelled to avoid eccentricity effects during loading.

- Zwick Universal Testing Machine

  This is an electronic device in the laboratory which was used to automatically apply the compressive loads during testing. The machine was connected to a computer whereby all the test properties were manually entered, and the experimental data were recorded during testing.

- Industrial vacuum cleaner

  An industrial vacuum cleaner, manufactured by Turner Morris in South Africa, was utilised to draw out the column material for post testing investigation purposes. The equipment of model AFM0C-04 consisted of 2 motors of 1200 Watts each. The use of a single motor provided lower suction power compared to if both motors were operated simultaneously. This was important to control the pressure which was needed for removal of the column material without disturbing the surrounding base soil. For all the tests, only the lower suction option was used.

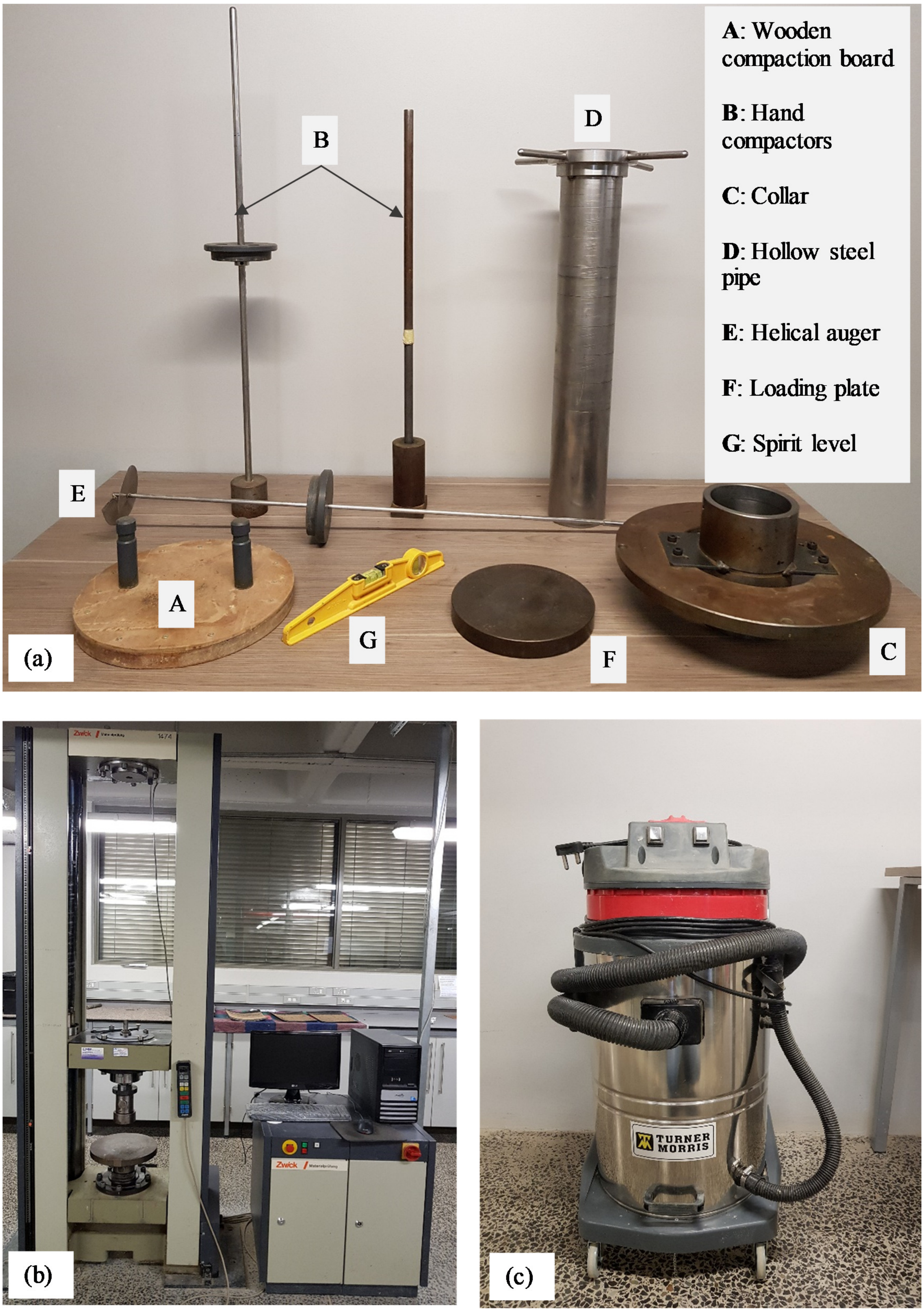


Figure 3.3: (a) Accessories used in test sample preparation, (b) The Zwick Universal Testing Machine and (c) Industrial vacuum cleaner

3.3 Testing programme and variables

Although several factors influence the performance of the granular column improved ground, selected ones were considered for this research. In the literature review, an in-depth description of the effect of each of these factors was presented. Since the concept of waste reinforced granular column involved several unknowns, different ways of incorporating the waste material within the column, as well as the form in which these wastes exist, was considered significant during test executions. In fact, 2 types of positioning were investigated for the column reinforcement: random mixing (R) and layering (L). Tests were classified under these two categories to ease the comparison of results thereby concluding which composite produced higher loading strengths. Random mixing simply implied that the PET waste was randomly mixed with the column material. On the other hand, layering referred to the placement of PET in layers, at different intervals, within the granular column. PET bottle wastes were used in the form of flakes (P) and fibres (F) to reinforce the columns. Besides these, geotextiles were additionally used as reinforcement in layering tests. These testing arrangements are better illustrated in Figure 3.4.

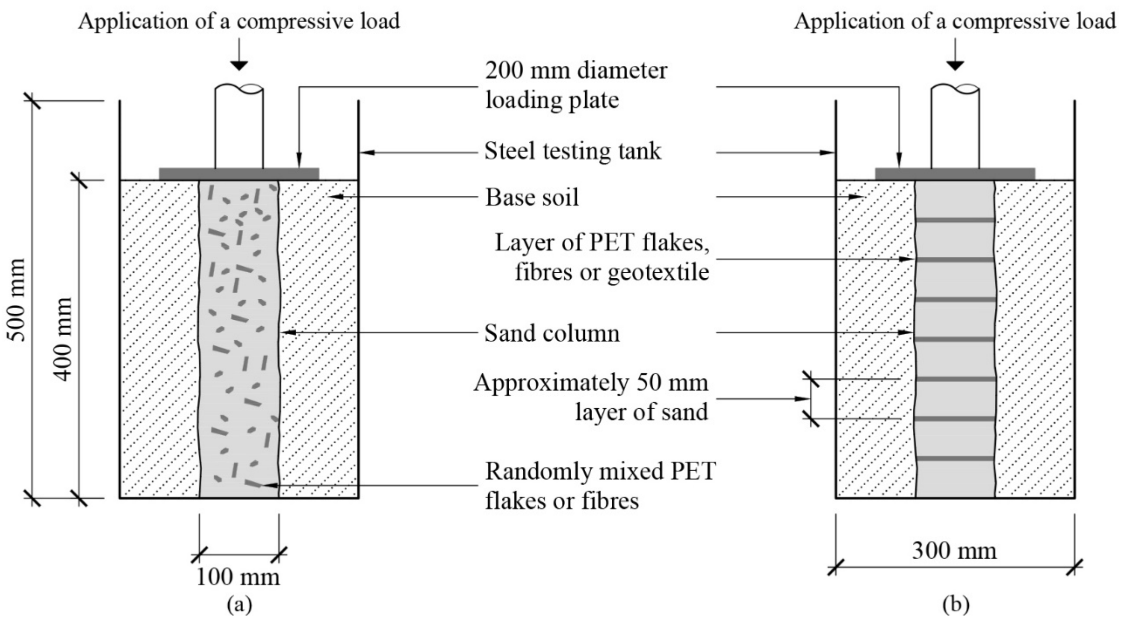


Figure 3.4: Two types of reinforcement arrangement within the granular columns (a) Random mixing and (b) Layering

Two types of geotextiles, manufactured from PET, were utilised; where the raw source of one (GW), referred to as Betatex, was from similar PET bottle waste flakes which was mentioned earlier, while the other was made from virgin PET (GV), known as Fibertex. These different forms of PET were then tested under the respective arrangement, while the other selected variables for this study were introduced. More specifically, the following were varied and each combination was investigated to generate the necessary results for analysis purposes: moisture content of the base material (optimum moisture content or liquid limit), the type of PET used (flakes, fibres, geotextiles manufactured from waste or virgin PET), the mass per unit weight of PET fibres or flakes used, the thickness of the geotextile used and the positioning of the PET (random mixing or layering). Besides these variables, all other parameters in the tests were kept constant such as length of the column, diameter of the column, size of loading plate, base material, granular material for column and rate of applied load during the tests.

Table 3.1 provides a summary of the variables used. With regards to random mixing, the concentration by weight of flakes were based on the range which was investigated in previous studies (Dutta & Sarda, 2007; Choudhary et al., 2010; Neopaney et al., 2012; Bhattarai et al., 2013; Laskar & Pal, 2013). However, in terms of the fibres, such concentrations could not be used since these elements tend to occupy much larger volumes than the flakes even at low weights; hence, the concentration by weight of fibres utilised in the random mixing arrangement was significantly low. In contrast, when these reinforcements were included in layers within the columns, their concentration by weight per each 50 mm layer of sand was determined by trial and error due to the lack of associated existing literature. The lowest content of reinforcement under the layering arrangement was estimated based on the minimum concentration by weight which was needed to just cover the sand surface for any particular layer. Thereafter, the higher contents were derived based on the experimental results obtained and by trial and error. With regards to the geotextiles, the masses per unit areas which are available on the market were studied. Subsequently, 3 masses for each geotextile were selected such that the thickness was not too small to avoid any possible damage during loading. However, it was also ensured that the material was not too thick which could possibly provide an overestimation of the performance of the RGC due to the much higher stiffness. Also, greater mass per unit areas implied thicker geotextiles which could possibly compress, thereby affecting the settlement being recorded for the column during the test.

Table 3.1: Summary of the variables used in this research

Variable	Description	Symbol	Value
Moisture content of base soil	Optimum moisture content (OMC)	M1	17.7 %
	Liquid limit (LL)	M2	37 %
Arrangement of reinforcement	Random mixing	R	-
	Layering	L	-
Reinforcement type	Flakes	P	Random mixing = 0.5, 1.0 and 2.5 % Layering = 2.2, 3.3 and 5.6 % (respective masses per layer are 20, 30 and 50 g)
	Fibres	F	Random mixing = 0.025, 0.05 and 0.1 % Layering = 0.28, 0.56 and 0.83 % (respective masses per layer are 2.5, 5 and 7.5 g)
	Geotextile (Betatex)	GW	Layering = Masses per unit area of 200, 400 and 600 g/m^2
	Geotextile (Fibretex)	GV	Layering = Masses per unit area of 200, 400 and 600 g/m^2

Figures 3.5 and 3.6 provide the codes designated to each experiment conducted, excluding those for the control experiments and for the unreinforced granular columns. In total, 42 laboratory tests were conducted for this research. Out of these, 2 were control experiments (unimproved base soil - tests M1 and M2) while another 2 (improved base soil with an ordinary sand column - tests M1-S and M2-S) were performed on unreinforced granular columns.

Repeatability tests were also performed at the onset of the testing exercise to ensure reproducibility of results from the methodology.

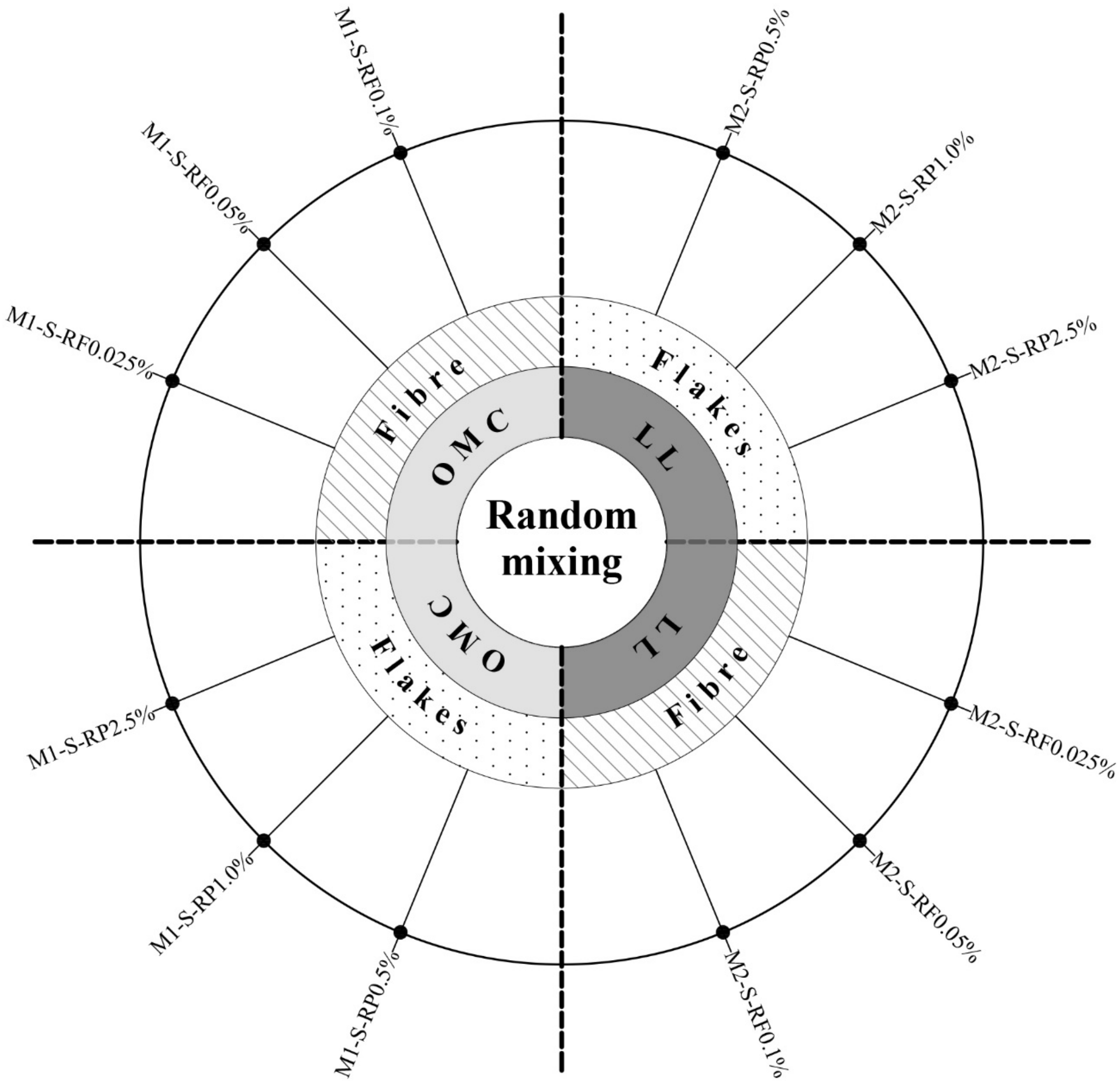


Figure 3.5: Experiments conducted for random distribution of the reinforcement material

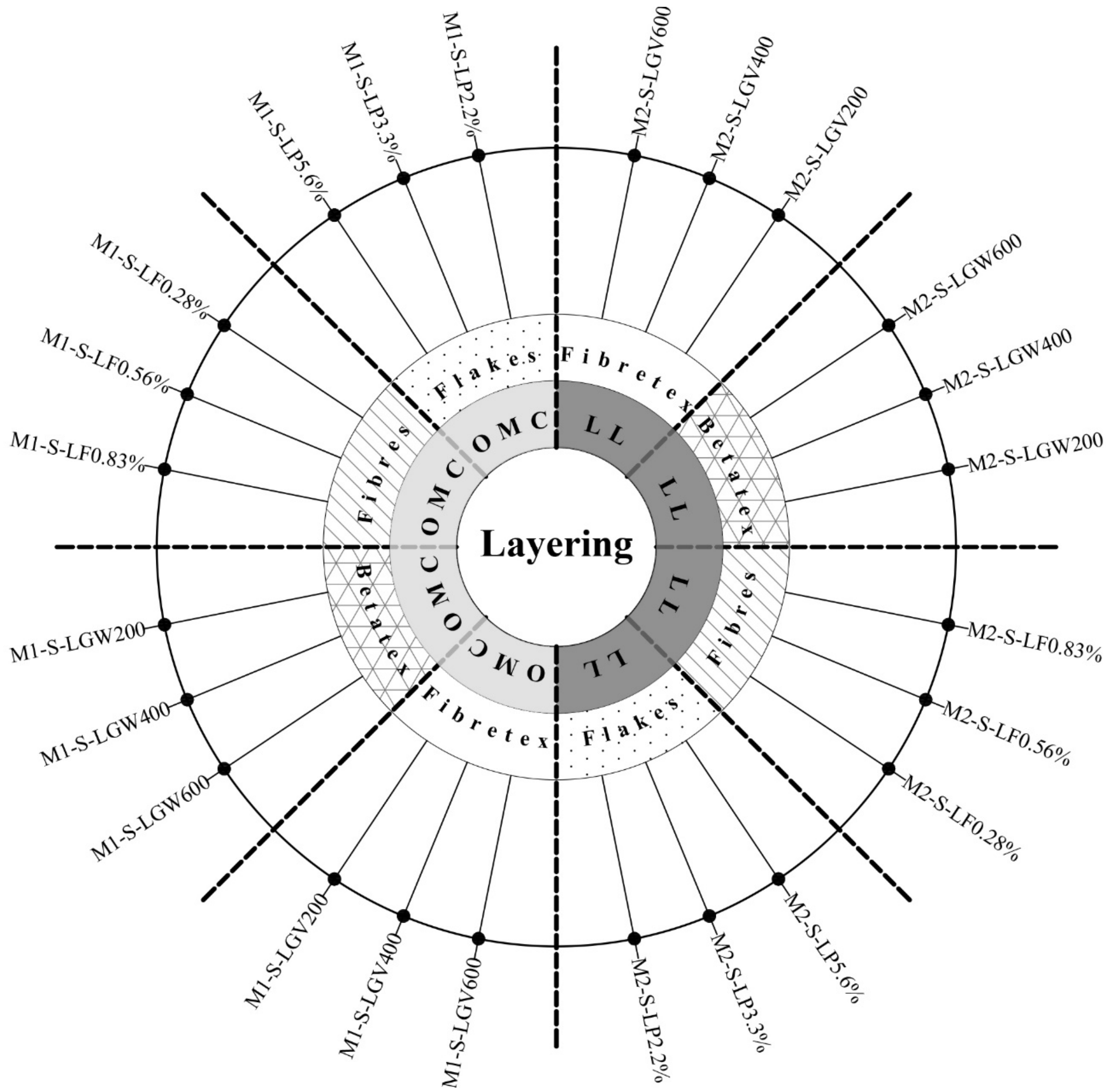


Figure 3.6: Experiments conducted on samples with reinforcement installed in layers

3.4 Experimental investigation

3.4.1 Research materials

3.4.1.1 Soils

Base material – Silt

A fine silt (referred to as Durbanville silt in this thesis) was used as the base material and it was sourced from the Phesantekraal quarry of Corobrik (large manufacturer of bricks) in the Durbanville region of Cape Town, South Africa. According to a geological investigation by Perold (2006), this area was described as being capped with silcrete and ferricrete. Occasional outcrops of the Malmesbury weathered shales (clay) were also observed which varied in colour (red, yellow and white to cream). The material which was collected for the experiments in this study was yellow and was described by Perold (2006) as being silty to clayey, and also as an extremely weathered ferruginous clay. While traces of feldspar were found to exist, the yellow colour was explained in terms of the presence of goethite. A semi quantitative mineralogical analysis of the clay fraction samples revealed that kaolinite, goethite and quartz were the major minerals present in the sample, although minor amounts of mica was also detected (Perold, 2006).

The fine soil from the quarry was chosen to represent a weak base soil which required improvement to sustain higher loads, under lower settlements. This material was preferred since it was easily available and accessible. Furthermore, the quality of the material was more controlled due to the quarry processes involved. A wet sieve analysis and hydrometer test performed on Durbanville silt yielded the particle size distribution graph as shown in Figure 3.7. From the Figure, it was noted that 79.4 % of the particles passed through the 0.075 mm sieve. Further characteristics tests on this material resulted in the mechanical properties presented in Table 3.2. By plotting the plasticity index and the liquid limit on the plasticity chart, the material was classified as a low plasticity silt.

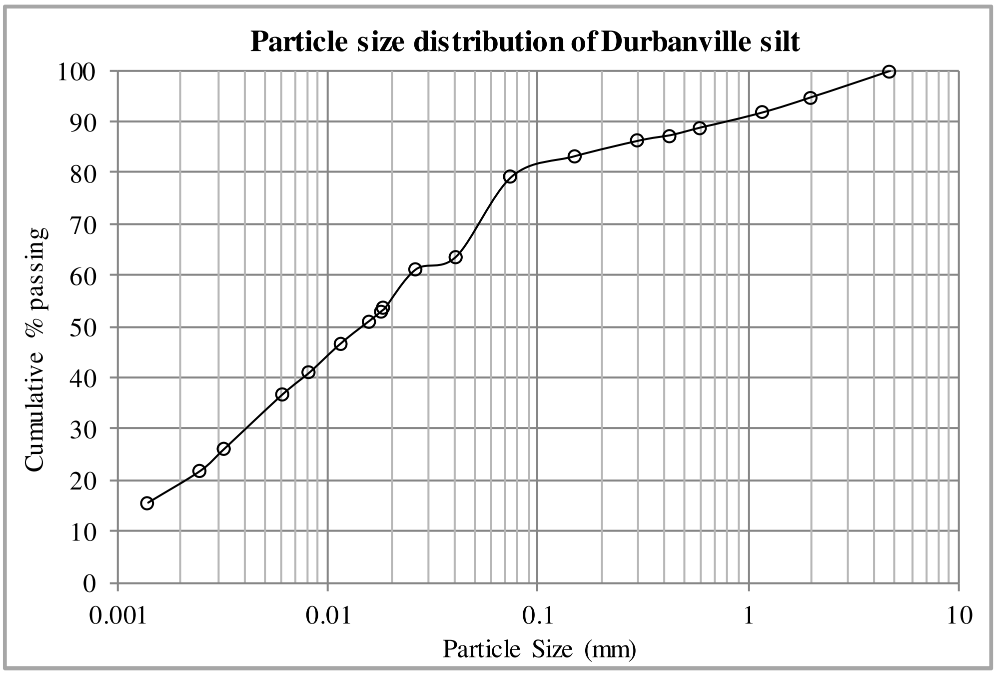


Figure 3.7: Particle size distribution of Durbanville silt (wet sieve analysis and hydrometer)

Table 3.2: Mechanical properties of Durbanville silt

Soil property	Unit	Value	Test method	Notes / Test codes
Specific gravity, G_s	Mg/m^3	2.71	Small Pycnometer method	BS 1377: Part 2: 1990
Natural moisture content	%	16.8	Oven drying method	BS 1377: Part 2: 1990
Liquid Limit (LL)	%	37	Atterberg Limits	BS 1377: Part 4: 1990
Plastic Limit	%	30.6	Atterberg Limits	BS 1377: Part 4: 1990
Plasticity Index	%	6.4	Atterberg Limits	BS 1377: Part 4: 1990
Optimum moisture content (OMC)	%	17.7	Standard Proctor test	BS 1377: Part 4: 1990
Maximum dry density	Mg/m^3	1.7	Standard Proctor test	BS 1377: Part 4: 1990
Angle of friction, ϕ_u: at OMC at LL	°	15.2 0	Triaxial test (UU)	BS 1377: Part 7: 1990
Cohesion, c_u: at OMC at LL	kN/m^2	3.98 6.42	Triaxial test (UU)	BS 1377: Part 7: 1990

Durbanville silt was obtained in a semi dry state from the quarry. Large batches were first visually inspected for the presence of any foreign elements such as stones, leaves or roots. Thereafter, they were oven dried at 105°C for 24 hours. This procedure was followed to prevent any possible moisture variation from affecting the ultimate water content of the prepared wet mix. Once dried, the material was left to cool down in the closed oven room, it was then sieved through a 4.25 mm sieve. This step was followed according to Ambily & Gandhi (2007) to eliminate any bigger lumps, which could possibly affect the distribution of water within the soil mass during the mixing process. After sieving, the finer fraction of the silt was stored in large 50l sealed plastic containers.

In the summary of the literature review, it was pointed out that granular columns have usually been studied when they were installed in base soils up to a maximum moisture content of approximately 1.4 times the liquid limit (McKelvey et al., 2004). Besides, the application of granular columns in wet and weak soils are quite popular. However, for the Durbanville silt, it was established through trials that the silt was too weak at these moisture contents and would, therefore, provide almost no confining stresses to support the columns. Hence, base soils at liquid limit (LL) was preferred. For a better understanding of the behaviour of the columns under the extreme conditions of wetness and dryness of the surrounding soil, the base soils at optimum moisture content (OMC) were also studied. The Durbanville silt used is later shown in Figure 3.9 after mixing it at the 2 required moisture contents (OMC and LL).

Column material - Sand

The primary column material used in this study was sand (referred to as Cape Flats sand) which was mined in the Cape Flats region of Cape Town, South Africa. Adelana, Xu & Vrbka (2010) explained that the Cape Flats is essentially sedimentary sand which overlies the Malmesbury shale, with a maximum thickness of up to 50 m in certain places. They further confirmed that there was practically no observation of any outcrops, despite of the relatively thin layer of sand, when compared to its large lateral spread. Hendey & Dingle (1983) described them as Cenozoic sediments of the Western Cape, which are more commonly referred to as the Sandveld Group. According to Adelana, Xu & Vrbka (2010), the Cape Flats sand are derived from 2 sources: (1) weathering, followed by deposition, of quartzite and sandstones of the Malmesbury Formation and the Table Mountain Group, under marine conditions, and (2) deposition of aeolian sand, from the beaches in the surroundings, over the marine sands.

This light grey sand was selected since the presence of the fines was minimal, in addition to the clean nature of the material. Besides, it was readily available in the locality and the associated cost was relatively low. From a dry sieve analysis, it was found that the sand particles were smaller than 2.36 mm. In fact, 98.9 % of the particles was smaller than 1.18 mm. Figure 3.8 illustrates the particle size distribution which was achieved when a dry sieve analysis was performed on a sample of the Cape Flats sand.

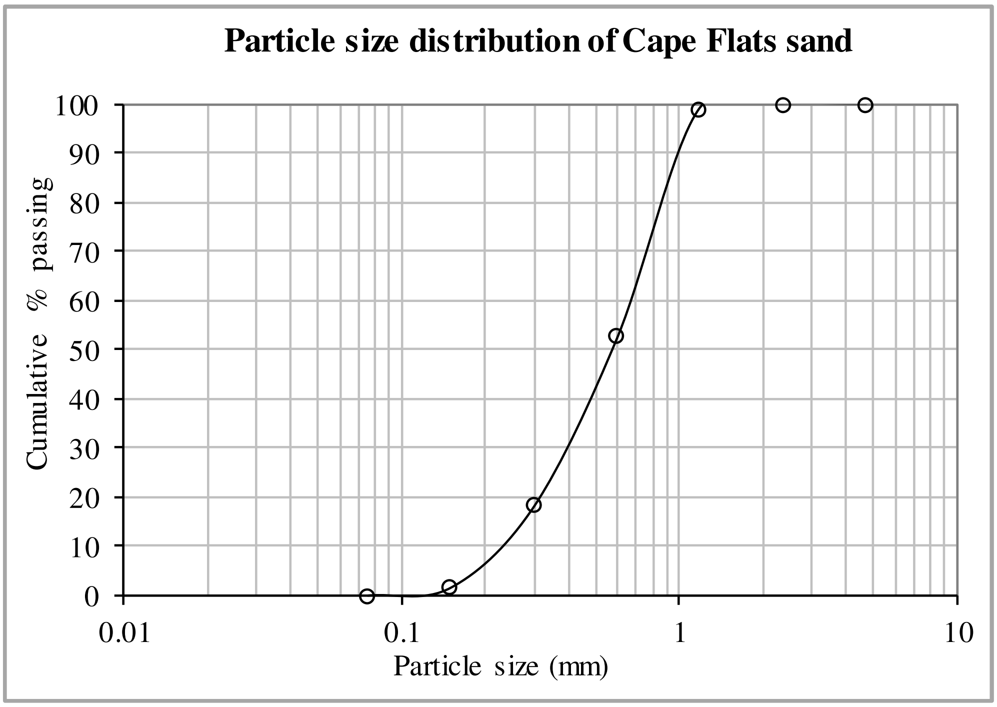


Figure 3.8: Particle size distribution curve for Cape Flats sand

From the information generated by the particle distribution curve, the sand was classified as poorly graded. Kalumba (1998) observed a sample of this type of sand and reported that the particles were generally round in shape. Consequently, the spaces in between the particles were expected to be larger compared to those in sands having angular particles or in well graded sands. Therefore, reinforcing of the columns was anticipated to reduce the volume of these spaces since the particles was expected to interlock around the extensible reinforcement materials thereby producing a much denser column. This sand was also used in the study conducted by Sobhee-Beetul (2012) whereby it was referred to as Cape Flats sand. Table 3.3 summarises the mechanical properties obtained from the characterisation tests which were performed on the sand (more details are presented in Appendix A).

Table 3.3: Mechanical properties of Cape Flats sand

Soil property	Unit	Value	Test method	Notes / Test codes
Specific gravity, G_s	Mg/m^3	2.70	Small Pycnometer method	BS 1377: Part 2: 1990
Natural moisture content	%	0.1	Oven drying method	BS 1377: Part 2: 1990
Optimum moisture content	%	12.5	Standard Proctor test	BS 1377: Part 4: 1990
Maximum dry density	Mg/m^3	1.796	Standard Proctor test	BS 1377: Part 4: 1990
D_{10} D_{30} D_{60}	mm	0.24 0.40 0.68	Dry sieve analysis	BS 1377: Part 2: 1990
Coefficient of uniformity, C_u	-	2.83	Dry sieve analysis	BS 1377: Part 2: 1990
Coefficient of curvature, C_c	-	0.98	Dry sieve analysis	BS 1377: Part 2: 1990
Angle of friction, ϕ	°	36	Direct shear method	BS 1377: Part 7: 1990
Cohesion, c	kN/m^2	5	Direct shear method	BS 1377: Part 7: 1990

Brown (1977) established a rating system to verify the suitability of a backfill material for vibro-replacement columns. He applied his project experience, together with the settling rate of the solid particles in water, and proposed the following equation and table to determine the suitability number (S_N) and the rating of a backfill (D_{10}, D_{20} and D_{50} are in mm and they are the particle sizes of 10, 20 and 50 % finer):

$$S_N = 1.7\sqrt{\frac{3}{(D_{50})^2} + \frac{1}{(D_{20})^2} + \frac{1}{(D_{10})^2}} \qquad \text{(Equation 3.1)}$$

Table 3.4: Suitability of backfill material (adapted from Brown, 1977)

Suitability Number (S_N)	0-10	10-20	20-30	30-40	>50
Rating	Excellent	Good	Fair	Poor	Unsuitable

Using this equation and the values of D_{10}, D_{20} and D_{50} (0.24, 0.32 and 0.58 mm) obtained from Figure 3.8, the suitability number and the rating of the backfill was determined using equation 3.1 and it was compared with the rating in Table 3.4. The suitability number of Cape Flats sand was found to be 10.2 and was therefore rated as a 'good' backfill material for the granular columns.

Since the sand was obtained in a clean and relatively dry state, the preparatory process involved was minimal. The sacks were emptied onto large metal trays which were placed in the oven for drying at a temperature of 105°C. Drying was necessary to minimise any overestimation of the strength of the columns; the low presence of water could possibly produce a more denser and stronger column. After 24 hours, the trays were removed, and the material was allowed to cool down in the closed oven room. Once room temperature was attained, the sand was stored in 50l airtight plastic containers for use during test sample preparation. Figure 3.9 illustrates the state of the prepared silt (at both moisture contents) and the sand.

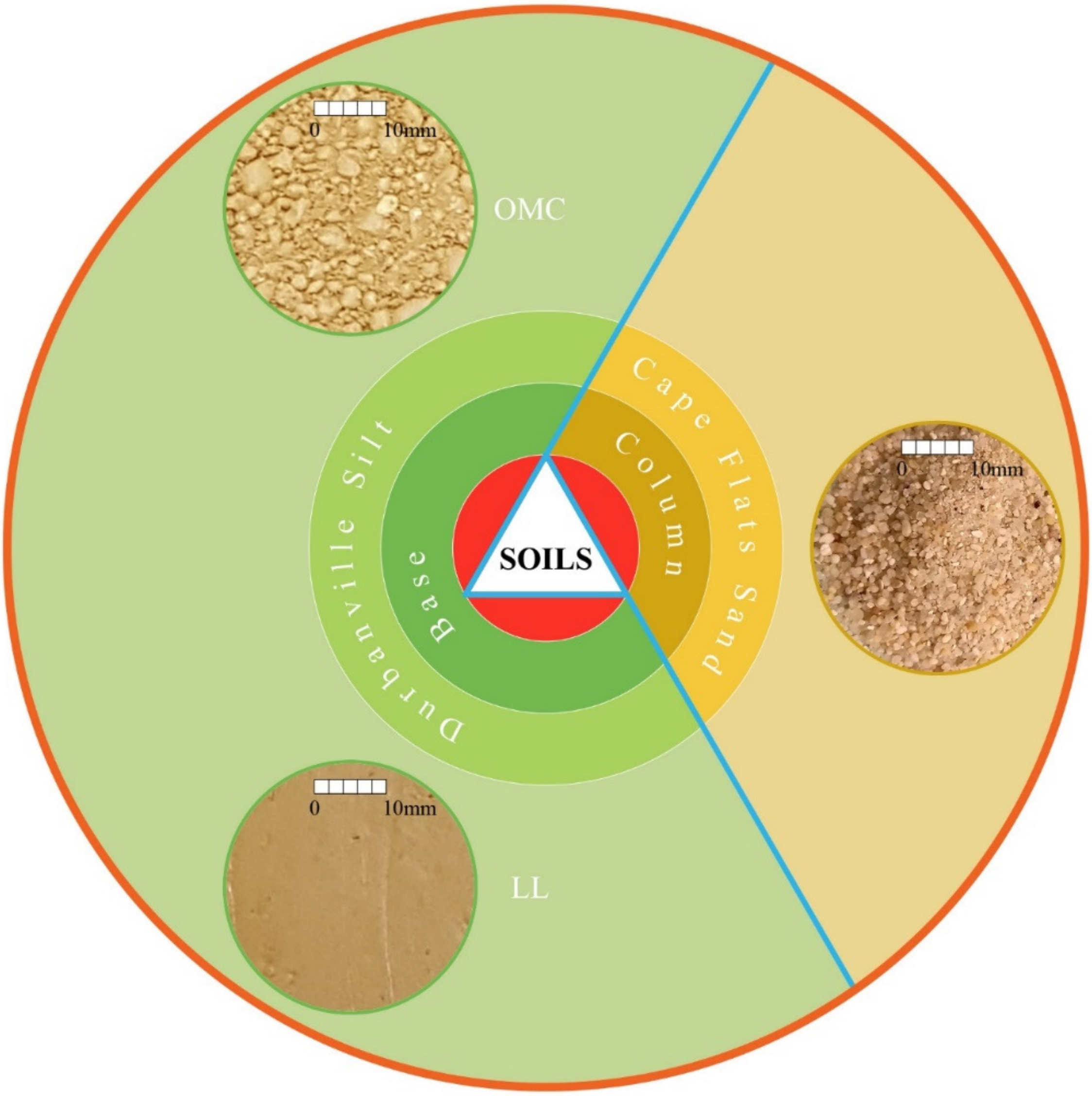


Figure 3.9: A pictorial representation of the prepared samples for both the silt (at OMC and LL) and the sand

3.4.1.2 PET derivatives as column reinforcement

The materials used for reinforcing the granular columns were selected based on the PET bottle recycling process. In this process, the first material obtained is usually the flakes followed by the fibres. These fibres are then used to manufacture several products, including geotextiles. Evidently, as the form of PET moves further away from the bottle state (within the recycling process), more energy is required, while simultaneously raising the product cost. In this research, flakes, fibres and geotextile (made from the fibres obtained during recycling) were used to investigate the behaviour of the reinforced columns when each of these were utilised.

A second geotextile, made from virgin PET fibres, was additionally considered as a potential reinforcement to allow for a comparison of performances achieved with both types of geotextiles. Each of these materials are subsequently described and images of each are given in Appendix C.

PET flakes

The flakes were sourced in a dry state from the Cape Town branch of Kaytech Engineered Fabrics, which is a local manufacturer and supplier of geosynthetics. In fact, these flakes are used by the same manufacturer to produce their Betatex geotextiles. While the flakes were predominantly green or colourless, a few brown and blue particles were spotted. The variation in colour of the flakes was attributed to the different pigmented PET bottles available on the local market. A sieve analysis performed on these irregular shaped flakes showed that the particles size varied between 0.6 and 9.5 mm, with 71.9 % of them passing through the 4.75 mm sieve. The particle size distribution of the PET flakes obtained from the dry sieve analysis is shown in Figure 3.10.

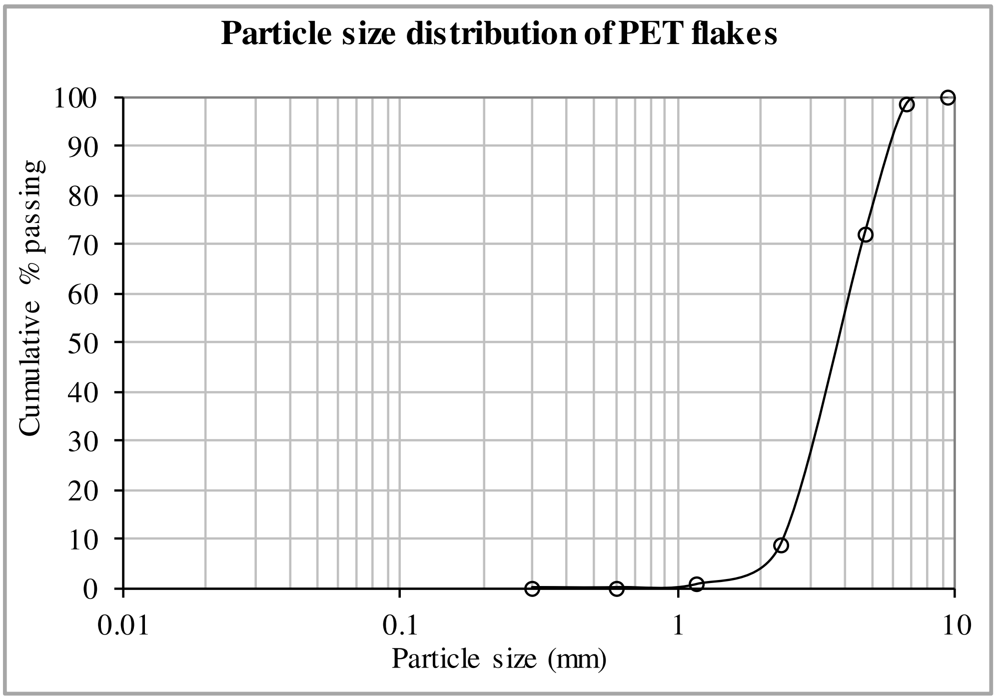


Figure 3.10: Particle size distribution of a PET flakes sample as obtained from the supplier

Besides sieving of the soils, the PET flakes were also sieved to disregard the finer (smaller than 2.36 mm) flakes present within the whole mass. This was desired since it was anticipated that the fine flakes could possibly fill in the voids present within the column thus resulting in a denser column rather than a reinforced one. Hence, large batches of flakes were sieved by means of a mechanical shaker to attain flakes passing through sieves of aperture sizes between 2.36 and 9.5 mm. The selection of the flakes size at this stage was dependent on the plastic dimensions which have been used in previous studies (Laskar & Pal, 2013; Luwalaga, 2015). Additionally, this selection was based on the size of crushed aggregates which has been used to form granular columns in previous studies (Ambily & Gandhi, 2007; Sobhee-Beetul, 2012). Once sieved, the flakes were stored in 20l closed plastic buckets for the intended use. Figure 3.11 provides a sample of the sieved flakes.

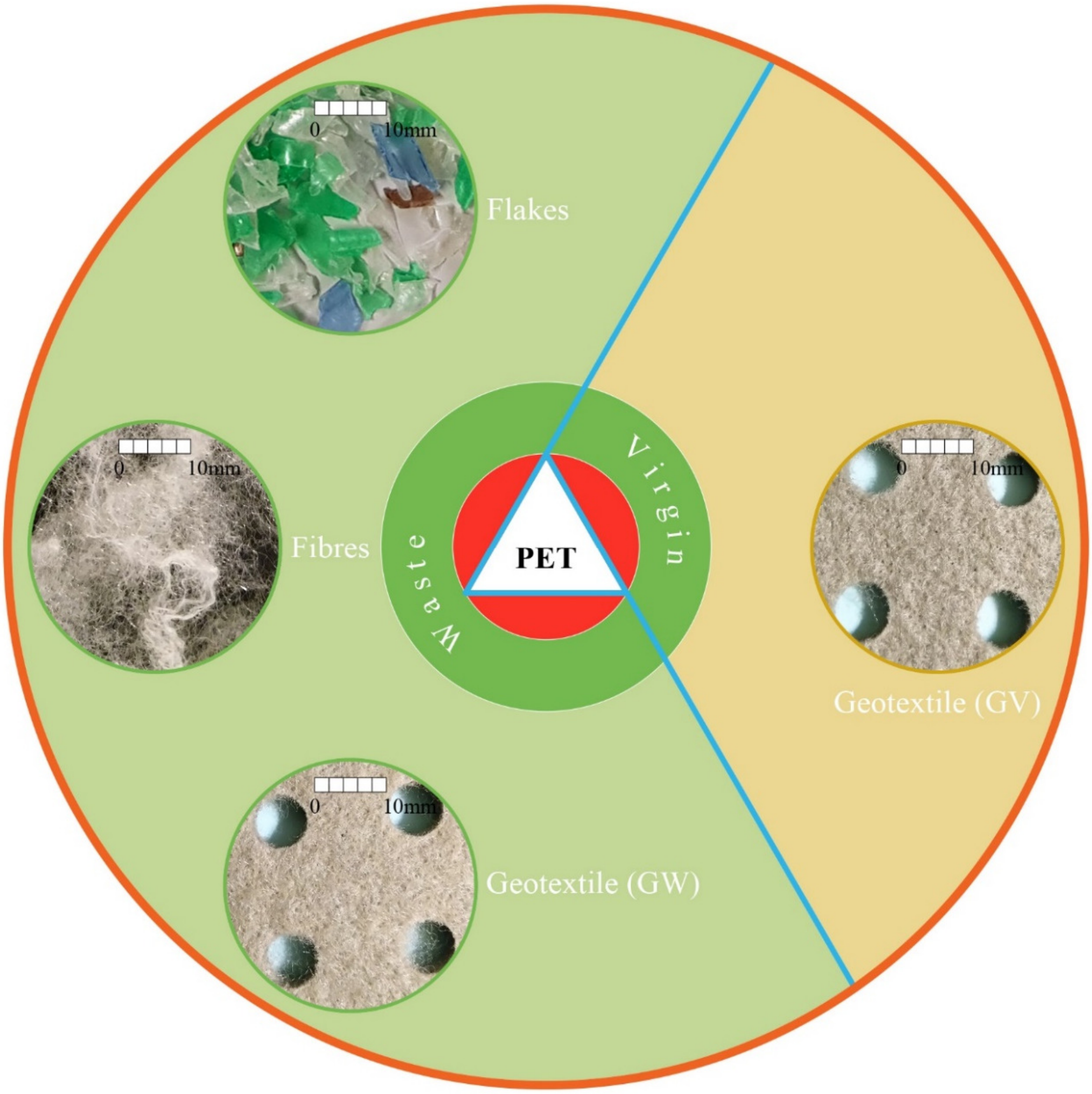


Figure 3.11: Forms of PET used in this study (include all types of PET)

PET fibres

The white fibres (shown in Figure 3.11), which were obtained after washing, heating and pelletizing the PET flakes, were also used as a reinforcement material. These were obtained from Fibertex South Africa, a local company which also produces non-woven geotextiles from these flakes by firstly converting it to fibres. The fibres typically appeared as candy floss and were remarkably light in weight.

Generally, PET fibres tend to form an entangled mass. Once they are pulled apart by hands, the same weight of fibre occupies a larger volume. For this study, 2 g of fibres was taken at a time, and they were carefully and manually separated to loosen up the entangled fibres. This procedure was done just before preparing the test sample for an experiment where fibre reinforcement was required since the fibres could not be worked onto and stored. Storing would involve some level of recompression under the fibres own weight, which would then result in a recurring drop in volume. Hence, the fibres were prepared just before being used in the columns. Two microscopic views of the fibres are given in Figure 3.12 where different measurements of the diameter of the fibre has been taken. The average diameter of the fibre was thus calculated as 24.6 µm.

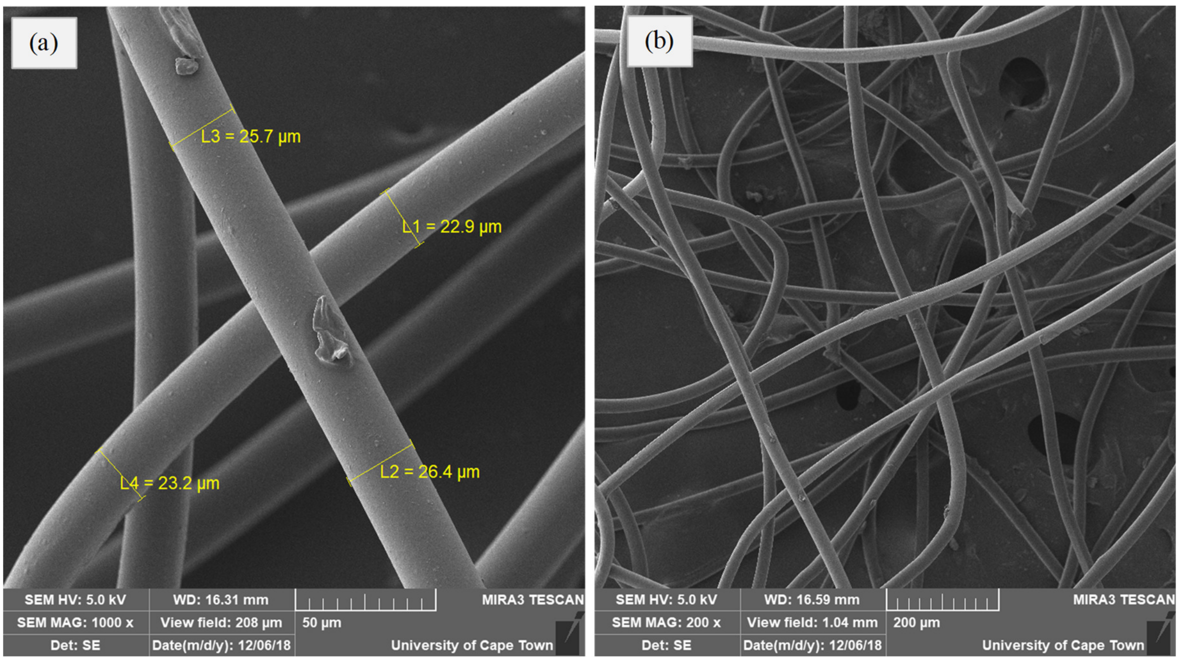


Figure 3.12: Microscopic view of a sample of the fibre (sourced from the Electron Microscope Unit at the University of Cape Town)

Geotextiles

The third and fourth materials used were non-woven geotextiles and they are depicted in Figure 3.11. Both geotextiles were supplied by Fibertex South Africa; one was made from the fibre obtained through the recycling of PET bottle wastes while the other was produced from virgin PET. These 2 types of geotextiles were considered to enable a comparison in their performance when included in the sand columns and tested under similar conditions. More specifically, it was important to compare the gain in improvement achieved through both samples in order to confirm the efficiency of the geotextile generated from the used PET bottles. Appendix B provides the properties and specifications of the geotextiles.

Both geotextiles, used to reinforce the columns, were cut into discs of 99 mm by means of a press tool to fit laterally at any horizontal cross-section within the column. The discs were made slightly smaller than the assumed column diameter of 100 mm to avoid bending of the geotextile along the edge of the column. Perforations of 7 mm in diameter were then made in regular patterns, at different positions on the circular material (Figure 3.13), by means of a puncher. Since the inclusion of the geotextile aimed at reinforcing the columns, the holes were deemed necessary to allow for better interlocking of the sand particles around the material, thereby improving the shear strength within the columns. In a previous research by Sobhee (2010), it was confirmed that perforations in reinforcing materials tend to improve the performance to a certain extent. Besides, geogrids also use this interlocking mechanism to provide a gain in strength of the weak soil. Hence, perforations were made in the geotextiles. Compared to geogrids where the apertures are squares to allow the material to perform biaxially, the holes were made circular in this instance to allow for the shearing effect of the reinforced sand to be equal in all directions. Figure 3.13 illustrates the pattern followed for punching on both types of geotextile discs.

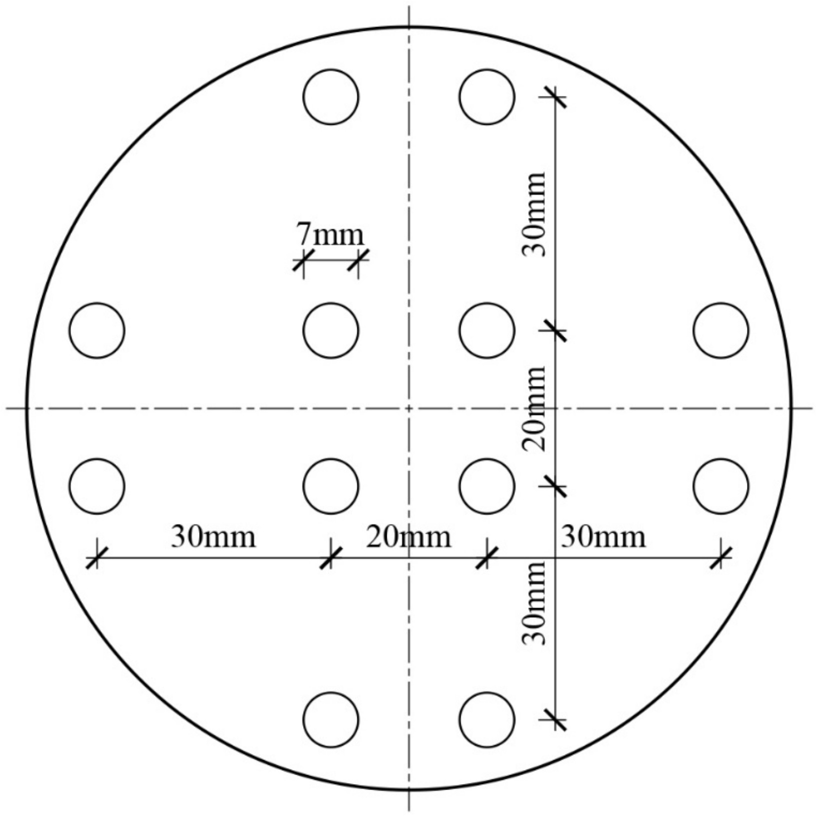


Figure 3.13: Typical perforations in a geotextile disc

3.4.2 Sample preparation and testing

3.4.2.1 Formation of the wet base material in the testing tank

To obtain the wet base material to be used for test sample preparation, 10 kg of the stored dry silt was weighed on a scale and it was poured into a 20l mechanical mixer. Tap water was measured on a balance and transferred into the mixer so as to produce a wet mix at either optimum moisture content (1.77 kg) or at liquid limit (3.70 kg). A large scoop was then used to carefully mix both products to avoid splashing of the fine material, at the onset of the machine; this was necessary to prevent any reduction in mass which could affect the blending accuracy.

After manually blending the silt with water, the machine was switched on and mixing was allowed for 5 minutes, with 3 stops in between to regather the material so as to obtain a homogeneous mass. A few trials were done, prior to the actual preparation, to precisely determine the duration and mode of mixing. These trials were beneficial since they helped avoid over mixing that would cause water loss due to evaporation as well as under mixing which would result in non-homogeneity. When the blend was ready, the mixer was switched off and it was emptied immediately in a plastic container which was subsequently closed tightly with a lid to prevent any water loss through evaporation. This process was repeated 5 times, each time storing the mix in the same storage container as the previous batch, to obtain 58.85 and 68.50 kg of wet silt at optimum moisture content and liquid limit, respectively; these weights corresponded to the amount of wet mix required to prepare one test specimen at the respective water contents. The wet silt was then covered with a double layer of HDPE plastic refuse bag and the container was closed with a lid to maintain the quantity of water in the wet silt. The stored material was left standing for 24 hours, to allow for even distribution of water within the silt, before being used for test sample preparation.

After 1 day, the specimen was prepared as described in the following paragraph, and it was either tested in an unimproved state or after being improved by granular columns. Before each test, the steel tank was wiped clean and dry and its inner surface was smeared with a thin coating of thick motor oil (viscosity grade – SAE 30) using a brush; this was necessary to reduce any friction between the silt and the wall of the container. Greasing served the additional purpose of corrosion prevention of the testing tank.

For tests at OMC, 5000 g of the wet silt was initially transferred in the tank carefully to form a layer on top of which the wooden compaction board was placed. A hand compactor of 2.5 kg was centrally dropped 15 times, through a height of 180 mm, on the board to uniformly compact the silt to a thickness of 50 mm. This process was followed 8 times until a silt bed of depth 400 mm was formed. During compaction, a total energy of 530 J was imparted to the silt bed, thereby resulting in an average bulk density of 1415 kg/m^3. Prior to testing, several trials were done on material preparation to establish the amount of energy required for adequately compacting the silt, thus forming a dense state without comprising the quality of the material. Lastly, a wide metal scraper, together with a spirit level, was used to produce a smooth and levelled surface at the top of the bed. For experiments conducted on silts at LL, the preparation stages differed slightly since compaction was not required due to the high degree of saturation of the mixture. Since the mix was very soft and contained a high volume of water, rapid loading from compaction of the material generated excess pore water pressures. As a result of these high forces in between the soil particles, the material failed to compact. Therefore, a predetermined mass of 5500 g of the wet silt was transferred to the empty tank and manual pressures were exerted onto it to compress the material to form the first layer of 50 mm. Compression was essential to expel maximum air pockets present within the sample. This procedure was repeated to obtain a bed of silt consisting of 8 layers, at a bulk density of 1556 kg/m^3, whereby the top layer was scraped smoothly and levelled. Figure 3.14 shows typically prepared silt beds at both OMC and LL.

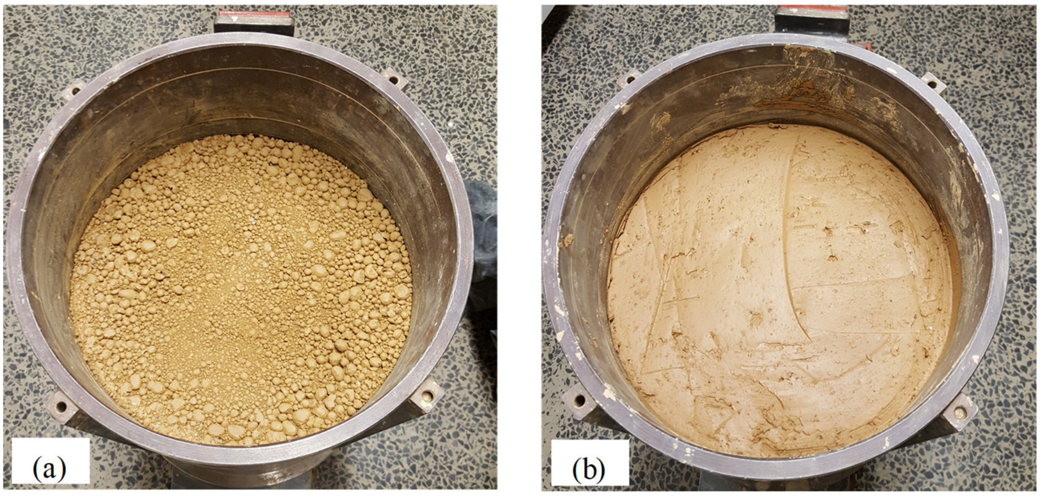


Figure 3.14: Prepared silt bed at (a) OMC and (b) LL

3.4.2.2 Column installation

Ordinary granular columns (OGC)

The column installation technique adopted in this research was primarily derived from Ambily & Gandhi (2007) and Sobhee-Beetul (2012). It was in fact a combination of the methods, which were proposed by Han (2015), and known as ‘sand compaction column’ and ‘rammed aggregate column’. After the preparation of the base material, the granular columns were installed by means of a replacement method in a pre-bored hole. At first, the collar was fitted by means of screws on the top of the tank. The outer surface of the hollow steel pipe was then lightly brushed with the motor oil, after which the tube was pushed down by hand carefully through the collar into the silt bed, until it touched the base of the container. Oiling was important to allow for smooth penetration and withdrawal of the pipe, without causing significant disturbances to the surrounding material. The helical auger was then inserted in the steel pipe to cut out all the silt present within the tube by manually turning it in a clockwise direction, while simultaneously pushing it down into the silt. After 2 turns, the auger was pulled out gently and the cut material was discarded, followed by further augering. Cutting and emptying was done in stages to prevent jamming of the auger due to the excessive suction pressure build up. After emptying all the silt within the pipe, the inner wall of the latter was cleaned with a long nylon brush, followed by a piece of clean cotton cloth which was attached to a wooden rod. This was done to prevent sand from getting trapped on the metal surface due to its wetness.

To form the unreinforced granular columns, a measured amount of sand was then poured into the hole and the pipe was carefully retracted by 35 mm. A 2.3 kg hand compactor was subsequently used to compact the column material by dropping the weight 12 times, through a height of 180 mm, to form a layer of thickness 50 mm. The generated energy from this degree of compaction was predetermined through trials to ensure that a dense column (the density varied slightly though for each test since compaction was affected by the type and quantity of the reinforcement within the columns – this was noted in terms of the masses of the sand and reinforcement required in each column) was formed, without over compacting which would cause crushing of the sand particles. This stage was repeated 7 more times until a column of length 400 mm was formed, which was levelled with the top surface of the silt bed; the 8th layer at the top was not compacted since a trial confirmed that the poor confinement from the surrounding soil in the upper most section of the base soil resulted in continuous bulging during compaction. In cases where the column was not completed, a small mass of sand was used to top up the column such that it was levelled with the surrounding silt. For each of these 7 layers, the hollow cylinder was retracted by 50 mm as opposed to 35 mm used in the first layer. This penetration of the cylinder was maintained for each layer to prevent and opening in the column which would encourage the surrounding material to collapse into the column, and thus interfering with its performance. Figure 3.15 shows the different stages involved in the column installation.

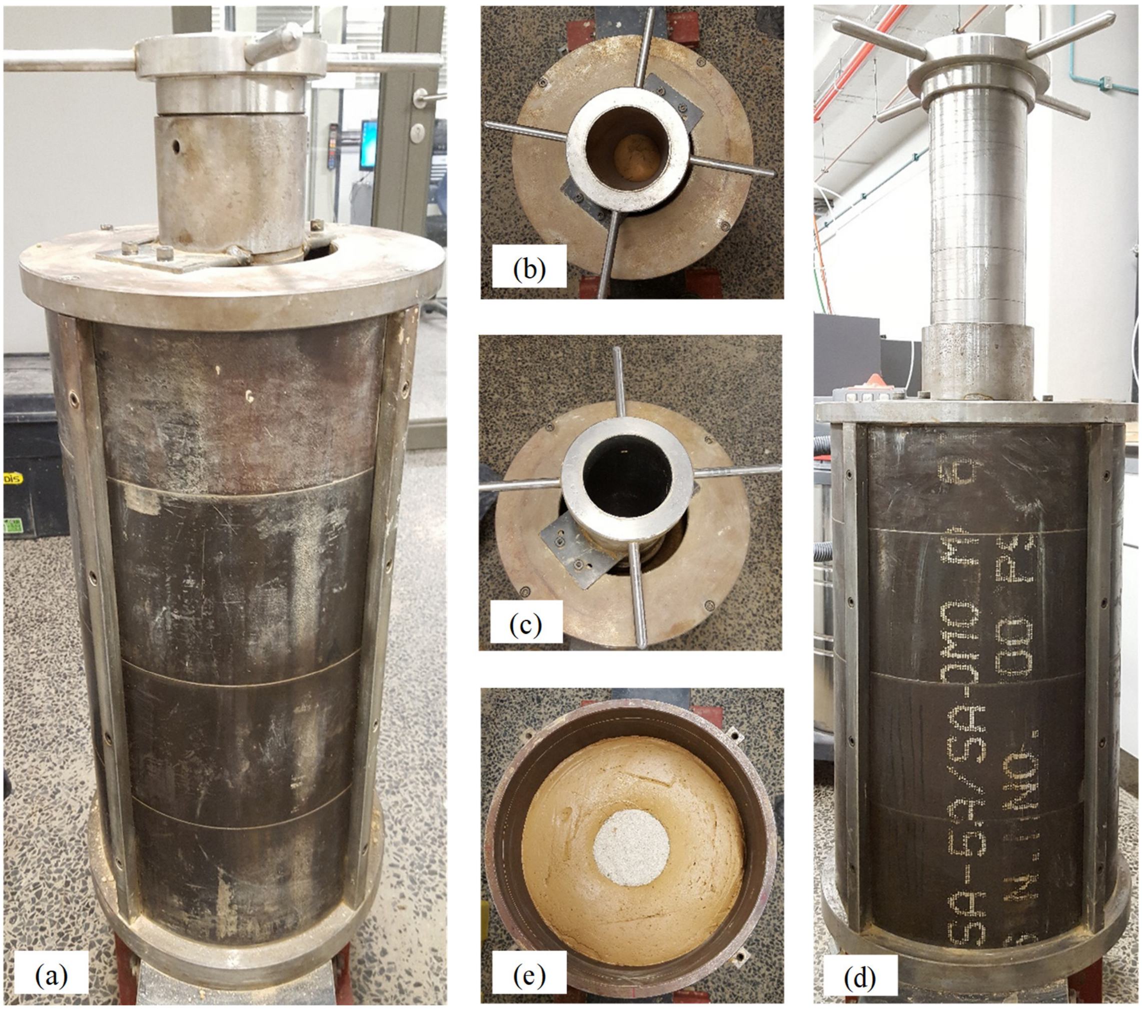


Figure 3.15: Stages in column installation within the base soil (a) hollow steel pipe pushed through the collar, (b) silt inside pipe to be removed using the auger, (c) inner surface of pipe cleaned by means of a nylon brush, (d) retraction of the pipe after a layer of sand has been poured, (e) column of length 400 mm formed

Reinforced granular columns (RGC)

Although the installation process was rather simple for sand columns, the degree of complexity increased when the reinforced granular columns were formed. Through some trials, it was observed that an easy approach to incorporate the reinforcement would be to keep the total composite mass of each layer in the column approximately the same, irrespective of the presence of the polymeric materials. The difference in thickness arising from the inclusions were then catered for at the top of the column, within the last layer. Depending on the moisture content of the base soil (OMC or LL), the composite masses of the columns varied such that these were always higher in LL tests due to the lower confinement which was provided by the surrounding material; therefore, pre-bulging occurred during the preparation which necessitated slightly larger quantities of the column materials to form it. These masses are presented at the end of this chapter. The following sub-sections describe how the procedures varied according to the arrangement and type of the reinforcement used.

Columns with randomly mixed reinforcement

Two types of reinforcement were used for the purpose of random mixing namely PET flakes and PET fibres. Both were installed following similar procedures to that employed in ordinary granular columns. Sand was measured, as per the predetermined amount, and was placed in 9 bowls whereby each one was used to form one 50 mm thick layer of the column. The corresponding masses of reinforcements required per each 50 mm column layer (prepared as per earlier description) were then measured and kept in small bowls. Before creating the hole in the silt bed, the reinforcement from 1 bowl was transferred into the respective container with the measured mass of sand; they were randomly mixed by means of a spatula. Figure 3.16 shows a typical randomly mixed sample of sand with the PET flakes.

Figure 3.16: Randomly mixed sample of sand with PET flakes

This composite material was then used to form the reinforced columns, following the same procedures which was adopted in the formation of the ordinary granular columns. As was mentioned earlier, the increase in thickness of the column layer was ultimately addressed when the column was nearly constructed. Before doing the last layer, the remaining depth to be filled was measured. If it was greater than the chosen layer thickness of 50 mm, an extra layer was added to the column. This step was not necessary in OGCs since the mass of the sand needed was predetermined; but, it was needed in RGCs since the presence of the reinforcement affected the composite mass. However, in case this depth was shorter than 50 mm, the randomly mixed composite was simply poured up to surface level and compaction was avoided. This is explained by the low lateral restraining forces which exists immediately below the surface. Any compaction done at such stages would basically result in significant bulging of the column. In a trial preparation on a silt bed at LL, it was observed that the column would not reach the same level as the top surface of the base material, irrespective of the extra composite mass being added to the column. The mix was then carefully emptied to observe the column behaviour whereby it was noted that bulging had significantly occurred prior to the application of the load. Furthermore, heaving of the silty surface was also detected. Hence, to limit the degree of protrusion of the column into the surrounding material, the top layer of the column was generally not compacted. This additionally explains the slight variation in density of the different columns.

Columns with layers of reinforcement

With regards to the arrangement of the reinforcements in layers, 4 types of materials were used. These were PET flakes, PET fibres, geotextile from a waste material and geotextile from a virgin material. In general, the installation procedure was similar to that of unreinforced sand columns comprising of several layers of compacted sand. However, in columns which were strengthened with layers of reinforcement, the latter was placed in between any 2 layers of sand. Through several trials and some mathematical calculations, the mass of sand to be added to form each layer was determined such that the first bottom layer of sand was always 900 g. Thereafter, the sand weight per layer was reduced to accommodate the inclusion of the reinforcements to form the rest of the column. Trials were initially performed to establish the respective masses of sand and reinforcement per layer. Similar to columns with randomly mixed reinforcement, the top layer within these columns was also not compacted to avoid significant bulging during preparation. Generally, it was noted that most columns constituted of 6 to 7 layers of reinforcement, depending on the thickness of the latter.

3.4.2.3 Testing and data acquisition

Compression tests formed a significant component of the experimental work in this research. Experiments were conducted on prepared samples by applying a displacement-controlled load through the Zwick machine. This machine was operated within a closed room due to its high sensitivity to displacement. When a test specimen was prepared, the bronze rollers on the trolley were lifted up (to allow for a smooth rolling of the tank), and the tank was gently pushed on to the loading apparatus.

Once on the machine, the tank was manually adjusted until centrally positioned to eliminate any occurrence of eccentric loading conditions. A rigid circular loading plate, of a diameter of 200 mm (twice the diameter of the column), was then placed on the centre top of the column and it was levelled by means of a spirit level. The diameter of the loading plate was based on that used by Sobhee-Beetul (2012); loading was applied to both the column and part of the surrounding material since a column loading scenario would result in lower load carrying capacities. A spacing cylinder was placed in the middle of the plate, followed by subsequent lowering of the loading platen of the machine such that they were very close to each other, but without making any point of contact. The Zwick machine was connected to a computer which operated by means of a specific program. Prior to starting the test, the program was initiated to capture a number of details regarding the experiment. This included information such as the test speed of 1.2 mm/min, the maximum allowable settlement of 50 mm, dimensions and mechanical properties of the loading plate and engineering properties of the sample. This compression rate was based on previous studies with similar criteria to allow for rapid loading, thus triggering undrained conditions (Sobhee-Beetul, 2012; Murugesan & Rajagopal, 2008). The approach was assumed to be reliable since it simulated a typical field condition immediately after the installation of the column and in the initial stages of loading when critical changes are experienced in terms of pore water pressures and column bulging (Weber, Laue & Springman, 2006). Therefore, this is usually considered to be the worst-case scenario of the effect of stresses in the field post-treatment with granular columns.

Apart from the loading rate, the maximum allowable settlement of 50 mm was also set based on the Eurocode 7 which suggests similar values for normal structures. When all the necessary values were captured into the file, the machine was switched on to allow loading of the sample while the computer recorded the stress-settlement behaviour simultaneously. Real time recording of readings, at 1 mm intervals, were subsequently used for describing and analysing the results. Figure 3.17 illustrates testing of a sample under a compressive force from the Zwick machine.

Figure 3.17: Compression test in progress on the Zwick machine (a) Experimental set-up, (b) Load being applied to a test specimen through a rigid loading plate

3.4.2.4 Physical modelling of the column deformation post-testing

This section describes how the deformation characteristics of each tested column was achieved by physically modelling each of them. Upon completion of any given test, the tank was gently rolled back onto the trolley to minimise any disturbance to the tested sample. The loading plate was then removed and by means of an industrial vacuum cleaner, the column material was cautiously drawn out to empty the space occupied by the column. This process was carefully done to minimise any disturbance to the surrounding silt, as well as to avoid collapsing of the silt into the opening. For the same reason, the vacuum cleaner was also operated at the lowest speed; only 1 motor of 1200 Watts was used. A soft brush and a spatula were then used to lightly clean the sides of the open hole to remove any trapped sand on the surface. Pre-measured equal masses of sand and plaster of Paris (2.5 kg each) were placed in a large metal dish and were immediately mixed manually with tap water, by means of a spatula, to form a soup like consistency. This mix was subsequently poured into the opening and left to solidify for 2 hours. After the curing time, the tank was emptied physically, and the formed column was yielded. This formation allowed for both visual observations (to identify any significant irregularities in the maximum bulging shape) and measurements recording to understand the behaviour of the columns under each testing condition. The observations were more specific with regards to bulging diameter and its corresponding position along the column. Although the stages involved in the post-testing exercise had to be completed relatively fast, intensive care was taken to lessen any possibilities of disturbance to the surrounding base soil. Figure 3.18 shows the different stages involved in the formation of the column of plaster of Paris, which was a physical representative of the deformation which was achieved at a settlement of 50 mm (additional information is provided in Appendix D).

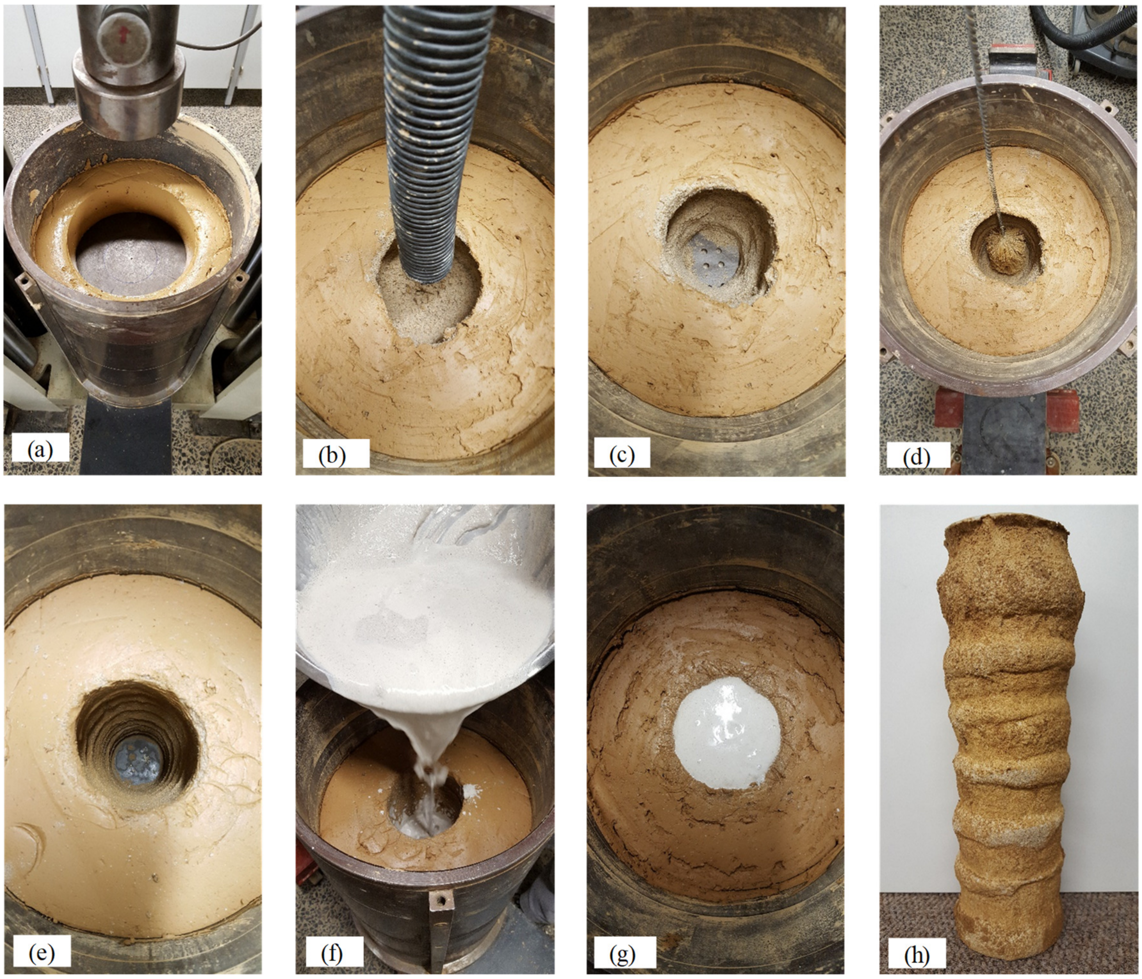


Figure 3.18: Stages in the formation of a typical plaster of Paris column (a) sample after testing, (b) vacuuming of column material, (c) exposure of the reinforcement material while vacuuming, (d) cleaning of the inner side of the column using a nylon brush, (e) sample after removal of column material, (f) pouring of the prepared plaster of Paris, (g) casted column left to solidify at room temperature, and (h) yielded column after 2 hours

3.4.2.5 Control tests

The control experiments in the testing programme involved the following: (1) base soil without any column, and (2) base soil with an ordinary granular column (OGC). For improvement determination, in terms of both load carrying capacity and settlement reduction, it was essential to establish bench mark results to compare with that obtained from the different reinforcement types and their arrangements. Hence, control tests were necessary, where 2 different types of such tests formed part of the experimental programme. Since the material used in this study had previously never been tested when improved with granular columns, the stress-strain relationship had to be established for unreinforced silt beds, at both OMC and LL. Therefore, these were the first 2 control tests of the investigation. Besides these tests, unreinforced granular column improved silt, at both OMC and LL, also formed part of the control experiments since they were required to determine the additional improvement achieved through the inclusion of the reinforcing elements. Thus, a total of 4 control tests were undertaken.

3.4.2.6 Quality control and reliability of laboratory tests

(a) Quality control

The laboratory procedures involved several steps whereby errors were prone to occur if the methodology adopted in each experiment was not consistent throughout the testing programme. To minimise the risk of errors, a quality control plan was drawn and always adhered to. The following are the precautionary measures which were taken to ensure negligible errors:

- Laboratory equipment such as the Zwick Universal Testing Machine and the weighing balance were calibrated before use.
- Identical mixing times were used for all mixes to avoid over or under mixing. This time was predetermined through some trials.
- The clay was oven dried and sieved using a mechanical shaker to avoid the presence of any larger particles, especially lumps. This was necessary to maintain the maximum particle size of the silt. It also avoided an uneven distribution of water which could have occurred with the presence of lumps.
- Once the wet clay was mixed, it was immediately transferred and stored in airtight containers to avoid any evaporation of water from the sample. Also, a fresh mixture was used for each test and small samples were taken before and after each experiment

for moisture content checks. This was done to ensure that all the tests were conducted at the same desired moisture content.

- Test specimens were prepared quickly to minimise any evaporation of water. To further reduce evaporation possibilities, all tests were conducted at the same room temperature of 20°C by pre-setting this through the air conditioning system.
- Test specimens were prepared at a rather fast pace especially when the granular columns were installed. This was to reduce the amount of water being drained from the surrounding clay into the column. All preparations were kept within relatively similar time frames with this respect.
- Care was taken to minimise any disturbances when transferring the test box from the trolley to the loading machine.
- The top surface of the base soil and the loading plate was levelled prior to each test to avoid eccentric loading.
- After the test, the tank was carefully loaded off the machine and vacuuming done immediately. When the column material was almost out, the premix for the plaster of Paris was simultaneously mixed with water and poured into the opening. This reduced the movement effect from the surrounding clay, or any potential collapse, due to the sudden drop in lateral stresses. Once mixed, the wet plaster of Paris with sand was transferred instantly into the opening to avoid any thickening of the wet mix prior to being used.
- Each column which was casted was left undisturbed for 2 hours to ensure a full development of strength, thereby diminishing the risks of breaking when being taken out of the test tank.

(b) Reliability of laboratory tests

Prior to testing, some trial experiments were performed to identify any possible sources of error as well as to refine the steps involved in sample preparation and testing. Initially, the results obtained were not as anticipated. Trials were then reconducted with slight adjustment in the procedures followed, and the results were compared to the preliminary ones. This was repeated a few times, following identical steps with one modification being introduced at a time, until the results were reproducible. To confirm the reliability of these results, few of the trial experiments were randomly selected and conducted again to ensure their repeatability; this was determined in terms of the repeatability standard deviation of the mean with an acceptable maximum level of 5 %. The basis of this percentage is later discussed in detail in Chapter 4, where the repeatability results are also presented.

3.4.3 Scale effect

Laboratory models for investigating granular columns are typically designed to replicate full scale models, as closely as possible. Small scale model testing has often been preferred due to their simplicity and the reasonable extent of information generated; however, their level of accuracy is often impacted by the scale effect (McKelvey & Sivakumar, 2000).

Al-Obaily (2017) pointed out that field tests have been implemented in previous studies to assess the load-settlement characteristics of reinforced granular columns. Although they were successful in addressing the concerns around scale effect in comparison to laboratory models, difficulties were faced with regards to managing the testing conditions as per the requirements. Thus, the findings from such tests can rather be misleading, besides the associated relatively high cost and time consumption (Al-Obaily, 2017). Hu (1995) stated that it is practically impossible to replicate identical parameters in the laboratory model as those in the prototype. If the stress levels similar to those in the prototype were to be attained and maintained, Schofield (1980), as cited by Hu (1995), proposed the use of a geotechnical centrifuge.

Ashour (2015) acknowledged that an appropriate factor should ideally be utilised to scale field situations for laboratory testing. Nevertheless, he claimed that it was impossible to maintain a scale factor in his physical model whereby all the governing parameters in the small scale are identical to those in the prototype. For tests conducted on sand columns installed in a clay, Hu (1995) adopted a dimensional analysis approach and provided a list of such parameters which he divided into 2 categories namely dominant variables and less significant quantities. The dominant variables included the following parameters: penetration of footing, diameter of footing (D_f), length of column (L_c), diameter of column (D_c), area replacement ratio (a_s), angle of internal friction for column material (ϕ'), elastic shear modulus of sand and the elastic shear modulus of clay. In contrast, these given parameters (when used in form of dimensionless quantities) were defined as less significant in terms of their effect on the load-settlement response: elastic shear modulus of sand, undrained shear stress of clay, unit weight of sand, unit weight of clay and average particle size of column material. Based on this analysis, Hu (1995) highlighted some dimensionless parameters which he satisfied in his laboratory model. These were as follows: L_c/D_c, L/D_f, a_s and ϕ'.

Miller (2002) claimed that there are limits to the similarities which can be achieved between a laboratory scaled model and a prototype constructed at full scale. According to Hughes & Withers (1974), the L_c/D_c ratio may be as low as 4 for a single column. This condition was considered in this study whereby the length to diameter ratio of the column was 4. In terms of the area replacement ratio, Barksdale & Bachus (1983) suggested that a range of between 10 to 35 % was commonly used in the field. Based on the diameters of the column and the testing tank adopted, the area replacement ratio was 11 % in the bench scale model. Dimensions selected for the tanks and column were discussed earlier in section 3.2.1. Overall, 2 dimensionless quantities (L_c/D_c and a_s) were adequately scaled down in this research. Due to the nature of the work whereby waste was used within the column, it was impossible to provide a scale between the laboratory and the field model; the selection of sizes and quantities of the reinforcement were principally based on previous studies which were presented in the literature review. As such, scale effects are expected in this regard. The results generated are anticipated to be used in further studies (especially in field tests) to account for such discrepancies.

3.4.4 Processing of experimental data

3.4.4.1 Test data

At the completion of a test, the test file was saved on the computer which was connected to the loading machine and the data was subsequently exported to an excel file for processing. The results obtained displayed the relationship between the applied load and the settlement. To generate the stress-settlement characteristics, this load was converted to the corresponding stress by the following equation:

$$Stress = \frac{Applied\ load}{Cross\text{−}sectional\ area\ of\ the\ loading\ plate} \qquad \text{(Equation 3.2)}$$

From the generated data, graphs were plotted, several calculations were performed, and different relationships were established. These are presented in Chapter 4, whereby descriptions are provided for the respective calculations required to produce each graph.

3.4.4.2 Post-testing information

While the stress-settlement characteristic for each test was electronically recorded, information pertaining to the column deformation behaviour was attained through the physical model (in the form of a column) which was casted with plaster of Paris. Using a measuring tape, the diameter and the height were measured at several intervals up along each plaster of Paris column. These measurements were taken immediately after the column formation to minimise any effect of shrinkage due to further drying of the plaster. The dimensions were then used to produce scaled drawings representing the column deformation, shown in Chapter 4. These illustrations aimed at identifying the largest bulge achieved in each test and the position at which it occurred along the column. The information facilitated the comparison of column distortions arising from the effect of the type of reinforcement used.

3.5 Summary

This chapter elaborated on the methodology adopted throughout this investigation. Throughout the study, a single type of base material (Durbanville silt) was used at 2 different degrees of wetness, OMC and LL. While the columns were each made of only one type of granular material (Cape Flats sand), 4 different types of reinforcements were used individually to further enhance the load-settlement characteristics of the improved silt bed. These were waste PET flakes, recycled PET fibres, and 2 types of geotextiles whereby one was manufactured from recycled PET and the other from virgin PET. The properties of each of these materials have been given. Besides the materials, the parameters governing the design and fabrication of the testing tank was also explained. A detailed procedure of the experimental set-up and testing was presented, in addition to the specification for the acquisition of data. Tables 3.5 and 3.6 summarise the experiments conducted to achieve the aims of this research. The masses of sand and reinforcement for each test are given together with their respective number of layers required to form the columns.

Table 3.5: Summary of tests conducted on columns with randomly mixed reinforcement

Moisture content	Test specimen	Column materials	Test code	Total mass of sand (g)	Total mass of reinforcement (g)	No of layers of sand containing randomly mixed reinforcement
Random Mixing						
OMC	Clay	N/A	M1	N/A	N/A	N/A
	Clay-Sand	Sand	M1-S	6756	0	7 + top up
	Clay-Sand-Plastic	Sand-Flakes	M1-S-RP0.5%	6656	33	7 + top up
			M1-S-RP1.0%	6014	60	7 + top up
			M1-S-RP2.5%	6371	163	7 + top up
	Clay-Sand-Fibre	Sand-Fibre	M1-S-RF0.025%	6300	1.61	7
			M1-S-RF0.05%	6035	3.05	6 + top up
			M1-S-RF0.1%	6266	6	6 + top up
LL	Clay	N/A	M2	N/A	N/A	
	Clay-Sand	Sand	M2-S	7453	0	8 + top up
	Clay-Sand-Plastic	Sand-Flakes	M2-S-RP0.5%	7047	35	7 + top up
			M2-S-RP1.0%	7345	74	8 + top up
			M2-S-RP2.5%	6766	174	7 + top up
	Clay-Sand-Fibre	Sand-Fibre	M2-S-RF0.025%	7605	1.9	8 + top up
			M2-S-RF0.05%	7428	3.7	8 + top up
			M2-S-RF0.1%	7361	7.4	8 + top up

Table 3.6: Summary of tests conducted on columns with layers of reinforcement

Moisture content	Test specimen	Column materials	Test code	Total mass of sand (g)	Total mass of reinforcement (g)	No of layers of sand	No of layers of reinforcement
Layering							
OMC	Clay-Sand-Plastic	Sand-Flakes	M1-S-LP2.2%	5961	120	6 + top up	6
			M1-S-LP3.3%	5790	180	6 + top up	6
			M1-S-LP5.6%	5099	250	5 + top up	5
	Clay-Sand-Fibre	Sand-Fibre	M1-S-LF0.28%	5707	15	6 + top up	6
			M1-S-LF0.56%	5546	25	6 + top up	5
			M1-S-LF0.83%	5164	37.5	5 + top up	5
	Clay-Sand-Betatex	Sand-Betatex	M1-S-LGW200	6066	8.7	6 + top up	6
			M1-S-LGW400	5950	17.34	6 + top up	6
			M1-S-LGW600	5741	26.04	6 + top up	6
	Clay-Sand-Fibretex	Sand-Fibretex	M1-S-LGV200	6061	8.7	6 + top up	6
			M1-S-LGV400	5772	17.34	6 + top up	6
			M1-S-LGV600	5753	26.04	6 + top up	6
LL	Clay-Sand-Plastic	Sand-Flakes	M2-S-LP2.2%	7192	140	8 + top up	7
			M2-S-LP3.3%	6120	180	7	6
			M2-S-LP5.6%	5522	300	6	6
	Clay-Sand-Fibre	Sand-Fibre	M2-S-LF0.28%	6809	15	7 + top up	6
			M2-S-LF0.56%	6353	27	7 + top up	6
			M2-S-LF0.83%	5365	45	6 + top up	6
	Clay-Sand-Betatex	Sand-Betatex	M2-S-LGW200	6621	8.7	7 + top up	6
			M2-S-LGW400	6234	17.34	7 + top up	6
			M2-S-LGW600	6042	26.04	7 + top up	6
	Clay-Sand-Fibretex	Sand-Fibretex	M2-S-LGV200	6487	8.7	7 + top up	6
			M2-S-LGV400	6242	17.34	7 + top up	6
			M2-S-LGV600	6123	26.04	7 + top up	6

Chapter

4

Results

4.1 Introduction

Two main aspects were considered as the outcome of the testing programme, namely: (1) stress-settlement characteristics, and (2) column deformation post-testing. While the first one was electronically captured on a computer, the second one involved physically producing a three-dimensional model to illustrate how the columns deformed under the different testing conditions. Chapter 4 presents the results obtained from this study in detail. The test codes which have been used on the graphs were described earlier in Table 3.1 of Chapter 3. The number of layers of the sand and reinforcements were also given in the same chapter (Tables 3.5 and 3.6). Analysis and discussion of the results are subsequently presented in Chapter 5. The following chart provides the structure for the presentation of the results.

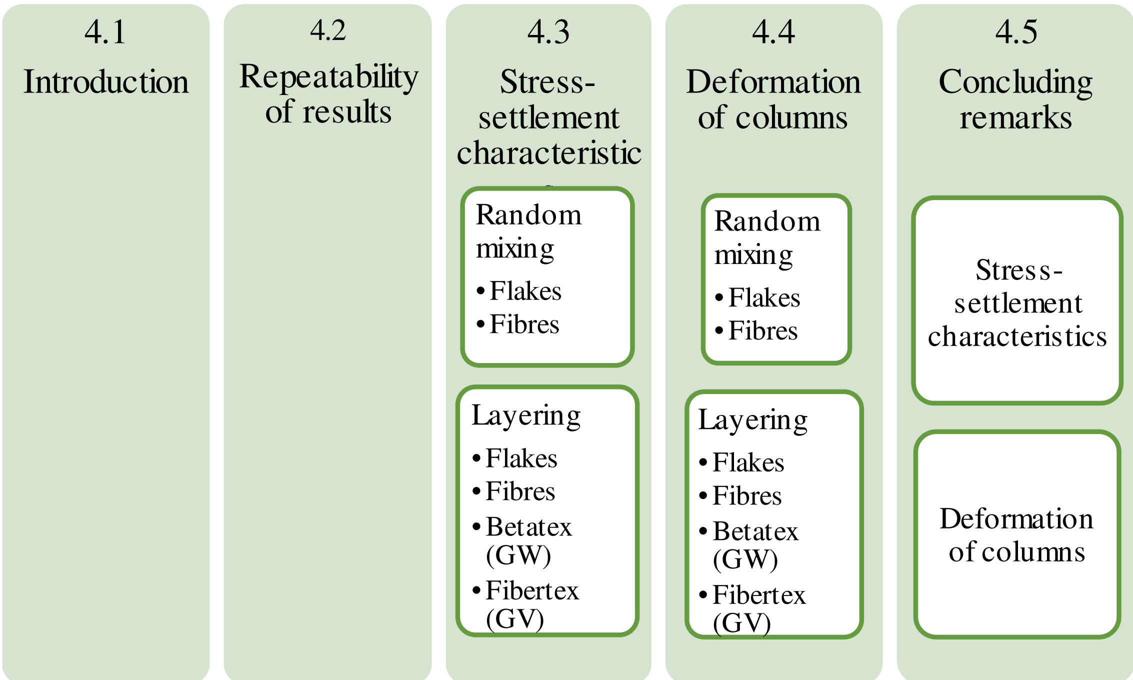


Figure 4.1: Structure followed for the presentation of the results in this chapter

4.2 Repeatability of results

Repeatability is a concept which has gained popularity for its importance in scientific experiments. It is commonly used to measure the precision of a test method, by conducting the same experiment multiple times. This measurement is obtained by applying the repeatability conditions which have been defined by the ASTM E177-14 as the *"conditions where independent test results are obtained with the same method on identical test items in the same laboratory by the same operator using the same equipment within short intervals of time"*.

According to the ASTM E177-14, repeatability comprises of 2 principal measures namely: repeatability standard deviation (S_r) and repeatability limit (r). The repeatability standard deviation measures the dispersion of the distribution of the test results which have been gathered under repeatability conditions and is essentially the standard deviation of these data. Under ideal conditions, the standard deviation should be zero. However, this is practically not achievable since there always exists a certain degree of variation, either in the material or in the sample preparation. Hence, the closer the standard deviation to zero, the more repeatable the testing results. The repeatability standard deviation is necessary to determine the repeatability limit which is given by the following equation:

$$r = 1.96\sqrt{2}\,S_r \approx 2.8S_r \qquad \text{(Equation 4.1)}$$

The repeatability limit is that value below which the absolute difference between two independent single tests results, which have been attained under repeatability conditions, may be expected to occur (ASTM E177-14). While this limit assumes a normal distribution of the data and compares only 2 independent readings, it is understood that 95 % of all pairs of the experimental results obtained under repeatability conditions will have an absolute difference which is lower than the repeatability limit. If a difference is larger than this limit, there is a 5 % chance that it might just be a random occurrence and not an actual difference in the testing procedure. The multiplier of 1.960 used in the equation is explained by the fact that 95 % of a normal distribution lies within 1.960 standard deviation of the mean. The testing procedures adopted was assessed in terms of repeatability only and not reproducibility since all the tests were conducted in the same laboratory and by the same operator. Therefore, the measurement of reproducibility was not required.

The repeatability relative standard deviation (RSD_r) is another simpler approach which is possibly more utilised with regards to the repeatability of data. This value is equal to the percentage ratio of the repeatability standard deviation to the mean. Since it is also derived from the standard deviation, it is also a measure of the variability of data. While an RSD_r of 5 % is relatively popular, it may also be lower than that or even higher. Usually, it is based on the acceptable level of variation in data, which is typically set within the laboratory. For the experiments conducted in this research, an RSD_r of 5 % was considered as the upper limit for which a data set was acceptable. Beyond this, it would imply that the repeatability of the testing procedure is low and therefore unacceptable. As such, a few trial experiments were conducted to refine the process that would yield high repeatability of the results.

Two main testing methods were followed to generate experimental data namely: (1) loading of the test specimen and (2) casting of the deformation of the columns. Therefore, a repeatability analysis was conducted on each procedure. The outcomes have been independently presented and discussed.

4.2.1 Measurement of the stress-settlement characteristics

Evaluation of the repeatability in terms of the stress-settlement characteristics generated were performed on 4 different types of tests (M2-S-RP0.5 %, M2-S-LGW200, M2-S-LGW400 and M2-S-LGW600). These experiments were randomly selected and each one was repeated 3 times, under repeatability conditions. Figure 4.2 presents the results for the stress-settlement characteristics obtained in all the tests. From the graphs, it appeared that the shapes of the curves were relatively consistent within each test series. No major discrepancy was observed. Therefore, in terms of the trends of the graphs, they appeared to be repeatable under identical conditions. For this reason, the repeatability analysis was entirely based on the maximum vertical applied stress which was attained at the completion of the respective tests, which corresponded to a settlement of 50 mm.

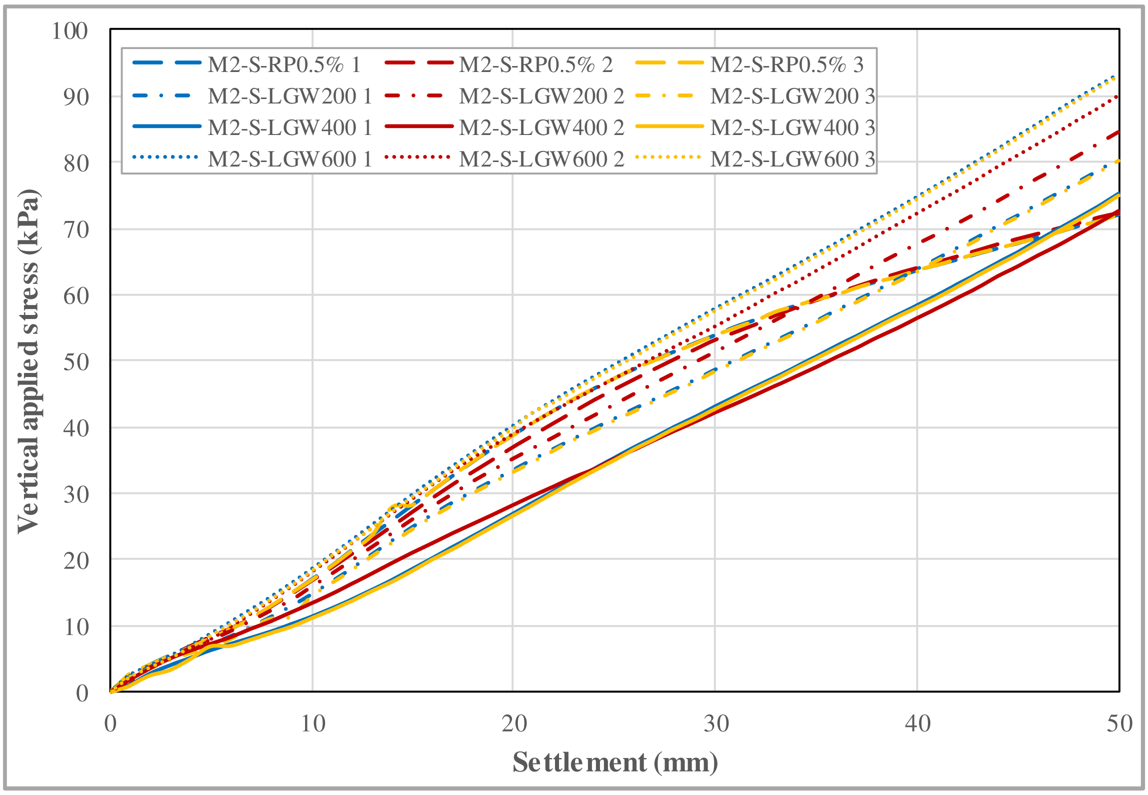


Figure 4.2: Repeatability stress-settlement results for 4 different test series

From Figure 4.2, the maximum vertical applied stress was captured for each test and these were used for the statistical analysis on repeatability of the testing procedure. Table 4.1 condenses the outcome of this analysis. Generally, the testing process appeared to have good repeatability since the standard deviation was reasonably low. Consequently, the percentage RSD_r was also low for all the test series, and as such they were all considered to be within the acceptable range of 5 %. Furthermore, when the computed repeatability limit for each test series was compared with the absolute difference between any pairs of the test results, 95 % of them had an absolute difference which was lower than this limit. From these findings, it was confirmed that the testing procedure followed in this research was highly repeatable.

Table 4.1: Statistical analysis of the 4 series of tests considered for repeatability (Stress-settlement results)

Test code	**M2-S-RP0.5 %**	**M2-S-LGW200**	**M2-S-LGW400**	**M2-S-LGW600**
Test data (kPa)				
Test 1 value	72.11	80.35	75.24	93.39
Test 2 value	72.26	84.60	72.68	90.22
Test 3 value	72.17	80.06	74.95	93.10
Mean (kPa)	72.18	81.67	74.29	92.24
Repeatability standard deviation, Sr	0.08	2.54	1.40	1.76
Repeatability relative standard deviation, RSD_r (%)	0.11	3.11	1.89	1.90
95 % Repeatability limit, $r = 2.8 \times S_r$	0.22	7.12	3.93	4.91

4.2.2 Measurement of the column deformation

Since the analysis of the deformation of the columns post-testing constituted a significant component in this research, it was compulsory to verify the repeatability of the test procedure followed to generate the columns. Therefore, 2 test series were randomly selected from those which were used in the repeatability analysis of the stress-settlement characteristics. These included the following tests: M2-S-RP0.5 % and M2-S-LGW400. Each of the tests was conducted 3 times, and a statistical analysis (similar to the one used in the previous section) was performed on the measured maximum bulging diameter which was acquired under repeatability conditions. Repeatability was also important with regards to the largest lateral deformation since the latter is considered as one of the most critical factors when predicting the failure of granular columns. Also, this diameter was one of the main laboratory outcomes of this investigation. Figure 4.3 presents the deformation of the columns for all the tests, while Table 4.2 summarises the findings of the statistical analysis. The detailed approach used to establish the largest bulge, is later described in section 4.4.

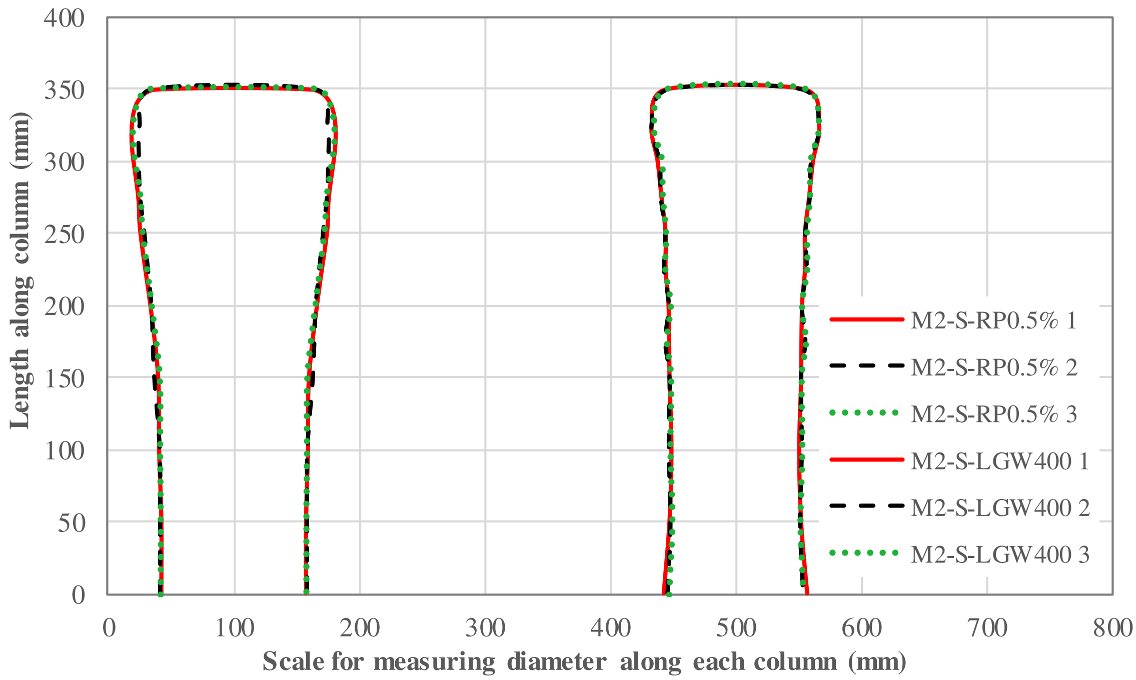


Figure 4.3: Repeatability column deformation results for 2 different test series (M2-S-RP and M2-S-LGW)

Table 4.2: Statistical analysis of the 2 series of tests considered for repeatability of the maximum bulging diameter (column deformation results)

Test Code	**M2-S-RP0.5 %**	**M2-S-LGW400**
Test data (mm)		
Test value 1	164	134
Test value 2	152	134
Test value 3	158	132
Mean (kPa)	158	133
standard deviation, Sr	6.00	1.15
Repeatability relative standard deviation, RSD_r (%)	3.80	0.87
95 % Repeatability limit, $r = 2.8 \times S_r$	16.80	3.23

From Table 4.2, it was evident that the method followed in generating these columns had a good repeatability. This was confirmed through the RSD_r values for both tests which were below the acceptable upper limit of 5 %. When both test series were compared, it was particularly noted that, although the RSD_r value was within the acceptable range for repeatability, it was significantly higher in the test which was performed on columns containing randomly mixed reinforcement. This larger variation was possibly due to the random mixing of the plastic, which did not guarantee a similar alignment of any one flake within the column for each test. Besides the repeatability relative standard deviation, the findings also indicated that 95 % of the absolute difference between any pairs of test results was lower than the repeatability limit, hence implying a good repeatability.

From the repeatability analysis performed in terms of both the maximum vertical applied stress and the largest bulging diameter, it was deduced that the procedures adopted to generate these outcomes were repeatable. Hence, although the remaining tests were performed a single time, their results were expected to be reliable.

4.3 Stress-settlement relationship of loaded granular columns

4.3.1 Presentation of experimental results

The stress-settlement figures presented in this section display the graphs for all the tests which were conducted. In all the figures, results have been repeatedly presented for both the unimproved and improved silt (with unreinforced sand column), besides those curves related to the different types, quantity and arrangement of the reinforcement. This was necessary to establish a point of reference for comparing the results achieved from each test. For any one figure, the quantity of the column reinforcement was varied while the following were kept constant: column length, (400 mm), column diameter (assumed as 100 mm), water content of base soil (OMC or LL), type of reinforcement (flakes, fibres, Betatex or Fibertex) and their arrangements (random mixing or layering). Since the behaviour of the columns differ at the 2 extremes of wetness of the base soil, the graphs for OMC and LL have been presented independently. Throughout the explanation of the results with regards to stress-settlement behaviour, the terms 'percentage improvement, improvement or enhancement' have been used. These, basically, refer to the percentage improvement achieved in the maximum vertical applied stress at a settlement of 50 mm (settlement based on Eurocode 7 for normal structures), and equation 4.2 was applied to determine this for each test:

$$\% \, improvement = \frac{S_{imp} - S_{unimp}}{S_{unimp}} \, x \, 100 \qquad \text{(Equation 4.2)}$$

where: S_{imp} = vertical applied stress at a settlement of 50 mm for an improved base soil

S_{unimp} = vertical applied stress at a settlement of 50 mm for an unimproved base soil

4.3.2 Random mixing

4.3.2.1 Flakes

Figure 4.4 presents the stress-settlement characteristics for OMC tests whereby the columns were randomly reinforced with PET flakes. From the plots, it was evident that each curve was smooth and followed the same trend, although the corresponding vertical applied stress differed at a maximum settlement of 50 mm for each one of them. Irrespective of the type of sand column used, that is unreinforced or reinforced, their inclusion in a random way generally produced an enhancement in the stress applied vertically to the test specimen at OMC, when compared to that of the pure silt bed (M1) at the same moisture content. For instance, the inclusion of a pure sand column (M1-S) resulted in a strength gain of 104 %, while that of the column with 0.5 % of flakes was found to be 109 %. In contrast, the other 2 tests on reinforced columns (concentrations of 1.0 and 2.5 %) showed a decline in the maximum vertical applied stress. Although slight variations in the maximum vertical stress was recorded, it was observed that the results for all the tests with sand columns was relatively closely spaced, with the respective percentage improvements ranging between 88 and 109 %.

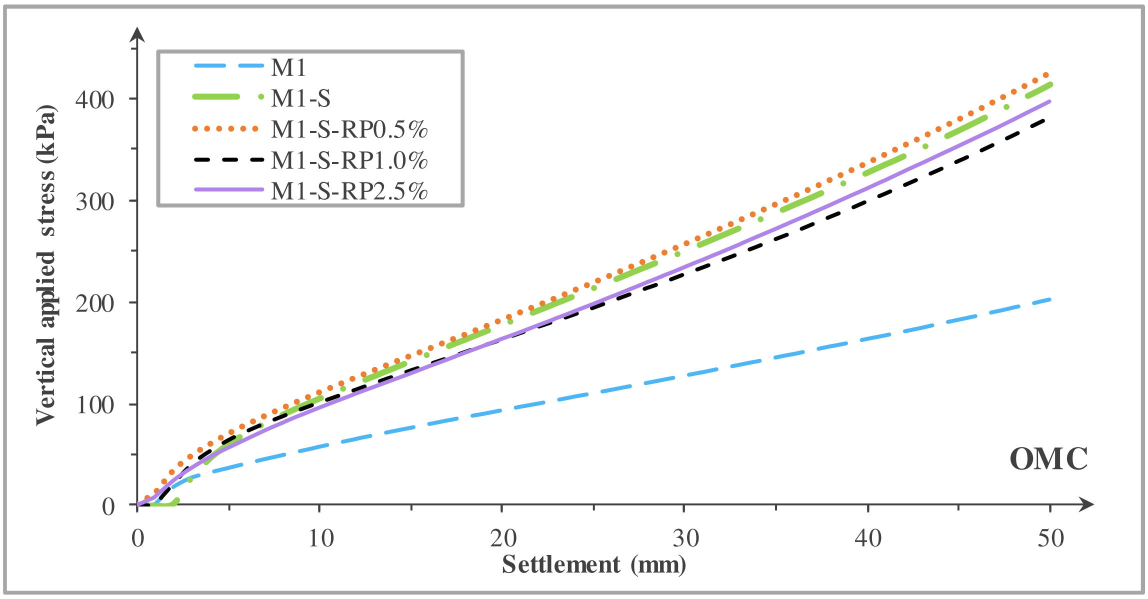


Figure 4.4: Stress-settlement relationship for tests conducted on a silt bed, improved by granular columns which were reinforced with randomly mixed PET flakes, at OMC

For the results of the LL tests, as shown in Figure 4.5, the shape of the curves differed slightly with regards to reinforced granular columns (RGC) such that they were steeper than those of the ordinary granular columns (OGC). It was also noted that the inclusion of the flakes, irrespective of their concentration, produced an increase in the maximum vertical applied stress. The corresponding percentage improvement was found to vary from 104 to 139 %, whereby a column which was internally reinforced by a flakes concentration of 0.5 % produced the highest performance. This is, in fact, almost doubled the improvement achieved with OGC when both types of columns are independently installed in a base soil at LL. From these observations, it was found that the percentage improvement in vertical applied stress decreased as the flakes concentration increased from 0.5 % to 2.5 %, when they were randomly mixed in the sand. Furthermore, it was also deduced that flakes inclusion resulted in better vertical loading strengths of the sand columns when installed in a base soil at LL, compared to that at OMC.

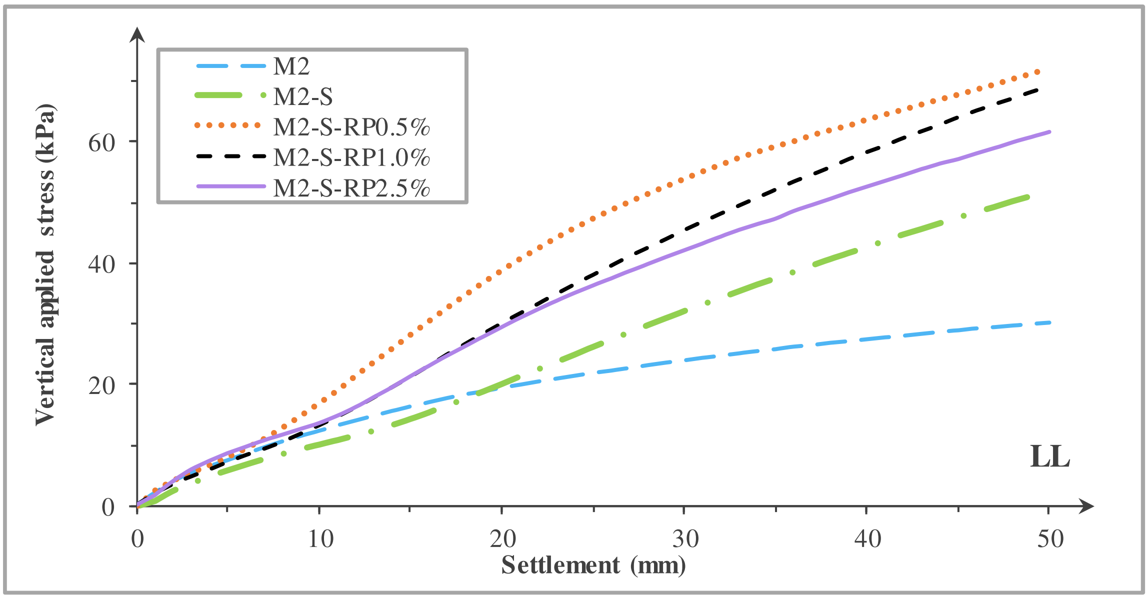


Figure 4.5: Stress-settlement relationship for tests conducted on a silt bed, improved by granular columns which were reinforced with randomly mixed PET flakes, at LL

4.3.2.2 Fibres

Figure 4.6 illustrates the stress-settlement characteristics of RGC with randomly mixed fibres, when installed in a base soil at OMC. The generated curves indicated that the inclusion of the fibres in the columns caused an increase in their load carrying capacity when compared to that of OGC. However, the lowest concentration (0.025 %) produced the highest improvement, which was calculated as 129 %. This quantitatively represented an extra gain of 25 % in vertical applied stress when compared to OGC. As the concentration of the fibres was amplified, a continuous drop in strength was recorded, with the highest concentration of 0.1 % producing an improvement of 111 %. Although this was the least performing column out of all the RGCs, it was still stronger than an OGC when tested under similar conditions.

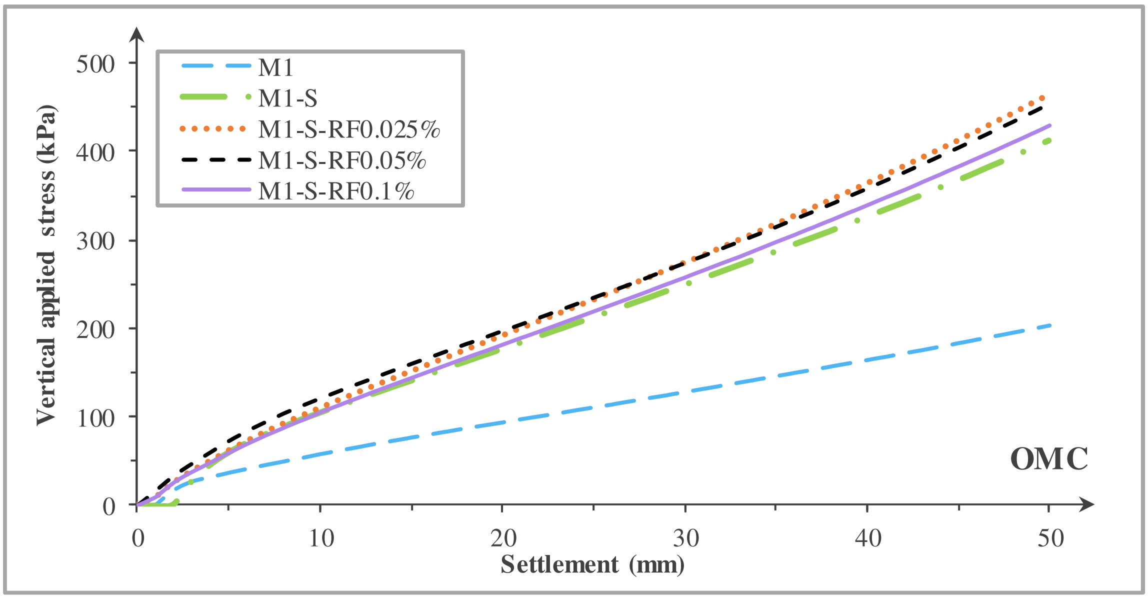


Figure 4.6: Stress-settlement relationship for tests conducted on a silt bed, improved by granular columns which were reinforced with randomly mixed PET fibres, at OMC

When the same type of RGCs were installed and tested in base soils at LL, their performance contrasted to those tested in base soils at OMC. This change in behaviour is demonstrated in Figure 4.7, where the curves clearly showed the distinct difference in maximum vertical applied stress at a settlement of 50 mm. It was generally found that an increase in the fibre concentration within the RGC produced an enhancement in the load carrying capacity of the column. More specifically, as the fibre content increased from 0.025 % to 0.1 %, the increase in vertical loading strength changed from 96 % to 244 %. Therefore, for these fibre concentrations which were investigated in a randomly mixed arrangement, 0.1 % produced the most significant gain in vertical loading strength, which was approximately equivalent to 3.4 times that of the OGC.

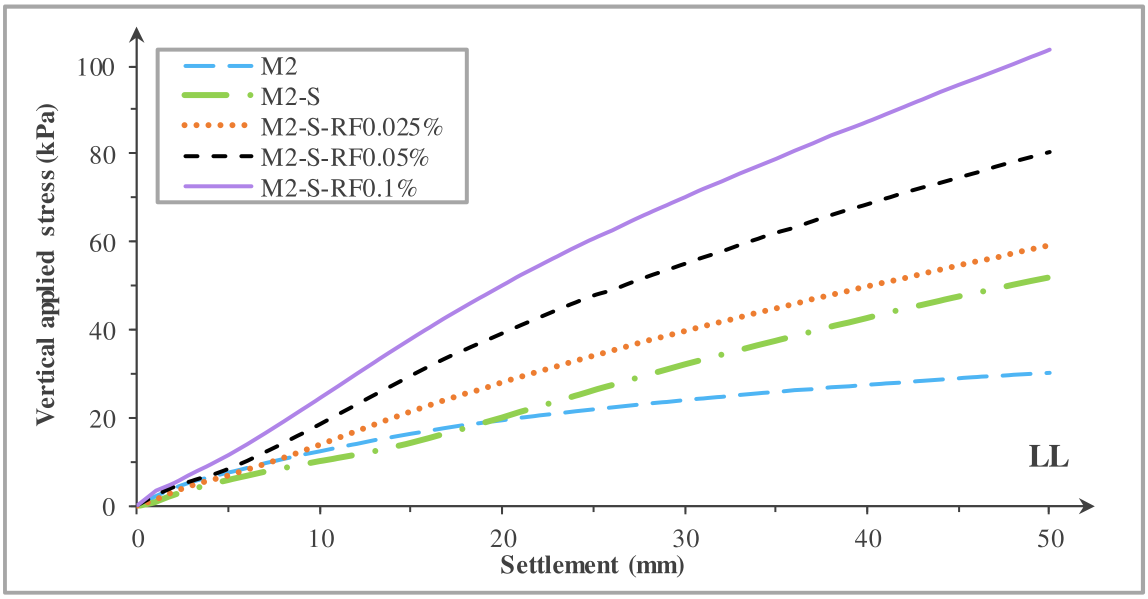


Figure 4.7: Stress-settlement relationship for tests conducted on a silt bed, improved by granular columns which were reinforced with randomly mixed PET fibres, at LL

4.3.3 Layering

4.3.3.1 Flakes

Figure 4.8 exemplifies the curve trends which were attained when columns reinforced with layers of flakes were installed in base soils at OMC. The RGCs (flakes content of 2.2 % and 5.6 %) mostly showed a decrease in the vertical applied stress at a settlement of 50 mm, when compared to the OGC. This represented an improvement in vertical loading strength of only 72 %. Generally, the OGC proved to be better performing under these conditions; where its percentage improvement in the load carrying capacity was calculated as 104 %, thus generating 32 % more strength than the weakest RGC. However, when a flake content of 3.3 % was utilised, this improvement was equal to 103 %, which was almost similar to that of an OGC.

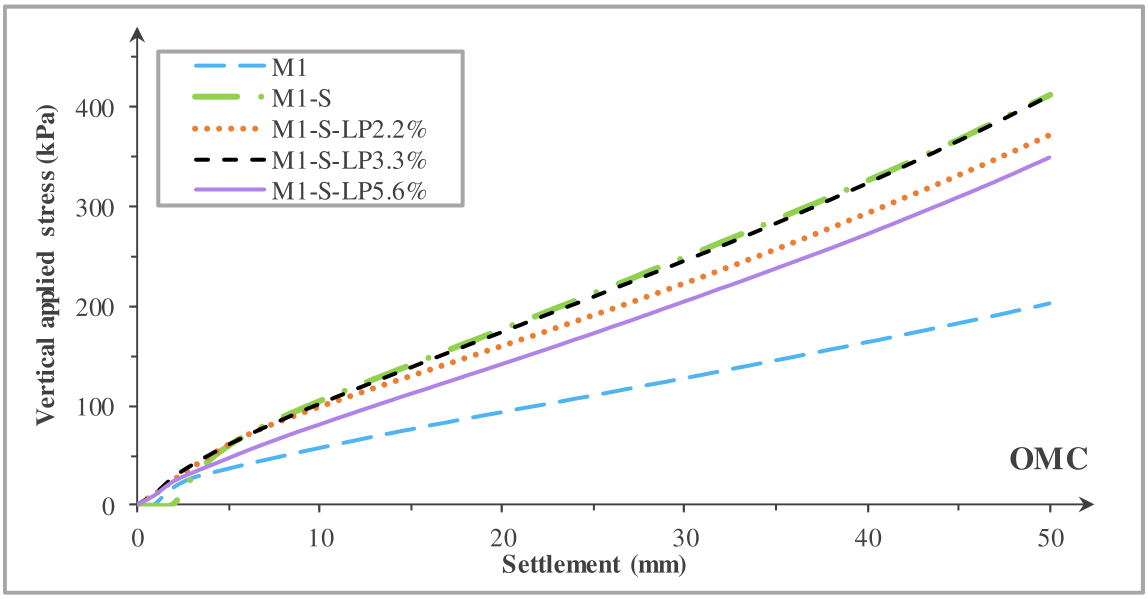


Figure 4.8: Stress-settlement relationship for tests conducted on a silt bed, improved by granular columns which were reinforced with layers of PET flakes, at OMC

In base soils prepared at LL, the inclusion of the flakes layers within the columns produced better strength gains than at OMC. This is illustrated in Figure 4.9, where the curves followed smooth paths. As the concentration of plastics increased from 2.2 to 5.6 % per layer, the gain in vertical applied stress (at a settlement of 50 mm) was found to rise from 76 to 143 %. These results proved that as the plastic content increased by a factor of 2.5, the improvement in load carrying capacity approximately doubled (1.9 times).

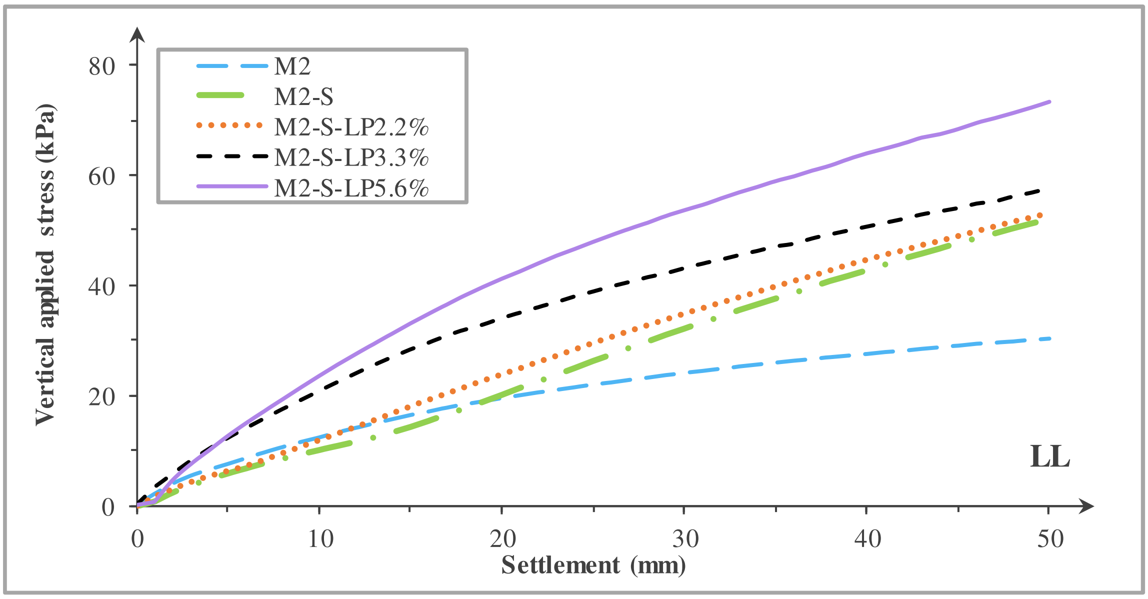


Figure 4.9: Stress-settlement relationship for tests conducted on a silt bed, improved by granular columns which were reinforced with layers of PET flakes, at LL

4.3.3.2 Fibres

Tests conducted with the layering arrangement of fibres within RGCs, which were installed in base soils at OMC, showed a general decrease in the maximum vertical applied stress when compared to that of the OGC. This observation is provided in Figure 4.10. In effect, as the fibre content was raised, a continuous subsequent drop in the load carrying capacity was recorded for a settlement of 50 mm, although a concentration of 0.28 % produced almost similar results to that of the OGC. While the improvement in loading strength of the OGC was calculated as 104 %, it was almost comparable to that of the RGC with 0.28 % of fibres, which was equal to 101 %. The lowest amelioration in strength was as low as 34 %, which was consequently due to the highest fibre content of 0.83 %.

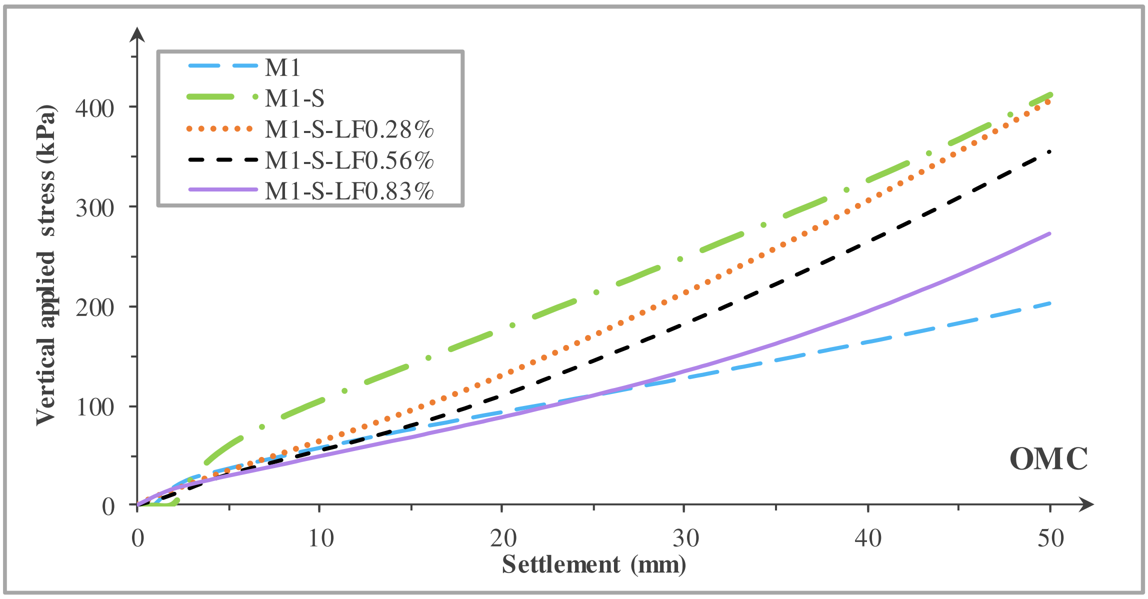


Figure 4.10: Stress-settlement relationship for tests conducted on a silt bed, improved by granular columns which were reinforced with layers of PET fibres, at OMC

In contrast to tests performed in base soils at OMC, the ones at LL demonstrated the opposite behaviour. From Figure 4.11, it is evident that the inclusion of the fibres in OGCs, to obtain RGCs, caused an enhancement in the maximum load carrying capacity. Initially, as the fibre was introduced at a concentration of 0.28 %, an augmentation of 93 % was recorded in the maximum vertical applied stress, when compared to that of the OGC which equated to 72 %. Thereafter, as the fibre content was increased from 0.28 to 0.56 %, a sharp gain in strength was detected which resulted in an improvement of 162 %. Further increase in the amount of the polymer did not provide higher load carrying capacities. In fact, a slight drop was observed which corresponded to 158 %. Therefore, the results confirmed that larger volumes of fibres under these testing conditions, beyond 0.56 %, did not necessarily enhance the column strength properties.

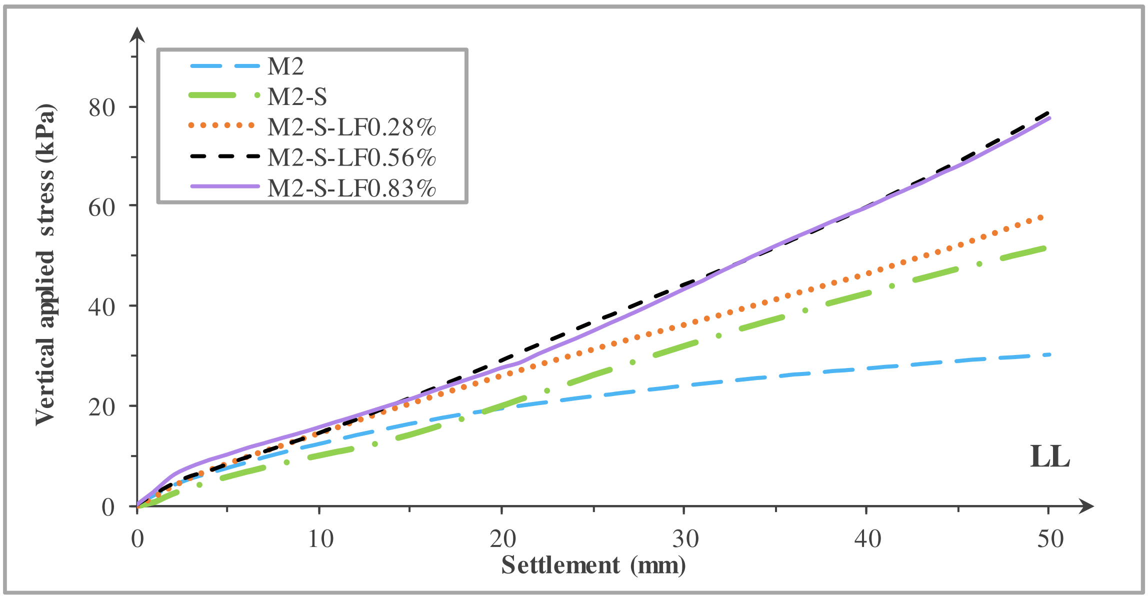


Figure 4.11: Stress-settlement relationship for tests conducted on a silt bed, improved by granular columns which were reinforced with layers of PET fibres, at LL

4.3.3.3 Betatex Geotextile (GW)

Figure 4.12 presents the stress-settlement characteristics for tests conducted on base soils at OMC, in which Betatex RGCs were installed. From the smooth trends of the curves, it is apparent that the inclusion of this geotextile typically resulted in an increase in load carrying capacity, with the GW400 providing the highest improvement in strength of 171 %. In comparison, the GW200 produced almost similar advancements in vertical loading strength and was found to be 169 %. Contrastively, as the mass per unit area increased beyond 400 g/m^2 and reached 600 g/m^2, the amelioration in maximum vertical applied stress declined to 119 %. This reflected an excess of 15 % when compared to that generated in the load carrying capacity of the OGC, which was calculated as 104 %, under similar testing conditions.

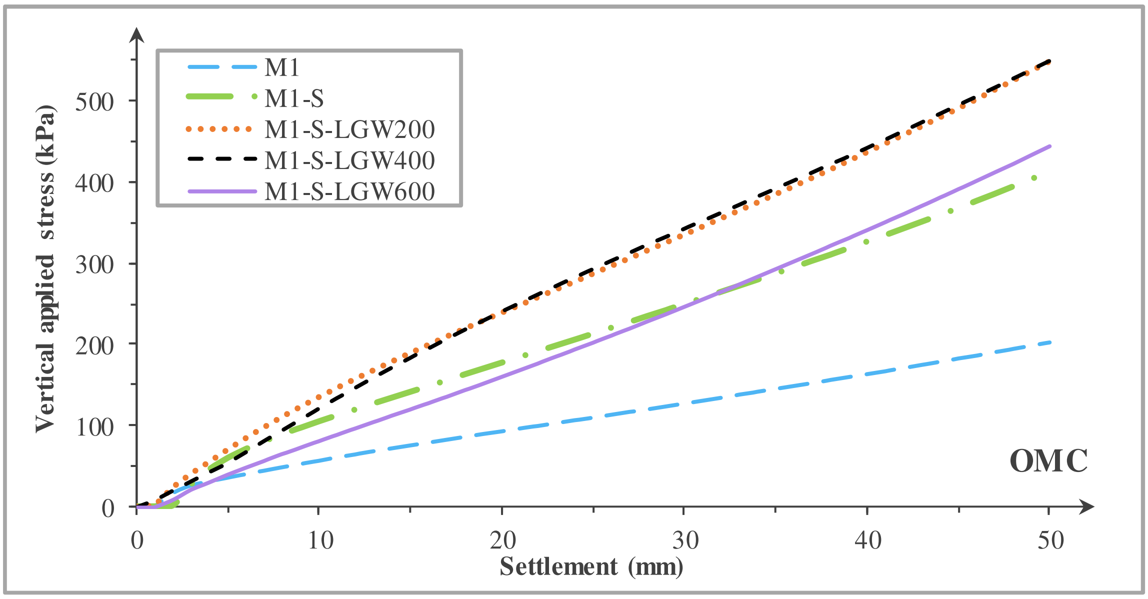


Figure 4.12: Stress-settlement relationship for tests conducted on a silt bed, improved by granular columns which were reinforced with layers of PET GW geotextile, at OMC

For the tests conducted in base soils at LL, the reinforcing of the columns by Betatex normally raised the maximum vertical applied stress of the columns, as shown in Figure 4.13. As the mass per unit area of the geotextile increased, a continuous elevation in the degree of improvement was also achieved, except for GW400 which appeared to be the weakest out of the 3 RGCs. Nevertheless, the column which was reinforced by GW400 was still stronger than the OGC, whereby their respective enhancements in load carrying capacity were 149 % and 72 %. This confirmed that even the weakest geotextile RGCs produce approximately 2 times the strength of an OGC when installed in a base soil at LL. Out of all the tests performed on Betatex RGCs, under these conditions, the column reinforced with GW600 was the strongest, with a gain in strength being equivalent to 210 %. Therefore, this column was almost 3 times stronger than the OGC.

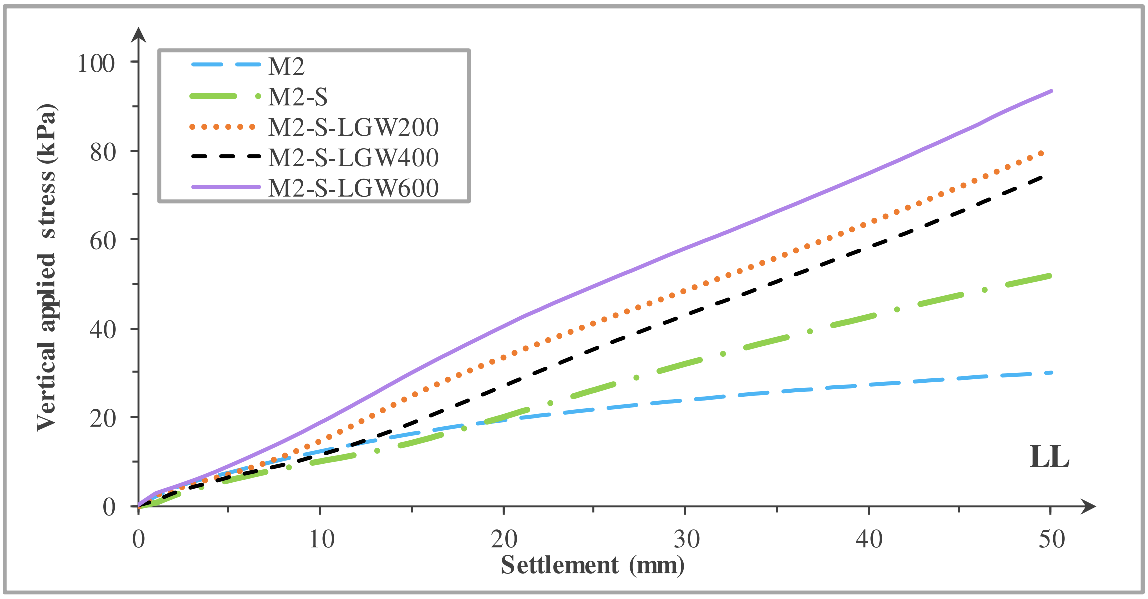


Figure 4.13: Stress-settlement relationship for tests conducted on a silt bed, improved by granular columns which were reinforced with layers of PET GW geotextile, at LL

4.3.3.4 Fibertex Geotextile (GV)

Figure 4.14 displays the relationships obtained between stress and settlement, when base soils at OMC were improved with Fibertex RGCs. From the smooth curves, it is clear that the addition of the GV geotextile to any pure sand column caused an amelioration in the maximum vertical applied stress, whereby GV200 was the best performing material with an improvement of 150 %. In contrast, GV400 and GV600 generated lower gain in stresses of 133 and 136 %, respectively. Actually, these 2 geotextiles produced almost similar results. Nevertheless, they were still stronger than an OGC which improved the vertical loading strength properties of the base soils by only 104 %.

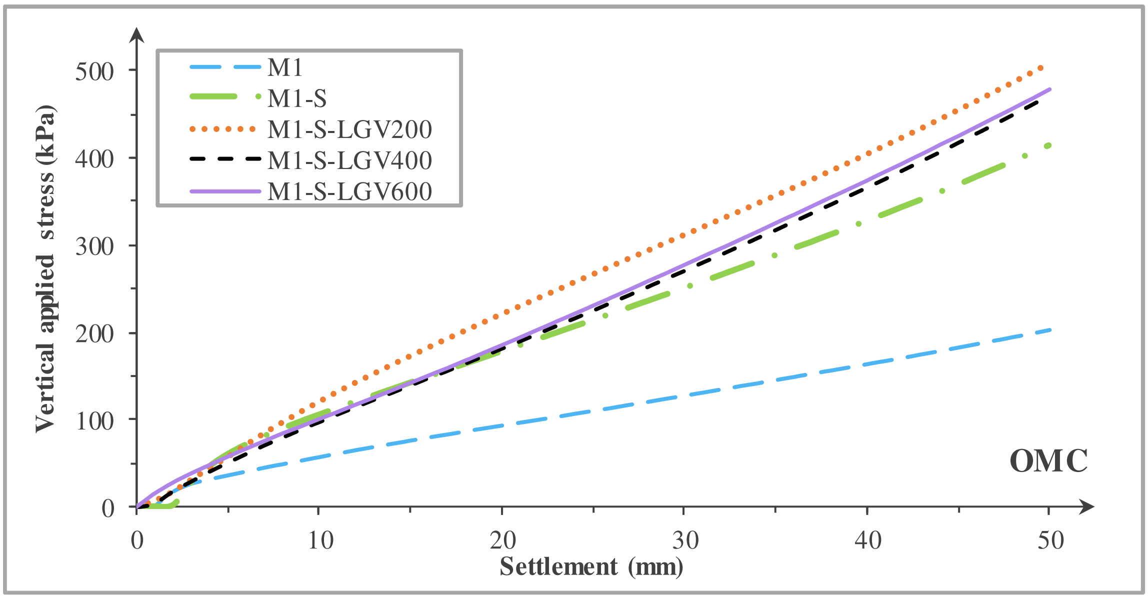


Figure 4.14: Stress-settlement relationship for tests conducted on a silt bed, improved by granular columns which were reinforced with layers of PET GV geotextile, at OMC

In comparison to tests performed on improved base soils at OMC, those at LL exhibited different behaviours, and are shown in Figure 4.15. For instance, GV600 was found to be the highest performing material under these conditions, with the load carrying capacity augmenting by 188 % compared to that of the OGC being 72 %. This showed that the inclusion of the GV600 geotextile in the layering arrangement, within a base soil at LL, produced 2.6 times the enhancement achieved through an OGC. With regards to GV200 and GV400, the respective gain in strength of 145 and 147 %, were almost similar with a small difference of only 2 %. However, they remained stronger than the OGC. The results confirmed that the percentage improvement achieved was impacted by the mass per unit area of the geotextile. As this mass was increased from 200 to 600 g/m^2, a gradual rise in the maximum vertical applied stress was recorded.

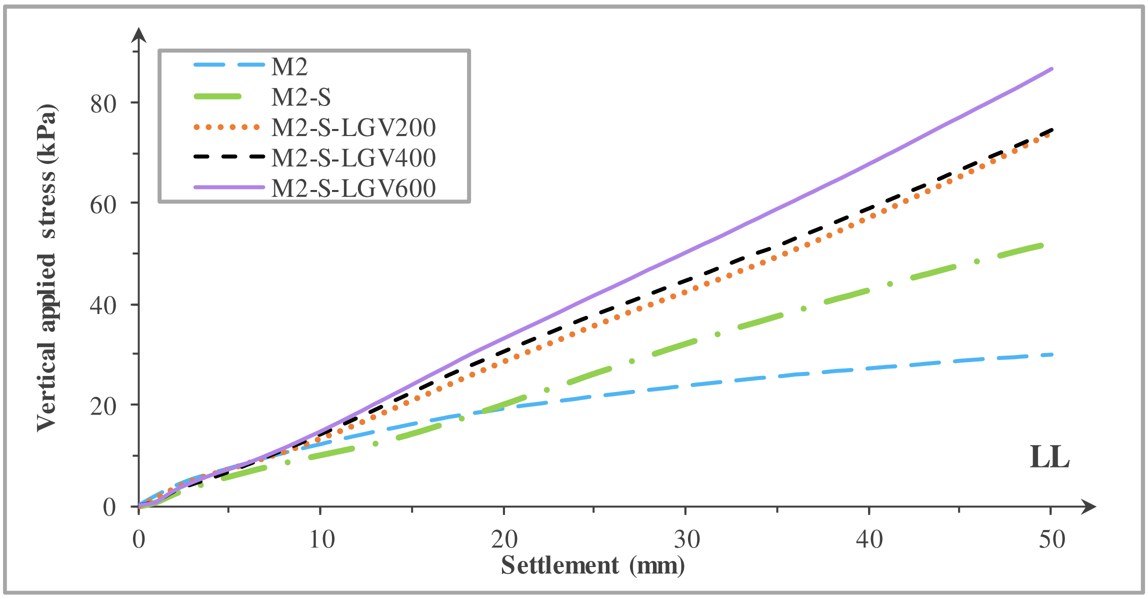
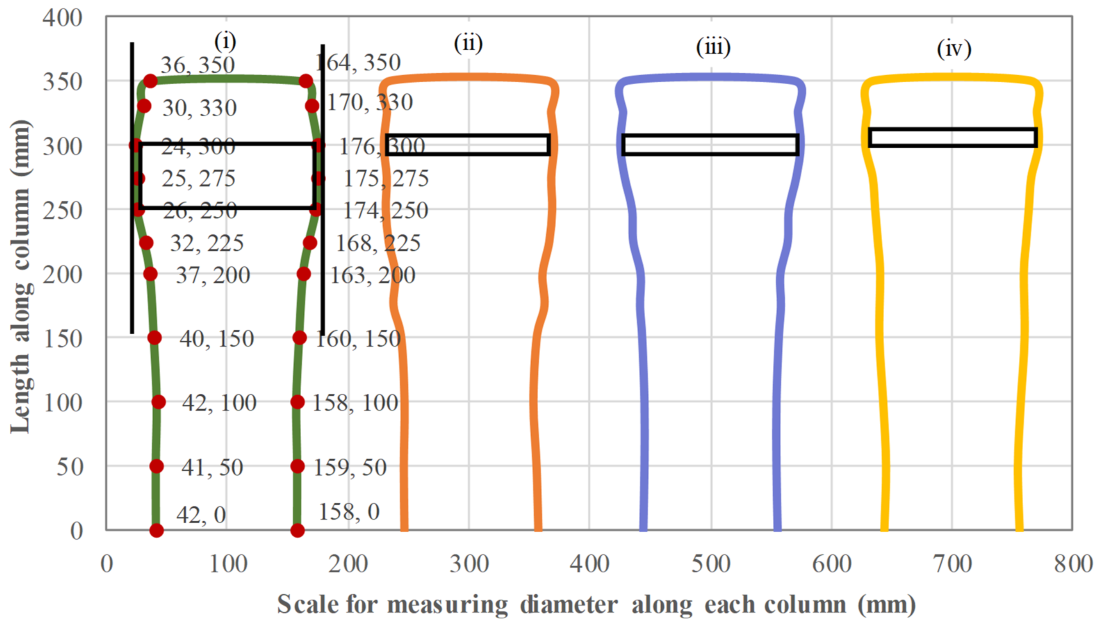


Figure 4.15: Stress-settlement relationship for tests conducted on a silt bed, improved by granular columns which were reinforced with layers of PET GV geotextile, at LL

4.4 Deformations of the tested columns

4.4.1 Processing of laboratory data

Post testing, the deformation of a column was achieved by physically modelling it; this was done by firstly emptying the column material and then filling it with a wet mix of plaster of Paris and sand. After casting, the mixture was allowed to set such that the deformed shape of the column was attained. Measurements of diameters and their corresponding positions along the casted column were manually taken using a measuring tape. These values were subsequently processed to graphically express the column shape post testing. This was necessary to accurately determine the maximum bulging diameter (D_B) which occurred in each tested column, as they were loaded up to a settlement of 50 mm. It further helped to observe the length, from the bottom of the column, to establish the span over which maximum bulging occurred. To establish these 2 values for each test, the diameter and length measurements taken at different intervals along the physical model were used to generate curves in an Excel spreadsheet. The column in Figure 4.16a(i) illustrates how a coordinate system was established such that the column was plotted as a single curve, by joining 22 data points (shown as the red dots), to demarcate the external surface of the column in two-dimension.



(a)

(b)

Figure 4.16: An illustration of the graphical plot system used in excel to establish the deformation of each column

In the graphical representation, a symmetrical deformation was considered for ease of plotting. This assumption was validated by the fact that maximum bulging would still be located at the same position, irrespective of any asymmetrical behaviour which might have happened during lateral deformations. Nevertheless, a pictorial representation of the physical model casted after each test was additionally presented beneath the graphical plot, when the results were explained. This was necessary to confirm that bulging did not excessively occur on any one side, since this could possibly indicate eccentricity of the vertically applied load. Generally, granular columns do not undergo perfectly symmetrical deformations since there are several factors in the ground which influence this behaviour. This occurrence is clearly demonstrated in the pictorial illustrations which are presented later in this section. Therefore, a picture is not adequate to determine the maximum bulging represented through the casted column since the latter is only represented 2-dimensionally. Hence, by assuming a symmetrical deformation for each column, and using the measured circumference of the actual physical model, the maximum bulging diameter for the respective column can be determined. This information can then be utilised for graphically establishing the largest bulge which was obtained in each test.

To reproduce the physical models of the deformed tested columns graphically, the starting point of the centre line from the bottom for each graphical column was carefully chosen along the x-axis (100, 300, 500 and 700 mm) to avoid overlapping while plotting them as a curve. From this centre line, the data points were obtained by subtracting the different radii from the x-value of the centreline, to obtain each of the left coordinates of the column. To determine the coordinates for the right side, the corresponding radii were added to the centreline value. The y-value of the coordinate was basically the length measured along the model, at the maximum radius on that point. For example, at a length of 50 mm along the first column (measured from the bottom of the column and along the y-axis of the graph), the radius was calculated to be 59 mm based on the measured circumference at this position along the modelled column, while the centreline was chosen at 100 mm on the x-axis. The left coordinate was determined as (41,50), while the right coordinate was (159, 50). This same procedure was applied to determine all the data points, for a two-dimensional representation, and they were joined by means of a curve to produce the outline of the physical model. From this, 2 vertical lines (shown in Figure 4.16a(i)) were drawn on each side of the column to establish maximum bulging, which was mathematically defined to be at the points where the curve just touched the lines. A block was then drawn within the graphical plot to demarcate the length span within which the highest bulging occurred. Since the study required comparison of the deformation characteristics for each test series, 4 graphical plots were produced to represent firstly the OGC as shown in Figure 4.16a(i)) under the specified testing conditions; the other 3 RGCs within that particular series were subsequently plotted on the same graph. Section 4.4, therefore, presents all the deformation curves for each physical model produced from the testing programme. The results for the tests conducted on base soils at OMC and LL, have purposely been presented independently since the associated behaviours of the columns differ significantly. Moreover, it was clearer to assess the degree of improvement achieved for each of the degree of wetness of the base soils.

4.4.2 Random mixing

4.4.2.1 Flakes

Figure 4.17 presents the lateral deformation patterns followed by the RGCs when they were reinforced with randomly mixed flakes, and the columns tested in base soils at OMC. In the figure, a flakes concentration of 0 % implied that it was an OGC. From the graphical plot of the physical model, it was seen that it's maximum bulging was 128 mm in diameter with the length span of bulging occurring between 235 and 275 mm. When flakes were introduced in the sand columns, the extent to which lateral expansion occurred changed such that 0.5 % and 2.5 % of flakes produced more bulging, which was determined as diameters of 135 and 137 mm, respectively. These represented corresponding increases of 5.5 % and 7 % in the diameter of the bulge. In contrast, the column with flakes content of 1 % demonstrated lower bulging, which was calculated as 125 mm; therefore, the bulge reduced by 2.3 %. While the maximum deformation differed in each testing conditions, the corresponding location also changed along the column. A concentration of 0.5 % raised the bulging to the highest position (between 270 to 285 mm) compared to 1.0 % flakes occupying the lowest position (235 to 250 mm). In the pictorial illustration in Figure 4.17(b), the graphical results obtained are evident. The column with 1.0 % flakes confirmed the smallest horizontal expansion.

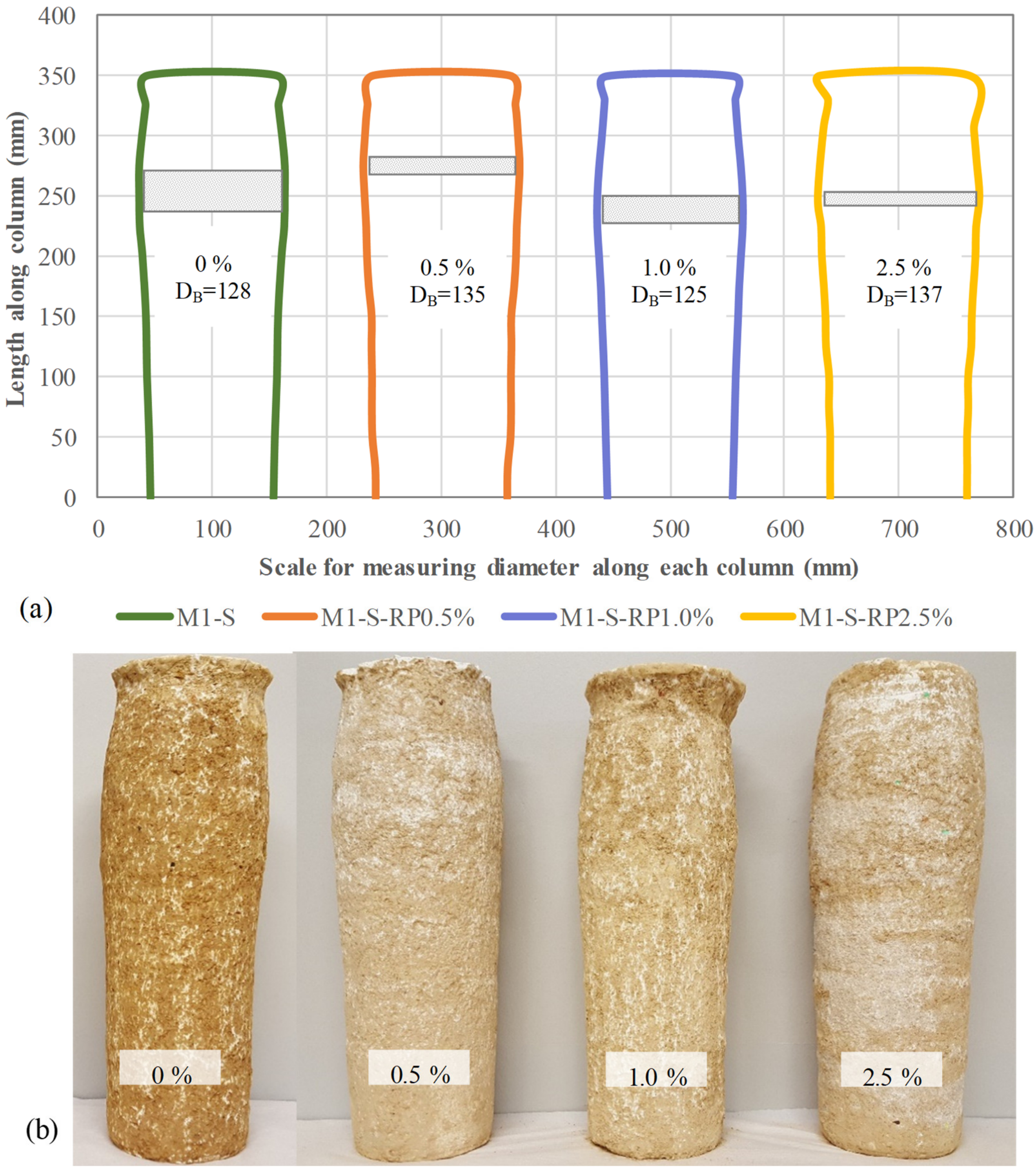
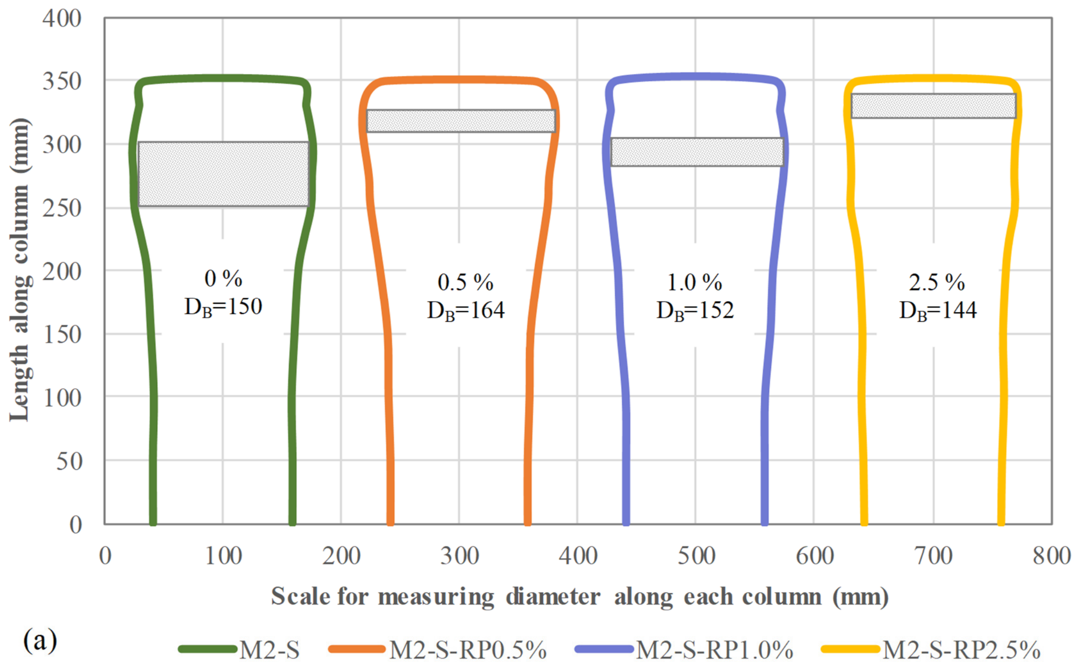


Figure 4.17: Deformations of tested columns reinforced with randomly mixed PET flakes, and installed in a base silt at OMC, (a) graphical plot and (b) pictorial illustration

Comparatively, for tests conducted in base soils at LL and with these types of polymeric arrangements, the deformation achieved in each column was remarkably different. The OGC, which is represented by 0 % of flakes in Figure 4.18, underwent a maximum deformation of 150 mm in diameter, within a length span of 250 and 300 mm. By introducing the randomly mixed flakes, this span reduced in length and was also located higher up in the column. In other words, maximum bulging occurred over a much shorter length in the RGCs. While a flakes content of 2.5 % produced the largest bulge at the highest position within the column

(325 to 340 mm), the 1.0 % of flakes resulted in the lowest placement (285 to 305 mm) of the highest bulging zone. Overall, when the RGCs were compared with the OGC, it appeared that the addition of the reinforcement reduced the length span over which the widest lateral spread occurred. In terms of the bulge size, at the lower concentration of flakes of 0.5 and 1.0 %, the diameter increased from 150 mm to 164 and 152 mm, respectively. However, when the content of the polymer was increased to 2.5 %, a drop in that dimension was recorded. This was equivalent to 144 mm and signified a reduction of 4 %, when compared to that of the OGC.



(a)

(b)

Figure 4.18: Deformations of tested columns reinforced with randomly mixed PET flakes, and installed in a base silt at LL, (a) graphical plot and (b) pictorial illustration

4.4.2.2 Fibres

In Figure 4.19, the deformation characteristics of the OGC and the fibre RGCs, tested in base silts at OMC are given. From the forms of the columns, in both the graphical and pictorial representations, it was evident that the deformation shape of the columns were relatively consistent. In fact, even the diameter of the columns was almost the same, and ranged between 126 and 130 mm. It was, therefore, confirmed that the inclusion of randomly placed fibres in these columns did not have any substantial effect on the size of the largest bulge. With regards to the position and length span of the bulging zone, an addition of 0.025 % of fibres did not show any remarkable effect. However, when the columns were reinforced with 0.05 and 0.1 % of fibres, this length and position of the zone of bulging changed from between 235 and 275 mm to 248 and 273 mm, and 230 and 260 mm, respectively. The length span in the OGC produced by the column with 0.05 % of fibres was shorter while the longest was noted in the 0.1 % fibre RGC.

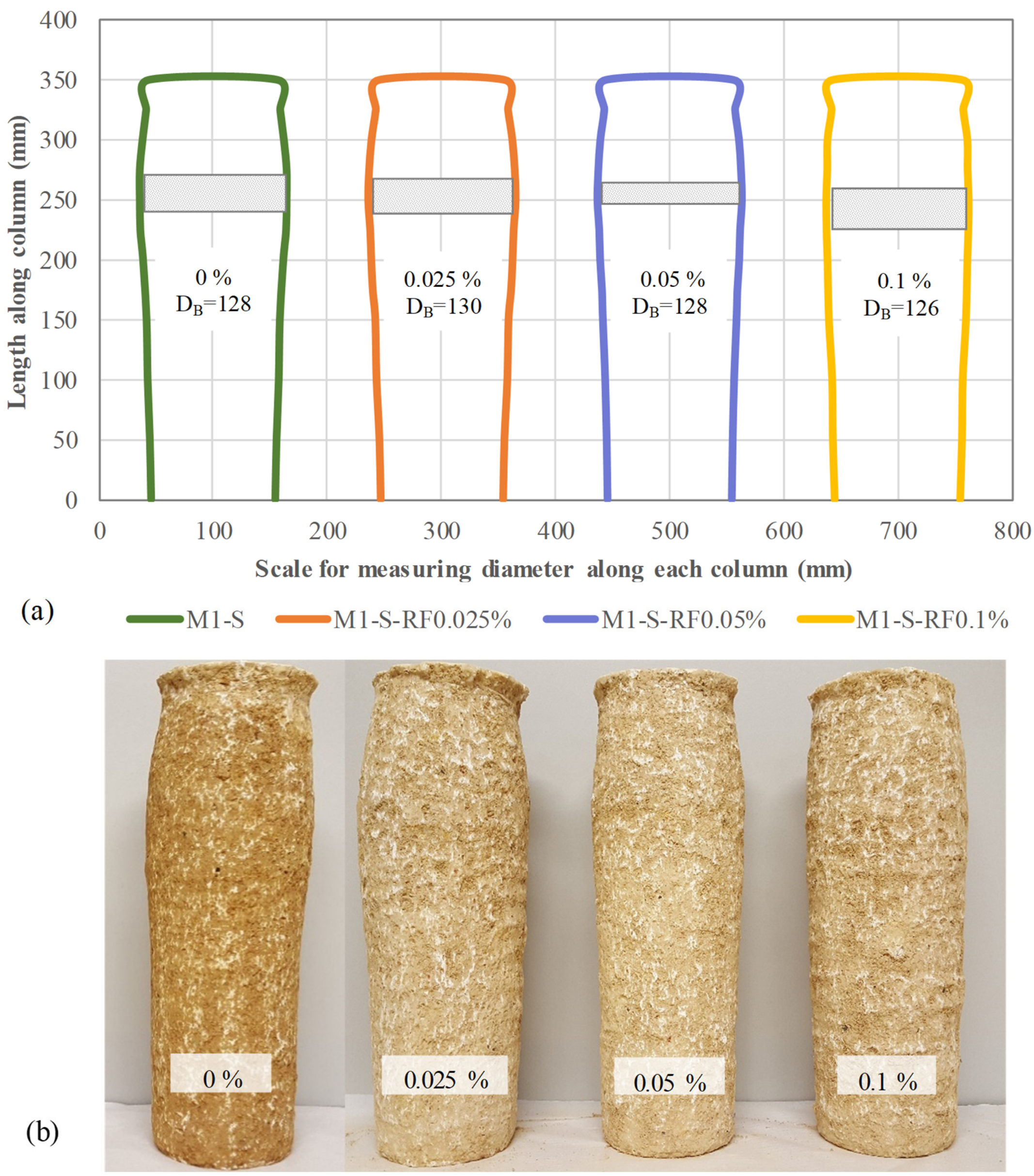


Figure 4.19: Deformations of tested columns reinforced with randomly mixed PET fibres, and installed in a base silt at OMC, (a) graphical plot and (b) pictorial illustration

For RGCs with randomly mixed fibres, and installed within a base soil at LL, the deformation of the columns was rather inconsistent. This is clearly depicted in Figure 4.20, whereby each column displayed differences in their shapes. Despite this irregularity, the maximum lateral expansion appeared to be lower in the RGCs than in the OGC, with a fibre concentration of 0.025 % producing the smallest bulge of 140 mm. In comparison to that of the OGC, this signified a reduction of approximately 7 %. However, as the fibre content increased from 0.025 % to 0.05 %, the column underwent higher bulging which was measured as 148 mm.

Beyond this, a further increase in the amount of fibre to 0.1 % resulted in a drop in the size of the bulge. In terms of the length span of the bulge and its position, all the RGCs shown in Figure 4.20 produced shorter spans of bulging and were all located nearly around the same position, between 295 and 310 mm.

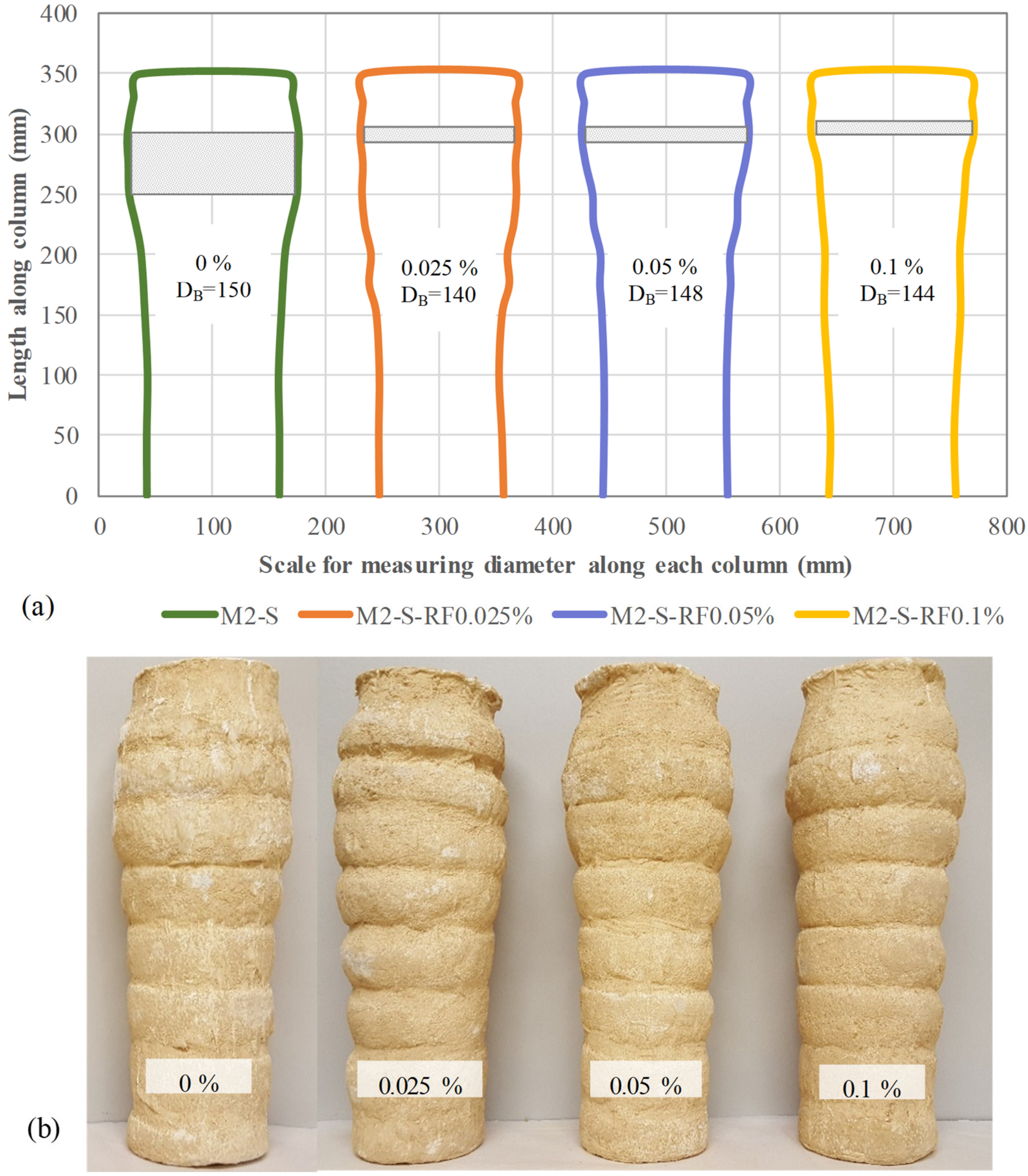


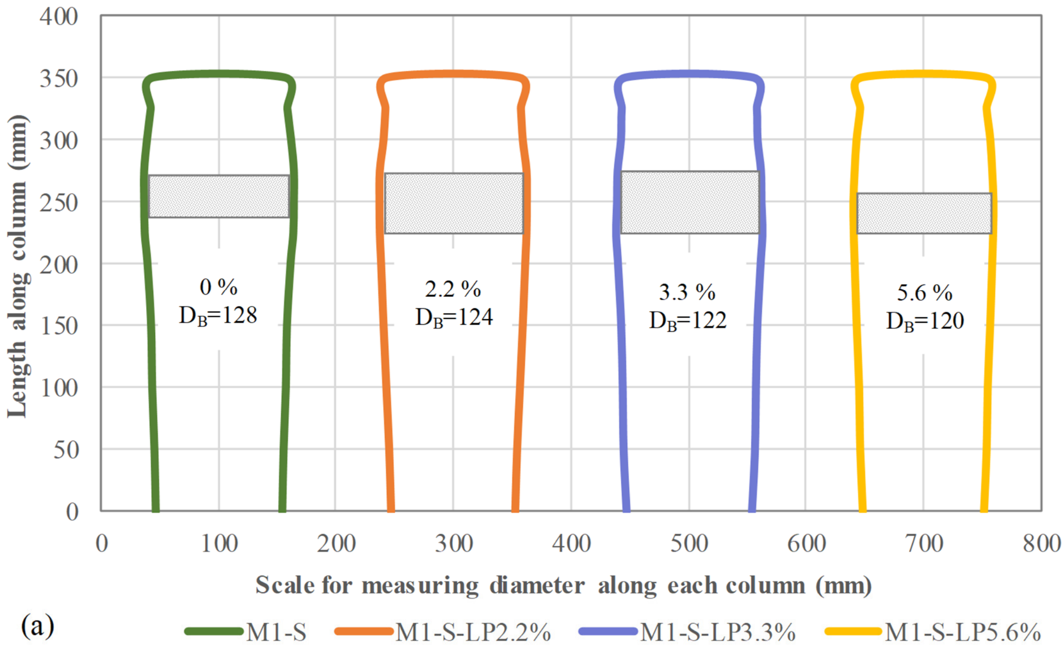
Figure 4.20: Deformations of tested columns reinforced with randomly mixed PET fibres, and installed in a base silt at LL, (a) graphical plot and (b) pictorial illustration

4.4.3 Layering

4.4.3.1 Flakes

In Figure 4.21, the deformation characteristics for columns reinforced with layers of fibres, and installed in base soils at OMC, are presented. In general, consistency is observed in their deformity. This is correspondingly confirmed in the pictorial representation. Furthermore, it is also evident that an increase in the flakes content resulted in a reduction of the bulging diameter, from 124 mm to 120 mm, when compared to that of the OGC which was found to be 128 mm. This reflected a maximum improvement of about 6 % in the deformation of the column with 5.6 % fibres. Although the deformation shape was relatively uniform, the length span along which maximum bulging occurred varied. For instance, at a flakes content of 2.2 and 3.3 %, the bulging spans were almost the same, although they had both increased when compared to that for the OGC. However, with the highest quantity of flakes, the length of the span was almost comparable to that of the OGC, although the largest bulging occurred at a slightly lower position in that RGC.

(a)



(b)

Figure 4.21: Deformations of tested columns reinforced with layers of PET flakes, and installed in a base silt at OMC, (a) graphical plot and (b) pictorial illustration

When the columns which were reinforced with layers of PET flakes were installed in base soils at LL, the deformation response were remarkably different compared to when they were installed at OMC. Besides the unalike shapes of the columns, the positions at which maximum bulging occurred were also dissimilar. Figure 4.22 illustrates these behaviours. While the columns deformed quite similarly in terms of the shape and the length span of the highest enlargement, at a flakes content of 0 and 2.2 %, the bulge diameter differed. For the OGC, this diameter was measured as 150 mm, while that for the RGC containing 2.2 % flakes was

142 mm. As the quantity of flakes increased from 2.2 %, the maximum bulging diameter decreased up to 138 mm, which corresponded to the 5.6 % concentration. In fact, a drastic reduction in the length span was also noted beyond 2.2 % of plastic, in addition to the higher placed positions along the columns. Irrespective of the column type in Figure 4.22, maximum deformation appeared to occur above a length of 250 mm up the columns.

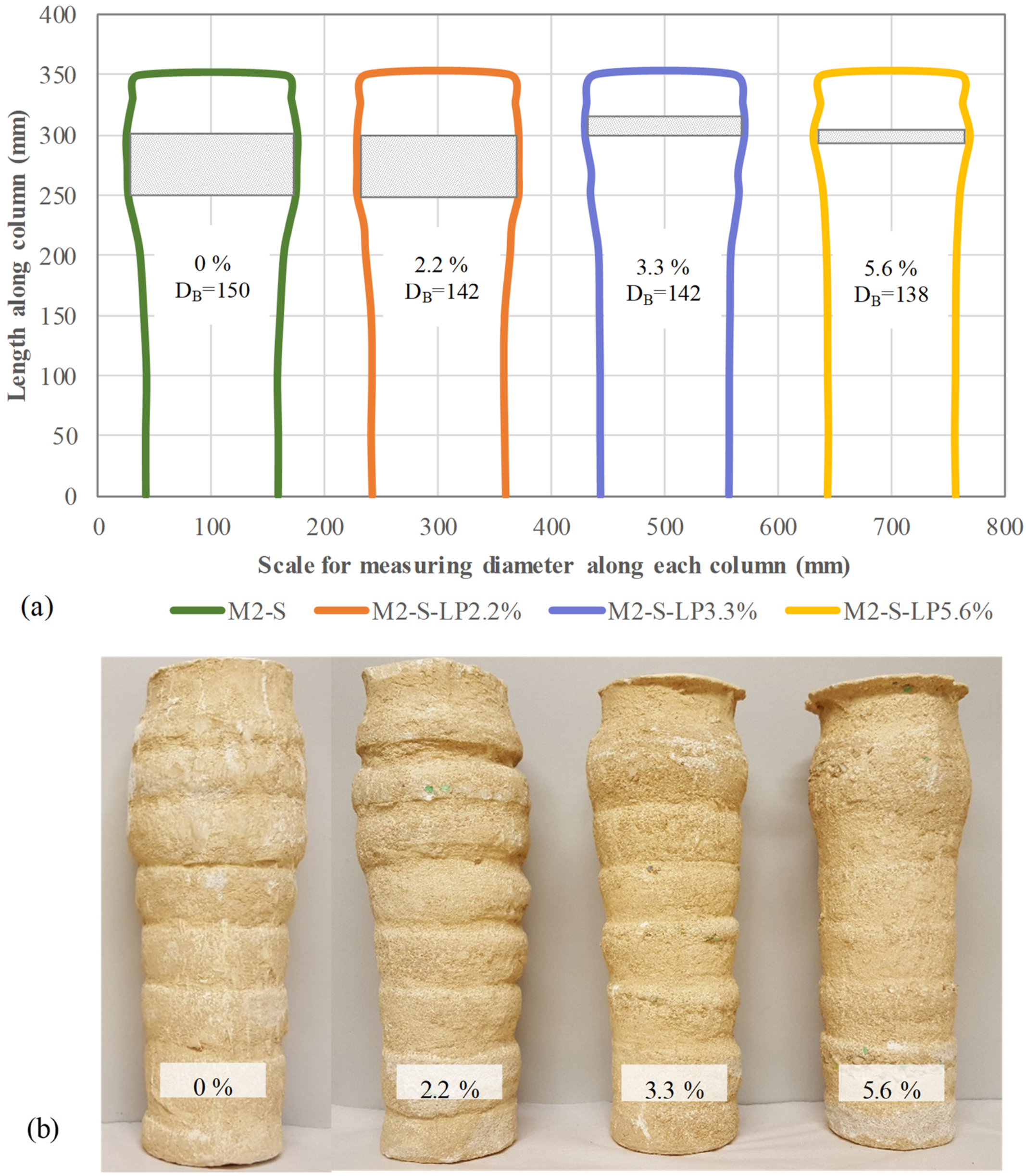
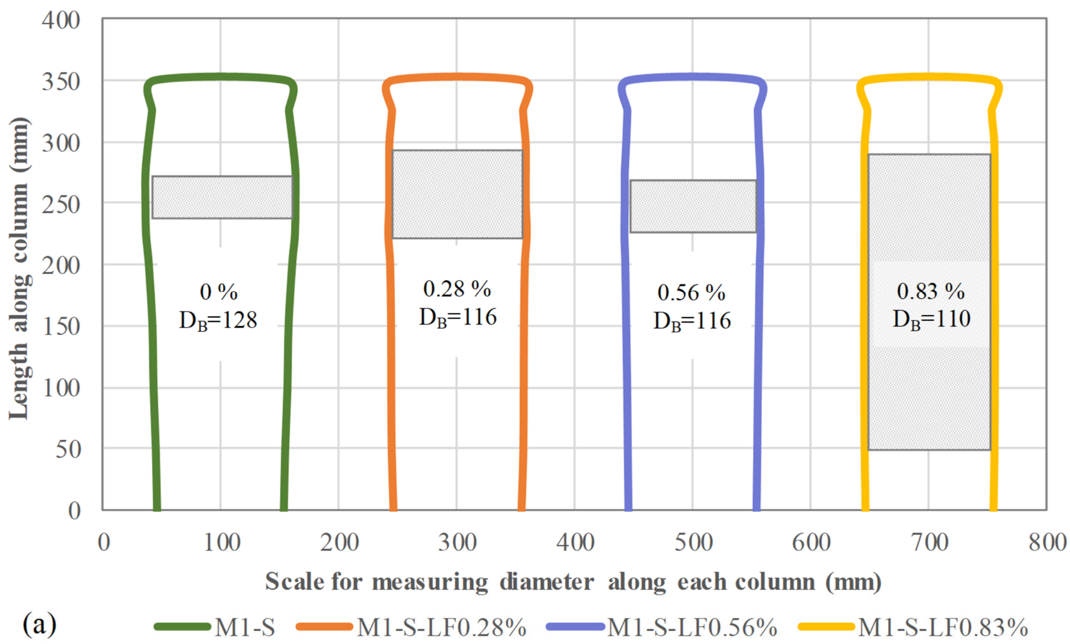


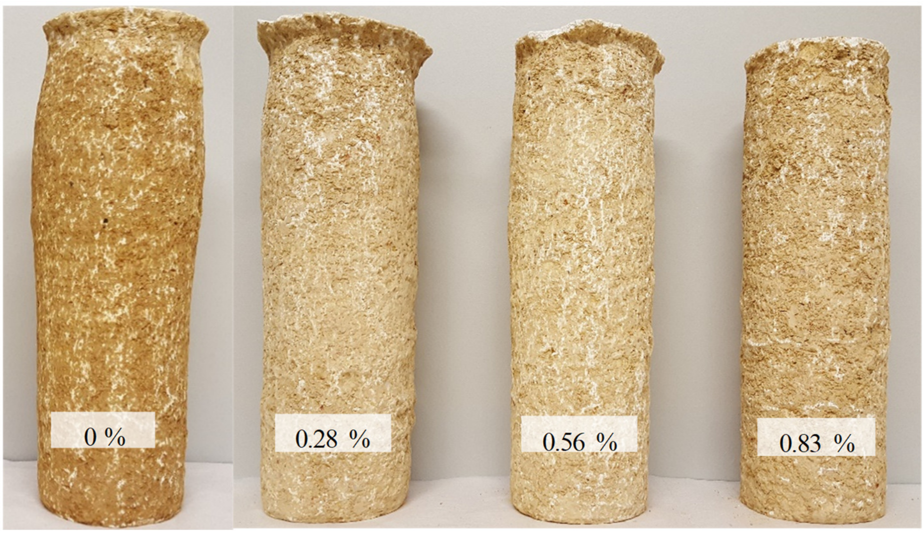
Figure 4.22: Deformations of tested columns reinforced with layers of PET flakes, and installed in a base silt at LL, (a) graphical plot and (b) pictorial illustration

4.4.3.2 Fibres

Figure 4.23 depicts the deformation characteristics of columns, which were installed in base soils at OMC, as they were reinforced with different quantities of fibres in a layering arrangement. From both the graphical and the pictorial representations, it was evident that the general inclusion of the reinforcement contributed to a reduction in the maximum bulge diameter. More specifically, a decrease in this diameter was recorded as the fibre content per layer was increased, whereby the columns with 0.28 and 0.56 % of fibres produced similar bulging diameters of 116 mm. This was calculated as a drop of 9 %, when compared to the largest lateral deformation achieved in the OGC. Out of the 3 RGCs, the highest improvement in bulging was found to be in the column with 0.83 % of fibre, and was quantified as 110 mm. This signified a lessening of 14 %, in contrast to the OGC which exhibited maximum horizontal enlargement of 128 mm. Although the columns experienced less enlargement with higher quantities of fibre, the length span over which this occurred appeared to be longer. Generally, the length span corresponding to maximum bulging were longer in all the RGCs in this series, compared to that in the OGC. In fact, it was apparent that the inclusion of the fibres produced more uniformly deformed columns, thereby increasing the length of which the largest lateral deformation occurred. Hence, in the column which was reinforced with 0.83 % of fibres, the length span was the longest (50 to 290 mm), although it displayed the least amount of bulging. This is clearly illustrated in Figure 4.23 (b).



(a)

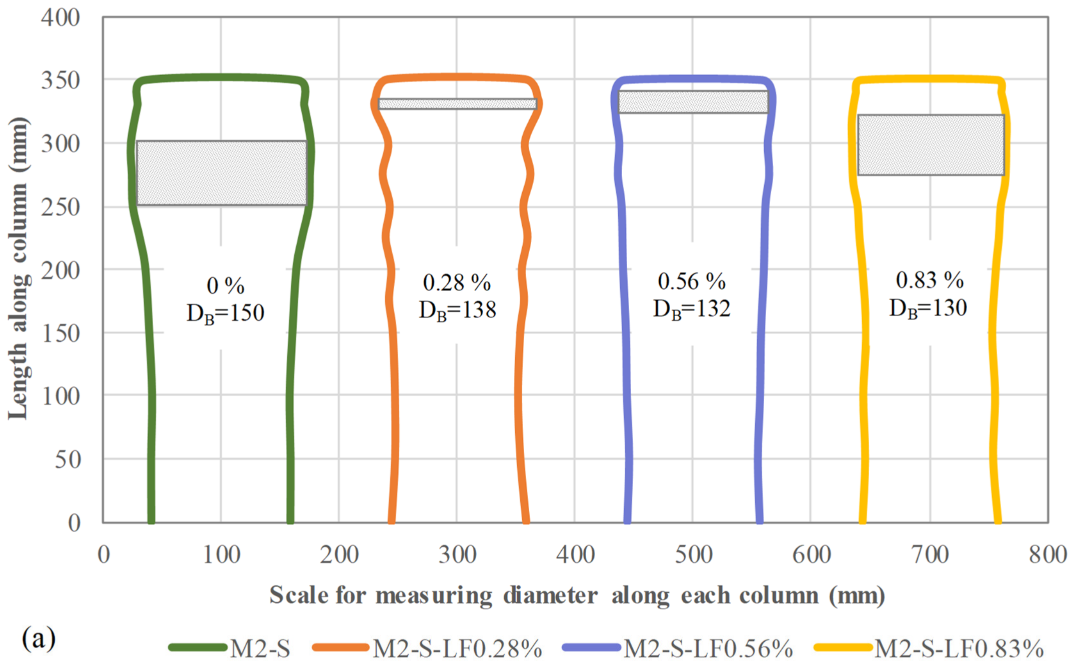


(b)

Figure 4.23: Deformations of tested columns reinforced with layers of PET fibres, and installed in a base silt at OMC, (a) graphical plot and (b) pictorial illustration

When the same types of columns were installed in base soils at LL, the results obtained were used to generate Figure 4.24. Although the moisture content of the base soil was changed, the trend was relatively similar with regards to the reduction in maximum bulging diameter, as the fibre content was increased. As the fibre concentration was augmented from 0 to 0.83 %, this diameter changed from 150 to 130 mm, which was calculated as a decline of approximately 13 %. Nevertheless, the deformation of the columns appeared to be rather non-uniform. In fact, each column followed a distinct shape, which was not comparable to each other.

Inconsistencies were also noted in terms of the length span and its position along the RGCs, although an increase in the amount of fibre resulted in a longer span over which the largest horizontal deformation occurred. Additionally, the lower the fibre content, the higher was the position of this length within the column; a fibre content of 0.28 % generated the shortest and highest positioned length span (325 to 330 mm).



(a)

(b)

Figure 4.24: Deformations of tested columns reinforced with layers of PET fibres, and installed in a base silt at LL, (a) graphical plot and (b) pictorial illustration

4.4.3.3 Betatex Geotextile (GW)

In Figure 4.25, the deformation behaviours of Betatex RGCs installed in base soils at OMC, are presented. Consistency was noted with regards to the shape of the deformed columns, in both the graphical and pictorial illustrations. Overall, the inclusion of the geotextile within the columns reduced the extent of bulging. More specifically, a progressive reduction in the maximum bulging diameter was recorded as the mass (and as such the thickness) of the geotextile was augmented from 200 to 600 g/m^2. The highest reduction, calculated as 6 %, in this diameter was noted in the column which was reinforced with the GW600 geotextile. Generally, it was established that the inclusion of these RGCs in base soils at OMC do not produce significantly high amelioration in terms of bulging minimisation. Moreover, the variation in the mass of the geotextile only produced small changes in the size of the deformed columns. For example, as the geotextile mass of 200 g/m^2 was doubled, the maximum enlargement only reduced by 2 mm. In the case where this mass was increased 3 times, maximum bulging only decreased by 4 mm. Therefore, it was apparent that for every increase of 200 g/m^2 in the mass of the geotextile, the largest lateral deformation reduced by only 2 mm. With regards to the length span which corresponded to this bulge, GW200 produced almost similar results as the OGC which was equivalent to a length of 40 mm. However, for the other 2 RGCs this length was additionally stretched, the longest of which was observed as 67 mm in the column with GW600. The position of this span for GW200 and GW600 was located at 230 to 270 mm and 223 to 290 mm, respectively. Therefore, it is evident that although there was an extension in the length span over which maximum bulging occurred as the mass of the geotextile was increased, the matching diameter was smaller, and the deformed columns displayed higher uniformity in their shapes.

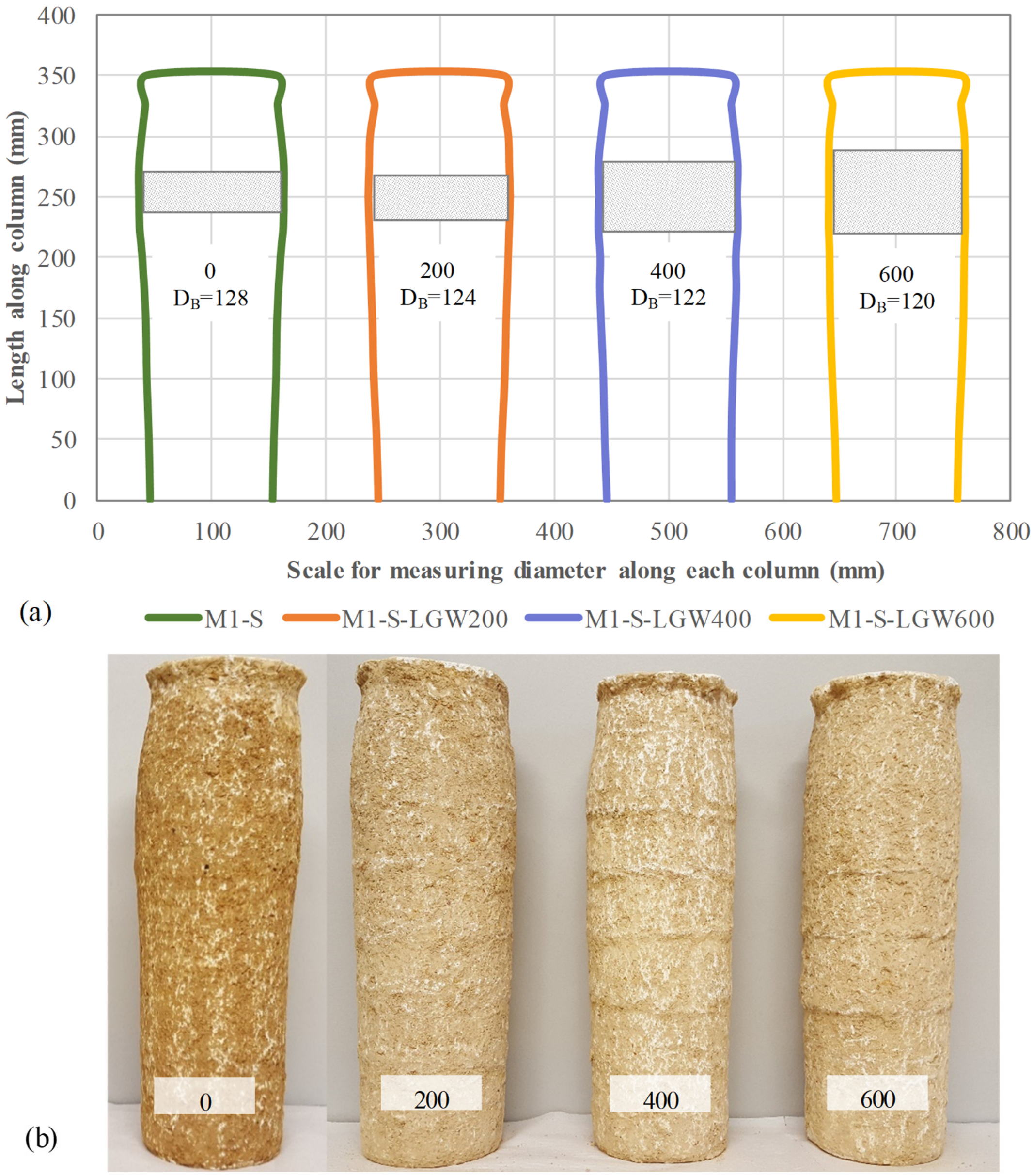


Figure 4.25: Deformations of tested columns reinforced with layers of Betatex geotextile (GW), and installed in a base silt at OMC, (a) graphical plot and (b) pictorial illustration

In contrast, when these RGCs were installed in base soils at LL, inconsistency was observed with regards to their deforming shapes, as shown in Figure 4.26. Nevertheless, the size of the maximum bulge appeared to be smaller in the RGCs. It was evident that an increase in the mass per unit area of the geotextile resulted in a reduction of the largest bulging diameter. In comparison to the OGC which exhibited this diameter as 150 mm, that of the lightest geotextile was 138 mm while a diameter of 124 mm was achieved for the heaviest reinforcement. These smaller diameters represented bulge reductions of 8 and 17 %, respectively. Irrespective of the differences noted in terms of the deformation shape and the corresponding lateral expansion, the position and length of the span appeared to be relatively similar. The length remained the same for all the columns, although the span positions slightly differed.

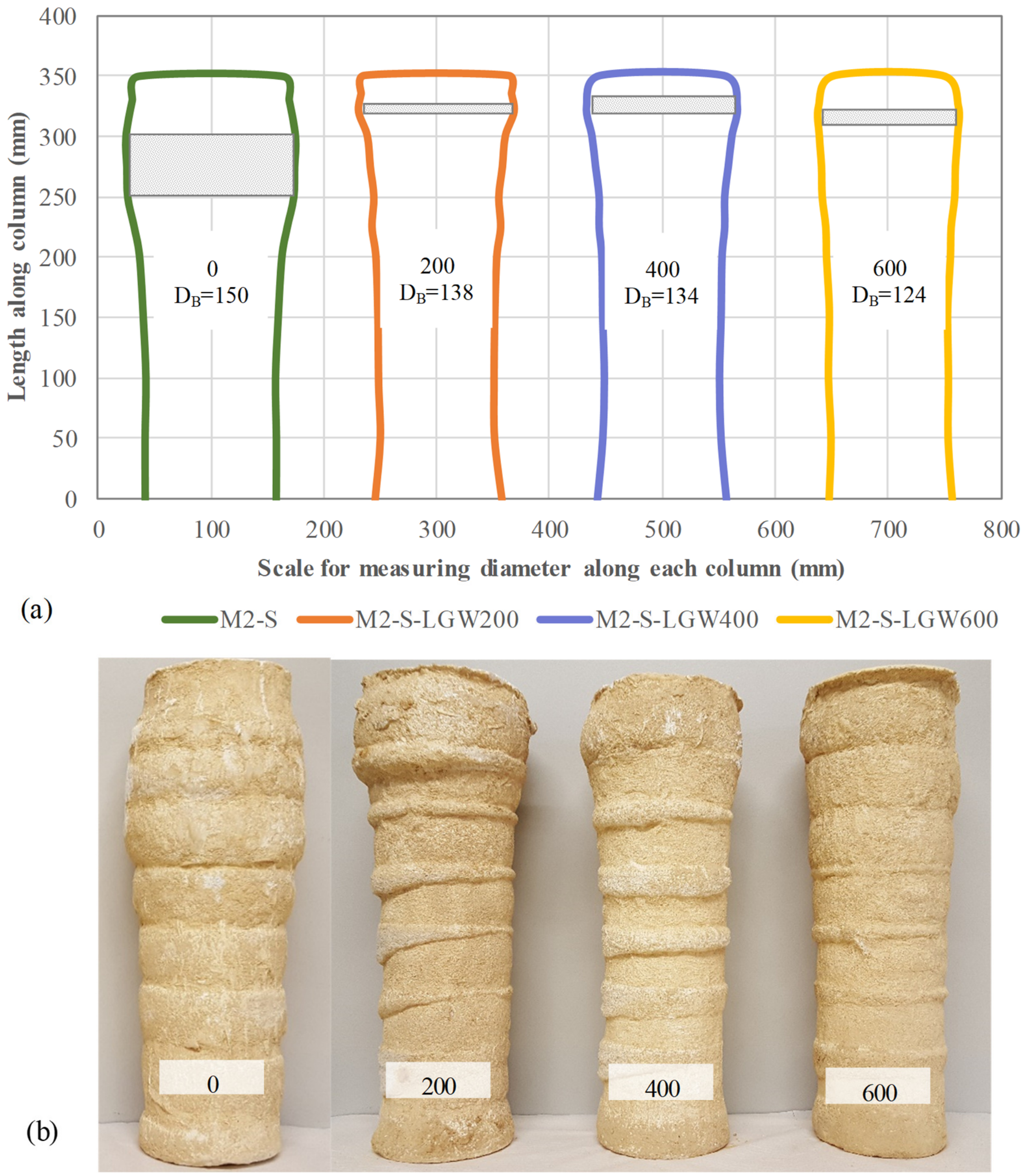
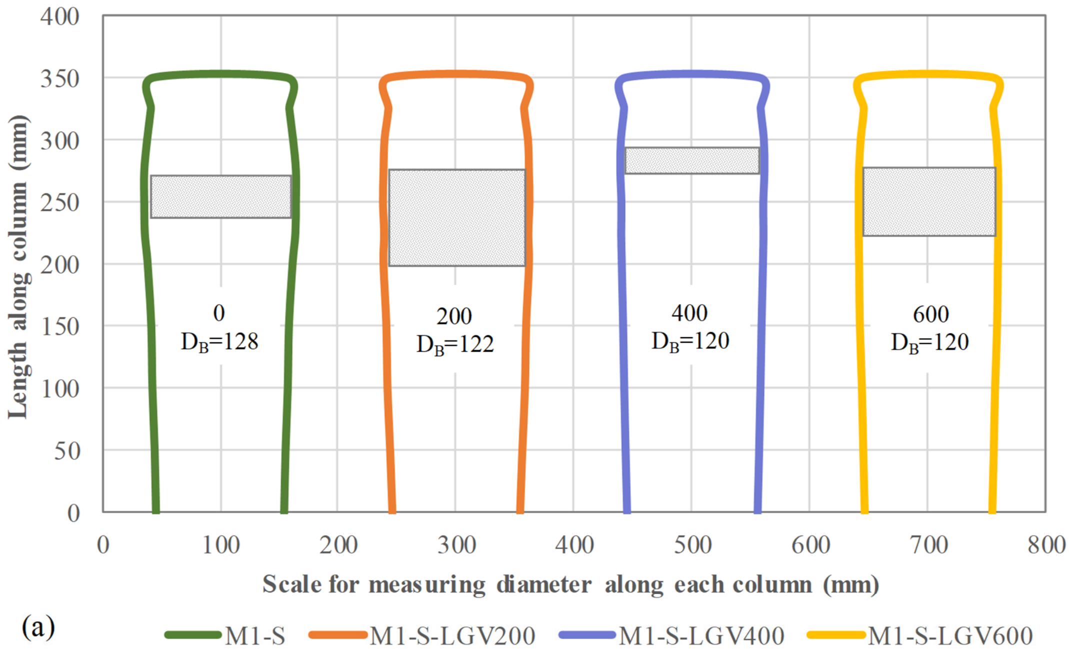


Figure 4.26: Deformations of tested columns reinforced with layers of Betatex geotextile (GW), and installed in a base silt at LL, (a) graphical plot and (b) pictorial illustration

4.4.3.4 Fibertex Geotextile (GV)

The inclusion of the Fibertex geotextile in the RGCs, which were installed in base soils at OMC, resulted in column deformations as shown in Figure 4.27. The highest reduction in maximum bulging was obtained in the column which was reinforced with the GV600 geotextile. This was calculated as a drop of 8 mm, or 6 %, when compared to the OGC with the largest diameter of 128 mm. It also appeared that the mass of the geotextile had negligible impact on the bulge reduction. For a geotextile mass of 200 g/m^2, the largest bulge was 122 mm. As this mass was increased 2 times, the highest lateral bulge decreased by 2 mm. Beyond this, a further raise in the mass resulted in a constant bulge size. Nevertheless, the length span of the bulge was particularly affected, and this was clearly seen through the irregularities in the length and position of the span over which bulging occurred in the columns. In the GV200 column, a significant increase in the length span was achieved which was equivalent to 75 mm compared to 40 mm in the OGC. However, in the GV400, the length span drastically reduced to 20 mm, and afterwards increased to 50 mm in the column with GV600. Evidently, the position of the length span of the bulge was also largely influenced.



(a)

(b)

Figure 4.27: Deformations of tested columns reinforced with layers of Fibertex geotextile (GV), and installed in a base silt at OMC, (a) graphical plot and (b) pictorial illustration

Figure 4.28 presents the results pertaining to the deformation of the Fibertex RGCs, which were installed in base soils at LL. The shape of the RGCs post-testing evidently differed to that of the OGC, such that the latter experienced the highest bulging. Interestingly, all the RGCs showed relatively similar bulging diameters, which varied from 132 mm (GV200) to 130 mm (GV600); the maximum bulge size for the column with GV400 was equal to that of the column with GV200. For a diameter of 130 mm, there was a remarkable drop of 13 % in the size of the largest lateral expansion. In terms of the corresponding position of maximum bulging along the column, the span was almost the same with a length varying between 6 and 8 mm. It is worth noting that this location was placed higher up in all the RGCs.

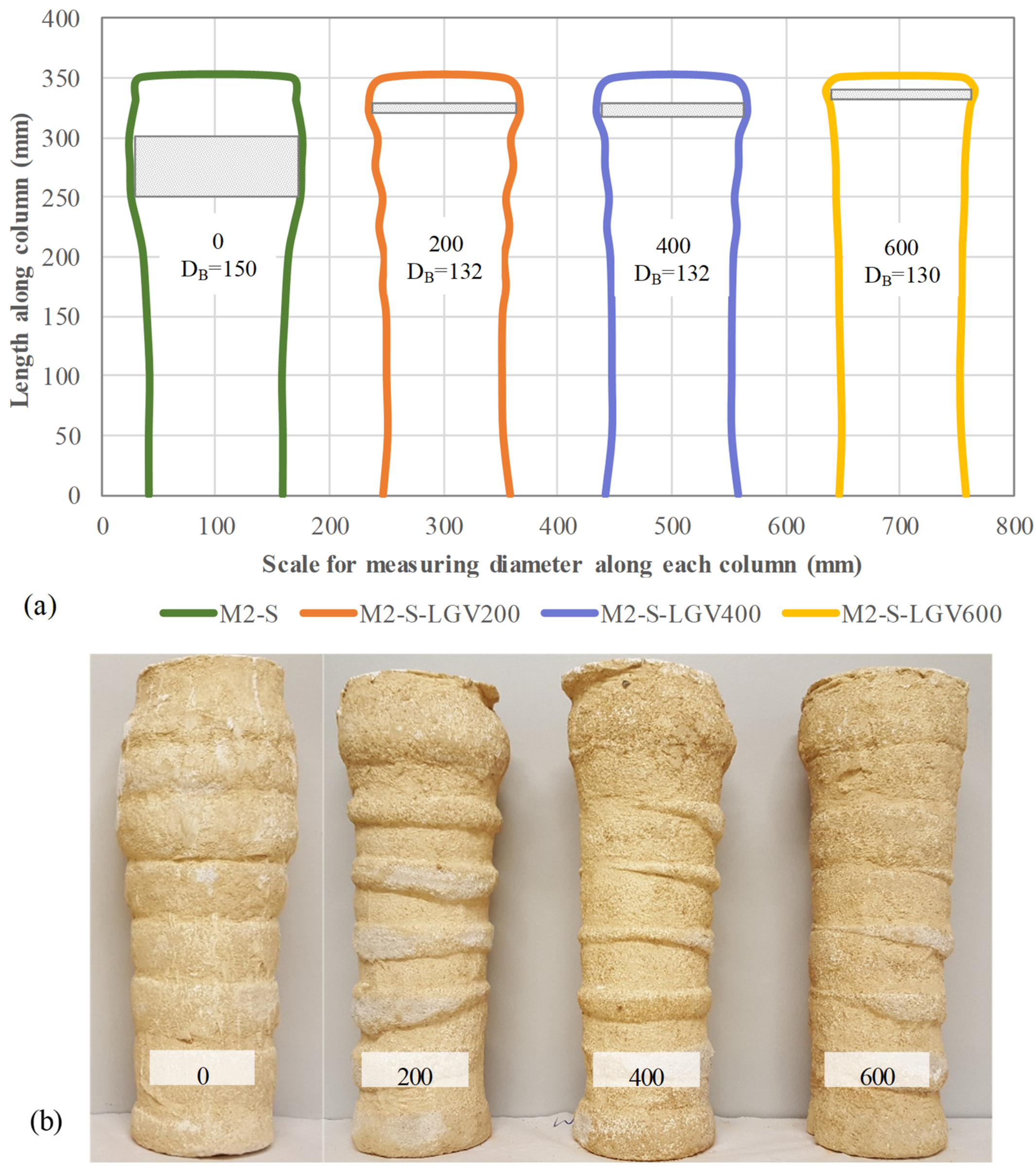


Figure 4.28: Deformations of tested columns reinforced with layers of Fibertex geotextile (GV), and installed in a base silt at LL, (a) graphical plot and (b) pictorial illustration

4.5 Conclusion

In this chapter, the results of the bench-scale experiments conducted on both OGCs and RGCs were given. In the testing process, different variables such as the moisture content of the base soil, the type of reinforcement, the quantity of reinforcement and the reinforcement placement were explored. However, other characteristics such as the column diameter, the type of sand used for the column, the base soil and the size of the model were kept constant throughout the testing programme. The outcome of these tests which were performed on columns, installed in base soils at both OMC and LL, were presented in 2 separate sections namely: (1) stress-settlement characteristics, and (2) deformation of the tested columns. Data acquired from the tests were used to produce the associated figures. The trends and changes brought about, from the different variables, in the generated figures were described in detail and the following principal values were obtained for each test: maximum vertical applied stress, largest bulging diameter and the length span over which that bulge occurred. These values condensed in Tables 4.3 to 4.6, were ultimately used in Chapter 5 for further analysis and discussion of the findings for better understanding of the mechanisms that caused the variations in the behaviour of the columns.

4.5.1 Stress-settlement characteristics

Tables 4.3 and 4.4 summarises the maximum vertical applied stress obtained for all the tests. The increase in this stress, which was obtained for each of the tests, has also been computed. This was calculated as a percentage in relation to the maximum vertical stress for the unimproved base soil.

Table 4.3: Summary of the percentage increase in maximum vertical applied stress for tests conducted on columns with randomly mixed reinforcement, when compared to that of an unimproved base soil

Moisture Content	Test specimen	Column materials	Test code	Maximum vertical applied stress (kPa)	Increase in maximum vertical applied stress (%)
Random Mixing					
OMC	Clay	N/A	M1	203	-
	Clay-Sand	Sand	M1-S	413	104
	Clay-Sand-Plastic	Sand-Plastic	M1-S-RP0.5%	425	109
			M1-S-RP1.0%	382	88
			M1-S-RP2.5%	397	95
	Clay-Sand-Fibre	Sand-Fibre	M1-S-RF0.025%	465	129
			M1-S-RF0.05%	453	123
			M1-S-RF0.1%	429	111
LL	Clay	N/A	M2	30	-
	Clay-Sand	Sand	M2-S	52	72
	Clay-Sand-Plastic	Sand-Plastic	M2-S-RP0.5%	72	139
			M2-S-RP1.0%	69	129
			M2-S-RP2.5%	61	104
	Clay-Sand-Fibre	Sand-Fibre	M2-S-RF0.025%	59	96
			M2-S-RF0.05%	80	166
			M2-S-RF0.1%	104	244

Table 4.4: Summary of the percentage increase in maximum vertical applied stress for tests conducted on columns with layers of reinforcement, when compared to that of an unimproved base soil

Moisture Content	Test specimen	Column materials	Test code	Maximum vertical applied stress (kPa)	Increase in maximum vertical applied stress (%)
Layering					
OMC	Clay-Sand-Plastic	Sand-Plastic	M1-S-LP2.2%	373	84
			M1-S-LP3.3%	412	103
			M1-S-LP5.6%	349	72
	Clay-Sand-Fibre	Sand-Fibre	M1-S-LF0.28%	408	101
			M1-S-LF0.56%	355	75
			M1-S-LF0.83%	272	34
	Clay-Sand-Betatex	Sand-Betatex	M1-S-LGW200	547	169
			M1-S-LGW400	550	171
			M1-S-LGW600	444	119
	Clay-Sand-Fibretex	Sand-Fibretex	M1-S-LGV200	508	150
			M1-S-LGV400	472	133
			M1-S-LGV600	478	136
LL	Clay-Sand-Plastic	Sand-Plastic	M2-S-LP2.2%	53	76
			M2-S-LP3.3%	57	91
			M2-S-LP5.6%	73	143
	Clay-Sand-Fibre	Sand-Fibre	M2-S-LF0.28%	58	93
			M2-S-LF0.56%	79	162
			M2-S-LF0.83%	78	158
	Clay-Sand-Betatex	Sand-Betatex	M2-S-LGW200	80	166
			M2-S-LGW400	75	149
			M2-S-LGW600	93	210
	Clay-Sand-Fibretex	Sand-Fibretex	M2-S-LGV200	74	145
			M2-S-LGV400	75	147
			M2-S-LGV600	87	188

4.5.2 Deformation of columns

The deformation characteristics of the columns in each test are presented in Tables 4.5 and 4.6. The increase or decrease in the maximum bulging diameter for each test was calculated in comparison to that of the OGC to understand the effect of introducing the reinforcement under certain conditions. The span over which the largest bulge occurred has also been given, from which the corresponding length has been calculated.

Table 4.5: Summary of the deformation characteristics for tests conducted on columns with randomly mixed reinforcement

Moisture Content	Test specimen	Column materials	Test code	Maximum bulging diameter, D_B (mm)	Increase (+ve) or decrease (-ve) in D_B compared to that in an OSC (%)	Position of length span L_B along column (mm)	Length of span, L_B (mm)
Random Mixing							
OMC	Clay	N/A	M1	-	-	-	-
	Clay-Sand	Sand	M1-S	128	-	235-275	40
	Clay-Sand-Plastic	Sand-Plastic	M1-S-RP0.5%	135	5.5	270-285	15
			M1-S-RP1.0%	125	-2.3	235-250	15
			M1-S-RP2.5%	137	7.0	245-255	10
	Clay-Sand-Fibre	Sand-Fibre	M1-S-RF0.025%	130	1.6	240-275	35
			M1-S-RF0.05%	128	0.0	248-273	25
			M1-S-RF0.1%	126	-1.6	230-260	30
LL	Clay	N/A	M2	-	-	-	-
	Clay-Sand	Sand	M2-S	150	-	250-300	50
	Clay-Sand-Plastic	Sand-Plastic	M2-S-RP0.5%	164	9.3	315-325	10
			M2-S-RP1.0%	152	1.3	285-305	20
			M2-S-RP2.5%	144	-4.0	325-340	15
	Clay-Sand-Fibre	Sand-Fibre	M2-S-RF0.025%	140	-6.7	295-305	10
			M2-S-RF0.05%	148	-1.3	295-305	10
			M2-S-RF0.1%	144	-4.0	300-310	10

Table 4.6: Summary of the deformation characteristics for tests conducted on columns with layers of reinforcement

Moisture Content	Test specimen	Column materials	Test code	Maximum bulging diameter, D_B (mm)	Increase (+ve) or decrease (-ve) in D_B compared to that in an OSC (%)	Extent of length span, L_B (mm)	Length span, L_B (mm)
Layering							
OMC	Clay-Sand-Plastic	Sand-Plastic	M1-S-LP2.2%	124	-3.1	225-275	50
			M1-S-LP3.3%	122	-4.7	225-275	50
			M1-S-LP5.6%	120	-6.3	225-258	33
	Clay-Sand-Fibre	Sand-Fibre	M1-S-LF0.28%	116	-9.4	225-295	70
			M1-S-LF0.56%	116	-9.4	230-270	40
			M1-S-LF0.83%	110	-14.1	50-290	240
	Clay-Sand-Betatex	Sand-Betatex	M1-S-LGW200	124	-3.1	230-270	40
			M1-S-LGW400	122	-4.7	223-280	57
			M1-S-LGW600	120	-6.3	223-290	67
	Clay-Sand-Fibretex	Sand-Fibretex	M1-S-LGV200	122	-4.7	200-275	75
			M1-S-LGV400	120	-6.3	275-295	20
			M1-S-LGV600	120	-6.3	225-275	50
LL	Clay-Sand-Plastic	Sand-Plastic	M2-S-LP2.2%	142	-5.3	250-300	50
			M2-S-LP3.3%	142	-5.3	300-318	18
			M2-S-LP5.6%	138	-8.0	295-305	10
	Clay-Sand-Fibre	Sand-Fibre	M2-S-LF0.28%	138	-8.0	325-330	5
			M2-S-LF0.56%	132	-12.0	325-340	15
			M2-S-LF0.83%	130	-13.3	275-320	45
	Clay-Sand-Betatex	Sand-Betatex	M2-S-LGW200	138	-8.0	320-330	10
			M2-S-LGW400	134	-10.7	325-335	10
			M2-S-LGW600	124	-17.3	315-325	10
	Clay-Sand-Fibretex	Sand-Fibretex	M2-S-LGV200	132	-12.0	322-328	6
			M2-S-LGV400	132	-12.0	320-328	8
			M2-S-LGV600	130	-13.3	337-343	6

Chapter 5

Analysis and discussions

5.1 Introduction

This chapter presents an analysis and discussion of the results described in Chapter 4. Since all graphs followed the same pattern, such that there was no dramatical deformation in the stress-settlement curves, all analyses were based on the highest settlement of 50 mm and the corresponding maximum vertical applied stress. With regards to the maximum lateral deformation of the column, the measurements provided in Chapter 4 were utilised to further assess the bulging behaviour of each RGC. Figure 5.1 summarises the sections covered in Chapter 5 to explain the results obtained in the present study. The analysis was executed from a geotechnical engineering perspective, although certain aspects of the environment was considered.

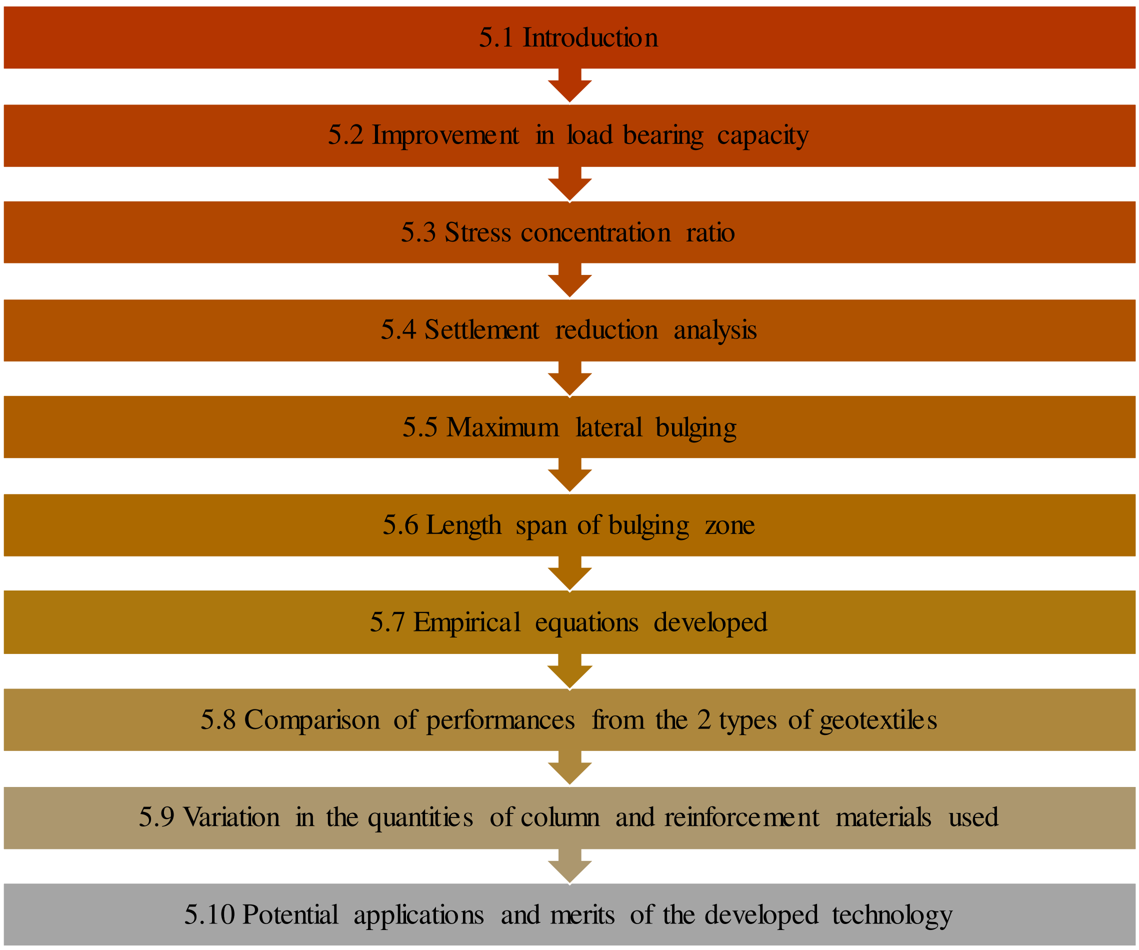


Figure 5.1: Layout of the analysis and discussion of results in Chapter 5

5.2 Improvement in load carrying capacity

The improvement in the maximum vertical applied load, at a settlement of 50 mm, was computed (using equation 4.2) as a percentage. A summary of the improvement recorded in each test was given earlier in Tables 4.3 and 4.4 and was rather reported as the increase in vertical applied stress. In this section, the percentage improvements are initially analysed and presented graphically in sub-sections 5.2.1 and 5.2.2. Discussions of the general observations are subsequently made in sub-section 5.2.3.

5.2.1 Random mixing

Figure 5.2 shows the highest percentage improvements achieved in all tests related to randomly mixed flakes, whereby the concentration of flakes as a percentage was measured by mass of each layer of sand of 50 mm within the column. From this bar chart, it is evident that tests conducted on RGCs, in base soils at LL (M2 tests), produced higher improvements than those executed at OMC. In fact, the highest enhancement (139 %) in maximum vertical applied stress corresponded to the tests on RGC, with flakes concentration of 0.5 %, in a base soil at LL. This was approximately twice the percentage increase, under similar conditions, if an OGC was utilised. Overall, it was apparent that a flakes concentration of 0.5 % produced the optimum increase in maximum vertical applied stress, irrespective of the moisture content of the base soil.

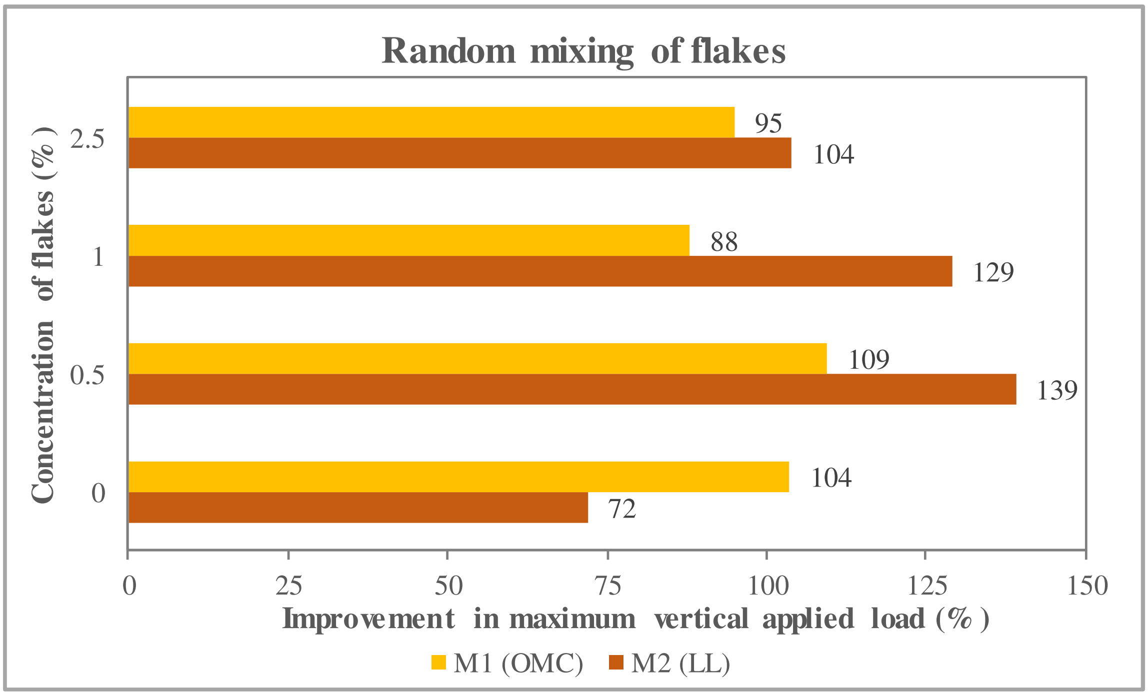


Figure 5.2: Percentage improvement recorded in tests conducted on samples improved by RGC containing randomly mixed flakes

The percentage improvements attained in the tests conducted on RGCs with randomly mixed fibres are illustrated in Figure 5.3. For tests in base soils at OMC, an optimum percentage improvement of 129 % was recorded for a RGC with a fibre concentration of 0.025 %. This was actually the smallest fibre content investigated. This generated an additional 25 % of advancement in the maximum vertical applied stress, when compared to an OGC under same conditions. Contrastively, tests conducted on RGCs in base soils at LL, demonstrated better response to the increase in fibre content. In effect, the largest concentration of fibres of 0.1 % resulted in the largest amelioration in vertical applied stress of 244 %. Compared to the test performed on an OGC, under similar environments, this was about 3.4 times that obtained for the OGC.

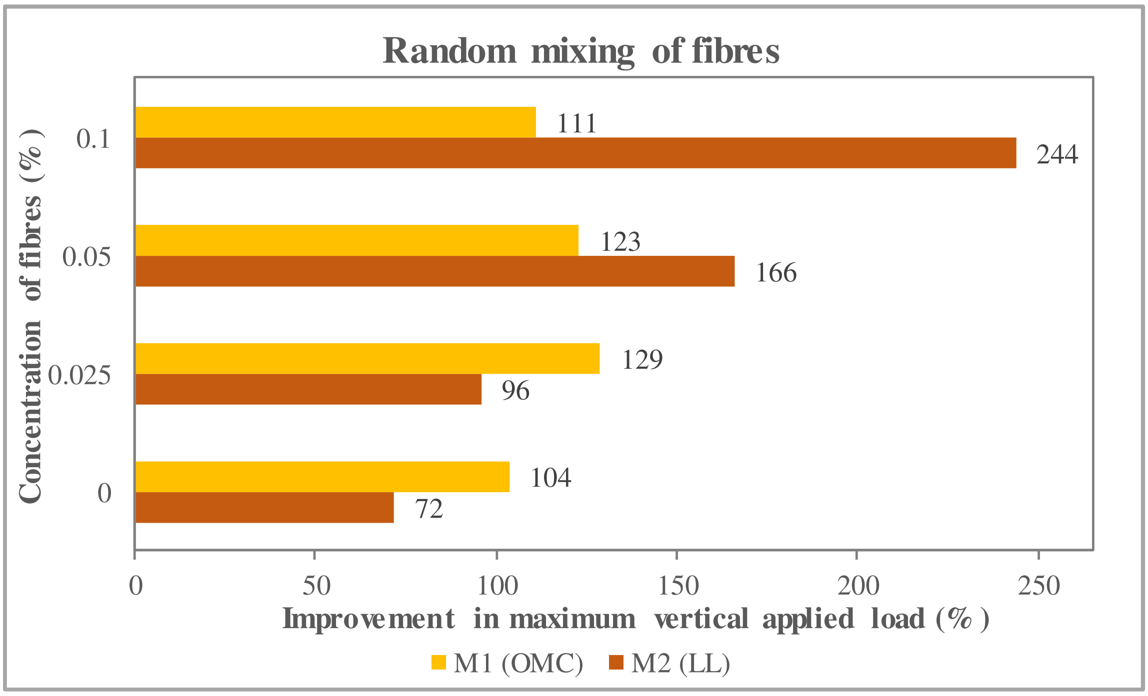


Figure 5.3: Percentage improvement recorded in tests conducted on samples improved by RGC containing randomly mixed fibres

5.2.2 Layering

In Figure 5.4, the improvements achieved are given for tests whereby flakes were included in the layering arrangement. The behaviour of the columns in terms of the maximum vertical applied stress is distinct, when installed in base soils at OMC and at LL. While a remarkable decrease in stress was recorded in tests conducted on base soils at OMC, an opposite observation was made in tests which were performed in the wetter base soil. The improvement in vertical stress was found to increase as the concentration of the flakes was raised; at a flakes concentration of 5.6 %, the improvement doubled in comparison to an OGC which was tested under similar conditions. For this test series, the highest improvement in OMC tests was generated in both the OGC or the RGC with 3.3 % of flakes while that in LL tests occurred in the RGC containing 5.6 % of flakes.

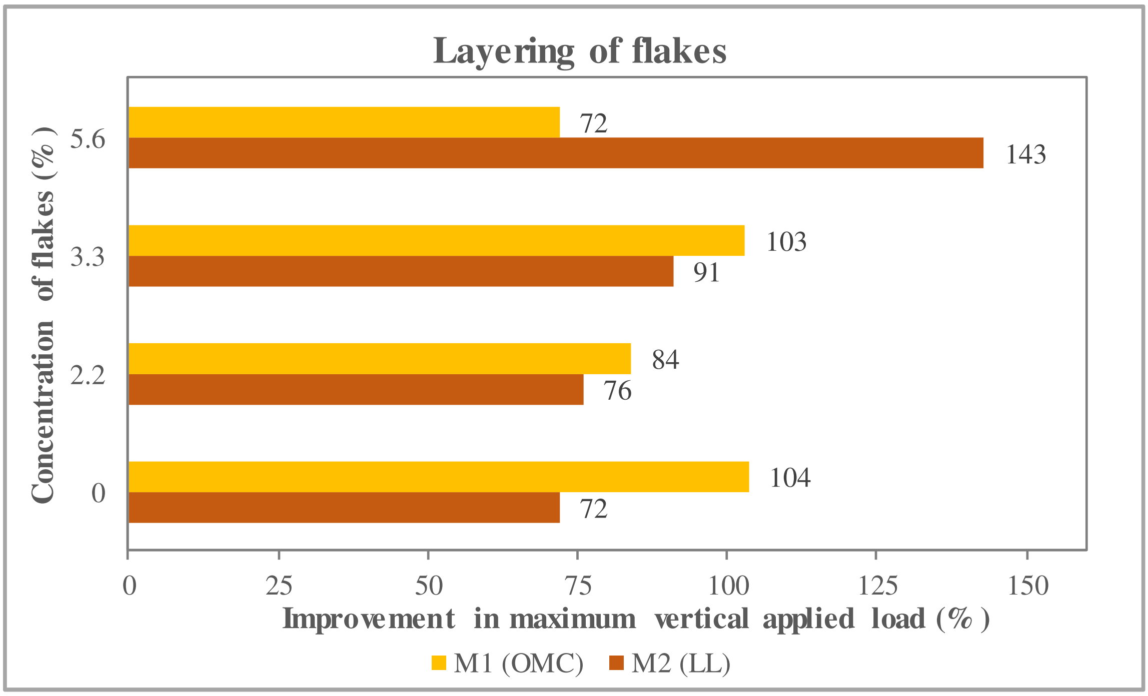


Figure 5.4: Percentage improvement recorded in tests conducted on samples improved by RGC containing layers of flakes

Figure 5.5 illustrates the percentage improvement which was achieved for the individual tests conducted on RGCs which consisted of layers of fibres. For this particular test series, it is evident that the addition of fibres in this arrangement, for the OMC tests, reduced the percentage improvement. Nevertheless, a small concentration of 0.28 % of fibres produced relatively similar gain in maximum vertical applied stress in the OMC tests when compared to that of the OGC under identical conditions. Contrastively, for the LL tests, it is apparent that there was an increase in the maximum vertical applied stress, with a drastic peak at a fibre concentration of 0.56 %. Beyond this point, a minor decline of 4 % (from 162 to 158 %) was recorded as the fibre content changed from 0.56 to 0.83 %.

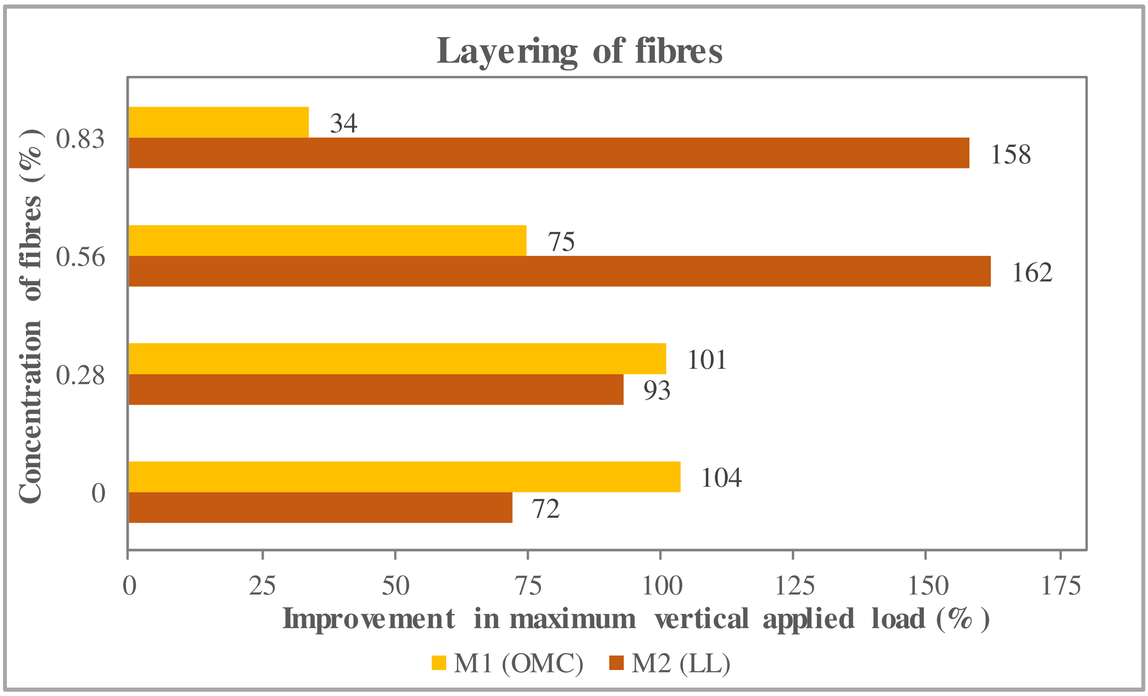


Figure 5.5: Percentage improvement recorded in tests conducted on samples improved by RGC containing layers of fibres

In Figure 5.6, it is observed that the introduction of Betatex layers in the RGCs raised the maximum vertical applied stress, irrespective of the moisture content of the base soil. For the OMC tests, the mass per unit area of 200 and 400 g/m^2 of the Betatex produced almost the same improvement. Beyond 400 g/m^2, a decrease was noted in the stress enhancement (at 600 g/m^2), although it was still higher than the stress gain in the OGC. In terms of the LL test series, this observation was dissimilar since the mass per unit area of 600 g/m^2 produced the most dramatic improvement of 210 %; this was almost 3 times that obtained for the OGC when tested under these conditions.

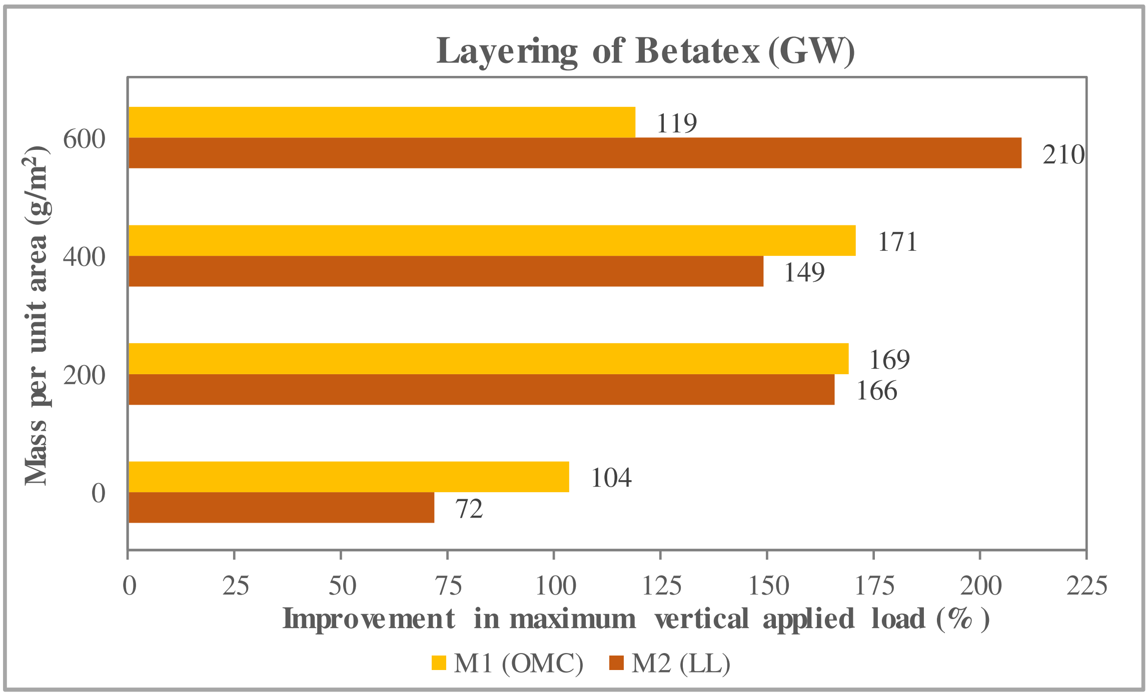


Figure 5.6: Percentage improvement recorded in tests conducted on samples improved by RGC containing layers of Betatex (GW)

In contrast to the test series with layers of Betatex, the improvement generated in the tests with layers of Fibertex (shown in Figure 5.7) have exhibited similar trends (although dissimilar values), except when the mass per unit area of 400 g/m^2 was used. Regardless of these observations, the inclusion of the Fibertex generally produced higher maximum vertical applied stresses in comparison to the OGCs, in base soils at both OMC and LL. For both OMC and LL, the respective optimum results of 150 and 188 % were obtained, which corresponded to mass per unit areas of 200 and 600 g/m^2.

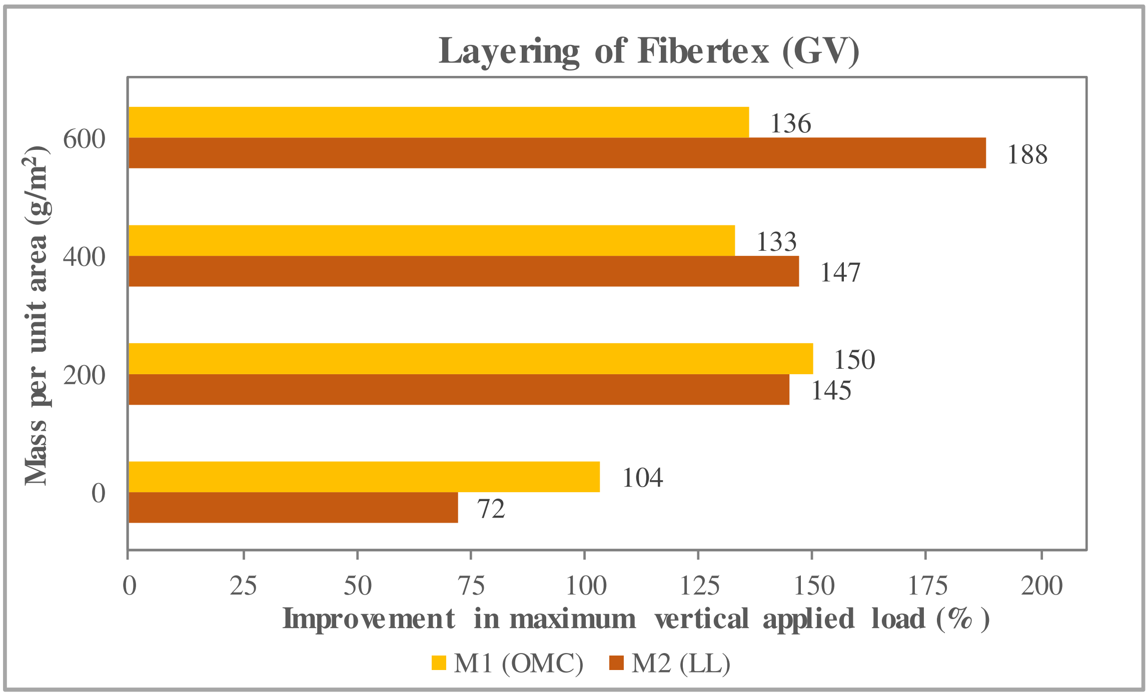


Figure 5.7: Percentage improvement recorded in tests conducted on samples improved by RGC containing layers of Fibertex (GV)

5.2.3 Discussions

An analysis of Figures 5.2 to 5.7 provided an in-depth understanding of the percentage improvement achieved, in terms of the maximum vertical applied stress at a settlement of 50 mm, for each test. Although the weight of the respective reinforcing materials shared no compatibility amongst each other, they were nevertheless selected based on the lowest required quantities to potentially generate a certain effect on the load carrying capacity and on the settlement reduction (the reasons behind the choice of the weights of the reinforcement were explained earlier in Chapter 3). Hence, this discussion drew a direct comparison of how the arrangement of the reinforcement influenced the results obtained.

In this investigation, flakes, fibres and geotextiles were the column reinforcement materials whereby the geotextiles (Betatex and Fibertex) were significantly stiffer than the others. From the bar charts presented in this section, it was apparent that the maximum improvement in load carrying capacity was generated in the test where the sand column installed in a base soil at LL was reinforced by 0.1 % of randomly mixed fibres. This corresponded to an amelioration of 244 % when compared to that of an unimproved base soil at LL. Closely related fibre concentration of 0.2 % generated optimum results in a study conducted by Al-Refeai (1992).

Additionally, it was often found that the stiffer the reinforcing material, the stronger the column formed. This was evident in the layering arrangement of the reinforcement in LL tests when the Betatex (GW600) and the Fibertex (GV600) geotextiles were utilised. The increase in the mass per unit area resulted in thicker geotextiles which were generally stiffer than the thinner ones. This can possibly explain the high gain in strength achieved when they were used to reinforce the granular columns since they could ameliorate the stiffness of the latter.

In terms of the stresses, relatively similar results to that obtained in this research were reported by Sharma, Phanikumar & Nagendra (2004) where a 258 % increase in vertical applied stress was obtained using layers of geogrids to internally reinforce granular columns (made from crushed stones). Rao, Kumar & Bindumadhava (1992) also claimed comparable results such that their bearing capacity was improved by 250 % when stones were mixed with the sand to form the columns. Contrastively, several other studies on reinforced granular columns have reported much lower enhancements in the load carrying capacity, ranging from 20 to 85 % (Ayadat, Hanna & Hamitouche, 2008; Tallapragada, Golait & Zade, 2011; Tandel, Solanki & Desai, 2014; Al-Obaily, 2017). In most of these studies, a relatively stiff material (often a geosynthetic) was used for reinforcement. In general, the literature review demonstrated that encased columns produced stronger RGCs. For instance, a study undertaken by Malarvizhi & Ilamparuthi (2004) highlighted improvement of 300 % in the bearing capacity of RGCs. Higher percentage, up to 500 %, had also been attained by Murugesan & Rajagopal (2008). The gain in strength in these studies were regularly explained in terms of the increase in stiffness of the column, arising from the inclusion of the reinforcements. Han (2015) also shared similar views.

Out of the 4 types of reinforcements tested and in the two arrangements, randomly mixed fibres generally appeared to be a good reinforcement material in RGCs when the columns were installed in a fully saturated soft base soil. From Figure 5.3 which shows results from tests using randomly mixed fibres, it was observed that as the fibre content was increased, greater amelioration in stress was obtained. However, when these fibres were introduced in layers, the resulting gain in strength was much lower. This behaviour can be explained in terms of the soil reinforcement theory by Vidal (1966). Vidal described the mechanism whereby the soil particles are tied together by the reinforcement such that the particles form an interlocking system around the reinforcement. This yields a form of pseudo-cohesion. As the vertical stress increased on the RGCs, frictional forces were generated along the soil-reinforcement interface, thereby creating tensile stresses within the fibres. However, in parallel, the column underwent a sort of compression to maintain equilibrium which was sustained as long as there was no slippage of the reinforcement. In the case of the randomly mixed fibres, their length and entangling nature throughout the sand in the column helped to restrict slippage, as opposed to when they were installed in layers. Hence, their random arrangement produced the highest gain in strength. With regards to randomly mixed flakes, lower improvement was generally achieved in comparison to that with fibres, since the particles were much shorter, straight and smooth, which rather favoured the occurrence of slippage at an earlier stage. For the OMC tests with the layering arrangements, the strength of the RGCs decreased as the fibre concentration was augmented. In the stiffer and dry base soils at OMC, the geotextiles appeared to be better performing; the Betatex GW400 produced an improvement of 171 % for OMC tests while the Fibertex GV200 generated a strength gain of 150 % for similar test conditions.

5.3 Effect of the variables on the stress concentration ratio

The stress concentration ratio, which was described in the literature review as the ratio of the stress in the column to that in the base soil, is important since it accounts for equal strain which occurs on both the column and the surrounding material. The stress within the column differs from that of the adjacent weaker soil since the stiffness of each of them is different. As such, most design methods introduce the stress concentration ratio (n) in their calculations when designing granular columns. Therefore, graphs pertaining to this ratio has been presented and discussed for each test series in this section. The ratio (n) was simply calculated by dividing the maximum vertical applied stress, at a settlement of 50 mm for any given test, by that for the unimproved base soil (as given in equation 5.1). As such, higher values of (n) are more desirable since they indicate better improvement in terms of strength. Subsections 5.3.1 and 5.3.2 have independently analysed the stress concentration ratios which were calculated for tests with the random mixing and layering arrangements of the reinforcement. A discussion of the overall observations is subsequently given in subsection 5.3.3.

$$Stress\ concentration\ ratio, n = \frac{stress\ exerted\ on\ improved\ base\ soil}{stress\ exerted\ on\ unimproved\ base\ soil} \qquad \text{(Equation 5.1)}$$

5.3.1 Random mixing

Figure 5.8 shows the trend in stress concentration ratio as the content of randomly mixed flakes was increased from 0 to 2.5 % for both OMC and LL tests. In the OMC tests, the change in stress concentration ratio appeared to be small, from 2 to 2.1 which corresponded to concentrations of 0 and 0.5 %. A concentration of 1 % resulted in a lower stress concentration ratio than that in an OGC, which was tested under identical conditions. However, as the content of flakes was further increased to 2.5 %, (n) was raised to 2. In LL tests, (n) increased to a peak value of 2.4 and subsequently decreased up to 2, for respective concentrations of 0 and 2.5 %. It was clearly seen that (n) was the same, for both OMC and LL tests, at a concentration of 2.5 %.

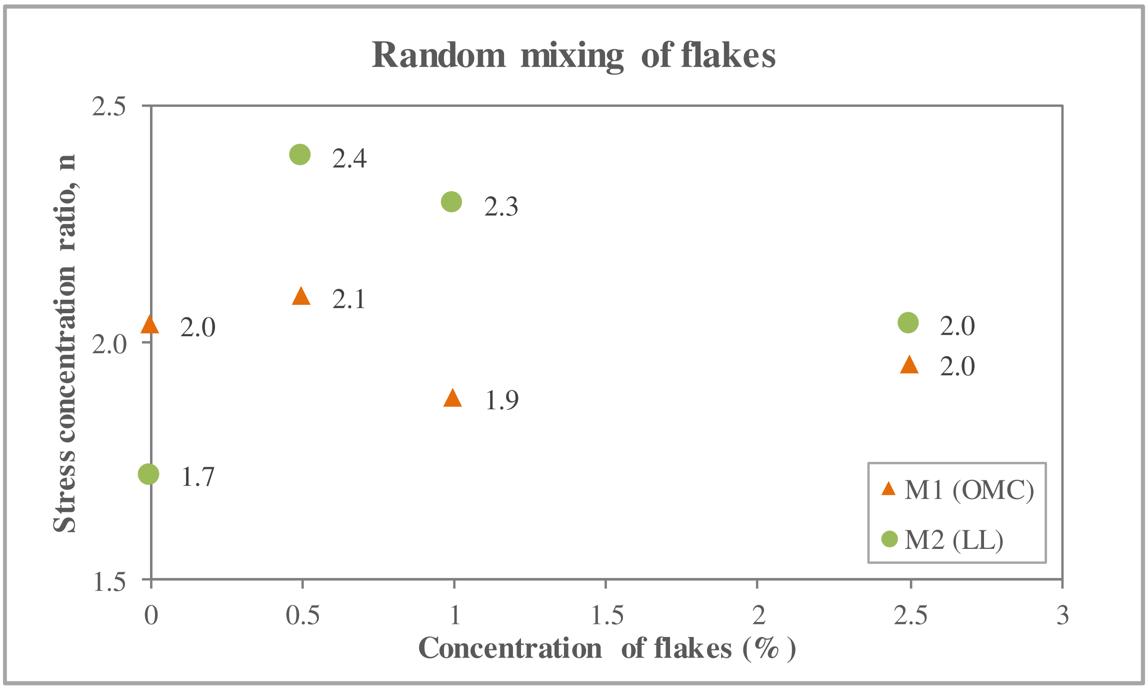


Figure 5.8: Effect of the concentration of flakes on the stress concentration ratio when randomly mixed in the granular columns

The effect of the concentration of fibres on the stress concentration ratio when fibres are randomly mixed in the granular columns, is illustrated in Figure 5.9. For the OMC tests, a relatively similar trend to that obtained for randomly mixed flakes was obtained. In this case, the variation in (n) remained small. However, a more dramatic pattern was generated by the LL tests. An increase in fibre content resulted in higher values of (n). Specifically, as the concentration of fibres was increased from 0 to 0.1 %, (n) changed respectively from 1.7 to

3.4. Therefore, this observation showed that the addition of 0.1 % of randomly mixed fibres to a granular column caused the stress concentration ratio to double.

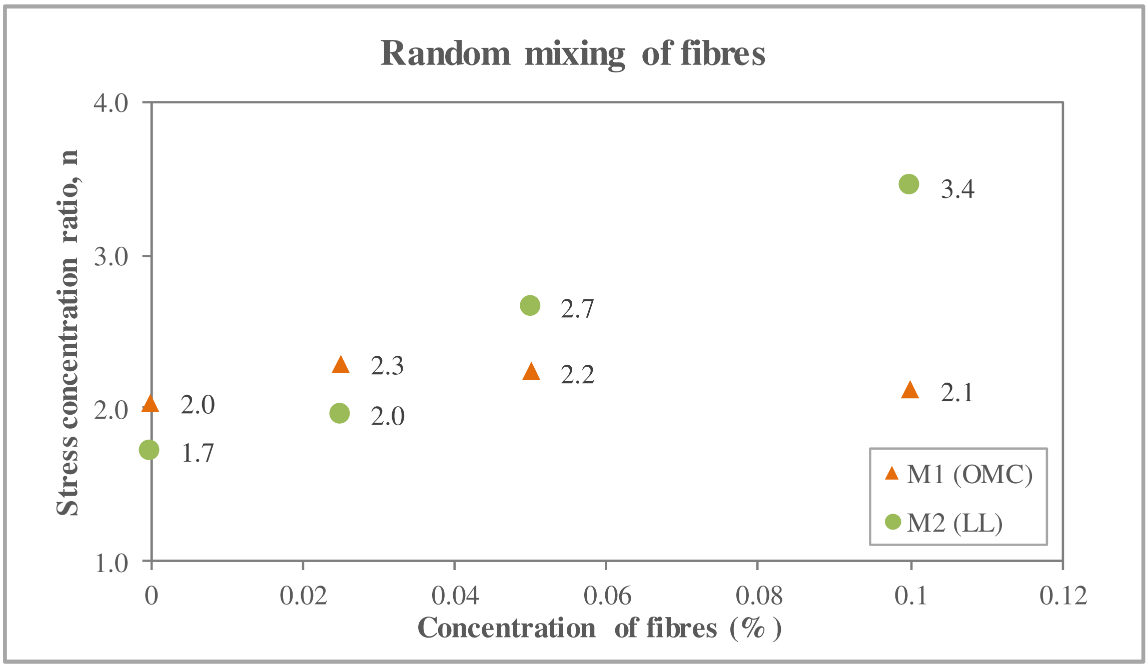


Figure 5.9: Effect of the concentration of fibres on the stress concentration ratio when randomly mixed in the granular columns

5.3.2 Layering

When layers of flakes were used to reinforce the granular columns, the effect on the stress concentration ratio differed. The trends for both OMC and LL tests are shown in Figure 5.10. From the generated information, it appeared that the variation in (n) was minimal in the OMC tests; (n) varied between 1.7 and 2 such that the OGC and the RGC with 3.3 % of flakes produced the highest values of (n). In contrast, the inclusion of the layers of flakes in LL tests demonstrated better improvement in the stress concentration ratios. As the concentration was progressively increased from 0 to 5.6 %, (n) changed from 1.7 to 2.4. Therefore, it is evident that RGC with layers of flakes respond better when improving wetter base soils.

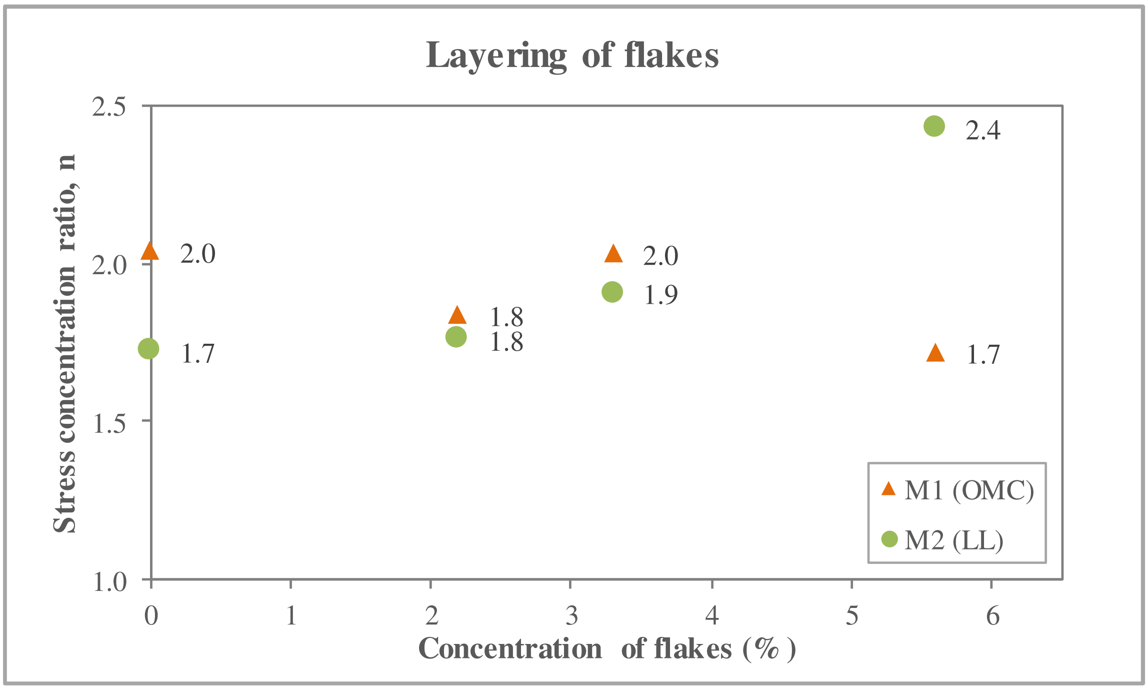


Figure 5.10: Effect of the concentration of flakes on the stress concentration ratio when included as layers in the granular columns

In Figure 5.11, the variation of (n) with increasing content of fibres has been shown where the fibres were installed in layers in the columns. For tests conducted on base soils at OMC, reinforcing of the column with fibres appeared to be meaningful only up to a concentration of 0.28 % since (n) remained as 2 which was similar to that in the OGC. Beyond this fibre content, (n) gradually decreased to 1.3 at a concentration of 0.83 %. For LL tests, a contrasting pattern was noted. As the fibre content was increased from 0 to 0.83 %, (n) changed from 1.7 to 2.6. Interestingly, both fibre contents of 0.56 and 0.83 % produced similar (n) values.

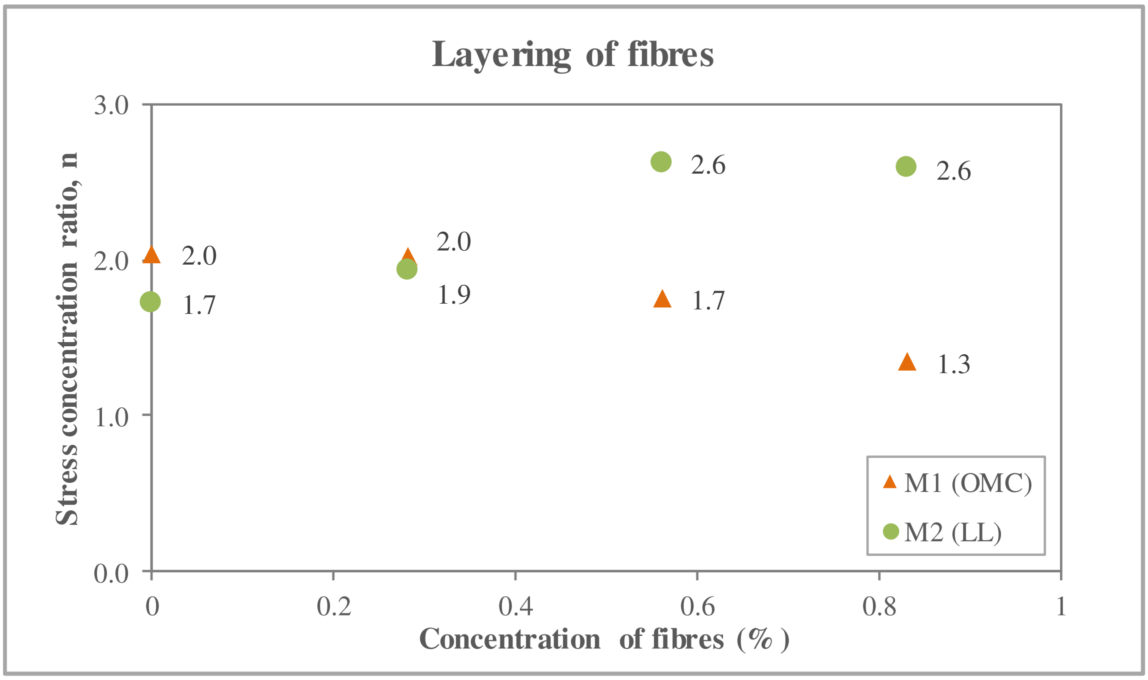


Figure 5.11: Effect of the concentration of fibres on the stress concentration ratio when included as layers in the granular columns

Figure 5.12 illustrates how the mass per unit area of Betatex (GW) affects the stress concentration ratio of the RGC, when the reinforcement was included in layers. Although the initial (n) value for OMC and LL tests were not the same, the inclusion of the geotextiles produced relatively comparable results, except for the largest mass per unit area of 600 g/m^2 (GW600). From GW400 to GW600, a decrease in (n) was recorded from 2.7 to 2.2 for the OMC tests. However, the opposite was recorded in the LL tests whereby (n) changed from 2.5 to 3.1 when GW600 was used instead of GW400.

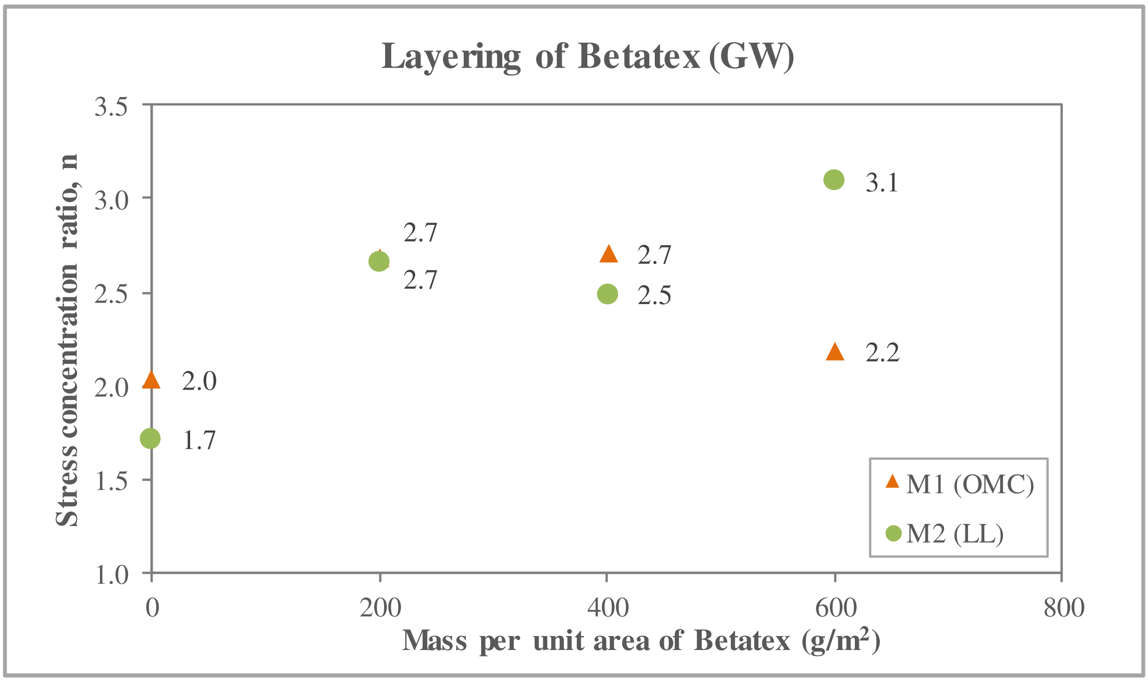


Figure 5.12: Effect of the mass per unit area of Betatex (GW) on the stress concentration ratio when included as layers in the granular columns

In Figure 5.13, the effect of mass per unit area of Fibertex (when they were placed in layers) on stress concentration ratio is illustrated. Similar trends in the variation in (n) was observed when compared to that in tests conducted on Betatex. Nevertheless, the values of (n) differed slightly. In fact, for the tests using Fibertex, (n) was often lower than that in tests with Betatex. In the OMC tests, the optimum (n) value of 2.4 was obtained for mass per unit areas of 200 and 400 g/m^2. However, in LL tests, the peak (n) value was recorded as 2.9 at a mass per unit area of 600 g/m^2.

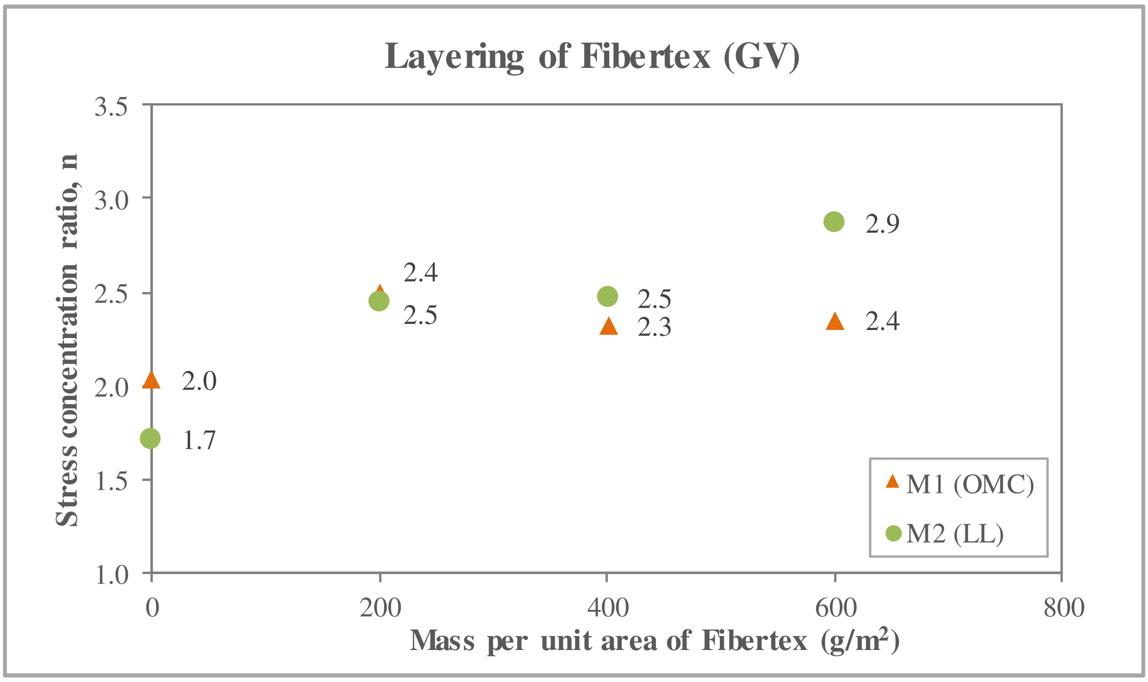


Figure 5.13: Effect of the mass per unit area of Fibertex (GV) on the stress concentration ratio when included as layers in the granular columns

5.3.3 Discussions

The stress concentration ratio is directly related to the load carrying capacities of the columns in the individual tests. Therefore, the observation in the behavioural response arising from the different testing configurations is similar to that which was discussed in subsection 5.2.3. Figures 5.8 to 5.13 displayed the stress concentration ratios in each test which were found to vary between 1.7 and 3.4. The value of (n) of 1.7 corresponded to the test on an OGC installed in a base soil at LL while (n) of 3.4 was obtained when the granular column was reinforced with 0.1 % of randomly mixed fibres and tested in a base soil of the same moisture content.

The highest (n) value evidently coincided with the maximum vertical applied stress which was discussed in the previous section. Since (n) is directly related to this stress, similar trends were observed when the mass of the reinforcement was raised. Upon application of the vertical stress, both the column and the base soil underwent equal deflection. To maintain equilibrium, a much higher stress was experienced in the RGC. As the column stiffness was increased through reinforcing the strength of the column was also enhanced, thereby resulting in the distinct high (n) value. In the random mixing arrangement, fibres seemingly produced stiffer column thereby allowing the later to support heavier loads, irrespective of the moisture content of the base soil. For this reason, fibres were better performing than the flakes. However, in the layering arrangement, the moisture content of the host soil influenced the load carrying capacity. For instance, in OMC tests, flakes resulted in higher load carrying capacities when compared to fibres. Fibres performed better than the flakes when placed in layers in the LL tests even though the most efficient type of reinforcement was generally the Betatex under these testing conditions. Since each layer of this geotextile was a single and rather stiff unit, compared to the flakes and the fibres, it was possibly more capable of transferring higher loads down the columns.

The stress concentration ratios of 1.7 to 3.4 attained in the tests were in line with previous research where n was found to vary between 1 and 5 (Vautrain, 1977; Barksdale & Bachus, 1983, Goughnour, 1983; Fattah et al. 2011 and Sobhee-Beetul, 2012). Much higher (n) values of up to 9 have also been reported (Aboshi et al., 1979; Bergado, Huat & Kalvade, 1987). However, these reported ratios were based on OGC as opposed to that for RGCs as presented in this investigation.

5.4 Settlement reduction analysis

The inclusion of the reinforcement in the granular columns produced a certain effect on the settlement of the composite ground. Barksdale & Bachus (1983) explained that shear strength is generated within granular columns as they are subjected to an overburden load. Concurrently, there appears to be a reduction in settlement of the improved ground when compared to that of its unimproved state. In the computation of settlement related to granular columns, a ratio of settlements is considered. In the literature review, the term settlement reduction ratio (SRR) was introduced and its relevance was explained. Through assumptions and manipulations of equations, Aboshi et al. (1979) proposed equation 2.22 (presented in the literature review) to calculate SRR, provided that the stress concentration ratio (n) (obtained from the previous section) and the area replacement ratio (a_s constant throughout this study and equal to 0.11 or 11 %) were known. Equation 2.22 was given as follows:

$$SRR = \frac{1}{1+(n-1)a_s}$$

The different SRR values computed were graphically represented in this section to understand how they were affected by the reinforcement, and their arrangement and quantities. The following sub-sections describe the observations which are ultimately discussed at the end of the section. Generally, a lower SRR value is preferred since it represents a reduction in settlement.

5.4.1 Random mixing

Figure 5.14 illustrates the SRR values obtained for the tests conducted on columns which were reinforced with randomly mixed flakes. From the trends in the figure, it appears that better settlement reduction was achieved in LL tests since SRR reduced to 0.87 at flakes concentrations of 0.5 and 1 %, from 0.93 in the OGC. Remarkably, settlement reduction was similar for both OMC and LL tests when a flakes content of 2.5 % was used. However, irrespective of the moisture content of the base soil, the lowest concentration of flakes of 0.5 % produced the largest reduction in settlement.

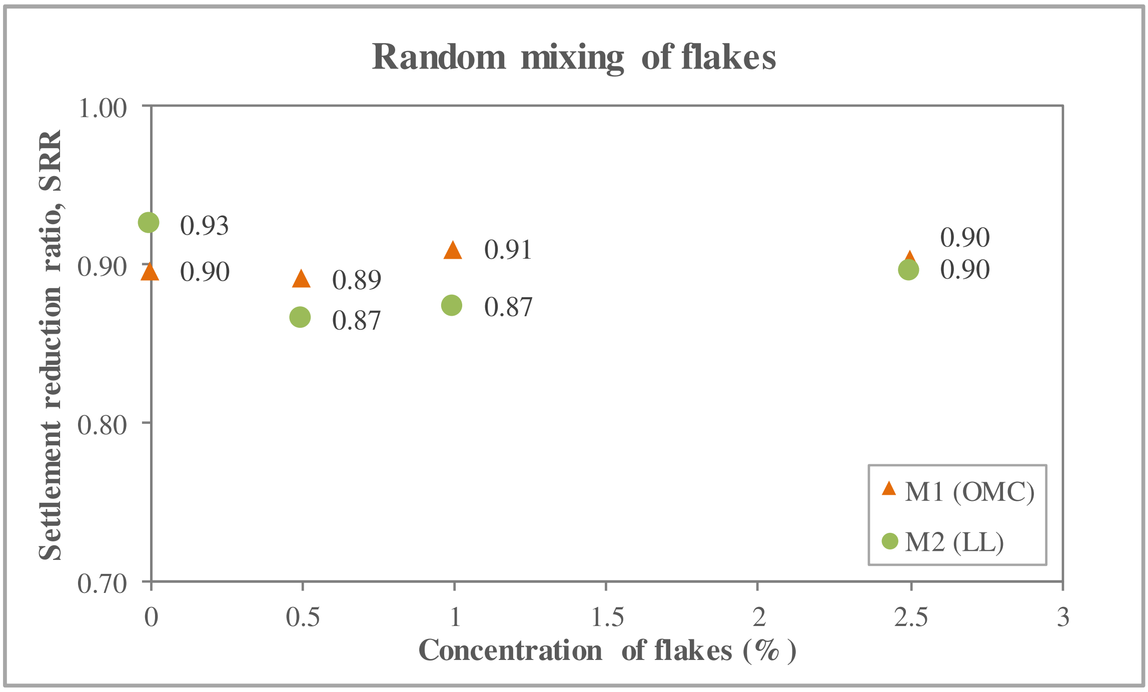


Figure 5.14: Effect of the concentration of flakes on the settlement reduction ratio when randomly mixed in the granular columns

In Figure 5.15, the relationship between SRR and the concentration of fibres is given for granular columns reinforced with randomly mixed fibres, and installed in base soils at OMC and LL. For OMC tests, the variation in SRR was quite small (between 0.87 and 0.90) when compared to that in LL tests (between 0.79 and 0.93). This was clearly seen in the figure where a dramatic drop in SRR was observed for LL tests as the concentration of fibres increased from 0 to 0.1 %. Therefore, the trend followed in both test series evidently differed since the data points in the OMC tests contrastively produced a less steep imaginary line of best fit; thus, indicating lower reductions in settlement.

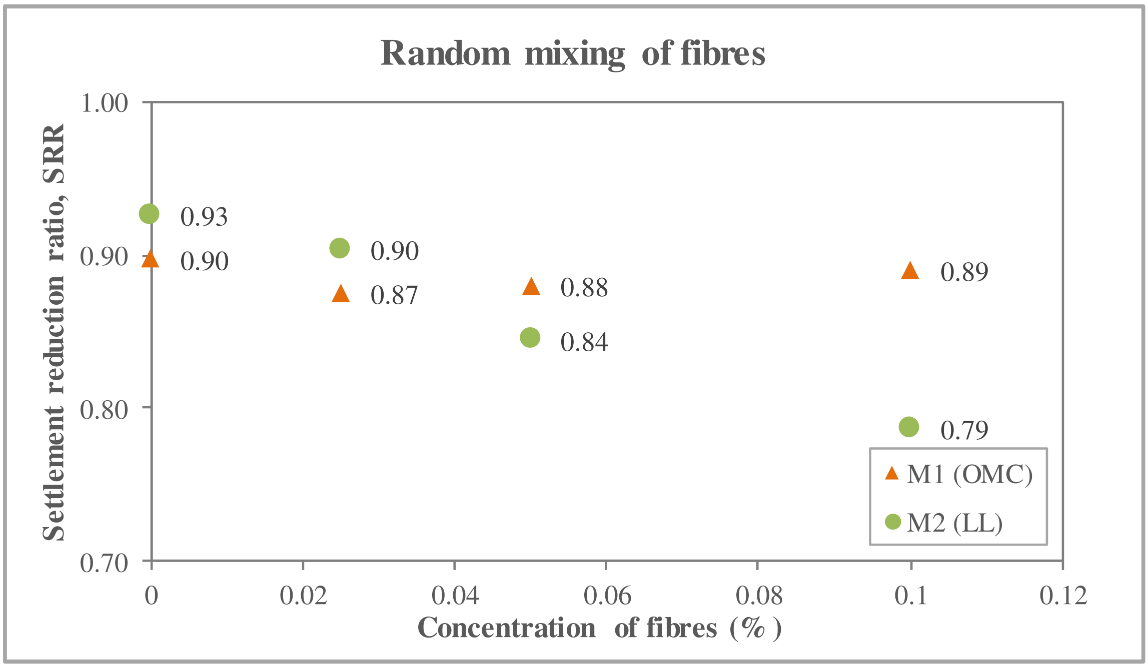


Figure 5.15: Effect of the concentration of fibres on the settlement reduction ratio when randomly mixed in the granular columns

5.4.2 Layering

When layers of flakes were used to reinforce the granular columns, in base soils at both OMC and LL, the relationship obtained between SRR and the flakes content is as shown in Figure 5.16. For OMC tests, reinforcing the columns with the flakes increased the degree of settlement, except for a concentration of 3.3 % where SRR was the same as for the OGC. However, in LL tests, a progressive drop in settlement was deduced since the SRR decreased as the flakes content was raised. As such, the optimum reduction in settlement was attained when the column was reinforced with a flakes concentration of 5.6 %.

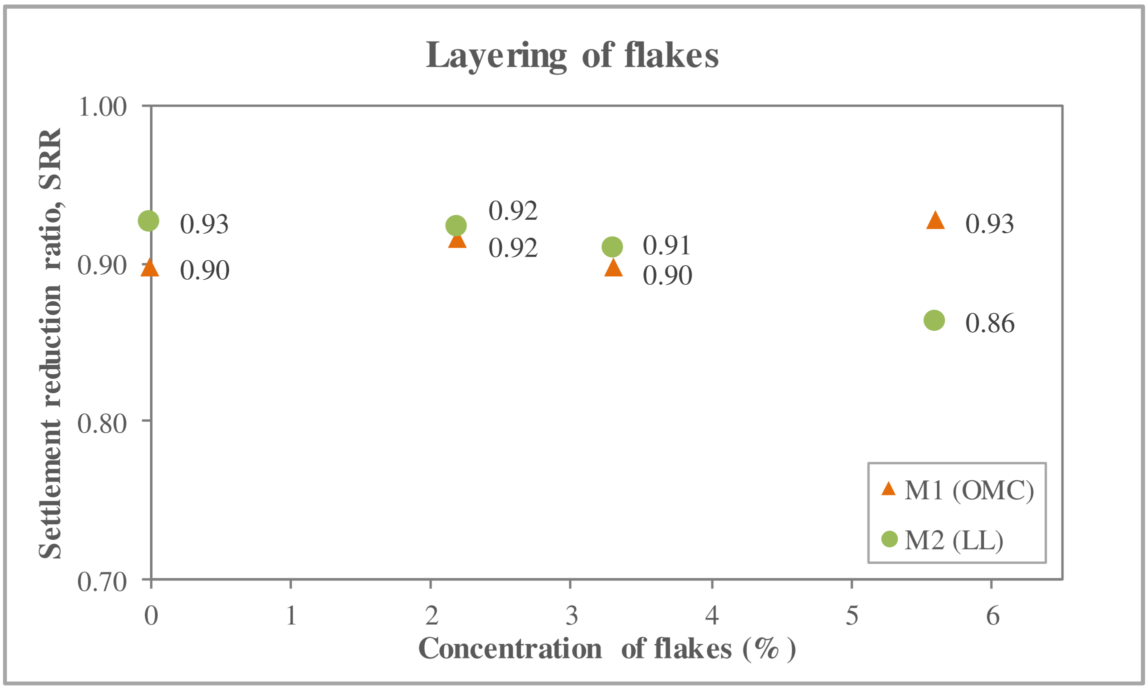


Figure 5.16: Effect of the concentration of flakes on the settlement reduction ratio when included as layers in the granular columns

An illustration of the effect of reinforcing granular columns with layers of fibres in the settlement reduction ratio is shown in Figure 5.17, for columns installed in base soils at OMC and LL. For the RGCs installed in a wet soil at OMC, the inclusion of fibres appeared to cause a negative effect on settlement reduction. As the fibre content augmented from 0 to 0.83 %, SRR gradually increased up to a maximum of 0.96 at the highest concentration. In contrast, LL tests demonstrated better response to settlement reduction with higher quantities of fibres. A change in SRR from 0.93 to 0.85 was recorded as the concentration was elevated from 0 to 0.83 %. However, it was worth noting that for a concentration of 0.56 % and beyond, SRR remained constant at 0.85.

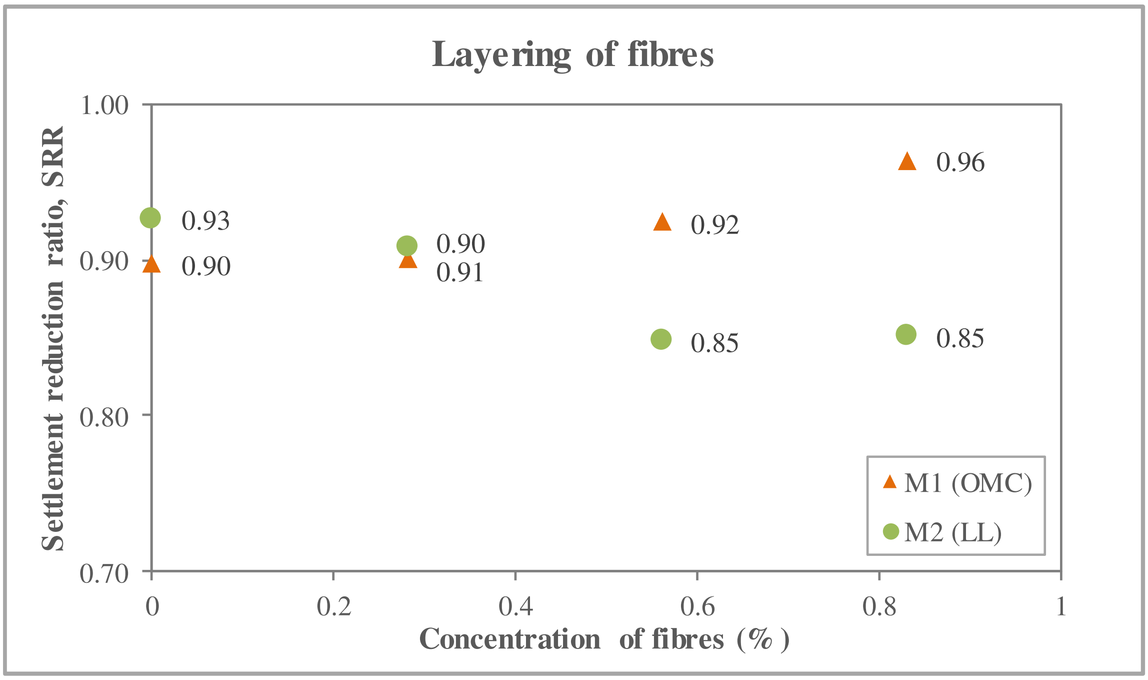


Figure 5.17: Effect of the concentration of fibres on the settlement reduction ratio when included as layers in the granular columns

In Figure 5.18, the relationship between SRR and the mass per unit area of Betatex is given when columns reinforced with this geotextile were installed in base soils at OMC or LL. The graphical plot confirmed that reinforcing granular columns with this geotextile generally reduce the degree of settlement of the composite ground. In OMC tests, SRR dropped from 0.90 to 0.84 when masses per unit area of 200 and 400 g/m2 were used compared to the OGC. Beyond these, SRR was again raised to reach 0.88 at 600 g/m2. Nevertheless, this value was still lower than that of the OGC. For the LL tests, the inclusion of the Betatex strongly reduced settlement especially when the thickest (600 g/m^2) geotextile was utilised, resulting in an SRR of 0.81.

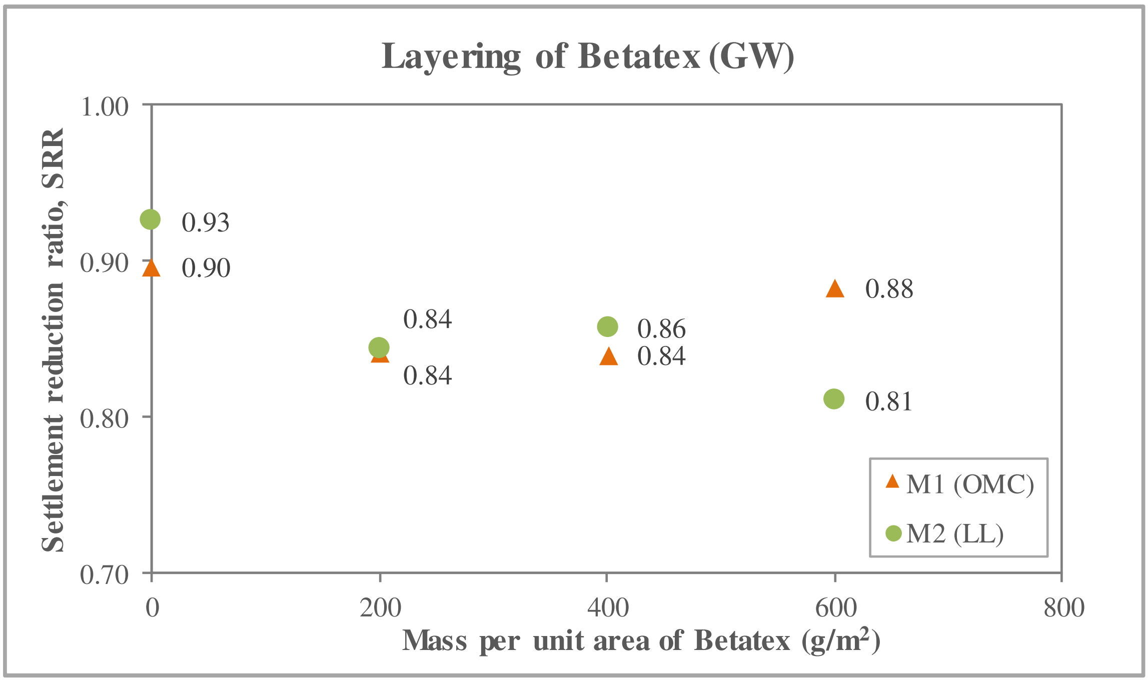


Figure 5.18: Effect of the mass per unit area of Betatex (GW) on the settlement reduction ratio when included as layers in the granular columns

When Fibertex (GV) was used, instead of Betatex (GW), Figure 5.19 was generated. For this material, the trends obtained in both OMC and LL tests are relatively similar. The only exception is noted at the mass per unit area of 600 g/m^2 whereby SRR was remarkably higher in the OMC test. Overall, GV600 in the LL tests demonstrated highest performance in terms of the settlement reduction with a corresponding SRR of 0.83.

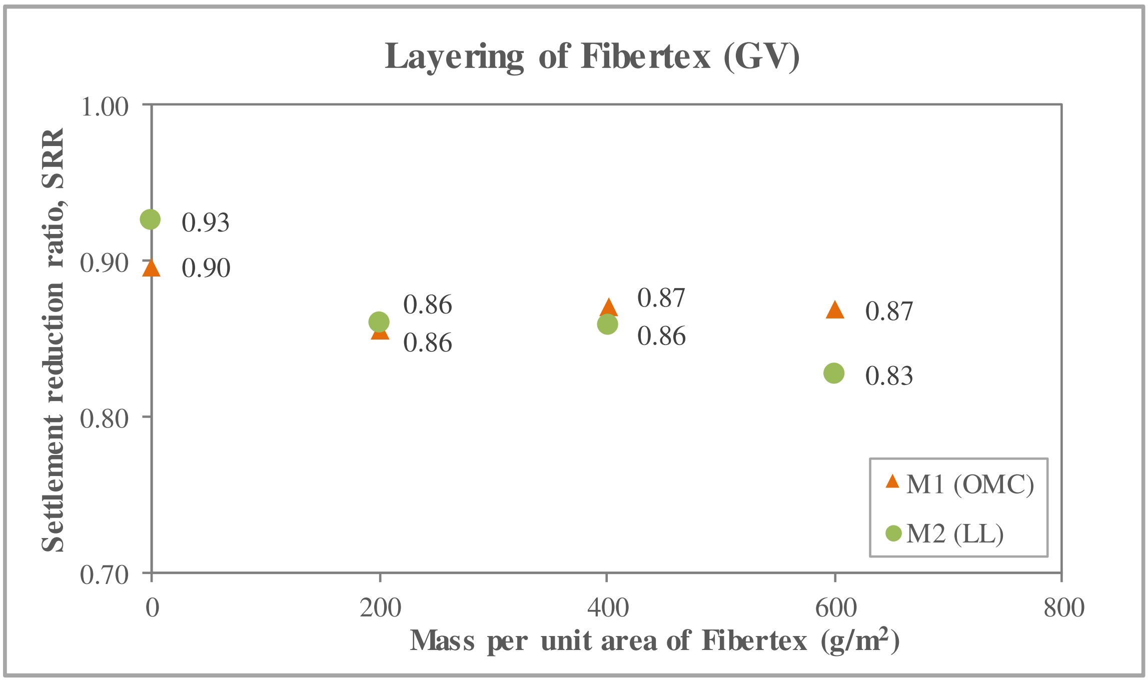


Figure 5.19: Effect of the mass per unit area of Fibertex (GV) on the settlement reduction ratio when included as layers in the granular columns

5.4.3 Discussions

In general, the granular column technique is highly efficient in settlement reduction (Goughnour, 1983; Van Impe, 1983; Ambily & Gandhi, 2007). This is largely due to the high stiffness attained in the composite ground as a result of the stronger column material utilised. Equations were presented in the literature review to compute the settlement achieved post treatment with granular columns. In the equations, it was noted that the settlement reduction ratio (SRR) shared a significant contribution. Therefore, in this section, the analysis was based on the SRR values which were computed from equation 2.22; the stress concentration ratios (n) which were determined in the previous section were used in this equation. The SRR values indicated that the lower they were, the smaller was the settlement attained.

Bergado, Alfaro & Chai (1991) compiled SRR values, as shown in Figure 5.20 from several past researches and claimed that they typically varied between 0 and 0.8. Shivashankar et al. (2011) also obtained results which were in line with Bergado, Alfaro & Chai (1991). A separate study conducted by Sobhee-Beetul (2012) reported the maximum SRR value as 0.65. In Figure 5.20, a red shaded rectangular box has been inserted to indicate the zone of variation of the SRR values attained (at an area replacement ratio of 0.11 which was used in this research) in comparison to the other studies suggested by Bergado, Alfaro & Chai (1991). The calculated SRR values from this study (between 0.79 and 0.93) appear to be relatable to other studies when additionally considering the stress concentration ratio (n) which varied between 1.7 and 3.4.

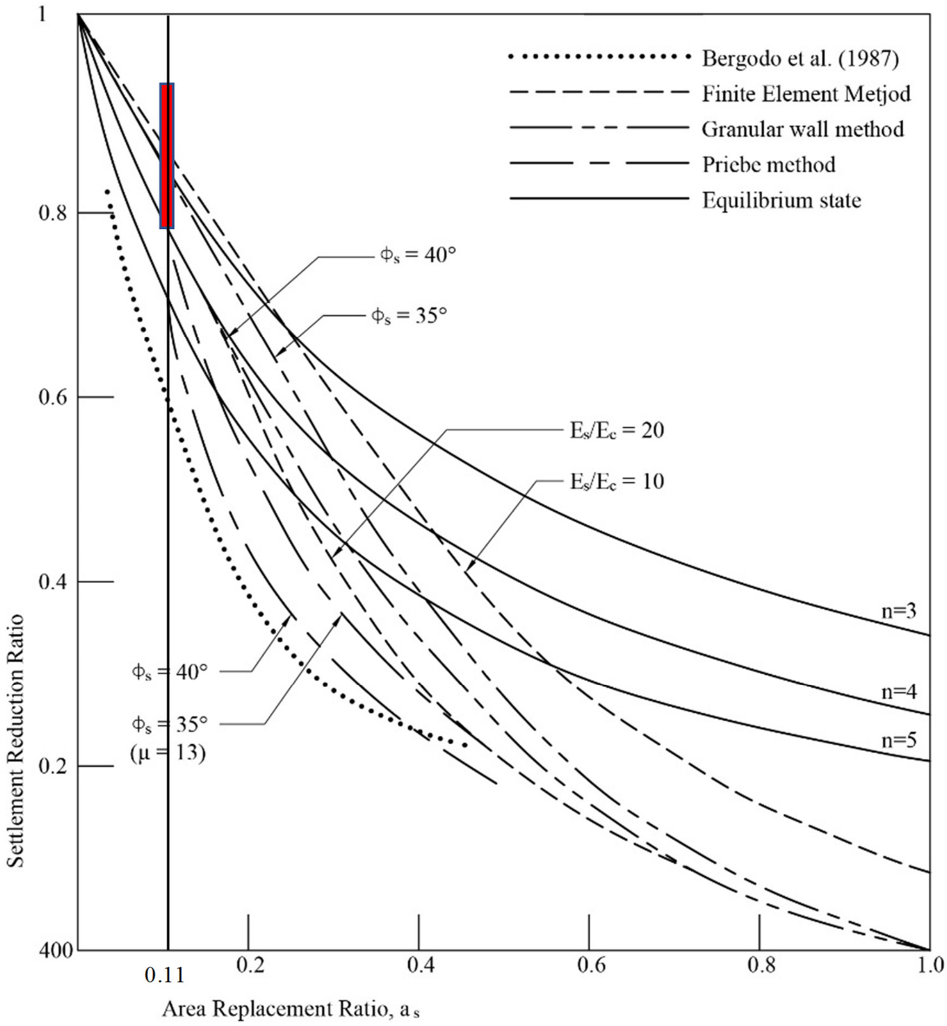


Figure 5.20: Compilation of SRR values from previous studies (adapted from Bergado, Alfaro & Chai, 1991)

When ordinary granular columns are reinforced, they can potentially cause an additional decrease in settlement of the improved ground, depending on the type and placement of the reinforcement (Tallapragada, Golait & Zade, 2011; Tandel, Solanki & Desai, 2014; Al-Obaily, 2017). This occurs due to an augmented stiffness or even density of the column (Malarvizhi & Ilamparuthi, 2004). Tallapragada, Golait & Zade (2011) reported a reduction in settlement between 11.9 to 41.5 %. Tandel, Solanki & Desai (2014) also claimed relatively similar settlement reductions which varied between 20 and 54 %.

From the analysis of the SRR values computed for the results obtained in this study, it appeared that randomly mixed fibres produced the highest reduction in settlement when used to reinforce columns which are installed in a base soil at LL. The corresponding SRR value for this test was 0.79, at a fibre concentration of 0.1 %. Comparatively, the SRR value of 0.81 produced by the 600 g/m^2 of Betatex in the LL test was relatively close. The largest settlement reduction from the fibres can be explained by the formation of a much stiffer and stronger mass within the column, as the sand particles interlock around the fibres. The spaces in between the particles are, therefore, largely filled in by the fibres. Furthermore, the presence of the fibres in this arrangement probably increased the shearing resistance which subsequently lowered the degree of settlement. Based on this analogy, fibres possibly produced the stiffest column when included in this arrangement and when the RGC was installed in a base soil at LL. Generally, in the wetter base soils, the RGCs tend to bear even higher loads than the surrounding soil in comparison to the columns installed in base soils at OMC. Consequently, the settlement is reduced. By reinforcing the column, it is able to take even higher loads and thus the settlement is further reduced.

5.5 Maximum lateral bulging behaviour for each tested column

The deformation characteristics of the columns in each test was graphically presented in Chapter 4 and the respective maximum lateral bulging was established through the plots. This section essentially analysed how the moisture content of the base soil affected this behaviour. During the analysis, each type of reinforcement and their arrangement within the column was independently studied. The deformation response to the amount of reinforcement was plotted together with that for the OGC. This was necessary to compare how the largest bulge was impacted by the inclusion of the reinforcement. Therefore, all comparisons were made using the OGC as the control for both the OMC and the LL tests. Subsections 5.5.1 and 5.5.2 presents the analysis related to maximum bulging of the columns in terms of the random mixing and layering arrangement, respectively. The observations made are afterwards discussed in subsection 5.5.3.

5.5.1 Random mixing

5.5.1.1 Flakes

In Figure 5.21, the effect of the concentration of flakes on the maximum lateral bulging is shown, when the RGCs are installed in base soils at OMC and LL. From the trends generated, it is evident that bulging was more prominent in LL tests compared to that in OMC tests. While the diameter of the largest bulge varied between 128 and 137 mm in the OMC tests, this range was between 144 and 164 mm in the LL tests. Between a flakes content of 0 and 1 %, both OMC and LL tests followed similar trends. Beyond that, at a flakes concentration of 2.5 %, the OMC test displayed an increase in the bulge size while a reduction was rather observed in the LL test. Conclusively, flakes concentrations of 1 and 2.5 % appeared to produce the corresponding lowest bulging diameters of 125 and 144 mm in OMC and LL tests, respectively.

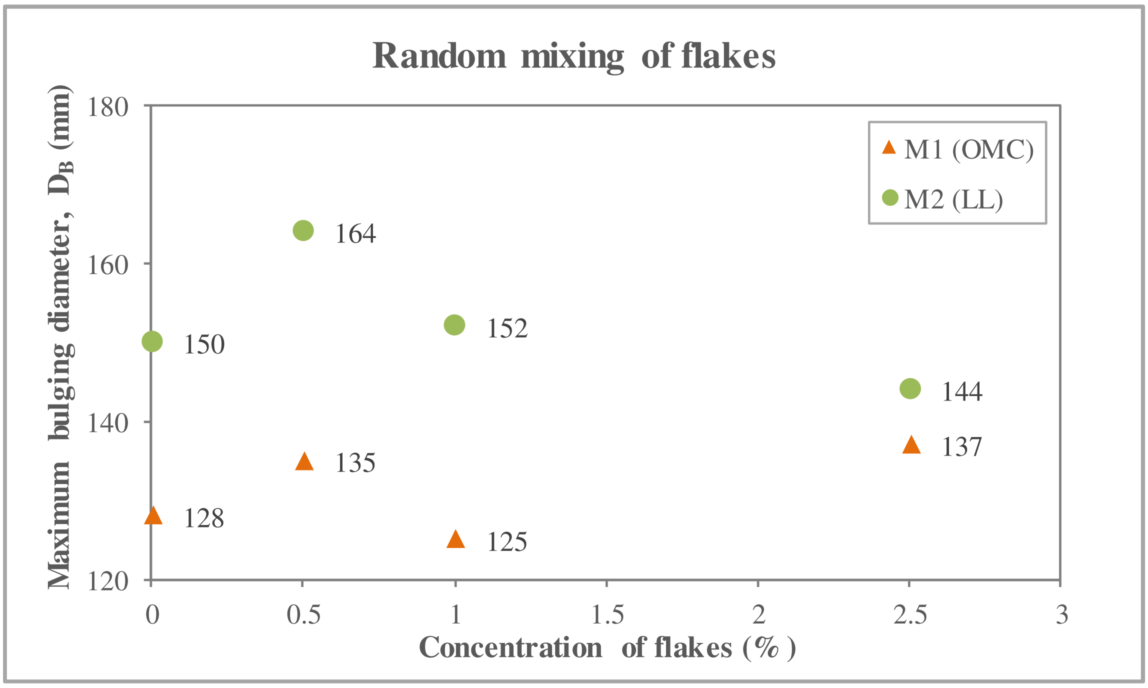


Figure 5.21: Effect of the concentration of flakes on the maximum bulging diameter when randomly mixed in the granular columns

5.5.1.2 Fibres

Figure 5.22 displays the maximum bulging diameters obtained for the OMC and LL tests when randomly mixed fibres were used to reinforce the columns. For the OMC test series, it was noted that bulging increased slightly (from 128 to 130 mm) with the addition of the lowest concentration of fibres of 0.025 %. As the amount of fibre was increased to 0.05 %, the size of the bulge reduced to 128 mm such that is was equal to that of the OGC. The highest fibre concentration of 0.1 % exhibited even lower lateral deformation. In the LL test series, the initial inclusion of 0.025 % of fibres in the column showed a decrease in the bulging diameter, from 150 to 140 mm. As the fibre content was further increased, a primary increase in the bulge size was recorded followed by a subsequent reduction to produce a diameter of 144 mm. In general, it was noted that the column with the highest amount of fibres (0.1 %) exhibited optimal results in the OMC tests while that in the LL tests corresponded to the column with the lowest reinforcement content (0.025 %).

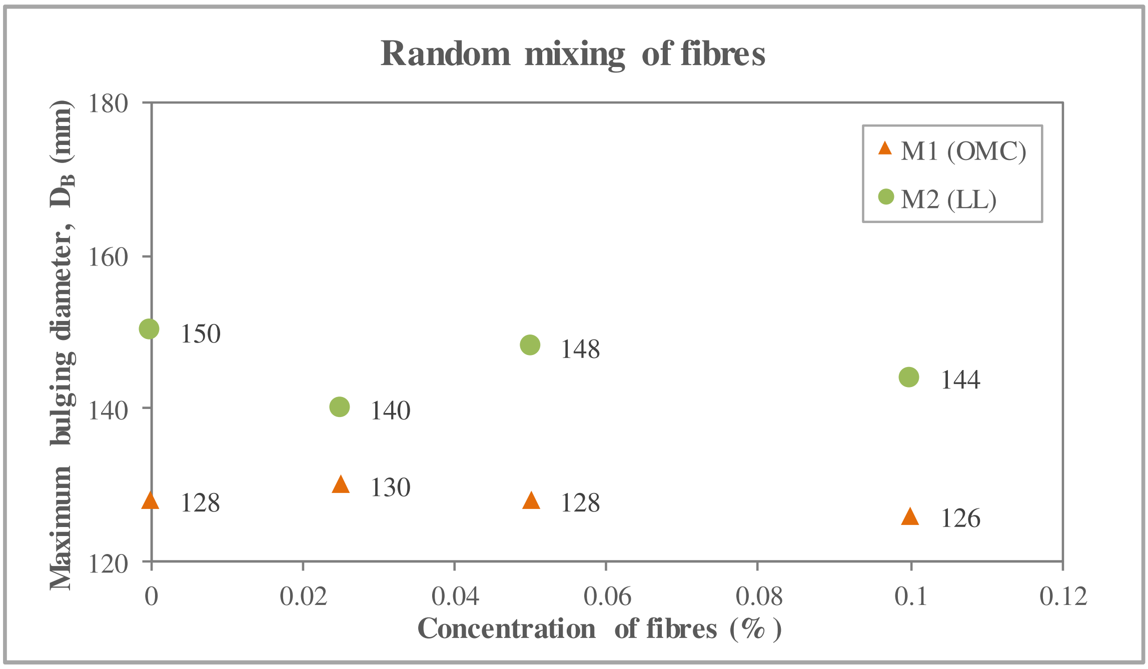


Figure 5.22: Effect of the concentration of fibres on the maximum bulging diameter when randomly mixed in the granular columns

5.5.2 Layering

5.5.2.1 Flakes

When flakes are included in the columns in the form of layers, the maximum lateral bulging in both the OMC and LL test series are shown in Figure 5.23. Relatively similar trends were recorded in both OMC and LL tests. Generally, it appeared that the inclusion of the fibres diminished the size of the bulge. More specifically, as the concentration of flakes was raised, the bulging diameter decreased. However, in the LL tests, a flakes content of 2.2 and 3.3 % produced identical maximum bulging. It was also worth noting that the progressive reduction in diameter was relatively low in the OMC tests. The optimum performance in the OMC and LL test series was, therefore, noted at a concentration of 5.6 % and with respective diameters of 120 and 138, respectively.

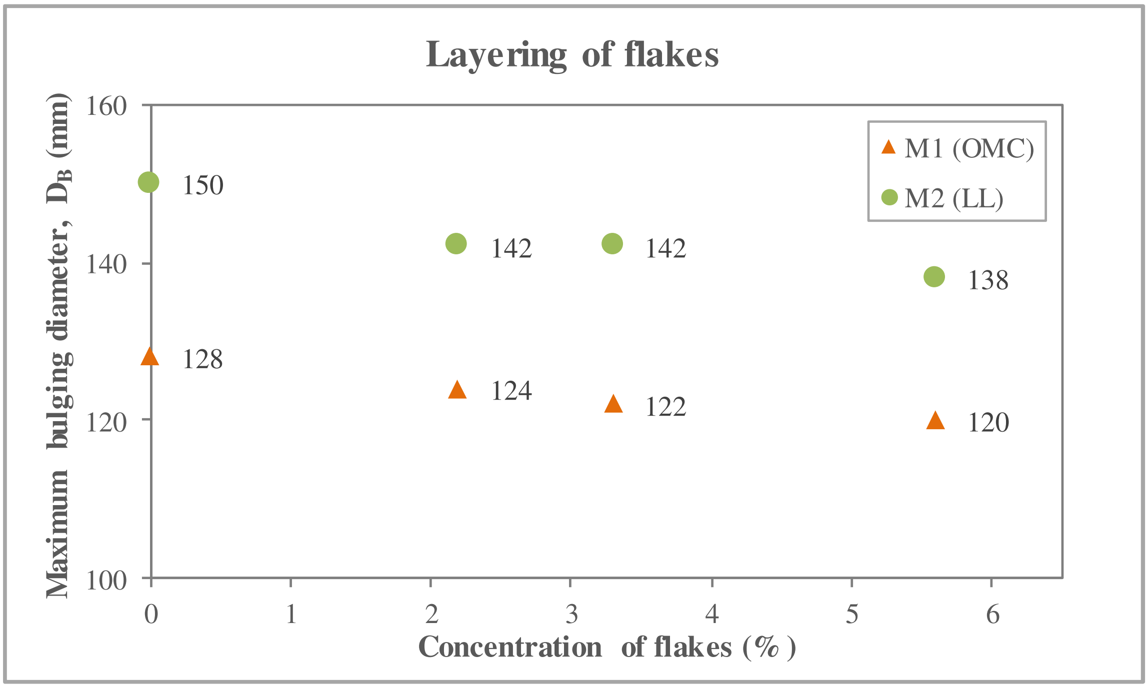


Figure 5.23: Effect of the concentration of flakes on the maximum bulging diameter when included as layers in the granular columns

5.5.2.2 Fibres

Figure 5.24 shows the maximum bulging diameter obtained for both OMC and LL tests, when the granular columns were reinforced with layers of fibres. Overall, it was evident that the inclusion of the fibres reduced the size of the largest bulge in each column, whereby a progressive increase in fibre content produced smaller lateral deformations. It was also noted that concentrations of 0.28 and 0.56 % produced an identical bulge diameter of 116 mm. For these tests conducted on columns reinforced with layers of fibres, the optimum results in both OMC and LL tests appeared to occur when a fibre content of 0.83 % was utilised, with respective bulging diameters of 110 and 130 mm.

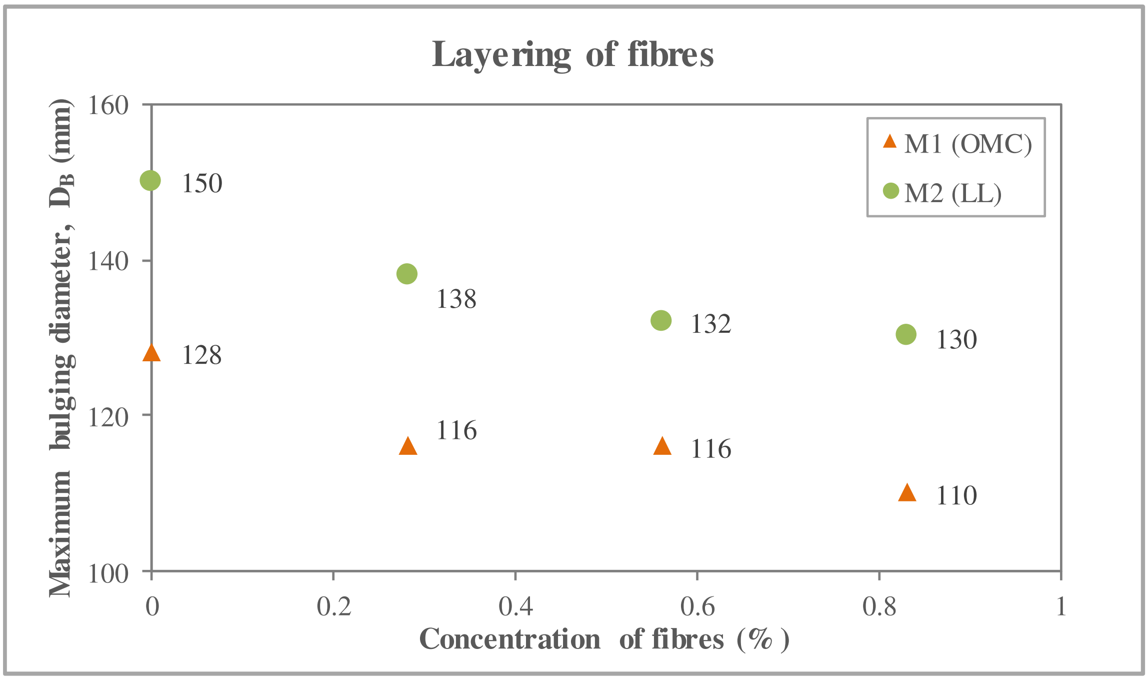


Figure 5.24: Effect of the concentration of fibres on the maximum bulging diameter when included as layers in the granular columns

5.5.2.3 Betatex (GW)

The inclusion of Betatex layers in the granular columns showed a continuous reduction in the bulge size as the mass per unit area of the geotextile was increased. This is clearly depicted in Figure 5.25 whereby the decrease in diameter appeared smaller in OMC tests compared to the LL ones. In fact, as the mass per unit area of the geotextile was increased from 0 to 600 g/m^2 (GW600), the total drop in bulging diameter was only 8 mm for the OMC tests compared to that in the LL tests which was calculated as 26 mm. Evidently, GW600 produced the lowest bulge size in both OMC and LL tests, which was equivalent to 120 and 124 mm, respectively.

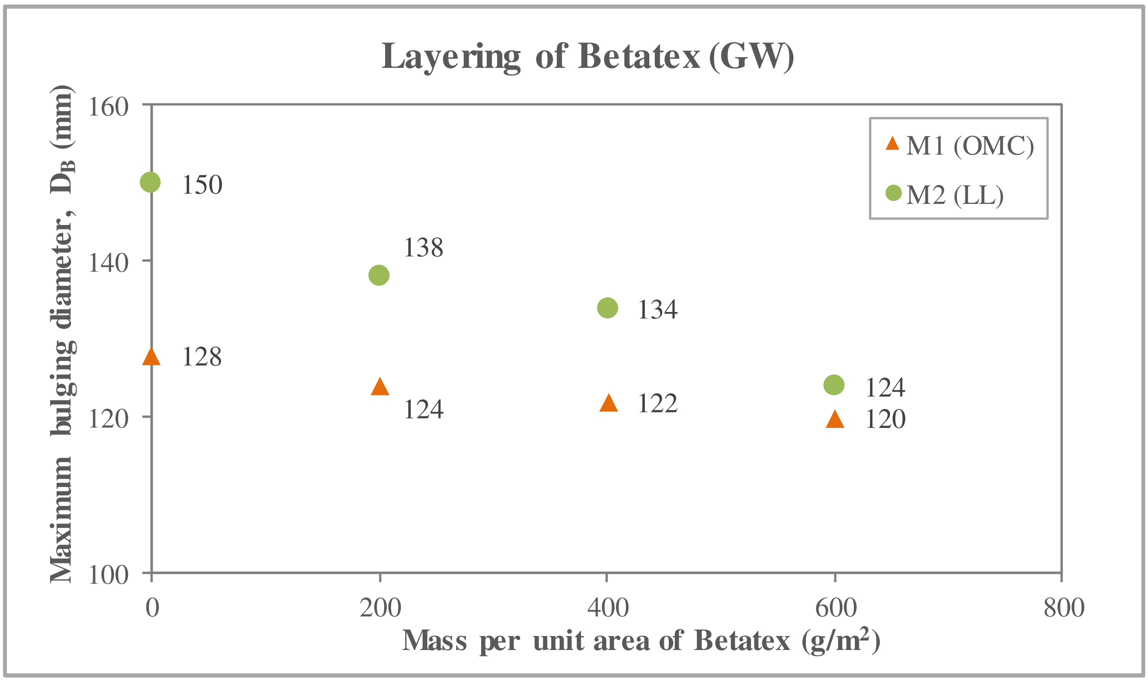


Figure 5.25: Effect of the Betatex (GW) geotextile on the maximum bulging diameter when included as layers in the granular columns

5.5.2.4 Fibertex (GV)

Compared to Figure 5.25, Figure 5.26 displays similar relationships except that Fibertex was used in the tests instead of Betatex. The inclusion of this geotextile demonstrated a decline in the size of the bulge as the mass per unit area of Fibertex was increased from 0 to 600 g/m^2, in both OMC and LL tests. In the OMC test series, the diameter was comparable when GV400 and GV600 were used and they coincidently represented the smallest diameter of 120 mm which was attained. However, in the LL test series, a similar behaviour was noted when GV200 and GV400 was used, with the diameters measuring 132 mm. The lowest lateral deformation in the LL tests was recorded as 130 mm, when a mass per unit area of 600 g/m^2 was utilised.

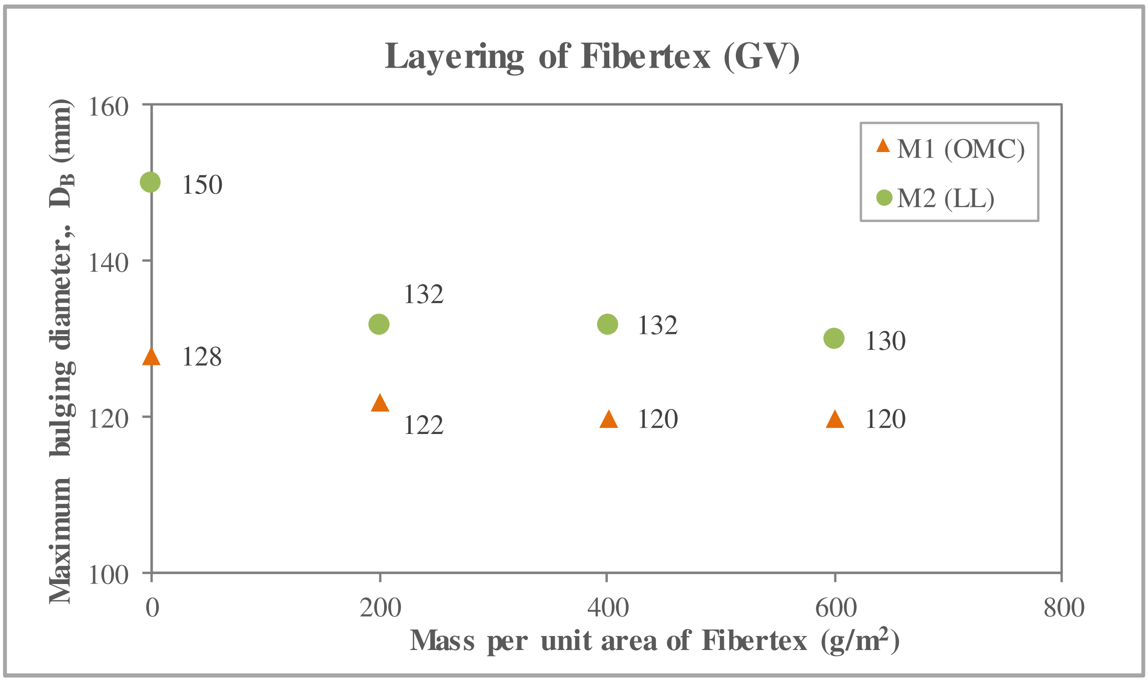


Figure 5.26: Effect of the Fibertex (GV) geotextile on the maximum bulging diameter when included as layers in the granular columns

5.5.3 Discussions

The illustrations used in this section only analysed maximum bulging in the columns (both OGC and RGC) post-testing, rather than comparing the maximum bulging diameters before (pre-bulging arising due to installation procedures) and after the experiments were conducted. This is because the deformation pattern of each installed column (before testing) varied remarkably due to the effect of each type of reinforcement and their arrangement. This imply that each column would have had to be modelled twice to obtain their deformation patterns before and after the tests. This was not practically possible since 2 specimens would have had to be prepared for any one test. To achieve such measurements, a sample would have been needed to purely cast a plaster of Paris column even without being tested; this would have given the maximum pre-bulging diameter which was only due to the installation processes for that respective test. It would then have been necessary to prepare a second sample and test it, following which the column would have been casted with plaster of Paris to establish the maximum bulging diameter post testing. This approach was not deemed accurate since the likelihood of the pre-bulging size before the test, in both samples for any one test, being equivalent was nearly impossible. The reason for this small possible difference was the reinforcements which would not necessarily be identical, or even precisely positioned within the columns, although the repeatability of producing identical test samples was significantly high.

Nevertheless, the bulging behaviour of the OGCs (in base soils at OMC and LL) were physically modelled to establish that there was a difference in the bulge size before and after the test. This step was also necessary to establish whether the diameter of the installed column was indeed 100 mm, as envisioned. The deformation characteristics of the OGCs (installed in base soils at both OMC and LL) which were generated from these models are shown in Figure 5.27. From the plotted columns, it was evident that the initial diameter of the column was not uniform, and it was clearly not equal to 100 mm as was assumed (slightly higher than 100 mm). Several studies (Hu, 1995; Malarvizhi & Ilamparuthi, 2004; Basu, 2009; Mekkiyah & Al-Saadi, 2016) have reported the maximum bulging diameter or even the L_c/D_c (L_c and D_c are the length and diameter of the column) ratio in terms of the initial diameter of the column, assuming that all the columns were of the same diameter before the experiment. This research indicated that this approach is not necessarily accurate and, results were rather reported as the maximum bulging diameter attained at the end of the test (at a settlement of 50 mm) and comparisons were also made accordingly.

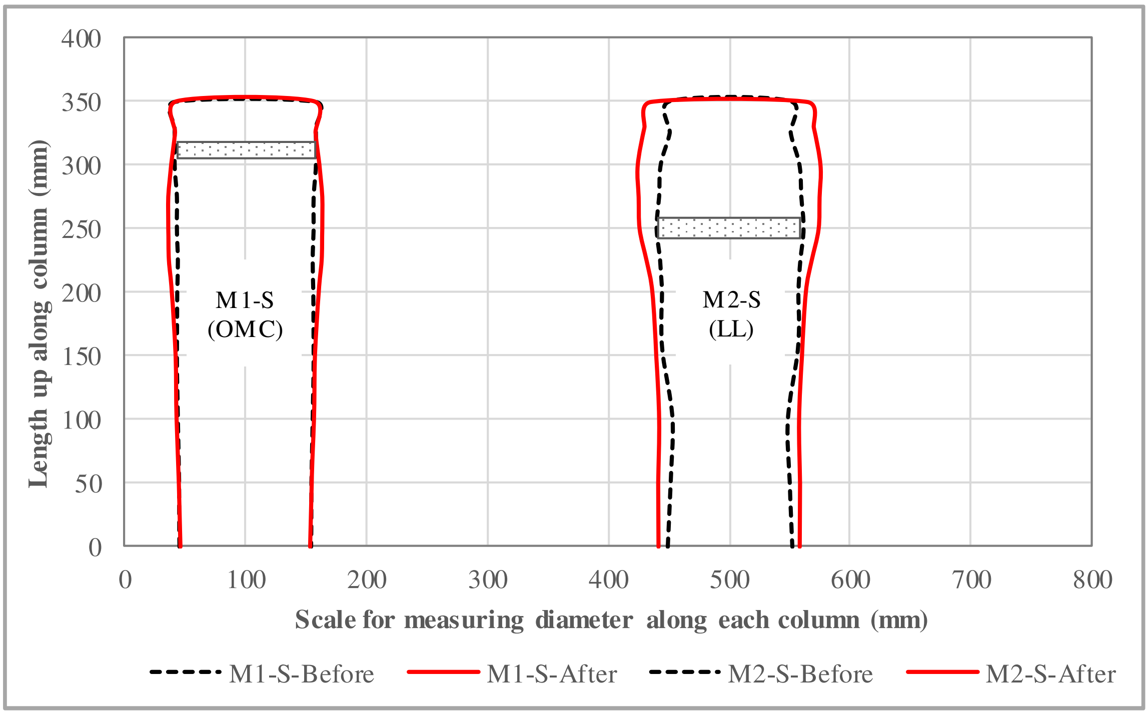


Figure 5.27: Deformation of OGCs before and after the test in base soils at OMC and LL

In the literature review, bulging was highlighted as one of the most common failure mechanisms in granular columns. Normally, as the column is loaded, it undergoes lateral deformation. This is possible since the granular particles are not bounded to form a single unit, compared to concrete piles. Bulging is typically more prominent when the moisture content of the base soil is high. This is explained by the low confining stresses provided by the host soil. As such, the load carrying capacity reduces while simultaneously increasing the settlement. Basu (2009) claimed that reinforcing of these columns, internally or encasing, can reduce the extent of bulging. Ali (2014) and Tandel, Solanki & Desai (2014) also shared similar views. In this research, a similar approach was adopted whereby internal reinforcing (internally incorporating the reinforcement in the column rather than encasing the latter) was the only method used for including the reinforcements.

Figures 5.21 to 5.26 summarised the different bulging diameters obtained in each test. Overall, bulging occurred in all the columns. This is in line with Barksdale & Bachus (1983) who claimed that failure occurs in bulging when the length of the column is greater than 3 times that of its diameter. In the present research, the column length was 4 times that of its assumed diameter of 100 mm. From the results, it was generally found that bulging was lower in columns which were installed in base soils at OMC compared to those in the LL tests. This can be explained by the higher confining stresses around the column, generated by the stiffer base soil at OMC. At LL, the base soil was much softer and therefore resulted in a less stiff soil bed. As such, the confinement provided to the column was much lower in LL tests. In the OMC tests, the smallest deformation of 110 mm was recorded in the test where layers of 0.83 % of fibres were used to reinforce the granular columns. This represented a drop of 18 mm in the diameter of the bulge, compared to that which was observed in the OGC (128 mm) under similar testing conditions. Basu (2009) also reported that the inclusion of fibres in granular columns reduced the bulging size. However, in Basu's study, the fibres were randomly mixed into the sand. In comparison to Basu's study, the deformation results achieved also confirmed that there was a reduction in the bulge diameter with the incorporation of the fibres. Nevertheless, the layering arrangement generally appeared to have a better impact than the random mixing one on bulge size reduction. This observation was also made for the other types of materials.

The lowering of this diameter due to the fibres can be explained by the heavy interlocking of particles around the reinforcement material. Essentially, the anticipated presence of void between the sand particles were probably occupied by the fibres which in turn produced a much stiffer and compacted column mass. As the column was vertically loaded, the stress within it increased to a point where it exceeded the surrounding confining stress, and therefore equilibrium was compromised. Since equilibrium was no longer applicable, and the lateral stress within the column was much higher than the confining stress from the base soil, the column material migrated outwards laterally. As the vertically applied stress was further raised, the column mass most likely experienced shearing as the confining stress was not adequate to resist the stress transfer from the column. Since previous research showed that soil reinforcement with PET fibres caused an increase in shear strength, the shear strength of the column was also increased when fibres were incorporated (Consoli et al., 2002). For this reason, the bulging diameter was lower (when fibres were used) since it restricted lateral outward movement of the column materials. With the layering arrangement, the quantity of the fibres used was much higher and was more concentrated at regular intervals. This probably favoured a much higher interlocking of the sand particles around them, thus increasing the shear strength of the column mass as opposed to when the reinforcements were randomly mixed. A relatively similar understanding may be applied to the flakes used for reinforcing the columns. However, the large smooth, and rigid surfaces of the flakes possibly reduced the interlocking potential. Consequently, the shear strength was lower, thereby generating higher lateral deformations than the columns reinforced with fibres.

In the LL tests, the lowest bulging diameter was 124 mm and it corresponded to the column which was reinforced by Betatex of a mass per unit area of 600 g/m^2. This signified a reduction of 26 mm in the diameter of the bulge compared to that of the OGC in the LL tests. The Betatex geotextile also behaved adequately as a reinforcement material. Due to its stiffness and rough surface, the sand particles were better able to bond to the surface, compared to the flakes with smoother surfaces. Therefore, the shear strength of the Betatex reinforced column was remarkably higher and restricted the extent of bulging. Nevertheless, overall bulging remained the lowest when fibres were used in the OMC tests.

5.6 Length span corresponding to the maximum lateral bulging zone

Previous studies have regularly pointed out that the maximum lateral bulging occurs at a certain position along the length of the column. However, it was found in this research that the largest bulge occurred within a particular zone rather than at a specific length. Therefore, the results presented in Chapter 4, pertaining to the deformation of the columns, were further analysed to generate Figures 5.28 to 5.33. The diameter and length of the maximum bulging zones were summarised in Tables 4.5 and 4.6. Box and whisker plots were preferred for this representation to enable an easy comparison of the length and position of the maximum bulging zone for every test. In each figure, the results for both OMC and LL tests were plotted on the same chart to easily understand how the moisture content affects the location and extent of highest bulging within each column. Box and whisker plots were considered appropriate since they clearly define the principal points of interest within the zone: lowest, median (or average) and highest lengths. On the given figures, only the y-axis was used for a scaled representation of each box and whisker plot which illustrates the total length of maximum bulging along the column (the scale on the y-axis does not graphically represent the entire column length since the base of the latter is equivalent to 0 mm on the y-axis while the top corresponds to 350 mm). However, the x-axis did not denote a measure for any parameters, but rather for positioning the individual plots (within a test series) next to each other for comparison purposes. Analysis of the results obtained in Chapter 4 are presented in terms of the random mixing and layering arrangements in sub-sections 5.6.1 and 5.6.2, respectively. The observations are then discussed in sub-section 5.6.3.

5.6.1 Random mixing

Figure 5.28 illustrates the position of the maximum bulging zone for both OMC and LL tests when the columns were reinforced with randomly mixed flakes. In the plots for the LL tests, it was evident that the zone of maximum bulging in the RGCs typically occurred within length spans (250 to 340 mm) closer to the top of the column. However, in OMC tests, the bulging zones were located further down (235 to 285 mm) the columns. Additionally, the length span in the LL test with OGC was longer than that of the same column when installed in a base soil at OMC. Overall, it was apparent that the inclusion of the reinforcement through the random mixing arrangement drastically reduced the length span of the zone of highest bulging. In the OMC and LL test series, the shortest length span was recorded when a granular column was reinforced with 2.5 and 0.5 % of randomly mixed flakes, respectively. Therefore, it can be said that higher flakes concentrations resulted in small bulging zones (245 to 255 mm) in OMC tests. Contrastively, small bulging zones (315 to 325 mm) were obtained with lower flakes content in LL tests.

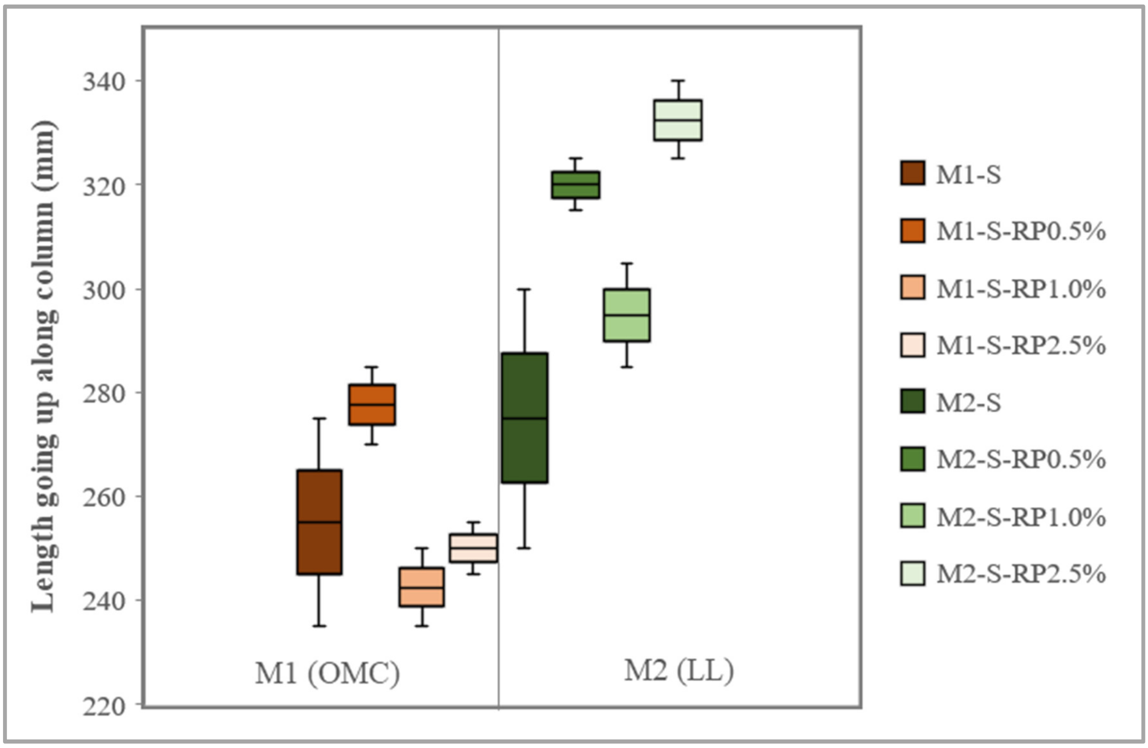


Figure 5.28: Position of the maximum bulging diameter along the columns reinforced with randomly mixed flakes and installed in base clays at both OMC and LL

When the granular columns were reinforced with randomly mixed fibres, and installed in base soils at OMC or LL, the length span obtained which corresponded to the largest deformation zone were as shown in Figure 5.29. Overall, it was observed that the length span in OMC tests were much longer compared to those in the LL tests. Also, in the OMC tests, this length reduced (from a length span of 40 to 25 mm)) as the fibre content was increased from 0 to 0.05 %. Beyond this concentration, the length span augmented to 30 mm although the zone appeared to be located further down along the column. A dissimilar observation was made in the LL test series. A drastic reduction in the length span was attained with the inclusion of the fibres. In fact, the length span was identical for fibre contents of 0.025 and 0.05 %. When a concentration of 0.1 % of the reinforcing material was used, the length span was almost the same. However, a slight difference was noted in the position of the largest deformation zone at this concentration such that the location of this area was higher up (300 to 310 mm) in the column.

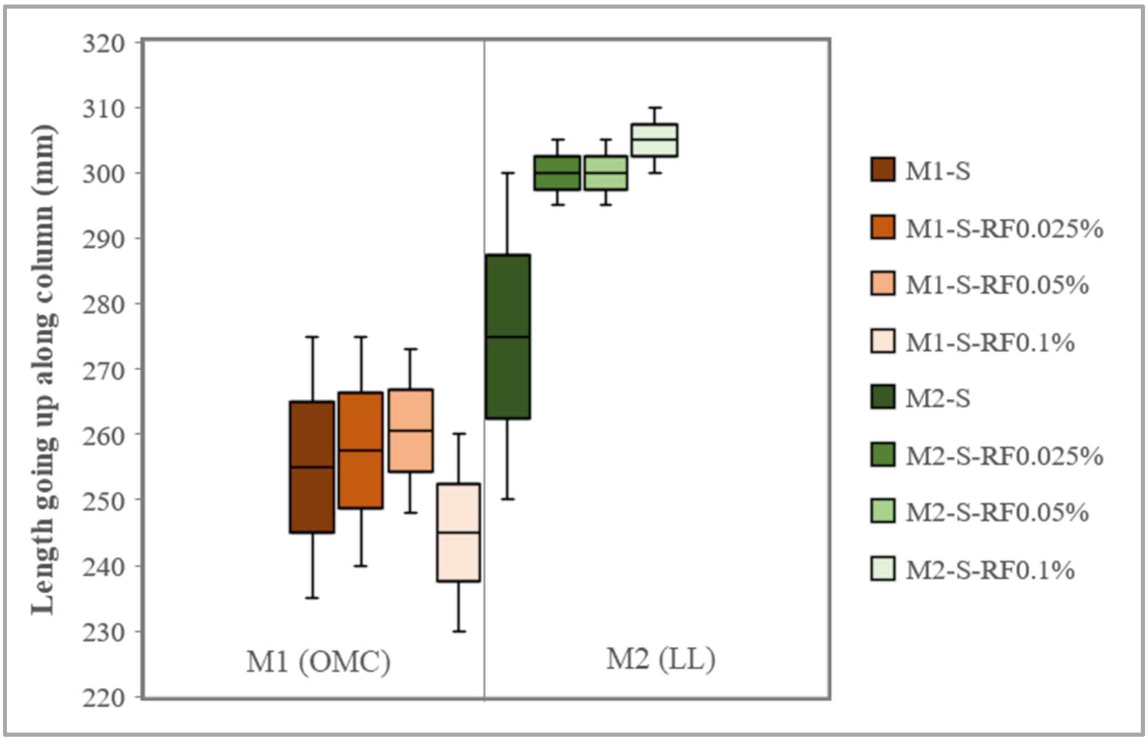


Figure 5.29: Position of the maximum bulging diameter along the columns reinforced with randomly mixed fibres and installed in base clays at both OMC and LL

5.6.2 Layering

When flakes were used in the layering arrangement, the deformation zones obtained in tests (for both OMC and LL tests) are as shown in Figure 5.30. In the OMC tests, flakes concentrations of 2.2 and 3.3 % caused the length span to extend from 40 to 50 mm while the position differed slightly compared to that in the OGC. At a higher flakes content of 5.6 %, the length span corresponding to the maximum deformation zone significantly reduced. However, the area was positioned further down along the column. In comparison to OMC tests, the zone of largest lateral deformation occurred higher in the columns for LL tests. Evidently, the smallest flakes content of 2.2 % did not have any impact on the position and length span of the bulging zone compared to that of the OGC. However, further increase in flakes concentration triggered a reduction in the length span and the position of the largest deformation area such that the latter was located higher up (225 to 275 mm) in the column.

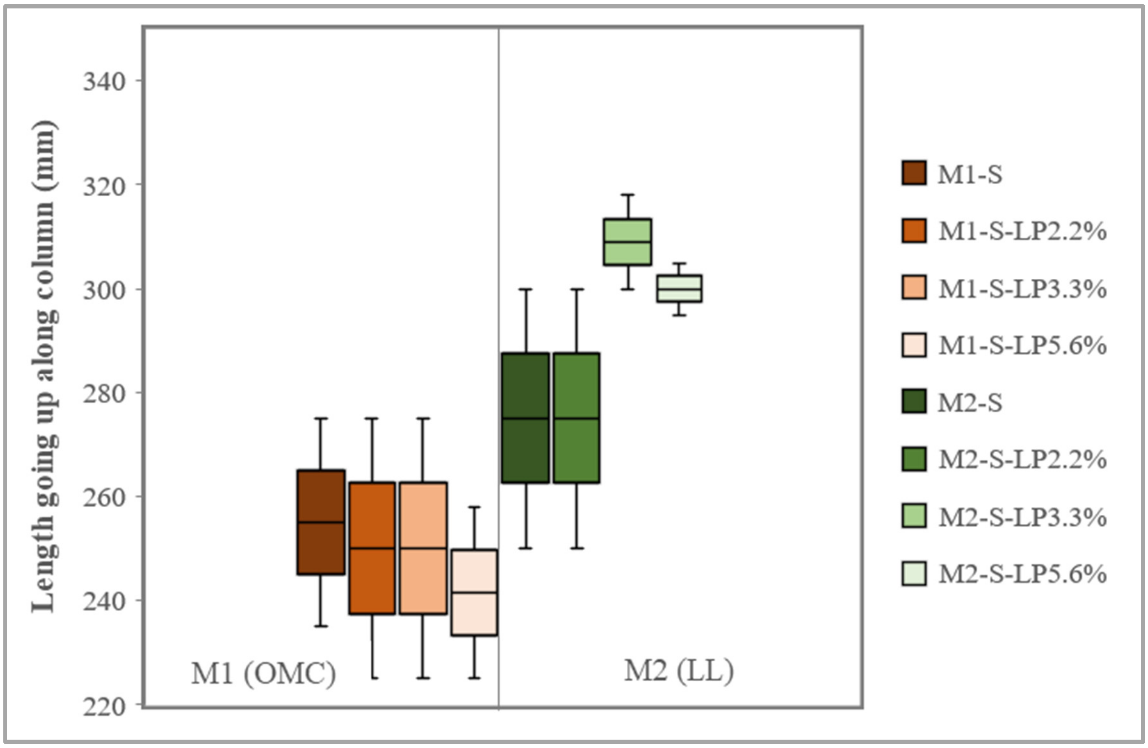


Figure 5.30: Position of the maximum bulging diameter along the columns reinforced with layers of flakes and installed in base clays at both OMC and LL

In Figure 5.31, the positions of the maximum bulging zones are shown for tests conducted on columns reinforced with layers of fibres. In general, it appeared that the length spans and their positions along the columns did not differ significantly. However, exceptions were noted when the highest and lowest concentrations of fibres were used in the OMC and LL tests, respectively. In the OMC test, a fibre concentration of 0.83 % caused a drastically long span (50 to 290 mm). This implied that bulging of the column for that test was more uniformly spread along the column (length span of 240 mm). However, in the LL test, the extremely short length span of 5 mm confirmed that the inclusion of the 0.28 % of fibres caused an insignificant amount of bulging. The trend in the LL tests showed that as the amount of fibres was raised from 0 to 0.28 %, the length span decreased (from 50 to 5 mm). Beyond a concentration of 0.28 % of fibres, the length span augmented with the addition of fibres.

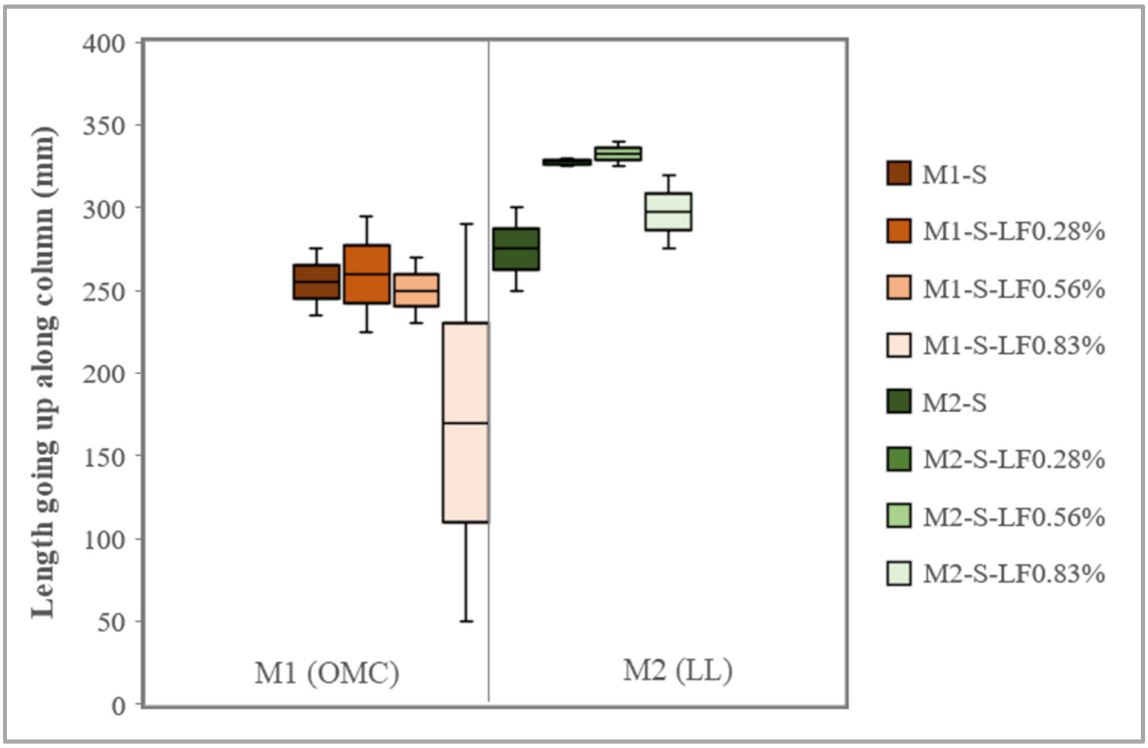


Figure 5.31: Position of the maximum bulging diameter along the columns reinforced with layers of fibres and installed in base clays at both OMC and LL

For the rather more stiffer materials, that is the Betatex and the Fibertex geotextiles, the associated maximum deformation zones and the corresponding length spans within the columns are shown in Figures 5.32 and 5.33. When the columns were reinforced with Betatex in the OMC tests, the positions of maximum bulging were fairly centred around the same location as shown in Figure 5.32 (between 200 and 295 mm). However, the length span appeared to increase as the mass per unit area of the geotextile was increased from 200 to 600 g/m^2. This implied that bulging was more consistently distributed along the columns. In the LL tests, where Betatex was used to reinforce the columns, an opposite behaviour was observed. The inclusion of the Betatex geotextile diminished the length span of the bulging zone, and simultaneously positioning it within the top one third segment of the column. This was noted in all the RGCs. Relatively similar behaviour was achieved in the LL tests (as shown in Figure 5.33) when the columns were reinforced with the Fibertex geotextile; the maximum bulging zones were located in the top third of the column. However, in the OMC tests, a drastic difference was observed in the results. The GV200 produced maximum bulging over a much longer length span (between 200 to 275 mm) compared to that of the OGC (between 235 to 275 mm) tested under the same conditions. As the mass per unit area was increased from 200 to 400 g/m^2, an extreme decrease in length span was obtained while the largest deformation occurred within a zone located higher up in the column. Upon further augmenting the mass per unit area to 600 g/m^2, the length span became longer again and was also situated further down in the column. This position and length were relatively comparable to the ones achieved in the OGC.

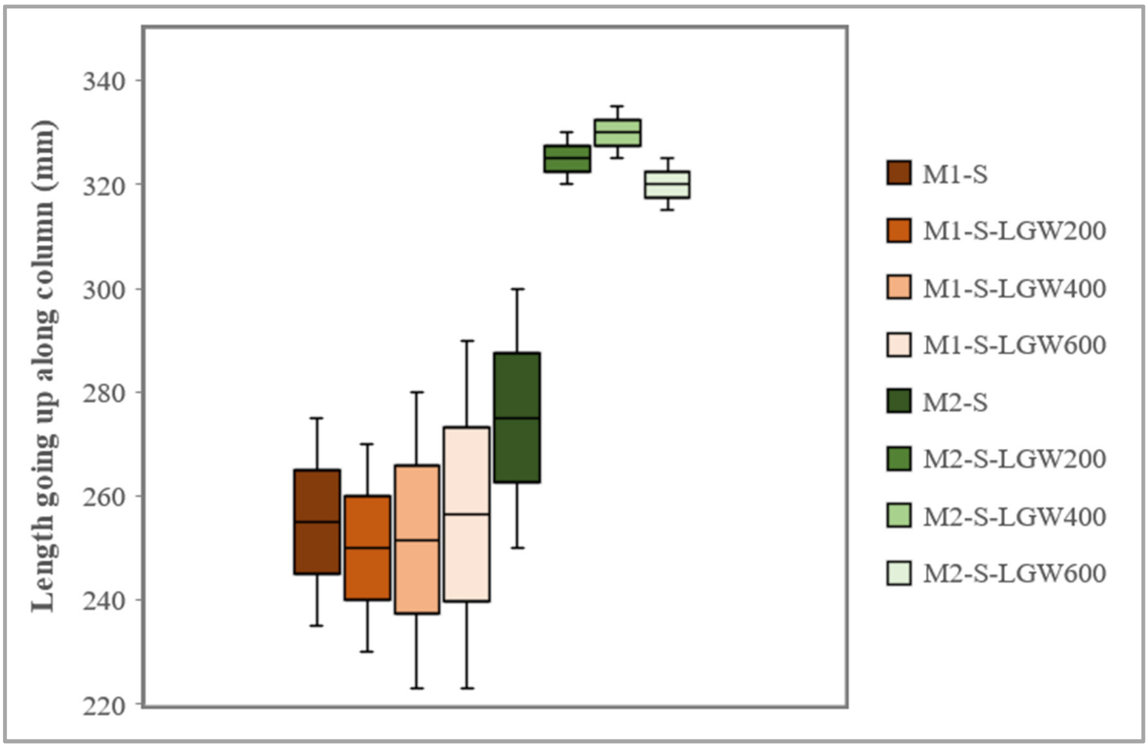


Figure 5.32: Position of the maximum bulging diameter along the columns reinforced with layers of the Betatex (GW) geotextile and installed in base clays at both OMC and LL

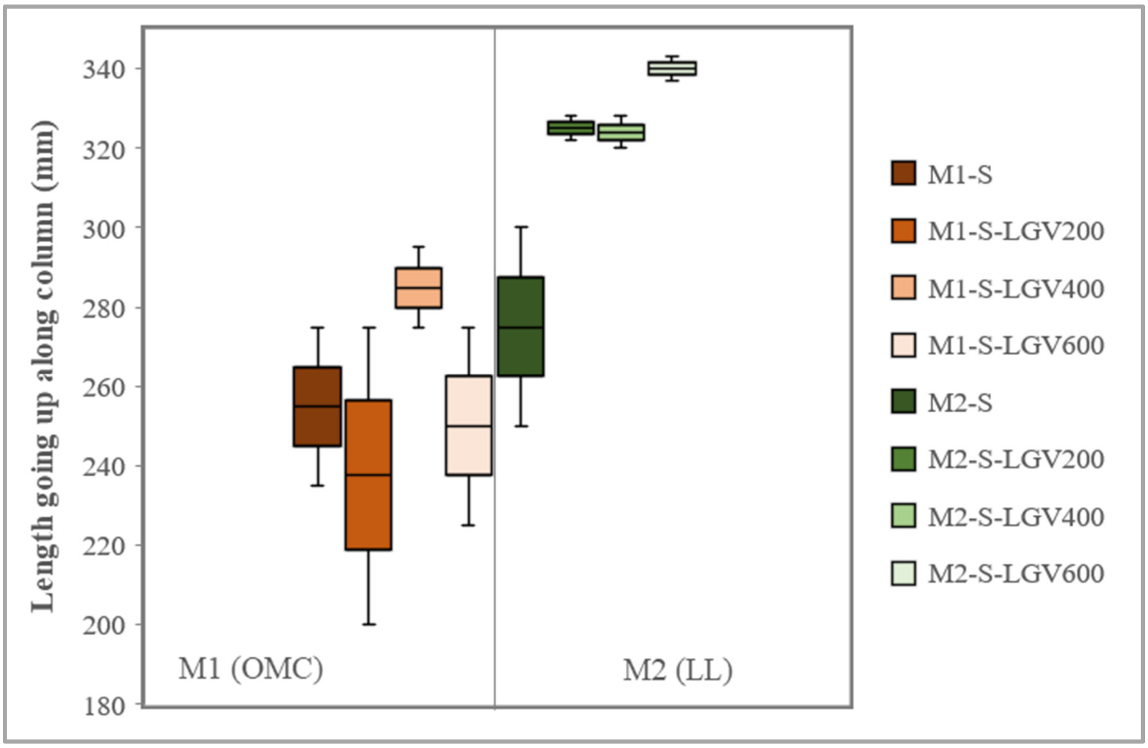


Figure 5.33: Position of the maximum bulging diameter along the columns reinforced with layers of the Fibertex (GV) geotextile and installed in base clays at both OMC and LL

5.6.3 Discussions

From the analyses presented in Figures 5.28 to 5.33, it is evident that the bulging response was dependent on the type, quantity and arrangement of the reinforcement. Beside these, the moisture content of the base soil also had a significant influence on the length span and the position of bulging. For instance, the length span in OMC tests generally appeared to be much longer compared to the ones in LL tests. Moreover, the position of the zones of maximum bulging in OMC tests were situated further down the columns in comparison to that in LL tests where they were located in the upper segment of the columns. Out of all the tests, the longest span of 240 mm (between 50 and 290 mm) was attained in the OMC test where the column was reinforced with layers of fibres (concentration of 0.83 %). In contrast, the shortest span of 5 mm (between 325 and 330 mm) was obtained in an LL test when the column was reinforced by layers of fibres with a corresponding concentration of 0.28 %. In terms of the position, bulging occurred at a highest position (between 337 and 347 mm) when a column which was laterally reinforced by the Fibertex geotextile and having a mass per unit area of 600 g/m^2, was used to improve the base soil at LL. The lowest location (between 50 and 290 mm) of bulging was, however, noted to be in the OMC test where the column was reinforced by layers of fibres at a concentration of 0.83 %.

The mechanism behind bulging and its corresponding position can be explained by the higher lateral confinement, from the host soil, which existed in the OMC tests. When the column was loaded, it initially restricted any lateral deformation. Beyond a certain point, when the vertical stress exceeded the confining stress, shearing was initiated. As a result, lateral strain occurred within the column which resulted in bulging. This analogy is almost comparable to that in the triaxial test; the only difference lies in the confining stresses which are not usually equal in all directions in the case of granular columns. This link between granular columns and triaxial testing was previously explained by Hughes & Withers (1974). In the LL tests, the stiffness of the host soil was significantly low compared to that in the OMC tests. Therefore, as the column was loaded, bulging occurred much faster and also at a higher position. Contrastively, in base soils at OMC, their higher stiffness allowed for a longer time before the column deformed since the confining stresses were better able to compensate for the vertically applied load. Consequently, the load was transferred further down the column which allowed for limited bulging. As such, longer spans of bulging zones were obtained in OMC tests compared to those in LL tests.

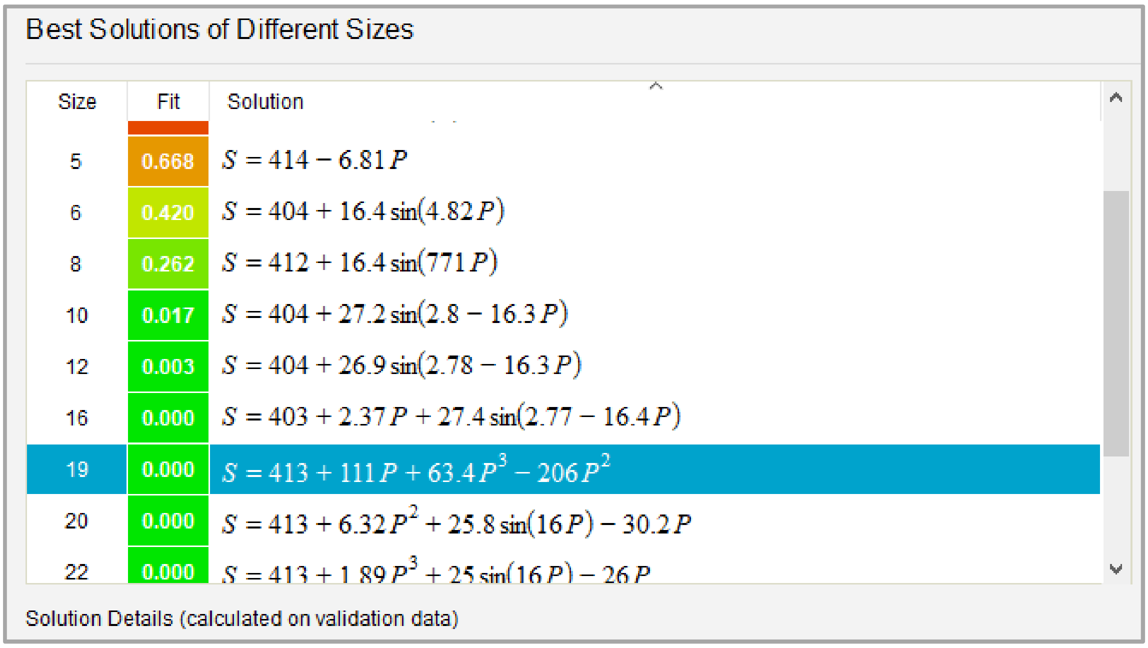
Studies pertaining to the length span of the maximum bulging zone is seemingly non-existent except for one by Basu (2009). Basu claimed that the length of bulging increased with an increase in fibre content. A similar finding was recorded in the present research; the longest span occurred when the highest concentration of fibre was used in the layering arrangement and for an OMC test.

Generally, researchers have reported the position of bulging at a certain depth within the column as opposed to over a span and stated that the maximum bulging typically occurred within the upper portion of the column length (McKelvey et al. 2004, Ali, Shahu & Sharma, 2010; Sobhee-Beetul, 2012). If the average of each length span in this investigation is considered, they are in agreement with the findings of these past studies. Nevertheless, it is worth noting that these were more likely obtained in the LL tests.

5.7 Empirical equations developed from the experimental results

After analysing the results, the following relationships were considered important to be established mathematically: stress-reinforcement content and bulging diameter- reinforcement content. This was necessary to allow for any future research or applications to predict the reinforcement content needed to generate the required stress at a maximum settlement of 50 mm. The equations were also anticipated to assist in determining the respective maximum bulging diameter from these quantities of reinforcement. Therefore, this section explains how the equations were obtained from a popular software known as Eureqa. The empirical equations which were generated from the laboratory results are also given for each relationship.

Eureqa is an Artificial Intelligence powered modelling engine which uses evolutionary search to derive mathematical equations based on experimental data sets. This software essentially performs symbolic regression analysis to obtain a mathematical solution which is capable of explaining the data generated from the experiments (Allgaier & McDevitt, 2018). During the analysis, several functions are generated such that it is highly probable for the following to be included: arithmetic, trigonometric, exponential and polynomial. Figure 5.34 illustrates an extract of a typical result obtained from an analysis of a set of experimental data in Eureqa. The solutions which relate stress (S) to reinforcement content (P) are given.



Best Solutions of Different Sizes

Size	Fit	Solution
5	0.668	$S = 414 - 6.81\,P$
6	0.420	$S = 404 + 16.4\sin(4.82\,P)$
8	0.262	$S = 412 + 16.4\sin(771\,P)$
10	0.017	$S = 404 + 27.2\sin(2.8 - 16.3\,P)$
12	0.003	$S = 404 + 26.9\sin(2.78 - 16.3\,P)$
16	0.000	$S = 403 + 2.37\,P + 27.4\sin(2.77 - 16.4\,P)$
19	0.000	$S = 413 + 111\,P + 63.4\,P^3 - 206\,P^2$
20	0.000	$S = 413 + 6.32\,P^2 + 25.8\sin(16\,P) - 30.2\,P$
22	0.000	$S = 413 + 1.89\,P^3 + 25\sin(16\,P) - 26\,P$

Solution Details (calculated on validation data)

Figure 5.34: An extract from the results generated by Eureqa when determining the equation for the relationship between stress and reinforcement content

Evidently, the equations were too many and only one needed to be selected. In this situation, the polynomial function of degree 3 was preferred (highlighted). This selection is principally based on the level of precision required in determining *S*. Since *S* was measured in kPa, lengthier equations which altered this value negligibly were not considered as they added onto the complexity of solving the equations, while not having a significant impact due to the high stresses which were being dealt with. Nevertheless, it was ensured that the fit (as shown in the figure) was 0. This simply referred to the iterations diverging to form a linear function whereby the observed (from the experiments) value was almost the same as the predicted (from the equation) one as shown in Figure 5.35. Besides, the statistical R^2 value and the correlation coefficient were also considered in choosing the equation. Both needed to be equal to 1 for an equation to be deemed acceptable.

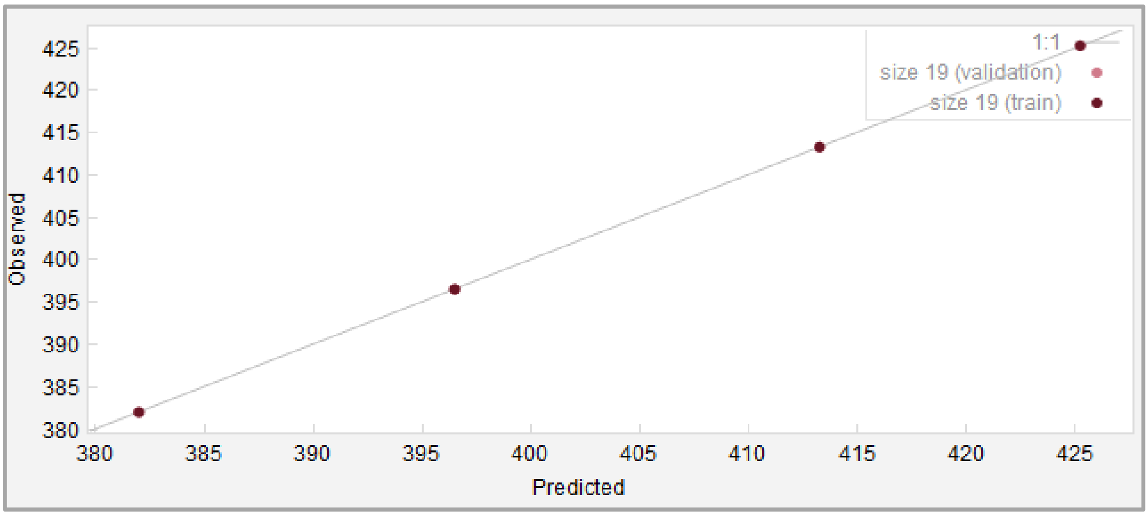


Figure 5.35: Divergence of the observed and the predicted values in Eureqa to form a linear equation

Analysis for each test series (M1-S-RP, M2-S-RP, M1-S-RF, M2-S-RF, M1-S-LP, M2-S-LP, M1-S-LF, M2-S-LF, M1-S-LGW, M2-S-LGW, M1-S-LGV, M2-S-LGV) was performed using Eureqa and the mathematical relationships which best fit the data was obtained. These are given in Tables 5.1 to 5.4. In all the equations, the R^2 value and the correlation coefficient was 1. Also, the reinforcement content varied in each equation, depending on the type and mass of reinforcement which was studied. Equations for the 2 types of arrangement (random mixing and layering) of the reinforcement are independently presented. In general, irrespective of the reinforcement arrangement, the moisture content of the base soil and the type of reinforcement, a cubic function was obtained for determining both the stress and the maximum bulging diameter, at a settlement of 50 mm. This is represented as a polynomial of order 3 and is of the form:

$$y(x) = ax^3 + bx^2 + cx + d \qquad \text{(Equation 5.1)}$$

In this equation, it is not possible for *a* to be zero if the cubic function is to be maintained. However, *b*, *c* and *d* may be equal to zero. From the cubic equations generated in this study, it was clear that *b*, *c* and *d* were always non-zero values.

Table 5.1: Mathematical relationships between the vertical stress and the reinforcement content for each test (random mixing)

Test series (Random)	**Equation for maximum vertical applied stress at a settlement of 50 mm (S in kPa)**	**Mean squared error**	**Comments**	**Equation No.**
M1-S-RP	$S = 413 + 111P - 206P^2 + 63.4P^3$	1.22×10^{-8}	P is the concentration of flakes and is equal to 0, 0.5, 1.0 or 2.5 %	5.2
M2-S-RP	$S = 51.9 + 73.1P - 74.7P^2 + 18.8P^3$	1.70×10^{-9}	P is the concentration of flakes and is equal to 0, 0.5, 1.0 or 2.5 %	5.3
M1-S-RF	$S = 413 + 3950F - 88300F^2 + 504000F^3$	4.31×10^{-9}	F is the concentration of fibres and is equal to 0, 0.025, 0.05 or 0.1 %.	5.4
M2-S-RF	$S = 51.9 - 187F + 23100F^2 - 161000F^3$	5.84×10^{-9}	F is the concentration of fibres and is equal to 0, 0.025, 0.05 or 0.1 %.	5.5

Table 5.2: Mathematical relationships between the vertical stress and the reinforcement content for each test (layering)

Test series (Layering)	Equation for maximum vertical applied stress at a settlement of 50 mm (S in kPa)	Mean squared error	Comments	Equation No.
M1-S-LP	$S = 413 - 99.4P + 50.5P^2 - 6.22P^3$	1.29×10^{-9}	P is the concentration of flakes and is equal to 0, 2.2, 3.3 or 5.6 %.	5.6
M2-S-LP	$S = 51.9 - 2.17P + 1.32P^2 - 0.046P^3$	1.24×10^{-9}	P is the concentration of flakes and is equal to 0, 2.2, 3.3 or 5.6 %.	5.7
M1-S-LF	$S = 413 + 79.2F - 385F^2 + 102F^3$	2.47×10^{-8}	F is the concentration of fibres and is equal to 0, 0.28, 0.56 or 0.83 %.	5.8
M2-S-LF	$S = 51.9 - 49.2F + 334F^2 - 286F^3$	1.03×10^{-9}	F is the concentration of fibres and is equal to 0, 0.28, 0.56 or 0.83 %.	5.9
M1-S-LGW	$S = 413 + 1.02W - 0.00187W^2 + 4.21 \times 10^{-7}W^3$	2.97×10^{-8}	W is the mass per unit area of the GW geotextile and is equal to 0, 200, 400 and 600 g/m^2.	5.10
M2-S-LGW	$S = 51.9 + 0.321W - 0.00113W^2 + 1.18 \times 10^{-6}W^3$	1.79×10^{-9}	W is the mass per unit area of the GW geotextile and is equal to 0, 200, 400 and 600 g/m^2.	5.11
M1-S-LGV	$S = 413 + 1.09V - 0.00382V^2 + 3.63 \times 10^{-6}V^3$	1.03×10^{-8}	W is the mass per unit area of the GV geotextile and is equal to 0, 200, 400 and 600 g/m^2.	5.12
M2-S-LGV	$S = 51.9 + 0.217V - 0.000671V^2 + 6.77 \times 10^{-7}V^3$	1.22×10^{-9}	W is the mass per unit area of the GV geotextile and is equal to 0, 200, 400 and 600 g/m^2.	5.13

Table 5.3: Mathematical relationships between the bulging diameter and the reinforcement content for each test (random mixing)

Test series (Random)	Equation for maximum bulging diameter at a settlement of 50 mm (D_B in mm)	Mean squared error	Comments	Equation No.
M1-S-RP	$D_B = 128 + 40.6P - 62.8P^2 + 19.2P^3$	5.56×10^{-10}	P is the concentration of flakes and is equal to 0, 0.5, 1.0 or 2.5 %	5.14
M2-S-RP	$D_B = 150 + 66.3P - 88.8P^2 + 24.5P^3$	1.47×10^{-9}	P is the concentration of flakes and is equal to 0, 0.5, 1.0 or 2.5 %	5.15
M1-S-RF	$D_B = 128 + 207F - 6000F^2 + 37300F^3$	4.59×10^{-11}	F is the concentration of fibres and is equal to 0, 0.025, 0.05 or 0.1 %.	5.16
M2-S-RF	$D_B = 150 - 1010F + 29200F^2 - 197000F^3$	1.98×10^{-10}	F is the concentration of fibres and is equal to 0, 0.025, 0.05 or 0.1 %.	5.17

Table 5.4: Mathematical relationships between the bulging diameter and the reinforcement content for each test (layering)

Test series (Layering)	Equation for maximum bulging diameter at a settlement of 50 mm (D_B in mm)	Mean squared error	Comments	Equation No.
M1-S-LP	$D_B = 128 - 1.46P - 0.274P^2 + 0.0498P^3$	1.56×10^{-10}	P is the concentration of flakes and is equal to 0, 2.2, 3.3 or 5.6 %.	5.18
M2-S-LP	$D_B = 150 - 8.15P + 2.69P^2 - 0.288P^3$	3.35×10^{-10}	P is the concentration of flakes and is equal to 0, 2.2, 3.3 or 5.6 %.	5.19
M1-S-LF	$D_B = 128 - 86.4F + 195F^2 - 141F^3$	3.10×10^{-10}	F is the concentration of fibres and is equal to 0, 0.28, 0.56 or 0.83 %.	5.20
M2-S-LF	$D_B = 150 - 56F + 51.2F^2 - 15.4F^3$	2.60×10^{-10}	F is the concentration of fibres and is equal to 0, 0.28, 0.56 or 0.83 %.	5.21
M1-S-LGW	$D_B = 128 - 0.0283W + 5 \times 10^{-5}W^2 - 4.17 \times 10^{-8}W^3$	6.52×10^{-11}	W is the mass per unit area of the GW geotextile and is equal to 0, 200, 400 and 600 g/m^2.	5.22
M2-S-LGW	$D_B = 150 - 0.103W + 2.75 \times 10^{-4}W^2 - 2.92 \times 10^{-7}W^3$	6.36×10^{-10}	W is the mass per unit area of the GW geotextile and is equal to 0, 200, 400 and 600 g/m^2.	5.23
M1-S-LGV	$D_B = 128 - 0.0433V + 7.5 \times 10^{-5}V^2 - 4.17 \times 10^{-8}V^3$	7.96×10^{-11}	W is the mass per unit area of the GV geotextile and is equal to 0, 200, 400 and 600 g/m^2.	5.24
M2-S-LGV	$D_B = 150 - 0.168V + 4.75 \times 10^{-4}V^2 - 4.17 \times 10^{-7}V^3$	5.94×10^{-10}	W is the mass per unit area of the GV geotextile and is equal to 0, 200, 400 and 600 g/m^2.	5.25

5.8 Comparison of performances of columns reinforced with the waste geotextile (Betatex) and the virgin geotextile (Fibertex)

5.8.1 Stress-settlement characteristics

Figures 5.36 and 5.37 show the stress-settlement characteristics for both OMC and LL tests when the Betatex (GW) and Fibertex (GW) geotextiles were used to laterally reinforce the granular columns. In general, similar trends were obtained with both types of geotextiles, irrespective of the moisture content of the base soil. Nevertheless, the maximum vertical applied stress at a settlement of 50 mm differed slightly.

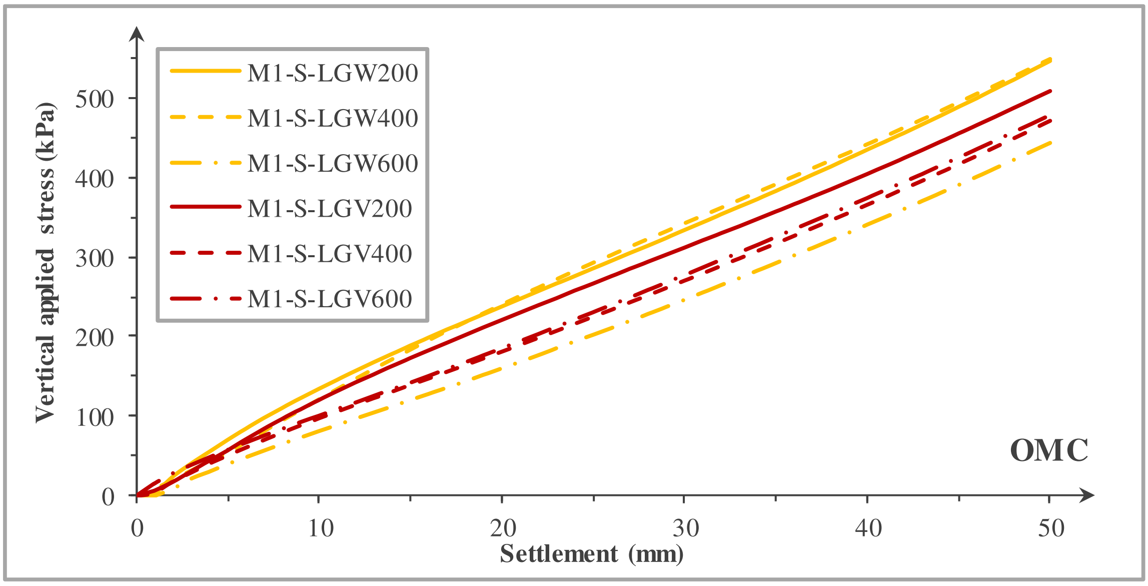


Figure 5.36: Comparison of the stress-settlement characteristics for the 2 types of geotextiles in the OMC tests

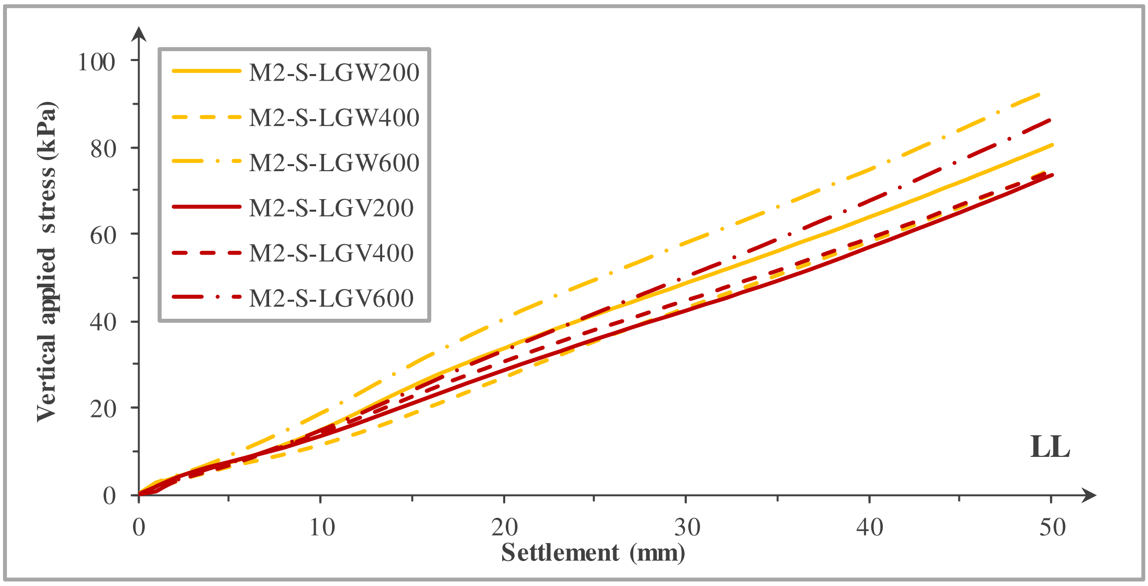


Figure 5.37: Comparison of the stress-settlement characteristics for the 2 types of geotextiles in the LL tests

From the plots, the Betatex geotextile appeared to produce higher stresses than the Fibertex in most cases; this observation was not expected since the manufacturers describe Fibertex as a generally stronger geotextile. The variation can possibly be explained by the fact that Betatex was generated from waste whereby the material properties are less controlled as opposed to the virgin PET used for manufacturing Fibertex. This is because it is practically impossible to determine the exact properties of Betatex since they are made from waste PET bottles which might have undergone the recycling process several times, and thus resulting in large variations in properties. In fact, within the same batch of bottles being recycled into fibres (to produce the geotextile), some might have been recycled a few times while others might be undergoing recycling for the first time; a certain amount of virgin PET may also be added (Khoramnejadian, 2011). Evidently, the strength characteristics differ in each bottle. As a result, the strength of the Betatex may also vary. Khoramnejadian (2011) explained that the performance of recycled plastic is generally lower than that of the virgin material since their properties are reduced during the recycling process. She further added that PET generally undergoes chemical and mechanical degradation during recycling, which in turn can decrease the following properties of recycled PET: physical, mechanical, chemical and rheological. Although this explanation implies that the anticipated vertical applied stresses should have been lower when Betatex was used, the results obtained contradicted this understanding.

5.8.2 Maximum bulging

In terms of the maximum bulging diameters recorded, Figure 5.38 summarises them for all the tests which were undertaken. It was noted that the maximum bulging diameters were generally lower in OMC tests than in LL tests, irrespective of the type of geotextile utilised. Overall, the trends seemed rather similar whereby the diameters were mostly larger when Betatex were used. However, an exception was noted in the LL tests when the geotextiles of mass per unit area of 600 g/m^2 were used; Fibertex produced larger maximum bulging diameters than Betatex in this case. At this mass per unit area, both types of geotextiles produced the same largest bulge size in the OMC tests. The short discrepancy may possibly be due to the material quality of Betatex. Due to the probable weakening of PET through recycling, as explained earlier, Betatex might have had a lower resistance to bulging compared to Fibertex. Consequently, the bulging diameters in tests with Betatex might have produced larger bulging.

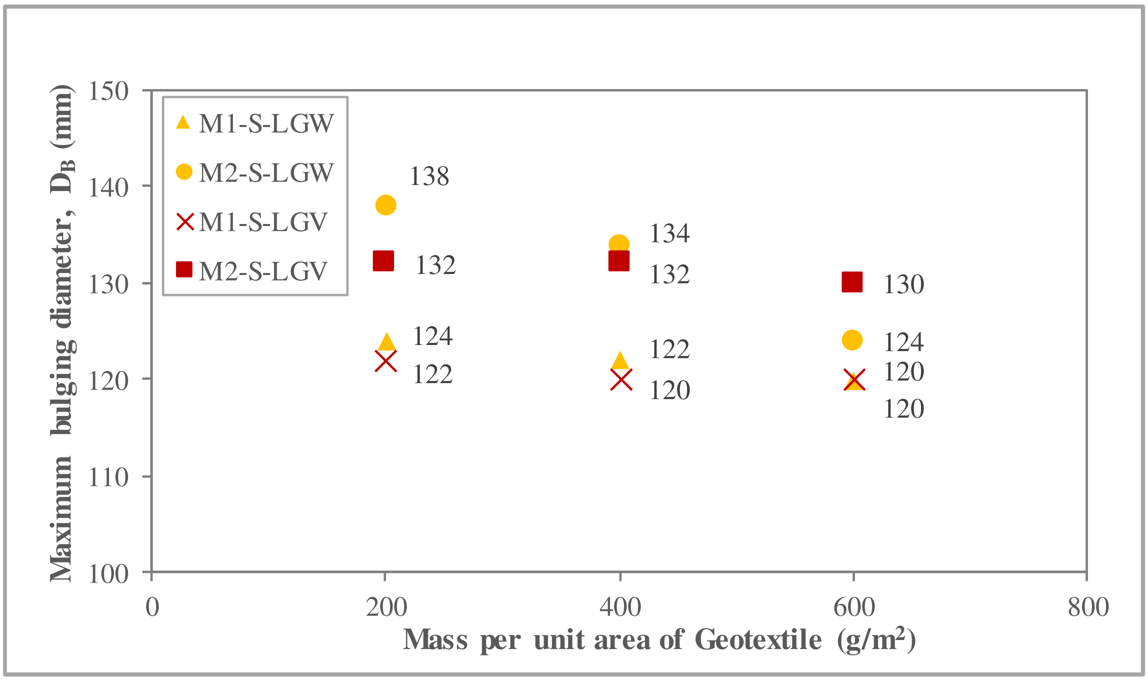


Figure 5.38: Comparison of the maximum bulging diameter for the 2 types of geotextiles in the LL tests

5.8.3 Length span of the maximum bulging zone

The length spans corresponding to the maximum bulging zones are shown in Figures 5.39 and 5.40 when both types of geotextiles were used, in the OMC and LL tests. Generally, the trends appeared to differ drastically. For instance, as the mass per unit area of Betatex was increased in the OMC tests (Figure 5.39), a gradual increase was also noted in the length span. This clearly showed that higher mass per unit areas of Betatex produced columns whereby bulging was more uniformly distributed within. A more irregular trend in the length span was obtained as the mass per unit area was augmented, when Fibertex was used in the OMC tests. In addition, the length of the spans displayed higher variations.

In the LL tests, the length spans attained with each type of geotextile were relatively similar. However, with regards to the trends of the position of the maximum bulging zone, the results exhibited even higher discrepancies. When Betatex was used, the position moved higher up as the mass per unit area was raised from 200 to 400 g/m^2. A further increase to 600 g/m^2 caused a drop in position which was even lower than that of the 200 g/m^2 Betatex. For identical masses of Fibertex, the position initially dropped slightly lower than that of the 200 g/m^2 geotextile, after which a significant rise was observed in the 600 g/m^2 one. Therefore, it can be deduced that Betatex behaved quite closely to Fibertex in terms of the stress-settlement characteristics. However, drastic observations were made with regards to maximum bulging diameter, and the corresponding length span and position of this zone.

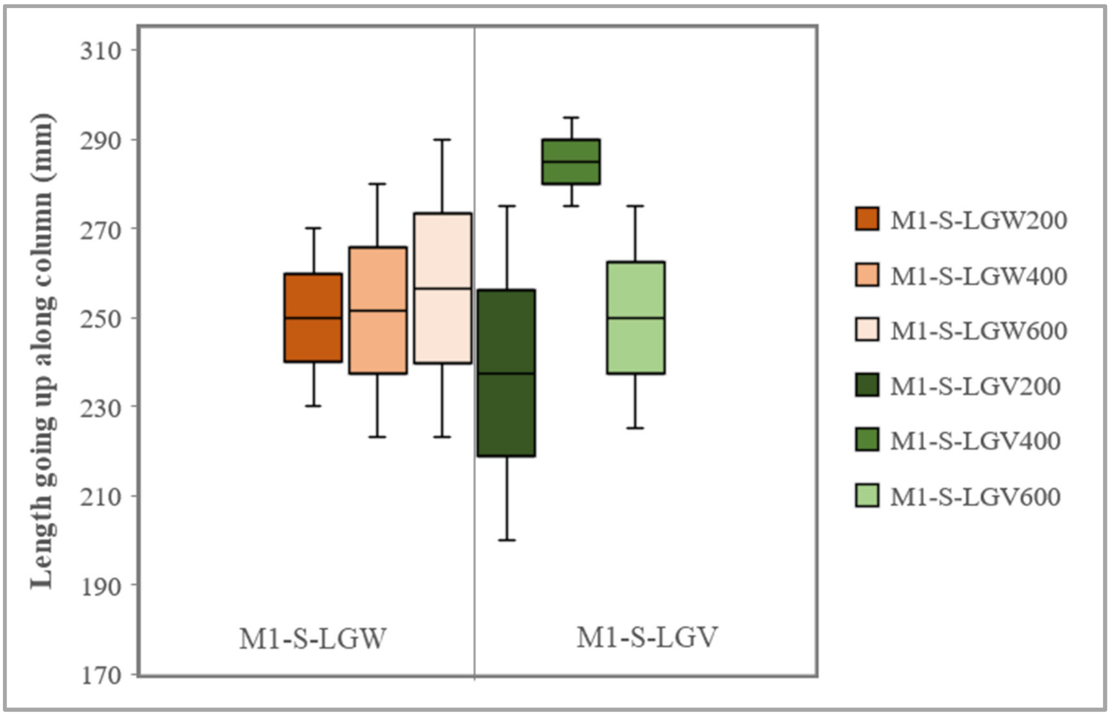


Figure 5.39: Comparison of the length span corresponding to the maximum bulging zone for the 2 types of geotextiles in the OMC tests

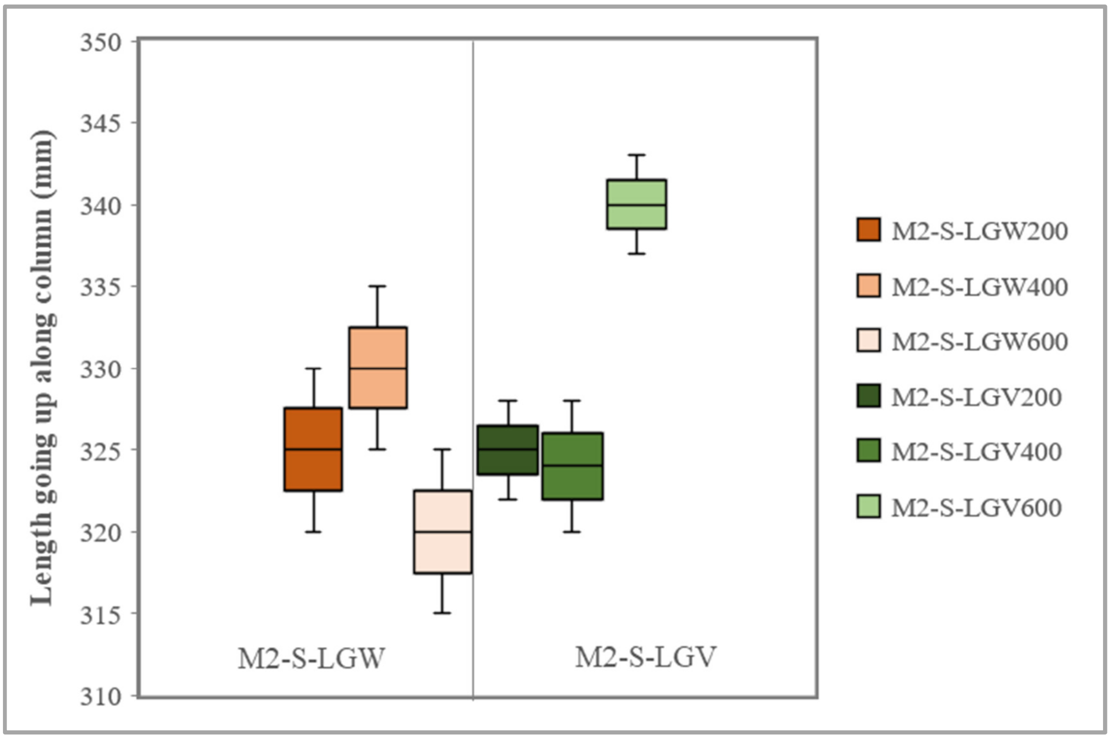


Figure 5.40: Comparison of the length span corresponding to the maximum bulging zone for the 2 types of geotextiles in the LL tests

5.9 Variation in the quantity of the column and reinforcement materials utilised

The density of the column impacts the behaviour of the improved ground since a low density increases the degree of settlement. In contrast, very dense columns reduce the settlement since the voids present between the particles are relatively low. Besides, denser columns are also able to sustain heavier loads. Generally, the column density is influenced by the type of column material, the degree of compaction and the moisture content of the surrounding soil which provides confinement to the column.

In this research, the column material varied since different composites were used in each test, depending on the type, concentration and arrangement of the reinforcements. As a result, the total mass of the column was also affected since the response of each type of reinforcement to compaction was not alike. Additionally, as was explained earlier, the level of bulging which occurred in the installation of each column was dissimilar, thereby causing a variation in the volume of each column. Due to the different volumes and masses, the density of the columns varied slightly.

Since the volume of each column was unknown before testing, this section concentrates on graphically presenting and analysing the masses of sand and reinforcement used in the individual tests, instead of discussing the densities. The outcome is necessary to create an understanding of the reduction in sand use when this technology is applied. From an environmental perspective, this is critically important since sand mining generates a significantly large amount of carbon dioxide emissions amongst other negative impacts such as deforestation, loss of biodiversity, soil erosion and acid drainage (Gavriletea, 2017). In view of creating innovative construction techniques which are less harmful to the environment, the analyses presented in this section will be informative to any potential users of this technology. Besides, the quantity of waste materials used is also given which helps in appreciating the probable contribution to PET bottles waste management through the application of waste reinforced granular columns.

5.9.1 Random mixing

Figures 5.41 and 5.42 illustrate the masses of sand and reinforcement used in each test (both OMC and LL tests) when the columns were reinforced with randomly mixed flakes and fibres, respectively. From the figures, it was evident that the mass of sand required to form a column in a base soil at LL was much higher compared to when it was installed in a base soil at OMC. In general, it was found that reinforcing caused a reduction in the amount of sand used to make the columns. However, a more remarkable drop in the mass of sand was noted when fibres were used in the OMC tests. Additionally, the mass of fibres which were used to achieve this reduction in the amount of sand was significantly lower than that of the flakes. This can probably be due to the higher volumes occupied by the fibres which have a low unit weight. Therefore, for the OMC tests, fibres appeared to be a better reinforcement in terms of diminishing the mass of sand utilised.

In contrast, flakes seemed to perform better in the sand use reduction, when the columns were tested in base soils at LL. Regardless of this observation, the mass of fibres used in the LL tests remained significantly lower than that of the flakes. Out of all the tests with randomly mixed reinforcement, a flakes content of 1.0 % produced the lowest sand mass of 6014 g compared to that of an OGC (in a base soil at OMC) which utilised 6756 g of sand. In the LL tests, the mass of sand for an OGC was 7453 g; a sharp drop to 6766 g was attained when a flakes concentration of 2.5 % was used.

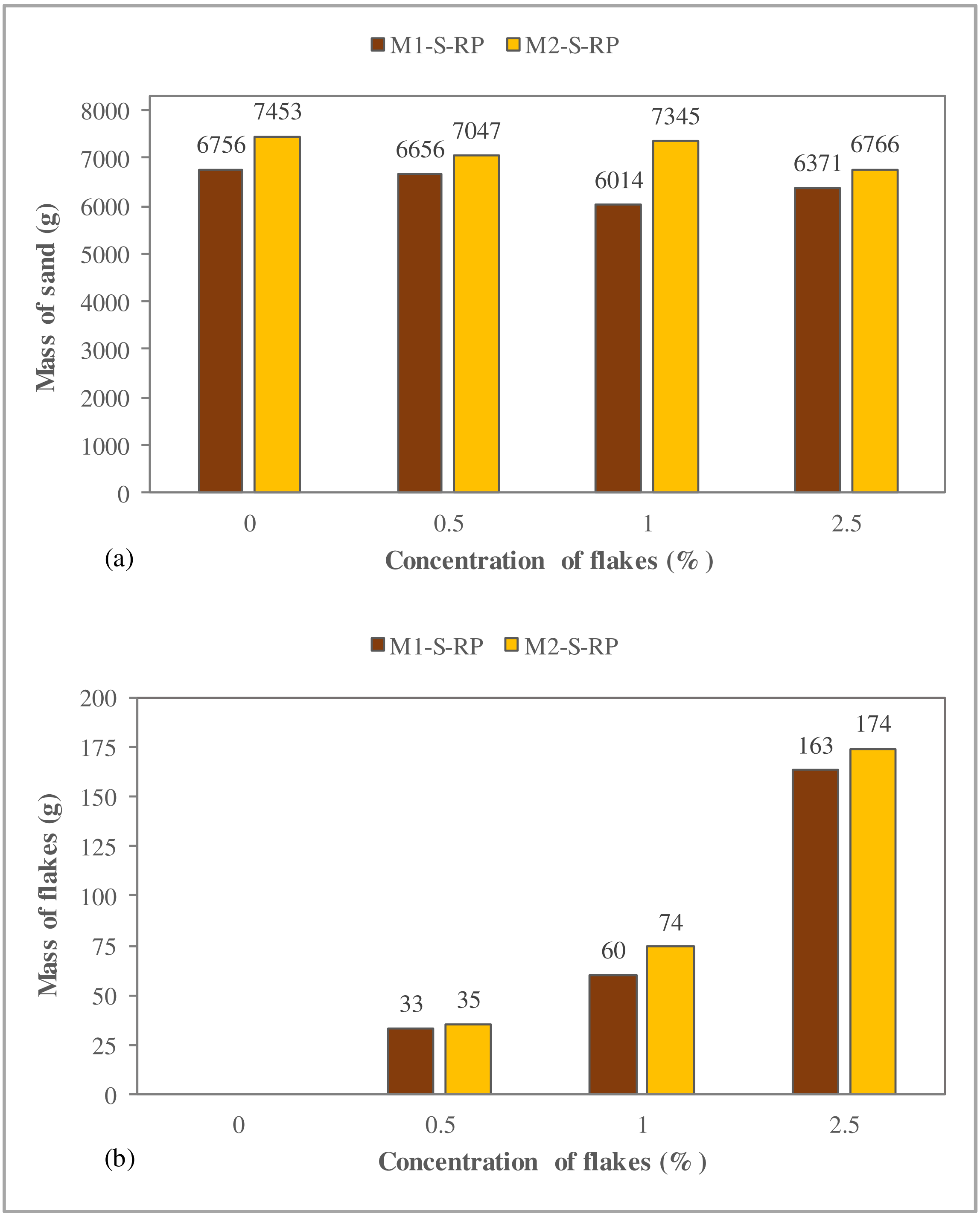


Figure 5.41: Masses of flakes and sand used in both OMC and LL tests where they were randomly mixed to reinforce the columns

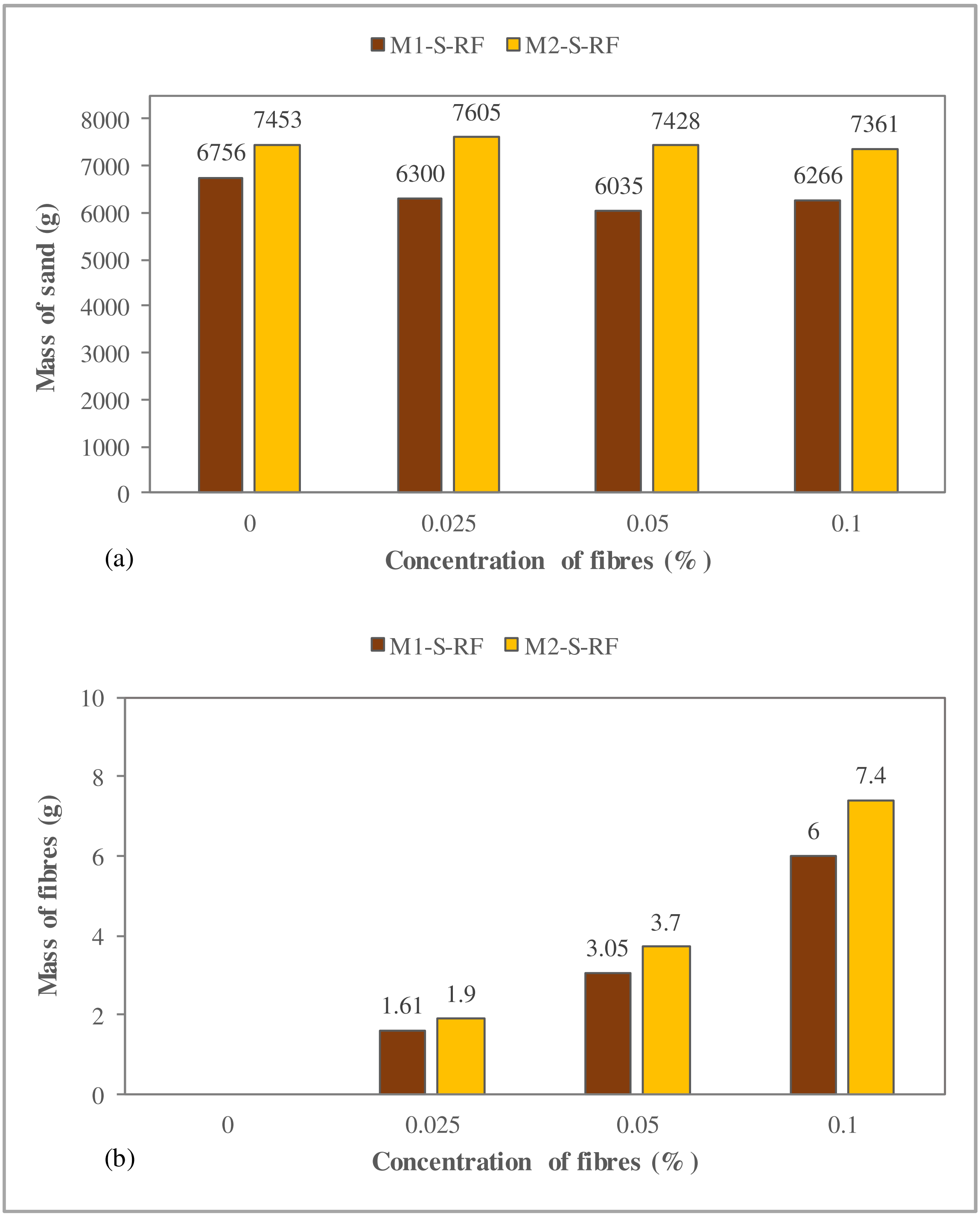


Figure 5.42: Masses of fibres and sand used in both OMC and LL tests where they were randomly mixed to reinforce the columns

5.9.2 Layering

Reinforcing of the columns (with flakes, fibres, Betatex and Fibertex) tested in base soils at OMC and LL, resulted in the masses of sand and reinforcement as shown in Figures 5.43 to

5.46. Overall, it was noted that flakes still caused the highest mass (300 g) of reinforcements used while both geotextiles produced the lowest mass of 8.7 g.

In the OMC tests, a flakes content of 5.6 % resulted in the largest saving of sand; the total mass of sand utilised was 5099 g compared to that in the OGC which was equivalent to 6756 g. This reduction was due to the 250 g of flakes which was used. Fibres, at a concentration of 0.83 % was the second material which also caused a significant drop in the amount of sand used; the masses of sand and fibres in this case corresponded to 5164 g and 37.5 g, respectively.

When the masses from the OMC tests were compared to those in the LL tests, it was apparent that the columns installed in the latter tests were generally heavier than those in the former ones. This was because the lower confinement from the wetter base soil possibly resulted in larger pre-bulging during the installation process. Consequently, the masses of sand required to form the columns were higher. It was worth noting that these larger masses did not necessarily imply denser columns since their volumes were also bigger than those of the columns in the OMC tests.

Overall, it was noted that the quantity of sand utilised decreased as the mass of reinforcement was augmented within each test series. Specifically, it was obvious that each type of reinforcement responded distinctively with regards to the reduction in sand used. Fibres were generally found to be a better performing material since only a small quantity of them was needed to cause a significant drop in the mass of sand, compared to flakes whose masses were the highest among all 4 types of reinforcement. A comparison of the columns prepared with Betatex and Fibertex also confirmed that the masses of these geotextiles were identical. Nevertheless, small and inconsistent differences were observed in the masses of sand required to form the columns reinforced with these geotextiles. Earlier, it was explained that the nature of Betatex, which was manufactured from PET bottle waste fibres was possibly the reason behind the irregularities due to their loss in properties during the recycling process.

Conclusively, it can be said that the 4 types of reinforcements may potentially be used with the intention of reducing the use of sand as a raw material while also contributing to the management of PET bottle waste destined to landfills. However, the different behaviours of the materials (which have been presented earlier) need to be properly understood, and a particular combination of the material can be opted for based on the engineering requirements.

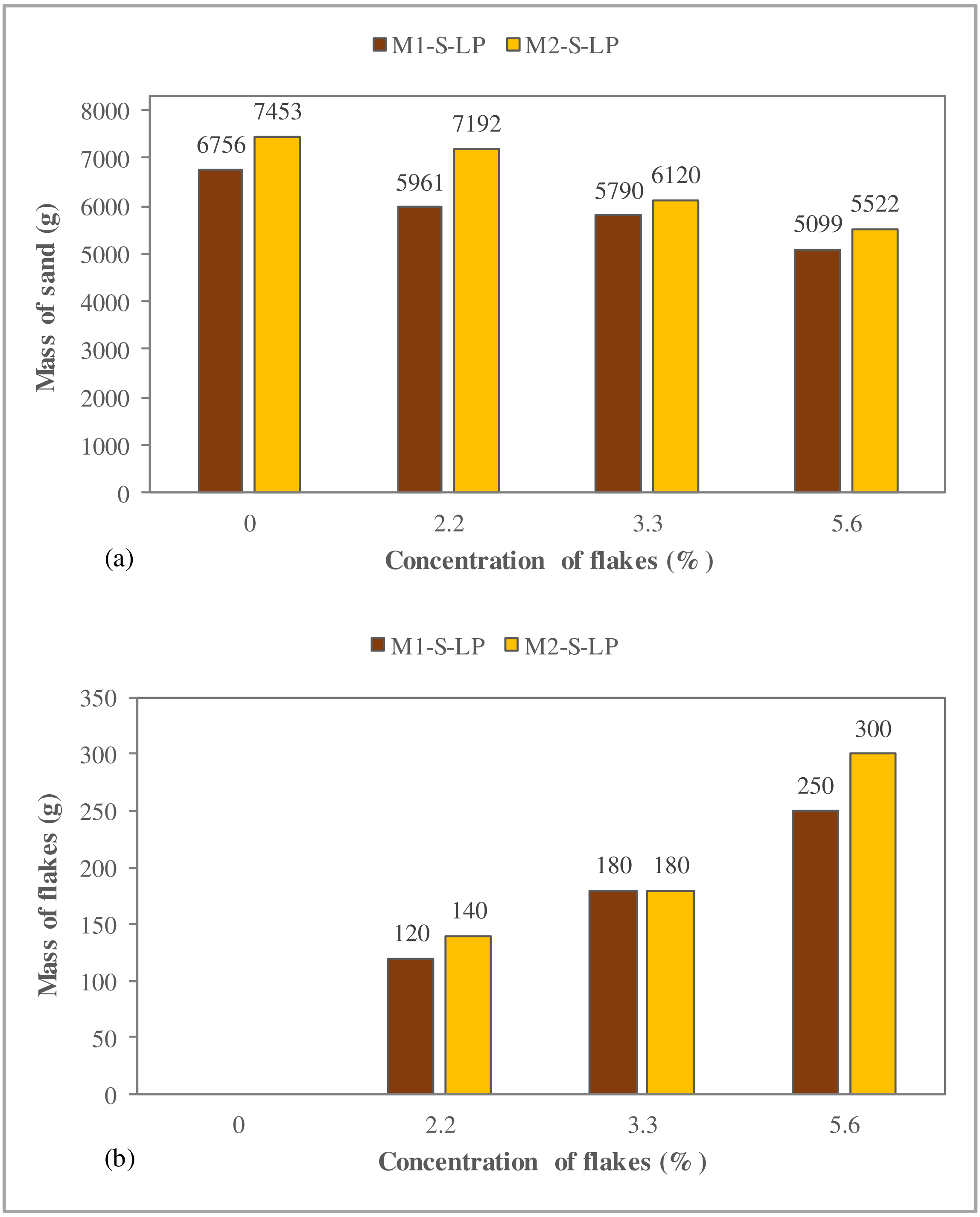


Figure 5.43: Masses of flakes and sand used in both OMC and LL tests where they were used in layers to reinforce the columns

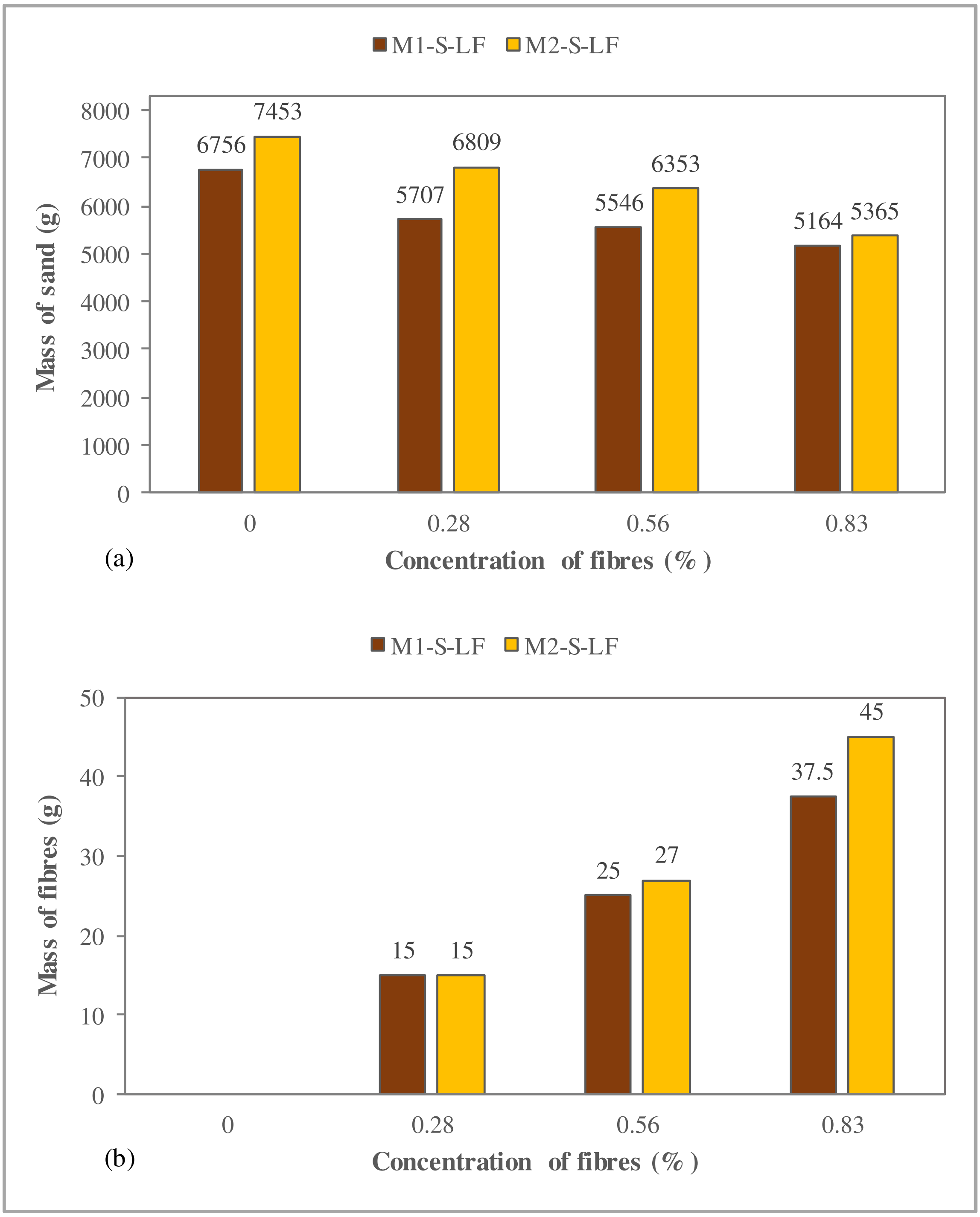


Figure 5.44: Masses of fibres and sand used in both OMC and LL tests where they were used in layers to reinforce the columns

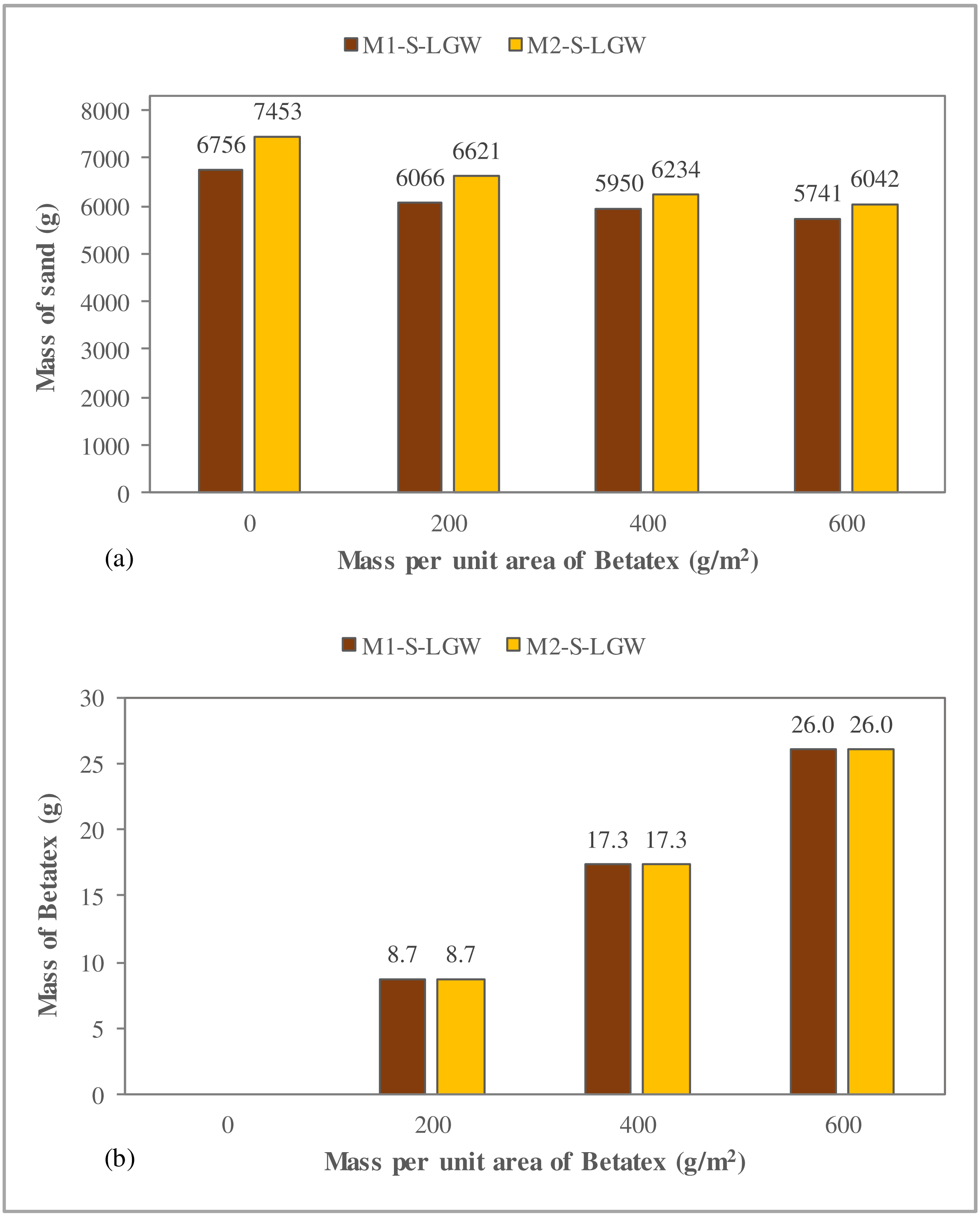


Figure 5.45: Masses of the Betatex geotextile (GW) and sand used in both OMC and LL tests where they were used in layers to reinforce the columns

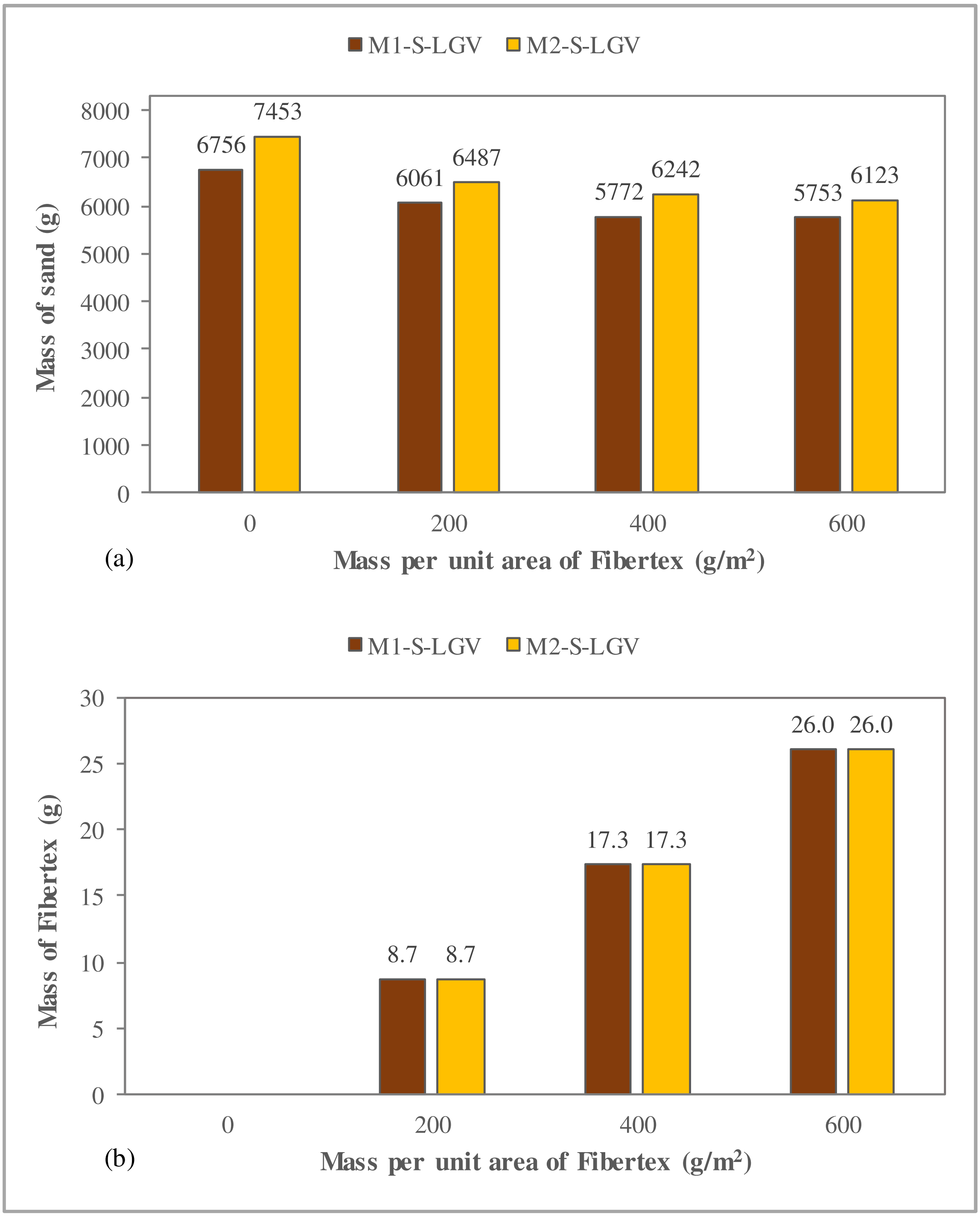


Figure 5.46: Masses of the Fibertex geotextile (GV) and sand used in both OMC and LL tests where they were used in layers to reinforce the columns

5.10 Potential applications and merits of the developed PET waste reinforced granular columns

Typical applications for granular columns were discussed earlier in the literature review where it was mentioned that this technology is common in the improvement of the foundation system to support low capacity structures such as embankments, low-rise buildings and oil tanks. In fact, several projects across the globe were highlighted to emphasise on the widespread application and efficiency of this technique in foreign countries. Nevertheless, it was pointed out that its use in South Africa remains nominal. In previous studies, it was proven that the method may be applied to the local soils, which encouraged further investigation on this subject.

Previous laboratory investigations on granular columns have continuously demonstrated the efficiency of this approach of ground improvement, while also generating more in-depth knowledge on the theory behind the performance of the columns. However, it was noted that the majority of the works were based on ordinary granular columns. Additionally, most of the tests conducted were on much smaller samples which further affected the representation of the field conditions. Nonetheless, the intensity of works in this area of research has facilitated the application of the technology in the field, although not all the methods have been successfully utilised due to the associated degree of sophistication with the process.

With regards to reinforced granular columns, the concept is relatively new. Several laboratory investigations have been undertaken using different types of reinforcement, out of which geosynthetics (geotextiles or geogrids) appeared to be more common. While encasement of the columns has been successfully employed in certain projects, other types of materials remain of interest within the laboratory environments. Also, no studies have been conducted with the environmental considerations such that waste PET bottle has been used as a reinforcing material but in different forms (flakes, fibres, geotextile). On the contrary, each researcher has come up with a different material and has, therefore, generated individual results. Consequently, there is a persistent lack of continuity in the information relating to reinforced granular columns which makes it difficult to attain the application stage.

In this research, it was also pointed out that the materials used in granular columns contributed to the carbon dioxide emissions when they are mined. Besides, they are natural resources which are rigorously being used in the construction industry. Replacing these partially with waste materials do not only reduce their use (and preserve them) and the emissions of carbon dioxide, but it is also beneficial to the solid waste management line.

By taking the above background into consideration, the technology researched in this study can provide various advantages. The charts and equations generated will allow future researchers or engineers to understand how each material behaved, under the different circumstances, should they be interested in pursuing further work in connection with RGCs.

In this investigation, laboratory scale experiments were conducted in a simple bespoke steel cylindrical tank. No specialised equipment was used which, therefore, makes it practical for extending this research area. Although the loading equipment was fully electronic and computerised, it is not explicitly required for applying the stress. Any other means of applying a displacement-controlled load can be used, irrespective of whether drained or undrained testing conditions are desired. The methodology followed was lengthy due to the different readings needed for the analyses. Yet, the steps were rather simple and easy to follow which allows for the whole procedure to be easily replicated.

The existing equipment which are used for the installation of granular columns can be used in the field. However, it must be ensured that a casing is pushed into the ground to act as support to the surrounding soil before digging of each hole is initiated. The equipment essentially operates in a similar way to that used to install sand compaction columns (discussed in Chapter 2). The only difference lies in the inclusion of the reinforcement when compared to the installation of a traditional granular column. When randomly mixed reinforcements (flakes or fibres) are used, they can be premixed off-site and stored in large sacks or containers to be transported to the construction site. In contrast, for the reinforcements to be introduced in layers, a modified light crane may be used whereby a predetermined mass per layer may be gently released on top of each layer of compacted sand. This complete procedure does not require major volumes of water, which makes it even more appealing to areas where access to water is limited.

During the installation process, a few considerations must be made to minimise any possible source of errors which would trigger a lower performance of the columns. For instance, during the withdrawal of the casing immediately before compaction, the granular material must be poured prior to that step. This prevents the surrounding soil from collapsing into the hole, thereby mixing with the column material and subsequently lowering the column strength. In terms of the reinforcements, random mixing must not be done and stored for long periods of time since the reinforcing members tend to segregate from the sand to form a bigger mass together. When layers of reinforcements are installed, it must be ensured that the total mass poured (fibres or flakes) is distributed rather evenly within a layer. Large differences in spreading will reduce the performance of the column since it will not be adequately reinforced in different directions. For a similar reason, care must be taken when geotextiles are placed into the columns to prevent it from folding.

The laboratory investigation conducted for this research has positively demonstrated the behaviour of each tested column. It was confirmed that reinforcing of the granular columns most definitely improved their stress-settlement and bulging characteristics. However, the quantity, type and arrangement of the reinforcement played a significant role in these performances. It is anticipated that the successful application of this technology will concurrently benefit several disciplines in South Africa.

While the principal purpose of the technology remains to cater for the construction industry, it will also be less harmful to the environment compared to certain techniques that are currently in use. In addition, the solid waste management line will also be dealing with lower volumes of PET bottles which are destined to landfills if not recycled. Consequently, smaller storage spaces will be required in landfills for storing the used PET bottles.

In 2017, the recycling rate of PET waste bottles in South Africa was 65 %. Several factors influence this which ultimately prevents a recycling rate of 100 %. Out of the many factors, the colour of the plastic played a major role since recycled colour bottles have a low market value. However, this study has confirmed that the colour of the bottles does not affect the performance of the columns significantly. Therefore, the construction industry is potentially an environment where these coloured bottles can be catered for.

Granular columns are often used to support embankments in temporary applications such as preloading to accelerate consolidation. It is believed that the reinforced granular columns may also be used in such applications. They will possibly further increase the rate of consolidation since they are more capable of withstanding higher loads.

In terms of cost effectiveness, granular columns are widely popular in this regard. By reinforcing them with waste, an additional reduction in cost may possibly be achieved. As such, this technology might be beneficial to the development of low cost housing. According to the constitution of the Republic of South Africa (Act 108 of. 1996: Chapter 1), "*Everyone has the right to have access to adequate housing"*. It further mentions that *"The state must* *take reasonable legislative and other measures, within its available resources to achieve the progressive realisation of this right"*. When South Africa became a democratic nation in 1994, the new government inherited a housing backlog which they have been addressing through legislation, policies and programmes such as the Reconstruction and Development Programme (RDP) (Sikota, 2015). However, the process is relatively complex since there are other issues to tackle in parallel such as access to the basic services like water, sanitation and electricity. Since problematic soils constitute a large portion of South Africa (AGIS, 2011; Diop et al., 2011), it is inevitable that a budget needs to be allocated for mitigating the associated hazards. Diop et al. (2011) claimed that the overall cost of housing development by the Department of Human Settlement in South Africa, whose primary responsibility is housing and urban development, can approximately increase by 20% when dealing with problematic soils. This increase in cost evidently hinders the number of dwelling allocations due to budget constraints. Under these circumstances where granular column is a viable option for dealing with the problematic soils, they can probably contribute to cost-cutting when establishing the foundation support for low budget houses.

Overall, RGCs seem to have numerous advantageous and, therefore, their application is strongly recommended in scenarios where they are deemed effective. To achieve this, further research needs to be undertaken to generate new knowledge in order to supplement the existing one.

Chapter

6

Conclusions and recommendations

6.1 Research summary

The granular column technology is a widely used method in geotechnical engineering to improve several properties of weak grounds namely: increase in bearing capacity, reduction in settlement, improvement in drainage and mitigation of liquefaction. Despite the high application of the method in foreign countries, the technique is rather unpopular in South Africa. Previous studies conducted on South African soils exhibited positive results. Hence, this study was undertaken. However, since reinforced granular columns are gaining more interest, also from a solid waste management point of view, reinforced granular columns with waste PET bottles (in the forms of flakes, fibres and geotextile) were proposed for this research. The principal aims were to further increase the load carrying capacity of the columns while reducing the settlement of the improved ground.

To achieve the main aims of this research, a comprehensive laboratory test programme was established to investigate the feasibility of the proposed technology when improving a wet soil bed (having low bearing strength) with a singular column. The following parameters were varied to understand their effects on the stress-settlement and deformation characteristics: moisture content of the base soil, type of the reinforcement, arrangement of the reinforcement and mass of the reinforcement. Other factors like column diameter, column length, testing tank dimension and type of sand for the columns were kept constant. Results were principally presented in terms of the stress-settlement and deformation characteristics for each test. Further analyses were made to better understand the effect of each variable on the different important aspects pertaining to the behaviour of granular columns. Overall, the results confirmed that the inclusion of the reinforcing members, which were generated from waste PET bottles, generally enhanced the performance of the columns. However, the form and concentration of the PET used (flakes, fibres or geotextiles) significantly influenced the degree of improvement.

6.2 Main conclusions

6.2.1 Improvement in load carrying capacity

- Randomly mixed flakes, compared to fibres, produced lower improvement in the load carrying capacity since the reinforcing elements were much shorter, straight and smooth. Consequently, slippage was more prominent when the flakes were used.

- Randomly mixed fibres generally produced higher improvements than layers of fibres, when installed in identical columns and tested under similar conditions. In fact, the vertical applied stress increased as the fibre content was augmented. For the same testing conditions, if layers of fibres were used, the strength of the RGC diminished as the fibre content increased.

- An optimum improvement of 244 % was generated in the load carrying capacity. This was obtained when a concentration of 0.1 % of randomly mixed fibres was used to reinforce the column installed in a base soil at liquid limit (LL). This was due to the soil reinforcement theory which discusses the interaction between the soil particles in the column and the fibres. The interlocking of the soil particles around the fibres resulted in frictional forces along the soil-reinforcement interface, which enhanced the shear strength of the column. Hence, the latter was better at sustaining higher vertical loads.

- For columns which were installed in base soils at OMC, the geotextiles appeared to be better performing than the flakes and the fibres. When respective masses per unit area of 200 and 400 g/m^2 of Betatex and Fibertex were included in the columns, the corresponding gains in vertical applied stress were 171 and 150 %.

6.2.2 Effect of the different variables on the stress concentration ratio

- The stress concentration ratios in the present study varied between 1.7 and 3.4; the lowest value corresponded to the OGC which was installed in a base soil at LL. In contrast, the highest value of 3.4 was obtained when a concentration of 0.1 % of randomly mixed fibres were used to reinforce a granular column in a base soil at LL. These values were dependent on the findings from the maximum vertical applied stress since the latter was used for the determination of (n). Hence, any trends observed in the improvement achieved with regards to the maximum vertical applied stresses also indicated similar patterns in the variation of (n).

6.2.3 Settlement reduction analysis

- In this investigation, the settlement reduction ratio (SRR) was calculated as a measure of the settlement reduction analysis. Generally, the lower the SRR, the larger the reduction in settlement. SRR varied between 0.79 and 0.93. When compared to the values obtained in previous research works, these were relatable.

- Randomly mixed fibres were found to cause the highest reduction in settlement, when the RGCs were installed in base soils at LL. In this instance, the largest concentration of fibres of 0.1 % produced the lowest SRR value of 0.79. The reason behind this decreased settlement was due to the formation of a much stiffer and stronger composite mass within the column, when the fibres were introduced. The gain in strength enhanced the column's resistance to lateral deformation, which was triggered by vertical strain.

- The Betatex geotextile, with a mass per unit area of 600 g/m^2, also produced a relatively high reduction in settlement and generated an SRR of 0.81 which was rather close to that of the column which was reinforced with randomly mixed fibres at a concentration of 0.1 %.

6.2.4 Maximum lateral bulging behaviour for each tested column

- Although, it was intended for a column of diameter 100 mm to be installed, modelling of the OGCs (in base soils at OMC and LL) showed that this dimension differed slightly once the column was formed. This was dependent on the moisture content of the host soil, as well as, the type, quantity and arrangement of the reinforcements. Hence, comparison of the bulging behaviour was performed based on the post-testing bulging diameters only.

- Compared to previous studies which have reported maximum bulging to occur at a certain depth within the column, this study confirmed that the largest lateral deformation occurred within a certain depth which was referred to as the maximum bulging zone. Maximum bulging in all the tests was considered to be that which was attained at a settlement of 50 mm.

- It appeared that the mechanism of failure in these columns, where the column length was 4 times that of its diameter, was essentially through bulging.

- Bulging was higher when the columns were installed in the wetter base soil (at LL). This was due to the lower confining stresses which were provided by the surrounding soil.

- Overall, the layering arrangement appeared to have a better impact on bulge size reduction, irrespective of the type of reinforcement used. However, this decrease was even more prominent in tests which were conducted on base soils at LL.

- The smallest deformation of 110 mm was achieved in an OMC test whereby the column was reinforced with a concentration of 0.83 % of layers of fibres. The inclusion of fibres frequently reduced the bulging diameter. This was due to the heavy interlocking of the sand particles around the fibres. The volume of empty spaces between the particles were, therefore, reduced. As such, the stiffness of each column was enhanced; thus, the extent of lateral movement was limited.

- In the LL tests, the smallest bulging diameter of 124 mm was obtained with the layering arrangement when the Betatex geotextile of a mass per unit area of 600 g/m^2 was used. The high stiffness and rough surface of the geotextile allowed better bonding with the sand particles which in turn reduced the range of bulging.

6.2.5 Length span corresponding to the maximum lateral bulging zone

- From the analyses, the bulging response was found to be dependent on the type, quantity and arrangement of the reinforcement. The moisture content of the base soil also had a major influence on the length spans of bulging and their respective positions within the columns.

- Generally, the zones of maximum bulging appeared to be located further down the columns when they were installed in base soils at OMC. In comparison, the positions of the maximum bulging zones were further up in the column when the moisture content of the base soils was wetter (at LL).

- Overall, the shortest span of between 325 and 330 mm was obtained when a column was reinforced by layers of fibres (concentration of 0.28 %) and installed in a base soil at LL. Contrastively, the longest span was between 50 and 290 mm; this occurred in an OMC test where a column was reinforced with layers of fibres at a concentration of 0.83 %.

- In terms of the position of bulging, the highest position (between 337 and 347 mm) occurred in a column which was reinforced by the Fibertex geotextile with a mass per unit area of 600 g/m^2 and installed in a base soil at LL. The lowest position (between 50 and 290 mm) was, however, noted in a base soil at OMC when the column was reinforced by layers of fibres at a concentration of 0.83 %. Although bulging in this instance appeared to be at the lowest position, it was also more uniformly distributed along the column which in turn made it more resistant to bulging.

6.2.6 Empirical equations developed from the experimental results

- To generate both the stress-reinforcement content and bulging diameter- reinforcement content relationships, the Eureqa software was utilised to generate the mathematical equations which governed these relationships. The software performed symbolic regression analyses on the experimental data and proposed several equations which could possibly dictate these relations. The preferred equations for the different test series were chosen from a possible list of equations such that they were the easiest to understand and to solve. However, it was ensured that the R^2 value and the correlation coefficient was 1 in each of the equations. Generally, all the equations which were selected appeared to be polynomials of degree 3 and were of the following form (a, b, c and d were non-zero values in all the equations, and they were dependent on factors such as moisture content of the base soil, type of reinforcement and the arrangement of the reinforcement):

$$y(x) = ax^3 + bx^2 + cx + d$$

6.2.7 Comparison of performances of columns reinforced with the waste geotextile (Betatex) and the virgin geotextile (Fibertex)

- With regards to the maximum bulging diameters, they were generally smaller in the OMC tests, irrespective of the type of the geotextile. However, Betatex had a higher tendency of producing larger diameters. During the recycling process of PET bottles (the raw material from which Betatex was manufactured), their physical or mechanical properties might have weakened. Consequently, Betatex might have had a lower resistance to bulging compared to Fibertex.

- In terms of the length spans corresponding to the maximum bulging zones, significant differences were noted from the two geotextiles. In the OMC test, Betatex appeared to produce columns where bulging was more uniformly distributed as their mass per unit area was augmented. In contrast, higher variations with Fibertex were obtained for the same test characteristics. In the LL tests, the lengths spans attained with both geotextiles were relatively similar. Nevertheless, drastic and irregular differences were noted with regards to the positions of the maximum bulging zones.

6.2.8 Variation in the quantity of the column and reinforcement materials utilised

- Overall, irrespective of the testing conditions, the inclusion of the reinforcements resulted in a decrease in the mass of sand needed to form the columns.

- When randomly mixed flakes and fibres were used to reinforce the columns, it was apparent that the mass of sand required to form a column in the wetter soil (at LL) was much higher compared to when it was installed in a drier soil (at OMC). In effect, columns installed in base soils at LL were much heavier than those in the drier base soils. This was because the low confining stresses from the host soils resulted in pre-bulging during the installation process. Hence, more sand was required to complete the formation of the columns.

- For the random mixing arrangement of the reinforcements, fibres appeared to cause the highest reduction in the mass of sand used to form the columns when the latter were installed in base soils at OMC. However, in the LL tests, flakes appeared to be a better performer than fibres.

- Generally, irrespective of the arrangement of the reinforcement and the moisture content of the base soil, the mass of fibres required in the tests seemed remarkably low. This was explained by the larger volumes which were occupied by fibres though their masses remained minimal. On the contrary, the masses of flakes remained highest under all conditions.

- In the layering arrangement, the lowest masses of reinforcement were generated from the geotextiles. However, the largest saving of sand occurred when layers of flakes at a concentration of 5.6 % were utilised, although the mass of flakes was significantly high.

- Conclusively, it was found that as the mass of reinforcement was raised, the quantity of sand required to form the columns decreased. Moreover, it was also evident that each type of reinforcement responded distinctively in terms of the reduction in sand achieved. However, fibres were identified as the better performing material since a minimal mass resulted in a significant drop in the mass of sand used.

6.3 Recommendations

Through this research, knowledge relating to reinforced granular columns has been extended. The laboratory investigations have provided insights into the stress-settlement and deformation characteristics of columns which have been reinforced with waste PET bottles in different forms (flakes, fibres, geotextile), and installed in a silt at different moisture contents. Although the generated information has answered several questions, several essential areas remain unanswered. Therefore, few recommendations have been made which may be worth considering for future work. These are as follows:

a) Previous studies have elaborated on the performance of singular columns in comparison to group ones. Columns in group tend to behave slightly different such that the load carrying capacity are typically higher. Therefore, it will be worth investigating the behavioural effect of the reinforced granular columns when installed in a group.

b) While this study only varied the type and quantity of reinforcement, their arrangement within the column and the moisture content of the base soil, there are several other factors which need to be investigated since they significantly influence both the stress-settlement and the deformation characteristics of the columns when subjected to vertical loading. These include the following: column length, number of layers of reinforcement, more varying reinforcement concentrations, area replacement ratios and the types of sand used for the columns and possibly the combined effect of random mixing and layering within an individual column.

c) Only laboratory tests were performed in this research. Although several factors were investigated, a more accurate and detailed solution is required; the findings are believed to assist in this area. To validate the results obtained, especially considering the scale effect, it will be necessary to conduct field studies. However, the methodology will more likely need some adjustments to ensure its compatibility with the equipment to be used on site.

d) Although granular columns are popular for the improvement of load carrying capacity and for the reduction of settlement, they are also widely used with an aim for drainage. Sand columns are often used for drainage purposes because of their high permeability. Normally, the columns are installed in soils with high water contents; upon loading, excess pore water pressures are generated which subsequently require an allowance for drainage. Hence, the drainage efficiency of the reinforced granular columns is critically important. Permeability tests may be performed in this regard to establish the permeability of the reinforced granular columns.

e) Once more investigations have been undertaken, it will be important to numerically model the behaviour of the granular columns using the experimental data. This will allow for future predictions of the performance of the columns under different conditions.

f) Since this study only performed undrained tests, it will be worth investigating the performance of the columns under drained conditions. This can be investigated simultaneously with the drainage efficiency of the columns since long term loading tests will be dependent on the draining ability of the column for optimised performance.

g) From an environmental point of view, analysis of the carbon dioxide emissions associated with the different forms of PET used must be researched and the outcome must be compared with that which is generated during sand mining. This is necessary to establish whether the technology is truly environmentally friendly.

h) Chemical and physical degradation (as a result of the sand-fibre interaction) of the fibres must be researched to identify their life time as well as any changes that can arise in their strength characteristics when they are left in the ground for a long time.

i) Since one of the purposes of the study was to establish a construction technique which will possibly contribute to the reduction of waste PET bottles in the landfills, it will be essential to conduct a study on the solid waste management cycle to identify if the proposed technology is indeed beneficial.

j) From the findings of this research, it is not possible to execute back-calculations since several parameters have not been investigated. One of the many factors are the shear strength parameters of the composite column material which were not investigated. Hence, it is recommended for shear tests to be conducted on the different configuration of the column materials to establish the associated friction angle and cohesion.

References

1. Abhijit, S. and Das, S.C. (2000) Interaction analysis of stone column groups in foundations, *Proceedings of the Indian Geotechnical Conference*, Bombay, India, pp. 279-284.

2. Aboshi, H., Ichimoto, E., Enoki, M. and Harada, K. (1979) The Compozer-A method to improve characteristics of soft clays by inclusion of large diameter sand columns, *Proceedings of the International Conference on Soil Reinforcement: Reinforced Earth and Other Techniques*, Vol. 1, Paris, pp. 211-216.

3. Adelana, S., Xu, Y. and Vrbka, P. (2010) A conceptual model for the development and management of the Cape Flats aquifer, South Africa, *Water SA*, Vol. 36, No. 4, pp. 461-474.

4. Afshar, J.N. and Ghazavi, M. (2014) Experimental studies on bearing capacity of geosynthetic reinforced column, *Arabian Journal for Science and Engineering*, Vol. 39, pp. 1559-1571.

5. AGIS. (2011) *Natural resources atlas: Soil classes.* [Online]. Available from: www.agis.agric.za [Accessed 5 July 2011].

6. Alamgir, M., Miura, N., Poorooshasb, H.B. and Madhav, M.R. (1996) Deformation analysis of soft ground reinforced by columnar inclusions, *Computers and Geotechnics*, Vol. 18, No 4, pp. 267-290.

7. Alexiew, D., Moormann, C. and Jud, H. (2009) Foundation of a coal/coke stockyard on soft soil with geotextile encased columns and horizontal reinforcement, *Proceedings of the 17th International Conference on soil Mechanics and Geotechnical Engineering*, Hamza, M. et al. (eds.).

8. Ali, K. (2014) Effect of encasement length on geosynthetic reinforced stone columns, *International Journal of Research in Engineering and Technology*, Vol. 3, Issue 6, pp. 72-75.

9. Ali, K., Shahu, J.T. and Sharma, K.G. (2010) Behaviour of reinforced stone columns in soft soils: An experimental study, *Indian Geotechnical Conference*, GEOtrendz, IGS Mumbai Chapter and IIT Bombay, India.

10. Allgaier, N., and McDevitt, R. (2018) *Reverse engineering the brain with Eureqa.* [Online]. Available from: http://www.uvm.edu/~nallgaie/research/RevEngBrain.pdf [Accessed on 27 December 2018].

11. Almeida, M.S.S., Hosseinpour, I., Riccio, M. and Alexiew, D. (2014) Behaviour of geotextile-encased granular columns supporting test embankment of soft deposit, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 040141 16, pp. 1-9.

12. Al-Obaidy, N. (2017) Treatment of collapsible soil using encased stone columns, PhD Thesis, University of Birmingham.

13. Al-Refeai, T.O. (1992) Strengthening of the soft soil by fiber-reinforced sand column, *Proceedings of the International Symposium on Earth Reinforcement Practice*, Fukoka, Japan, Vol. 1, pp. 677-682.

14. Al-Waily, M.J.M. (2012) Effect of area replacement ratio on bearing capacity of soil treated with stone column, *Journal of Kerbala University*, Vol. 10, No 4, pp. 280-290.

15. Andrady, A.L. (2011) Microplastics in the marine environment, *Marine Pollution Bulletin*, Vol. 62, pp. 1596–1605.

16. Andreou, P., Frikha, W., Frank, R., Canou, J., Papadopoulos, V. and Dupla, J-C. (2008) Experimental study on sand and gravel columns in clay, *Proceedings of the Institution of Civil Engineers*, Ground Improvement 161, Issue GI4, pp. 189-198.

17. Ambily, A.P. and Gandhi, S.R. (2007) Behaviour of Stone Columns Based on Experimental and FEM Analysis, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 133, No 4, pp. 405-415.

18. Arman, H., Firat, S., Vural, I. and Gunduz, Z. (2009) Soil and foundation stability improvement by stone column: A case study in Adapazari city, Turkey, *Scientific Research and Essay – Academic Journals*, Vol. 4, No 10, pp. 972–983.

19. ASTM E177-14, *Standard practice for use of the terms precision and bias in ASTM test Methods*, ASTM International, United States.

20. Awaja, F. and Pavel, D. (2005) Recycling of PET, *European Polymer Journal*, Vol. 41, pp. 1453–1477.

21. Ayadat, T., Hanna, A.M. and Hamitouche, A. (2008) Soil improvement by internally reinforced stone columns, *Proceedings of the Institution of Civil Engineers*, Ground Improvement 161, Issue GI2, pp.55-63.

22. Bachus, R. C. (1989) Design Methodology for Foundations on Stone Columns, *Proceedings of the ASCE Conference on Current Principles and Practices in Foundation Engineering*, Vol. 1, Evanston, IL, USA.

23. Balaam, N. P., Poulos, H. G., and Brown, P. T. (1978) Settlement analysis of soft clays reinforced with granular piles, *Proceedings of the 5th Asian Conference on Soil Engineering*, Bangkok, Thailand, pp. 81–92.

24. Balfour Beatty Ground Engineering. (2018) *Vibro stone columns, Technique sheet*. [Online]. Available from: https://www.balfourbeatty.com/media/29501/vibro-stone-columns.pdf [Accessed 31 May 2018].

25. Barksdale, R.D. (1987) State of the Art for Design and Construction of Sand Compaction Piles, *Technical Report REMR-GT-4*, Georgia Institute of Technology, prepared for Department of the Army, US Army Corps of Engineers, Washington, DC.

26. Barksdale, R.D. and Bachus, R.C. (1983) *Design and Construction of Stone Columns, Federal Highway Administration*, Washington DC, Final Report SCEGIT, pp. 83-104.

27. Basu, P. (2009) Behaviour of sand-fiber mixed granular piles, PhD Thesis, Indian Institute of Technology Roorkee, India.

28. Bauman, V. and Bauer, G.E. (1974) The performance of foundations on various soils stabilized by the vibro-compaction method, *Canadian Geotechnical Journal*, Vol. 11, Issue 4, pp. 509–530.

29. Bell, F.G. (2004) *Engineering Geology and construction*, Spon Press, London, pp. 392.

30. Benson, C.H. and Khire, M.V. (1993) Soil reinforcement with strips of reclaimed HDPE, *Proceedings of the Geosynthetics '93 conference*, Canada. pp. 935–948.

31. Bergado, D.T., Rantucci, G. and Widdodo, S. (1984) Full scale load test of granular piles and drains in soft Bangkok clay, *Proceeding of the International Conference on In Situ Soil and Rock Reinforcement*, Paris, pp. 111-118.

32. Bergado, D.T., Huat, S.H. and Kalvade, S. (1987) Improvement of soft Bangkok clay using granular piles in subsiding environment, *Proceedings of the 5th International Geotechnical Seminar on Case histories in soft clay*, Singapore, pp. 219-226.

33. Bergado, D.T., Alfaro, M.C. and Chai, J.C. (1991) The granular pile: Its present state and future prospects for improvement of soft Bangkok clay, *Geotechnical Engineering*, Vol. 22.

34. Bergado, D.T., Anderson, L.R., Miura, N. and Balasubramaniam, A.S. (1996) *Soft ground improvement in lowland and other environments*, ASCE press, New York, USA, pp. 427.

35. Bergeret, A., Ferry, L. and Ienny, P. (2009) Influence of the fibre/matrix interface on ageing mechanisms of glass fibre reinforced thermoplastic composites (PA-6,6, PET, PBT) in a hygrothermal environment, *Polymer Degradation and Stability*, Vol. 94, pp. 1315–1324.

36. Bhattarai, P., Bharat Kumar, A.V.A., Santosh, K., Manikanta, T.C. and Tejeswini, K. (2013) Engineering behaviour of soil reinforced with plastic strips, *International Journal of Civil, Structural, Environmental and Infrastructure Engineering Research and Development*, Vol. 3, Issue 2, pp. 83-88.

37. Bonaparte, R., Holtz, R.D. and Giroud, J.P. (1987) Soil reinforcement design using geotextiles and geogrids, *Geotextile Testing and the Design Engineer*, ASTM Special Technical Publication 952, Fluet, J.E. (ed.), pp. 69-116.

38. Bonaparte, R. and Schmertmann, G.R. (1987) Reinforcement extensibility in reinforced soil wall design, *The application of polymeric reinforcement in soil retaining structures*, McGown, A. and Jarrett, P.M. (eds.), pp. 410.

39. Bonhomme, S.; Cuer, A.; Delort, A.-M.; Lemaire, J.; Sancelme, M.; Scott, G. (2003) Environmental degradation of polyethylene, *Polymer Degradation and Stabilility*, Vol. 81, pp. 441–452.

40. Bowles, J.E. (1997) *Foundation Analysis and Design*, Fifth Edition, McGraw-Hill, Singapore.

41. Brauns J. (1978) Initial bearing capacity of stone columns and sand piles, *Soil reinforcing and stabilizing techniques in engineering practice*, Sydney I, pp. 497–512.

42. Broms, B. (1979) Problem and solution to construction on soft clay, *Proceedings of the 6th Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Singapore, Vol. 2, pp. 3–40.

43. Brown, R.E. (1977) Vibroflotation compaction of cohesionless soils, *Journal of the Geotechnical Engineering Division*, ASCE, Vol.103, Issue 12. Pp. 1437-1451.

44. Castro, J. and Sagaseta, C. (2009) Consolidation around stone columns—influence of column deformation, *International Journal for Numerical and Analytical Methods in Geomechanics*, Vol. 33, Issue 7, pp. 851–877.

45. Castro, J. and Sagaseta, C. (2011) Consolidation and deformation around stone columns: Numerical evaluation of analytical solutions, *Computers and Geotechnics*, Vol. 8, pp. 354–362.

46. Chawla, G.R., Raju, V.R. and Krishna, Y.H. (2010) Some environmental benefits of dry vibro stone columns in a gas based power plant project, *Indian Geotechnical Conference*, GEOtrendz, India.

47. Chebet, F.C., Kalumba, D. and Avutia, D. (2012) Investigating the effect of plastic shopping bag waste material on load bearing capacity of foundation soils in Civil Engineering, *21st WasteCon Conference and Exhibition*, ICC East London, South Africa.

48. Chiu, S.J. and Cheng, W.H. (1999) Thermal degradation and catalytic cracking of poly(ethylene terephthalate), *Polymer Degradation and Stability*, Vol. 63, pp. 407-412.

49. Choudhary, A.K., Jha, J.N. and Gill, K.S. (2010) A study on CBR behaviour of waste plastic strip reinforced soil, *Emirates Journal for Engineering Research*, Vol. 15, Issue 1, pp. 51-57.

50. Christopher et al. (1989) *Reinforced soil structures, Vol. 2: Summary of research and systems information, Federal Highway Administration Office of Engineering and Highway Operations Research and Development*, McLean, Virginia.

51. Consoli, N., Montardo, J., Prietto, P. and Pasa, G. (2002) Engineering Behavior of a Sand Reinforced with Plastic Waste, *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 128, Issue 6, pp. 462–472.

52. CSIR. (2011) *Municipal waste management - good practices*, Edition 1, CSIR, Pretoria, ISBN No: 978-0-7988-5596-9.

53. Das, R., Majhi, K., Khatun, C. and Maiti, A. (2017) Soil stabilization using plastic strips of varied sizes by enhancing the bearing capacity, *International Journal of Scientific & Engineering Research*, Vol. 8, Issue 3, pp. 74-79.

54. Datye, K. R. (1982) Settlement and Bearing Capacity of Foundation System with Stone Columns, *Proceedings of the ASCE Symposium on Recent Developments in Ground Improvement Techniques*, Bangkok.

55. Datye, K.R. and Madhav, M.R. (1988) Case histories of foundations with stone columns, *Proceedings of the Second International Conference on Case Histories in Geotechnical Engineering*, St. Louis, Mo., pp. 1075-1086.

56. Dave, T.N. and Thaker, T.P. (2017) Reuse of plastic waste in foundation soil reinforcement application, *Proceedings of the 19th International Conference on soil Mechanics and Geotechnical Engineering*, Seoul, pp. 3369-3372.

57. Diop, S., Stapelberg, F., Tegegn, K., Ngubelanga, S. and Heath, L. (2011) *A review on problem soils in South Africa*, Council for Geoscience, Western Cape Unit, South Africa, Report number: 2011-0062.

58. Dutta, R.K. and Sarda, V.K. (2007) CBR behaviour of waste plastic strip-reinforced stone dust/fly ash overlying saturated clay, *Turkish Journal of Engineering and Environmental Sciences*, Vol. 31, pp. 171-182.

59. Edil, T.B., and Bosscher, P.J. (1994) Engineering properties of tire chips and soil mixtures, *ASTM Geotechnical Testing Journal*, Vol. 17, Issue 4, pp. 453–464.

60. Etezad, M., Hanna, A.M. and Ayadat (2015) Bearing capacity of a group of stone columns in soft soil, *International Journal of Geomechanics*, Vol. 15, Issue 2.

61. Eurocode 7. *Geotechnical Design-Part 1: General rules*, Annex H, British Standard, BS EN 1997-1:2004.

62. European Union (2010) *Being wise with waste: The EU's approach to waste management*, Belgium.

63. Fattah, M.Y., Shlash, K.T. and Al-Waily, M.J.M. (2011) Stress concentration ratio of model stone columns in soft clays, *Geotechnical Testing Journal*, Vol. 34, No. 1.

64. Franki, A Keller Company. (2018) *Vibro replacement*. [Online]. Available from: http://www.franki.co.za/products/ground-improvement/vibro-replacement/ [Accessed 31 May 2018].

65. Franklin Associates (2011) *Cradle-to-Gate Life Cycle Inventory of Nine Plastic Resins and Four Polyurethane Precursors*, The Plastics Division of the American Chemistry Council, Prairie Village, KS, USA.

66. Forrest, M. (2016) *Recycling of polyethylene terephthalate*, Smithers Rapra Technology Ltd, UK.

67. Gavriletea, M.D. (2017) Environmental impacts of sand exploitation, Analysis of sand market, *Sustainability*, Vol. 9, Issue 7.

68. Ghanti, R. and Kashliwal, A. (2008) *Ground Improvement Techniques – with a focussed study on stone columns*, Dura Build Care Pvt Ltd., VIT University, India. [Online]. Available from: http://www.durabuildcare.com/pdf/Ultimate%20Bearing%20Capacity%20of%20a%20SINGLE%20STONE%20COLUMN.pdf [Accessed 6 April 2011].

69. Gibson, R.E. and Anderson, W.F. (1961) In situ measurements of soil properties with the pressuremeter, *Civil Engineering and Public Works Review*, Vol. 56, No 658.

70. Goughnour, R.R. (1983) Settlement of vertically loaded stone columns in soft ground, *Proceedings of the 8th European CSMFE*, Helsinki, pp.235-240 Rathmeyer, H.G. and Saari, K.H.O. (eds.), AA Balkema.

71. Greenwood, D.A. (1970) Mechanical Improvement of soils below ground surface, *Conference on Ground Engineering*, Institution of Civil Engineers, London, pp. 11-22.

72. Griffith, C.J. (1991) *Soil improvement through vibro-compaction and vibro-replacement*, University of Maryland, Dept. of Civil Engineering, Unpublished.

73. Guo, W. D. and Qian, H. J. (1990) New methods for calculating bearing capacity of granular pile foundations, *Ground Improvement*, Vol. 1, Issue 1, pp. 38–46 (in Chinese).

74. Han, J. (1992) Stone column techniques—general report, *Proceedings of the 3rd Chinese Soil Improvement Conference*, Qengwangdao, China.

75. Han, J. (2010) Consolidation settlement of stone column reinforced foundations in soft soils, Invited paper, New Technologies on Soft Soils, *Proceedings of Symposium on New Techniques for Design and Construction on Soft Clays*, M. Almeida (ed.), Brazil, pp. 167–179.

76. Han, J. (2015) *Principles and practice of ground improvement*, John Wiley & Sons, Hoboken, New Jersey, pp. 218.

77. Hendey, Q.B. and Dingle, R.V. (1983) *Onshore sedimentary phosphate deposits in South Western Africa*, Technical Report, Joint Geological Survey/University of Cape Town Marine Geoscience Unit 14, pp. 27-40.

78. Hermanova, S., Smejkalova, P., Merna, J. and Zarevucka, M. (2015) Biodegradation of waste PET based copolyesters in thermophilic anaerobic sludge, *Polymer Degradation and Stability*, Vol. 111, pp. 176-184.

79. Home Building Manual, (2014), South Africa. [Online]. Available from: http://www.nhbrc.org.za/wp-content/uploads/2014/11/Home-Building-Manual-2014-final-Oct-G.pdf [Accessed on 7 January 2019]

80. Hu, W. (1995) Physical modelling of group behaviour of stone column foundations, PhD Thesis, University of Glasgow.

81. Hughes, J.M.O. and Withers, N.J. (1974) Reinforcing of soft cohesive soils with stone columns, *Ground Engineering*, Vol 7, No 3, pp. 42-49.

82. Hughes, J.M.O., Withers, N.J., Greenwood, D.A. (1975) A field trial of the reinforcing effect of a stone column in soil, *Geotechnique*, Vol. 25, Issue 1, pp. 31–44.

83. Isaac, D.S. and Madhavan, S.G. (2009) Suitability of Different Materials for Stone Columns Construction, *Electronic Journal of Geotechnical Engineering*, Vol. 14, Bund. M, pp. 1-12.

84. Jessberger, H.L., Jagow-Klaff, R. and Braun, B. (2003) Ground Freezing, Smoltczyk, U., (ed.), *Geotechnical Engineering Handbook*, Vol. 2: Procedures, pp.117-167, Ernst & Sohn, Berlin.

85. Jiang, Y., Han, J., and Zheng, G. (2013) Numerical analysis of consolidation of soft soils fully-penetrated by deep-mixed columns, *KSCE Journal of Civil Engineering*, Vol. 17, Issue 1, pp. 96–105.

86. John, N.W. M. (1987) *Geotextile*, Chapman and Hall, New York.

87. Jones, C.J.F.P. (1988) *Earth reinforcement and soil structures*, Butterworth & Co. Ltd, England, Chapters 1-4.

88. Khoramnejadian, S. (2011) Improve properties of recycled polyethylene terephethalat (PET) by polycarbonate, *Asian Journal of Research in Chemistry*, Vol. 4, Issue 10, pp. 1539-1541.

89. Killeen, M. M. and McCabe, B. A. (2014) Settlement performance of pad footings on soft clay supported by stone columns: A numerical study, *Soils and Foundations*, Vol. 54, Issue 4, pp. 760–776.

90. Kint, D. and Muñoz-Guerra, S. (1999) A review on the potential biodegradability of poly(ethylene terephthalate), *Polymer International*, Vol. 48, Issue, 5, pp. 346–352.

91. Komolprasert, V. and Bailey, A. (2008) Recycled plastics for food applications: improving safety and quality, *Environmentally Compatible Food Packaging, A volume in Woodhead Publishing Series in Food Science, Technology and Nutrition*, Chiellini, E. (ed), pp. 326-350.

92. Krishna, A.M. and Madhav, M.R. (2009) Treatment of loose to medium dense sands by granular piles: Improved SPT 'N' values, *Geotechnical and Geological Engineering*, Vol. 27, pp. 455-459.

93. Kruger, J.J., Guyot, C. and Morizot, J.C. (1980) The dynamic substitution method, *International Conference on Compaction*, Paris
Editions Anciens ENPC, Laboratoire Central des Ponts et Chaussees, Ecole Nationale des Ponts et Chaussees.

94. Laskar, A. and Pal, S.K. (2013) Effects of waste plastic fibres on compaction and consolidation behaviour of reinforced soil, *Electronic Journal of Geotechnical Engineering*, Vol. 18, Bund, H, pp. 1547-1558.

95. Levchik, S.V. and Weil, E.D. (2004) A review on thermal decomposition and combustion of thermoplastic polyesters, *Polymer for Advanced Technologies*, Vol.15, pp. 691–700.

96. Luwalaga, J.G. (2015) Analysing the behaviour of soil reinforced with polyethylene terephthalate (PET) plastic waste, MEng. Thesis, Faculty of Engineering, Stellenbosch University.

97. Madhav, M.R. (1982) Recent developments in the use and analysis of granular piles, *Syposium on Recent Developments in Ground Improvement Techniques*, Bangkok, pp. 117-129.

98. Madun, A., Meghzili, S.A., Tajudin, S.A.A., Yusof, M.F., Zainalabidin, M.H., Al-Gheethi, A.A., Md Dan, M.F. and Ismail, M.A.M. (2018) Mathematical solution of

the stone column effect on the load bearing capacity and settlement using numerical analysis, *Journal of Physics: Conference Series*, 995 012036.

99. Mahoney, D.P. and Kupec, J. (2014) Stone columns ground improvement field trial: A Christchurch case study, *NZSEE Conference*, New Zealand.

100. Malarvizhi, S.N. and Ilamparuthi, K. (2004) Load versus Settlement of Clay bed Stabilized with Stone & Reinforced Stone Columns, *Proceedings of the 3rd Asian Regional Conference on Geosynthetics*, GeoAsia, Seoul, Korea, pp. 322–329.

101. Marandi, S.M., Bagheripour, M.H., Rahgozar, R. and Zare, H. (2008) Strength and ductility of randomly distributed palm fibres reinforced silty-sand soils, *American Journal of Applied Sciences*, Vol. 5, No 3, pp. 209-220.

102. Maurya, R.R., Sharma, B.V.R. and Naresh, D.N. (2005) Footing load tests on single and group of stone columns, *Proceedings of the 16th International Conference on soil Mechanics and Geotechnical Engineering*, pp. 1385-1388.

103. Mekkiyah, H.M. and Al-Saadi, S.Z. (2016) Experimental study for granular column and fiber granular column, *Journal of Civil Engineering Research*, Vol. 6, No 5, pp. 199-127.

104. McCabe, B.A., McNeill, J.A., Black, J.A. (2007) Ground Improvement using the vibro-stone column technique, *Joint meeting of Engineers Ireland West Region and the Geotechnical Society of Ireland*, NUI Galway, The Institution of Engineers of Ireland.

105. McCabe, B.A., Nimmons, G.J. and Egan, D. (2009) A review of field performance of stone columns in soft soils, *Proceedings of the Institution of Civil Engineers, Geotechnical Engineering*, Vol. 162, Issue GE6, pp. 323-334.

106. McKelvey, D. and Sivakumar, V. (2000) A review of the performance of vibro stone column foundations, *3rd International Conference on Ground Improvement Techniques*, Singapore, pp. 245-254.

107. McKelvey, D., Sivakumar, V., Bell, A. and Graham, J. (2004) Modelling vibrated stone columns in soft clay, *Proceedings of the Institution of Civil Engineers, Geotechnical Engineering*, Vol. 157, Issue GE3, pp. 137-149.

108. Miller, H., (2002), Modelling the collapse of metastable loess soils, PhD thesis, The Nottingham Trent University, UK.

109. Mirafi, (2010), *Geosynthetics for soil reinforcements*, Ten Cate Nicolon. [Online]. Available from: http://www.tencate.com/TenCate/Geosynthetics/documents/Brochures/BRO_soilrein.pdf [Accessed 9 June 2010).

110. Miraftab, M. and Lickfold, A. (2008) Utilization of carpet waste in reinforcement of substandard soils, *Journal of industrial textiles*, Vol. 38, pp.167-174.

111. Mitchell, A.J. (1990) *Formulation and production of carbonated soft drinks*, Blackie and Son Ltd, Glasgow and London.

112. Mitchell, J.K. and Katti, R.K. (1981) Soil Improvement - State-of-the Art Report, *Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering*, Stockholm, Vol.4, pp. 509-565.

113. Mitra, S. and Chattopadhyay, B. C. (1999) Stone columns and design limitations, *Proceedings of the Indian Geotechnical Conference*, Calcutta, India, pp. 201–205.

114. Mokashi, S.L., Paliwal, S.T., and Bapaye, D.R. (1976) Use of stone columns for strengthening soft foundation clay, *Proceedings of the Central Board of Irrigation and Power*, 45th Annual Research Session, Vol. 3—Soils and Concrete, pp. 61–68.

115. Mori, H. (1979) Some case records of stability and settlement of embankment on the soft ground, *Proceedings of the 6th Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Singapore, Vol. 2, pp. 169–189.

116. Mueller R.J. (2006) Biological degradation of synthetic polyesters—enzymes as potential catalysts for polyester recycling, *Process Biochemistry*, Vol. 41, pp. 2124–2128.

117. Müller, R.J., Kleeberg, I. and Deckwer, W.D. (2001) Biodegradation of polyesters containing aromatic constituents, *Journal of Biotechnology*, Vol. 86, pp. 87–95.

118. Munfakh, G.A., Sarkar, S.K. and Caslelli, R.J. (1983) Performance of test embankment founded on stone columns, *Proceedings of the International Conference on Advances in Piling and Ground Treatment for Foundations*, pp. 259-265.

119. Murugesan, S. and Rajagopal, K. (2008) Performance of encased stone columns and design guidelines for construction on soft clay soils, *Proceedings of the 4th Asian Regional Conference on geosynthetics*, Shanghai, China.

120. Mustapha, A.M. (2008) Bamboo as Soil Reinforcement: A Laboratory Trial, *Leonardo Journal of Sciences*, Issue 13, pp. 69-77.

121. Najjar, S.S. (2013) A state-of-the-art review of stone/sand column reinforced clay systems, *Geotechnical and Geological Engineering*, Vol. 31, pp. 355-386.

122. Neopaney, M., Wangchuk, U.K. and Tenzin, S. (2012) Stabilization of soil using plastic wastes, *International Journal of Emerging Trends in Engineering and Development*, Vol. 2, Issue 2, pp. 461-466.

123. NWMS. (2011) *National waste management strategy*, Department of Environmental Affairs, Republic of South Africa.

124. Perold, J. (2006) Ceramic parameters in the financial evaluation of brick clay deposits, with reference to two South African examples, MSc. Thesis, Faculty of Natural and Agricultural Sciences, University of Pretoria.

125. PETCO. (2014) Annual report celebrating PETCO. [Online]. Available from: https://www.petcodb.co.za/ag3nt/media/media_items/2015//1433755684.pdf [Accessed 3 July 2016)

126. PETCO. (2018a) SA PET industry hits new high with 2.15 billion bottles recycled. [Online]. Available from: http://petco.co.za/sa-pet-industry-hits-new-high-2-15-billion-bottles-recycled/ [Accessed 31 July 2018].

127. PETCO. (2018b) [Online]. Available from: www.petco.co.za [Accessed 1 August 2018].

128. PETCO (2018c) Plastics packaging recyclability by design. [Online]. Available from: https://petco.co.za/wp-content/uploads/2016/08/PETCO_Design-4-Recycling-Guide.pdf [Accessed 10 August 2018]

129. PETRA, (2018) PET resin facts at a glance. [Online]. Available from: http://www.petresin.org/pdf/PETResinFactsataGlance.pdf [Accessed 31 July 2018].

130. Pivarc, J. (2011) Stone columns – Determination of the soil improvement factor, *Slovak Journal of Civil Engineering*, Vol. 19, No. 3, pp. 17-21.

131. PLASTICS SA. (2015) *The plastic identification code*. [Online]. Available from: http://www.plasticsinfo.co.za/2014/10/14/zero-plastics-to-landfill/ [Accessed on 3 July 2016].

132. PLASTICS SA. (2016a) *Zero plastics to landfill: Seven key areas identified*. [Online]. Available from: http://www.plasticsinfo.co.za/2014/10/14/zero-plastics-to-landfill/ [Accessed on 3 July 2016).

133. PLASTICS SA. (2016b) *Plastics SA releases 2015 plastics recycling figures*, Press release. [Online]. Available from: http://www.plasticsinfo.co.za/wp-content/uploads/2016/05/Plastics-recycling-figures-2015-1.pdf [Accessed on 19 January 2019].

134. PLASTICS SA (2019) *2017/2018 Annual Review*. [Online]. Available from: http://www.plasticsinfo.co.za/annual-reports/ [Accessed on 7 January 2019]

135. Pokharel, G. (1995) Deformation and ultimate load of reinforced soil structures - Theory and Experiment, PhD Thesis, Department of Civil Engineering, Nagoya University, Japan.

136. Priebe, H.J. (1995) The design of vibro replacement, *Ground Engineering*, Vol. 28, No 10, pp. 31-37.

137. Pulko, B. and Majes, B. (2005) Simple and accurate prediction of settlements of stone column reinforced soil, *Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering*, Osaka, Japan, pp. 1401–1404.

138. Pulko, B. and Majes, B. (2006) Analytical method for the analysis of stone-columns according to the Rowe dilatancy theory, *Acta Geotechnica Slovenica*, Vol. 1, pp. 37-45.

139. Purushothama Raj, P. (2005) *Ground Improvement Techniques*, pp.142, First Edition, Laxmi Publications (P) Ltd, New Delhi, India.

140. Raju, V.R. (2003) Ground improvement techniques for railway embankments, Keller (M) Sdn Bhd, Malaysia, Technical paper 10-59E. [Online]. Available from: http://www.fenixdigital.com.br/teste/keller/images/10-59E.pdf [Accessed on 25 June 2019]

141. Raju, V.R. and Valluri, S. (2008) Practical applications of ground improvement, *Symposium on Engineering of Ground & Environmental Geotechniques*, (S EG2), Hyderabad, India.

142. Rao, B.G. and Bhandari, R.K. (1977) Reinforcing of non-cohesive soil by granular piles, *Proceedings of the 6th ARC on SMFE*, Singapore, pp. 175-178.

143. Rao, P.J., Kumar S. and Bindumadhava (1992) Experimental studies on stone columns, *Indian Geotechnical Conference*, pp. 97-107.

144. Rao, N.B.S. and Nayak, G. (1995) Model studies on partially confined sand column using geo-grid tube, *Indian Geotechnical Journal*, Vol. 25, No. 3, pp. 365-378.

145. Saha, A. and Das, S.C. (2000) Interaction analysis of stone column groups in foundations, *Indian Geotechnical Conference*, pp. 279-284.

146. Saha, B. and Ghoshal, A.K. (2005) Thermal degradation kinetics of poly(ethylene terephthalate) from waste soft drinks bottles, *Chemical Engineering Journal*, Vol. 111, pp. 39-43.

147. Saran, S. (2010) *Reinforced soil and its engineering applications*, International Publishing House Pvt. Ltd, India, pp.1.

148. Saroglou, H., Antoniou, A.A. and Pateras, S.K. (2008) Ground improvement of clayey soil formations using stone columns: A case study from Greece, *The 12th International Conference of International Association for Computer Methods and Advances in Geomechanics (IACMAG)*, India.

149. Schlosser, F. and Bastick, M. (1991) *Reinforced Earth, Foundation Engineering Handbook*, 2nd edition, Springer Science + Business media New York, Fang, H (ed), pp. 778-795.

150. Schlosser, F. and Delage, P. (1987) Reinforced Soil Retaining Structures and Polymeric Material, *Advanced Research Workshop on the Application of Polymeric Reinforcement in Soil Retaining Structures*, NATO ASI Series E147, Kingston, Canada, pp. 3-65.

151. Seymour, R.B. (1989) Polymer science before & after 1989: notable developments during the lifetime of Maurtis Dekker, *Journal of Macromolecular Science-Chemistry*, Vol. 26, pp. 1023–1032.

152. Shah, A.A., Hasan, F., Hameed, A. and Ahmed, S. (2008) Biological degradation of plastics: A comprehensive review, *Biotechnology Advances*, Vol. 26, pp. 246-265.

153. Sharma, R.S., Phanikumar, B.R. and Nagendra, G. (2004) Compressive load response of granular piles reinforced with geo-grids, *Canadian Geotechnical Journal*, Vol. 41, No.1, pp. 187-192.

154. Shivashankar, R., Dheerendra Babu, M.R., Nayak, S. and Rajathkumar, V. (2011) Experimental studies on behaviour of stone columns in layered soils, *Geotechnical and Geological Engineering*, Vol. 29, pp. 749-757.

155. Shroff, A.V. and Patel, B.R. (2003) Study on composite stone column in soft kaolinitic clay, *Proceedings of the Indian Geotechnical Conference*, Roorkee, pp. 325-327.

156. Sinha, V., Patel, M.R. and Patel, J.V. (2010) PET waste management by chemical recycling: A review. *Journal of Polymers and the Environment*, Vol. 18, pp. 8–25.

157. Sivakumar, B.G.L., Chouksey, L., Anoosha, G. and Geetha, M.K. (2010) Strength and compressibility response of plastic waste mixed soil, *Proceedings of the Indian Geotechnical Conference*, GEOtrendz, IGS Mumbai Chapter and IIT Bombay, pp. 553-556.

158. Smadi, M. (2016) *Ground improvement methods using column type techniques*. [Online]. Available from: https://docs.lib.purdue.edu/cgi/viewcontent.cgi?referer=https://www.google.co.za/&httpsredir=1&article=3993&context=roadschool [Accessed 27 June 2018].

159. Sobhee, L. (2010) Soil reinforcement using perforated plastic (polyethylene) waste, BSc Thesis, University of Cape Town.

160. Sobhee-Beetul, L. (2012) An investigation into using rammed stone columns for the improvement of a South African silty clay, MSc thesis, University of Cape Town.

161. Sobhee-Beetul, L. and Kalumba, D. (2011) Soil reinforcement using perforated plastic shopping bags, *Proceedings of the Young Geotechnical Engineers Conference*, Kruger National Park, South Africa.

162. Sobhee-Beetul, L. and Kalumba, D. (2012) An investigation into using stone columns in the improvement of marginal sites in South Africa, *International Conference on Ground Improvement and Ground Control*, Indraratna, B., Rujikiatkamjorn, C. and Vinod, J.S., University of Wollongong, Australia.

163. Som, N.N. and Das, S.C. (2006) *Theory and Practice of Foundation Design*, pp. 315, PHI Learning Pvt. Ltd

164. Sondermann, W. and Wehr, W. (2004) Deep vibro techniques, in: Moseley, M.P. and Kirsch, K. (eds.) *Ground improvement*. Oxon: Spon Press.

165. Tallapragada, K.R., Golait, Y.S. and Zade, A.S. (2011) Improvement of bearing capacity of soft soil using stone column with and without encasement of geosynthetics, *International Journal of Science and Advanced Technology*, Vol. 1, No 7, pp. 50-59.

166. Tandel, Y.K., Solanki, C.H. and Desai, A.K. (2014) Field behaviour geotextile reinforced sand column, *Geomechanics and Engineering*, Vol. 6, No. 2.

167. Van Der Westhuizen, V., Parrock, A. (2010) Stone column construction at O.R. Tambo International Airport's Midfield Terminal Development, *SAICE*, Vol. 18 No 6, pp. 45-49.

168. Van Impe, W. (1983) Improvement of settlement behaviour of soft layers by means of stone columns, *8th European Conference on Soil Mechanics. and Foundation Engineering*, Helsinki.

169. Van Impe, W.F., Madhav, M.R., VANDERCRUYSSEN, J.P. (1997) Considerations in stone column design, Ground Improvement Geosystems, in: Davies, M.C.R., Schlosser, F. (eds.) *Ground Improvement Geosystems: Densification and Reinforcement*. London: Thomas Telford Publishing on behalf of the British Geotechnical Society London.

170. Vautrain, J. (1977) Mur en Terre Armee Sur Colonnes Ballastees, *Proceedings of the International Symposium on Soft Clay*, Bangkok, Thailand.

171. VGNL. (2011) Vibroflotation method of ground improvement, *Vibroflotation & Geotechnical (Nig.)* Ltd. [Online]. Available from: http://vibroflotation-ng.com/VGNL%20Profile.pdf [Accessed on 13 April 2011).

172. Vidal, H. (1966) "La Terre Armée", *Annales de L'Institute Technique du Bâtiment et des Travaux Publics*, No. 223-224, pp. 888-938.

173. Watts, K.S., Johnson, D., Wood, L.A., Saadi, A. (2000) Instrumental trial of vibro ground treatment supporting strip foundations in a variable fill. *Geotechnique*, Vol. 50, Issue 6, pp. 699–709.

174. Webb, H.K., Arnott, J., Crawford, R.J. and Ivanova, E.P. (2013) Plastic degradation and its environmental implications with special reference to poly(ethylene terephthalate), *Polymers*, Vol. 5, pp. 1-18.

175. Weber, T. M., Laue, J. and Springman, S. (2006) Centrifuge modelling of sand compaction piles in soft clay under embankment load, *Proceedings of the 6th International Conference on Physical Modelling in Geotechnics*, Kowloon, Hong Kong, pp. 603-606.

176. Williams, C.L., Chang, C.C., Do, P., Nikbin, N., Caratzoulas, S., Vlachos, D.G., Lobo, R.F., Fan, W. and Dauenhauer, P.J. (2012) Cycloaddition of biomass-derived furans for catalytic production of renewable p-xylene, *ACS Catalysis*, Vol. 2, pp. 935–939.

177. Wood, L.A., Johnson, D., Watts, K.S. and Saadi, A. (1996) Performance of strip footings on fill materials reinforced by stone columns, *Structural Engineer*, Vol. 74, No 16, pp. 265-271.

178. Wroth, C.P. and Hughes, J.M.O. (1973) An instrument for the in situ testing of soft soils, *Proceedings of the 8th International Conference on Soil Mechanics And Foundation Engineering*, Moscow, Vol. 1, pp. 487-494.

179. Wu, C. and Hong, Y. (2008) The behaviour of a laminated reinforced granular column, *Geotextiles and Geomembranes*, Vol. 26, Issue 4, pp. 302-316.

180. Yamada-Onodera, K.; Mukumoto, H.; Katsuyaya, Y.; Saiganji, A.; Tani, Y. (2001) Degradation of polyethylene by a fungus, Penicillium simplicissimum YK, *Polymer Degradation and Stability*, Vol. 72, pp. 323–327.

181. Zahmatkesh, A. and Choobbasti, A.J. (2010) Investigation of bearing capacity and settlement of strip footing on clay reinforced with stone columns, *Australian Journal of Basic and Applied Sciences*, Vol. 4, No. 8, pp. 3658-3668.

182. Zhang, J., Wang, X., Gong, J. and Gu, Z. (2004) A study on the biodegradability of polyethylene terephthalate fiber and diethylene glycol terephthalate, *Journal of Applied Polymer Science*, Vol. 93, pp. 1089–1096.

183. Zheng, Y.; Yanful, E.K.; Bassi, A.S. (2005) A review of plastic waste biodegradation, *Critical Reviews in Biotechnology*, Vol. 25, pp. 243–250.

184. Zornberg, J.G., Cabral, A.R. and Viratjandr., C. (2004) Behaviour of tire shred – sand mixtures, *Canadian Geotechnical Journal*, Vol. 41, pp. 227-241.

185. Zukri, A. and Nazir, R. (2018) Sustainable materials used as stone column filler: A short review, *IOP Conference Series: Materials Science and Engineering*, 342 012001.

Appendix

Appendix A: Characterisation Tests

Appendix B: Geotextiles properties

Appendix C: Reinforcements used in the tests

Appendix D: Additional pictures for the process of physically modelling the deformation of the columns

Appendix A: Characterisation Tests

A.1: Particle size distribution

Table A.1: Particle size distribution data for Durbanville silt

Test	Aperture size (mm)	Percentage passing (%)
Wet sieve Analysis	4.750	100.0
	2.000	94.9
	1.180	91.9
	0.600	89.0
	0.425	87.5
	0.300	86.4
	0.150	83.4
	0.075	79.4
Hydrometer	0.0405	63.7
	0.0260	61.2
	0.0186	53.7
	0.0179	53.1
	0.0157	51.2
	0.0115	46.9
	0.0082	41.2
	0.0061	36.9
	0.0032	26.2
	0.0025	21.9
	0.0014	15.6

Table A.2: Particle size distribution data for Cape Flats sand

Sieve opening (mm)	Mass retained (g)	Mass retained (%)	Cumulative retained (%)	Cumulative % passing (%)
4.75	0.0	0.0	0.0	100.0
2.36	0.0	0.0	0.0	100.0
1.18	13.8	1.1	1.1	98.9
0.6	583.4	46.2	47.3	52.7
0.3	434.3	34.4	81.6	18.4
0.15	210.4	16.6	98.3	1.7
0.075	21.0	1.7	99.9	0.1
PAN	1.0	0.1		
Total	**1263.9**			

Table A.3: Particle size distribution data for PET flakes

Sieve opening (mm)	Mass retained (g)	Mass retained (%)	Cumulative retained (%)	Cumulative % passing (%)
9.5	0.1	0.1	0.1	99.9
6.7	2.9	1.7	1.7	98.3
4.75	46.1	26.4	28.1	71.9
2.36	110.5	63.2	91.3	8.7
1.18	14.0	8.0	99.3	0.7
0.6	1.3	0.7	100.1	-0.1
0.3	0.0	0.0	100.0	0.0
PAN	0.0	0.0		
Total	**174.8**			

A.2: Specific gravity determination

Table A.4: Specific gravity data for Durbanville silt

Durbanville silt				
Pyknometer no		1	2	3
Mass of bottle + soil + water (g)	m3	91.817	88.119	89.471
Mass of bottle + soil (g)	m2	41.887	40.425	42.508
Mass of bottle full of water (g)	m4	87.46	84.463	84.572
Mass of bottle (g)	m1	34.995	34.622	34.729
Mass of soil (g)	m2 - m1	6.892	5.803	7.779
Mass of water in full bottle (g)	m4 - m1	52.465	49.841	49.843
Mass of water used (g)	m3 - m2	49.93	47.694	46.963
Volume of soil particles	(m4 - m1)- (m3 - m2)	2.535	2.147	2.88
Particle density (Mg/m^3)	(m2-m1)/{ (m4 -m1) - (m3 -m2)}	2.72	2.70	2.70
Average Value (Mg/m^3)		2.71		

Table A.5: Specific gravity data for Cape Flats sand

Cape Flats sand				
Pyknometer no		1	2	3
Mass of bottle + soil + water (g)	m3	93.985	93.928	94.423
Mass of bottle + soil (g)	m2	49.516	49.959	50.018
Mass of bottle full of water (g)	m4	84.916	84.154	84.725
Mass of bottle (g)	m1	35.049	34.362	34.718
Mass of soil (g)	m2 - m1	14.467	15.597	15.3
Mass of water in full bottle (g)	m4 - m1	49.867	49.792	50.007
Mass of water used (g)	m3 - m2	44.469	43.969	44.405
Volume of soil particles	(m4 - m1)- (m3 - m2)	5.398	5.823	5.602
Particle density (Mg/m^3)	(m2-m1)/{ (m4 -m1) - (m3 -m2)}	2.68	2.68	2.73
Average Value (Mg/m^3)		2.70		

A.3: Natural moisture content

Table A.6: Natural moisture content of Durbanville silt

Durbanville silt			
Sample No	**1**	**2**	**3**
Mass of tin (g)	9.6	9.7	9.6
Mass of tin + wet soil (g)	29	24.6	21
Mass of tin + dry soil (g)	26.2	22.4	19.4
Mass of water (g)	2.8	2.2	1.6
Mass of dry soil (g)	16.6	12.7	9.8
Moisture content (%)	16.9	17.3	16.3
Average moisture content (%)		16.8	

Table A.7: Natural moisture content of Cape Flats sand

Cape Flats sand			
Sample No	**1**	**2**	**3**
Mass of tin (g)	9.6	9.8	9.5
Mass of tin + wet soil (g)	33.4	29	35.9
Mass of tin + dry soil (g)	33.3	29	35.9
Mass of water (g)	0.1	0	0
Mass of dry soil (g)	23.7	19.2	26.4
Moisture content (%)	0.4	0.0	0.0
Average moisture content (%)		0.1	

A.4: Atterberg limit tests

Table A.8: Plastic limit data for Durbanville silt

Plastic Limit of Durbanville silt			
Sample No	**1**	**2**	**3**
Mass of tin (g)	8.195	8.062	8.119
Mass of tin + wet soil (g)	11.017	11.32	10.92
Mass of tin + dry soil (g)	10.331	10.576	10.27
Mass of water (g)	0.686	0.744	0.65
Mass of dry soil (g)	2.136	2.514	2.151
Moisture content (%)	32.1	29.6	30.2
Average moisture content or Plastic Limit (%)		30.6	

Table A.9: Liquid limit data for Durbanville silt

Liquid Limit of Durbanville silt			
No of drops	34	24	18
Sample No	**1**	**2**	**3**
Mass of tin (g)	8.218	8.19	8.2
Mass of tin + wet soil (g)	13.768	13.207	13.179
Mass of tin + dry soil (g)	12.193	11.816	11.897
Mass of water (g)	1.575	1.391	1.282
Mass of dry soil (g)	3.975	3.626	3.697
Moisture content (%)	39.6	38.4	34.7

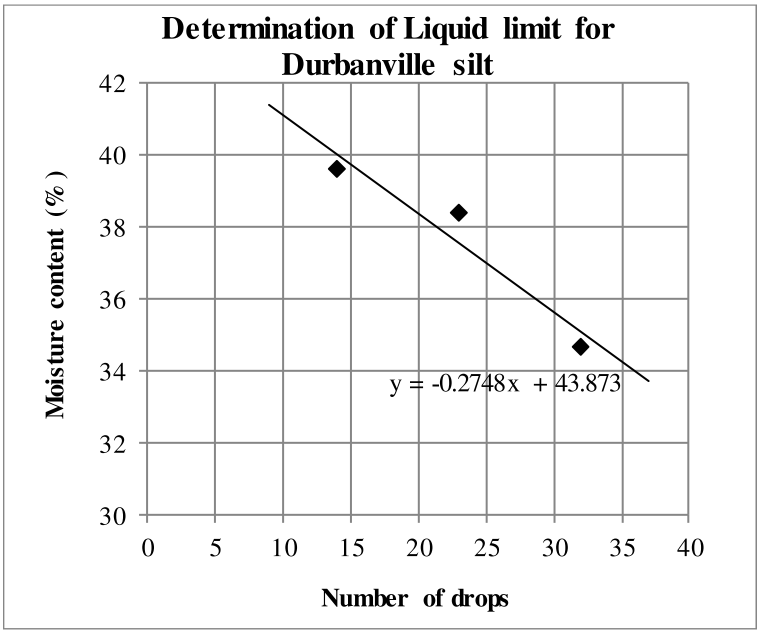


Figure A.1: Liquid limit determination of Durbanville silt

A.5: Determination of the optimum moisture content (OMC) and maximum dry density (MDD) from the Proctor test

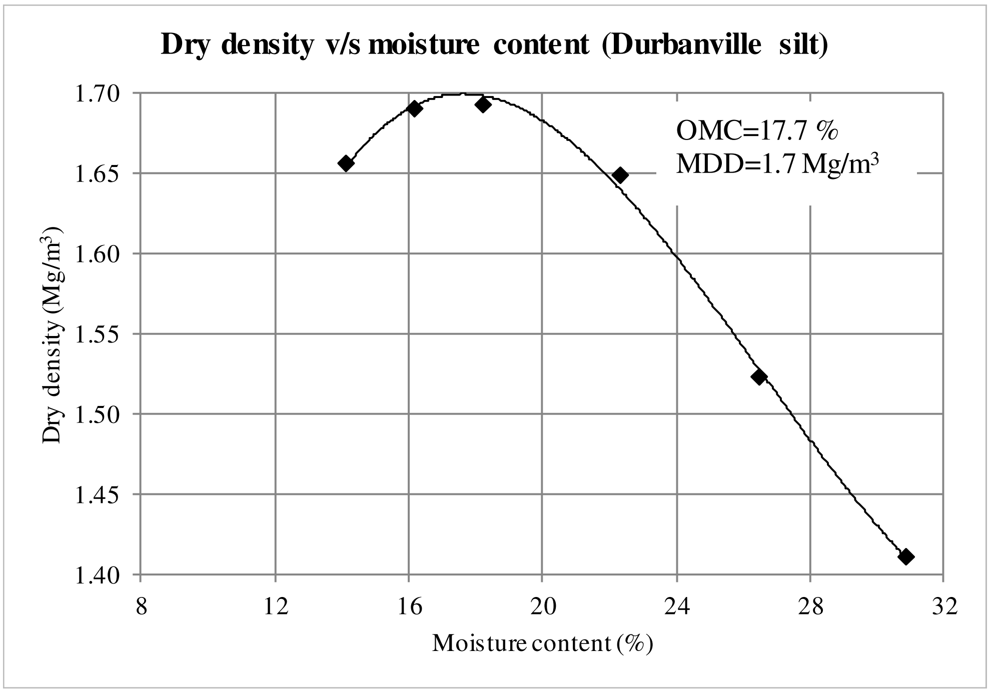


Figure A.2: Dry density and moisture content relationship for Durbanville silt

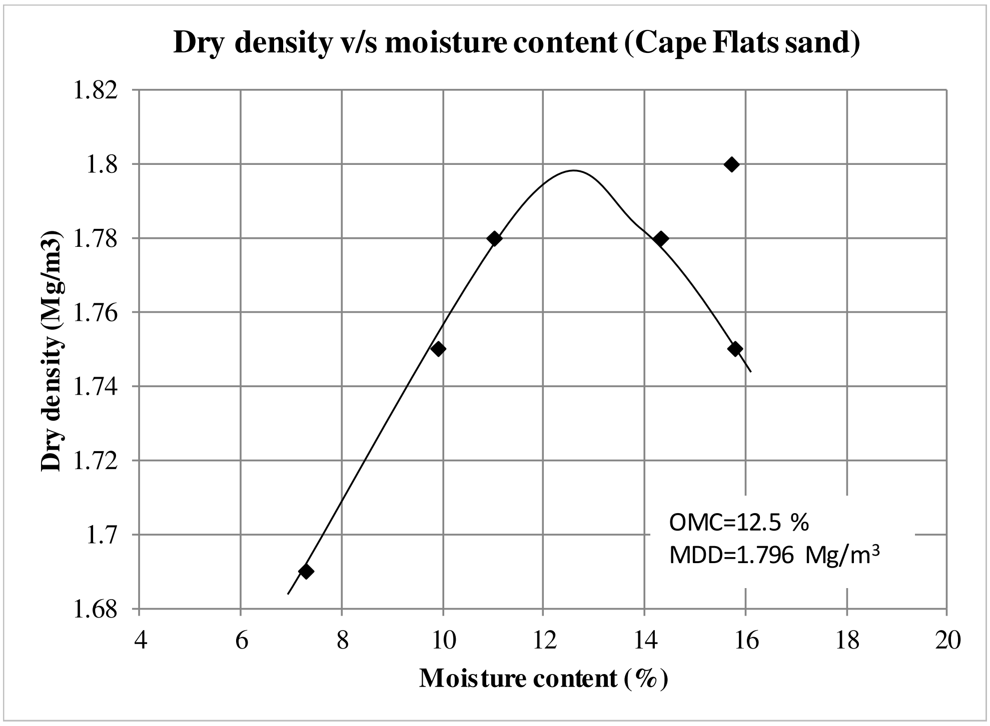


Figure A.3: Dry density and moisture content relationship for Cape Flats sand

A.6: Determination of the shear strength parameters

Table A.10: Shear strength parameters (obtained from triaxial tests) of the Durbanville silt at optimum moisture content (OMC) and liquid limit (LL)

	OMC			LL		
Vertical effective consolidation stress (kPa)	50.17	99.65	199.6	49.2	100.3	199.6
Deviator stress at failure (kPa)	41.2	89.72	149.9	10.56	6.316	2.393
Normal stress (kPa)	91.37	189.37	349.5	59.76	106.616	201.993
Friction angle	15.2			-1.5		
Cohesion (kPa)	3.98			6.42		

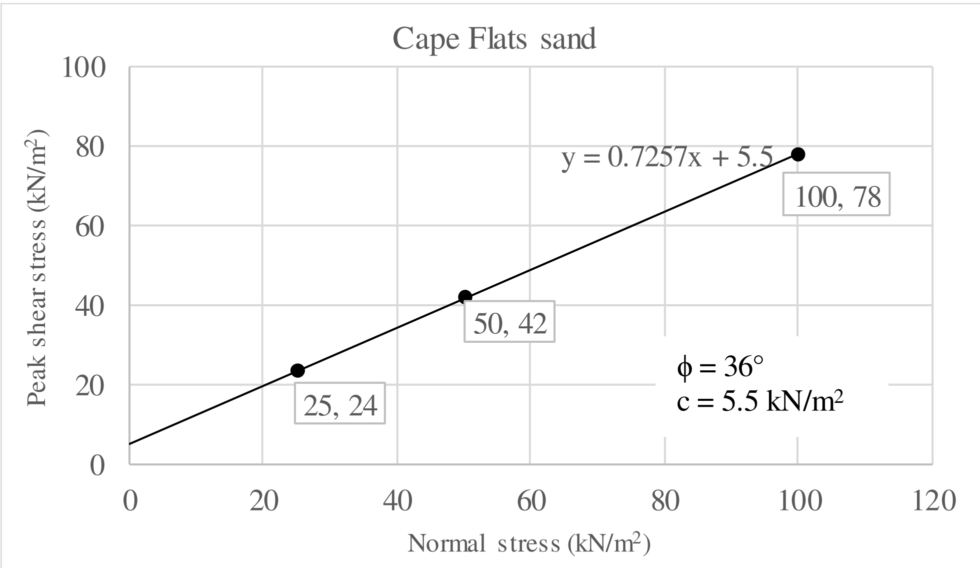


Figure A.4: Shear strength parameters of the Cape Flats sand (obtained from direct shear tests)

Appendix B: Geotextiles properties

B.1: Betatex specifications

Betatex 200M sa

Fibertex Geotextiles Africa
GEOSYNTHETIC PRODUCTS

Betatex geotextiles are used in building and construction works for separation, filtration, drainage and protection. Betatex geotextiles are manufactured from recycled polyester fibres with added UV stabiliser. The basic strength of the Betatex geotextiles is obtained by needle punching the polyester fibres, which provides the elastic bonding. Betatex is best used where no specification is stated and for domestic DIY applications.

SPECIFICATIONS

Doc No.: FGA 6095 (SANS) Rev 00 – 03.2016
F_11_7015 REV: 00 Date: 16.03.2016

Physical Properties				
Thickness	At 2 kPa	mm	1.1	SANS 9863:2013
Mass		g/m^2	200	SANS 9864:2013
Mechanical Properties				
Static Puncture Strength	CBR Test	N	1 500	SANS 12236:2013
Elongation at break		%	>45	SANS 12236:2013
Tensile Strength	MD/CMD	kN/m	9.5/9.5	SANS 1525:2013
Elongation at Break		%	>40	SANS 1525:2013
Dynamic Cone Drop		mm	28	SANS 13433:2013
Hydraulic Properties				
Water Flow	50mm Water Head	$l/s/m^2$	50	SANS 11058:2013
Permeability	50mm Water Head	m/s	0.05	SANS 11058:2013
Permittivity	50mm Water Head	sec^{-1}	1.00	SANS 11058:2013
Pore Size	$0_{90\%}$	micron	70	SANS 12956:2013
Roll Dimensions				
Widths	Standard	m	1.3, 1.7, 2.6, 5.2	
Length		m	150	
Roll Diameter	Approx.	Cm	41	

*An "**M**" in the Fibertex product code indicates it is needle punched only, and has not undergone thermal treatment.*

Fibertex geotextiles are manufactured to ISO 9001:2008 quality management procedures.
The above values represent Mean values based on current production test results.

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Figure B.1: Specification sheet for the Betatex with mass per unit area of 200 g/m^2

Betatex 400M sa

Fibertex Geotextiles Africa
GEOSYNTHETIC PRODUCTS

Betatex geotextiles are used in building and construction works for separation, filtration, drainage and protection. Betatex geotextiles are manufactured from recycled polyester fibres with added UV stabiliser. The basic strength of the Betatex geotextiles is obtained by needle punching the polyester fibres, which provides the elastic bonding. Betatex is best used where no specification is stated and for domestic DIY applications.

SPECIFICATIONS

Doc No.: FGA 6114 (SANS) Rev 00 – 03.2016
F_11_7015 REV: 00 Date: 16.03.2016

Physical Properties				
Thickness	At 2 kPa	mm	3.0	SANS 9863:2013
Mass		g/m^2	400	SANS 9864:2013
Mechanical Properties				
Static Puncture Strength	CBR Test	N	3 000	SANS 12236:2013
Elongation at break		%	>45	SANS 12236:2013
Tensile Strength	MD/CMD	kN/m	20.0/20.0	SANS 1525:2013
Elongation at Break		%	>40	SANS 1525:2013
Dynamic Cone Drop		mm	17	SANS 13433:2013
Hydraulic Properties				
Water Flow	50mm Water Head	$l/s/m^2$	34	SANS 11058:2013
Permeability	50mm Water Head	m/s	0.03	SANS 11058:2013
Permittivity	50mm Water Head	sec^{-1}	0.68	SANS 11058:2013
Pore Size	$O_{90\%}$	micron	70	SANS 12956:2013
Roll Dimensions				
Widths	Maximum	m	5.2	
Length		m	100	
Roll Diameter	Approx.	cm	42	

*An "**M**" in the Fibertex product code indicates it is needle punched only, and has not undergone thermal treatment.*

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Figure B.2: Specification sheet for the Betatex with mass per unit area of 400 g/m^2

Betatex 600M sa

Fibertex
Geotextiles Africa
GEOSYNTHETIC PRODUCTS

Betatex geotextiles are used in building and construction works for separation, filtration, drainage and protection. Betatex geotextiles are manufactured from recycled polyester fibres with added UV stabiliser. The basic strength of the Betatex geotextiles is obtained by needle punching the polyester fibres, which provides the elastic bonding. Betatex is best used where no specification is stated and for domestic DIY applications.

SPECIFICATIONS

Doc No.: FGA 6129 (SANS) Rev 00 – 03.2016
F_11_7015 REV: 00 Date: 16.03.2016

Physical Properties				
Thickness	At 2 kPa	mm	3.0	SANS 9863:2013
Mass		g/m^2	600	SANS 9864:2013
Mechanical Properties				
Static Puncture Strength	CBR Test	N	4 500	SANS 12236:2013
Elongation at break		%	>45	SANS 12236:2013
Tensile Strength	MD/CMD	kN/m	27.5/27.5	SANS 1525:2013
Elongation at Break		%	>40	SANS 1525:2013
Dynamic Cone Drop		mm	4	SANS 13433:2013
Hydraulic Properties				
Water Flow	50mm Water Head	$l/s/m^2$	32	SANS 11058:2013
Permeability	50mm Water Head	m/s	0.03	SANS 11058:2013
Permittivity	50mm Water Head	sec^{-1}	0.64	SANS 11058:2013
Pore Size	$O_{90\%}$	micron	70	SANS 12956:2013
Roll Dimensions				
Widths	Maximum	m	5.2	
Length		m	50	
Roll Diameter	Approx.	cm	43	

*An "**M**" in the Fibertex product code indicates it is needle punched only, and has not undergone thermal treatment.*

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Figure B.3: Specification sheet for the Betatex with mass per unit area of 600 g/m^2

B.2: Fibertex specifications

FIBERTEX F34 SA

Fibertex SOUTH AFRICA

Fibertex geotextiles are used in building and construction works for separation, filtration, drainage, protection, stabilisation and reinforcement. Fibertex geotextiles are manufactured from virgin polypropylene fibres with added UV stabiliser. The basic strength of the Fibertex geotextiles is obtained by needle punching the polypropylene fibres, which provides strong elastic bonding. Fibertex is highly durable and resistant to all natural occurring soil alkalis and acids.

SPECIFICATIONS

Physical Properties			
Unit Weight (Mass)	EN ISO 9864	g/m^2	200
Thickness at 2 kPa	EN ISO 9863	mm	1.0
UV Stability	ASTM D7238	%	70
	EN 12224	To be covered within 1 month after installation.	
Life Expectancy	Predicted to be durable for a service life of 100 years in natural soils with $4 \leq pH \leq 9$ and soil temperatures ≤ 25°C on the basis of the results of test method EN ISO 13438 procedure A		
Mechanical Properties			
Static Puncture Strength (CBR Test)	EN ISO 12236	N	2600
Tensile Strength (MD/CMD)	EN ISO 10319	kN/m	15.0/15.0
Elongation at Break		%	40 – 65
Dynamic Cone Drop	EN ISO 13433	mm	17
Hydraulic Properties			
Flow Rate (Δh = 50 mm)	EN ISO 11058	$l/m^2/s$	54
Permeability (Δh = 50 mm)		m/s	0.05
Pore Size, O_{90W}	EN ISO 12956	µm	90
Roll Dimensions			
Widths	-	m	6.0
Length	-	m	150

Fibertex geotextiles are manufactured to ISO 9001:2008 quality management procedures.

The above values represent typical values based on current production test results.

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** GEOTEXTILES – GEOGRIDS – SUBSOIL DRAINAGE PIPE & FITTINGS – GEOCELLS – COMPOSITE DRAINAGE SYTEMS – GABIONS & MATTRESSES – CUSPATED SHEETS – GEOBAGS – GCLs – GEOMEMBRANE

Figure B.4: Specification sheet for the Fibertex with mass per unit area of 200 g/m^2

FIBERTEX F400M SA

Fibertex SOUTH AFRICA

Fibertex geotextiles are used in building and construction works for separation, filtration, drainage, protection, stabilisation and reinforcement. Fibertex geotextiles are manufactured from virgin polypropylene fibres with added UV stabiliser. The basic strength of the Fibertex geotextiles is obtained by needle punching the polypropylene fibres, which provides strong elastic bonding. Fibertex is highly durable and resistant to all natural occurring soil alkalis and acids.

SPECIFICATIONS

Physical Properties			
Unit Weight (Mass)	EN ISO 9864	g/m^2	400
Thickness at 2 kPa	EN ISO 9863	mm	3.4
UV Stability	ASTM D7238	%	70
	EN 12224	To be covered within 1 month after installation.	
Life Expectancy	Predicted to be durable for a service life of 100 years in natural soils with $4 \leq pH \leq 9$ and soil temperatures $\leq 25°C$ on the basis of the results of test method EN ISO 13438 procedure A		
Mechanical Properties			
Static Puncture Strength (CBR Test)	EN ISO 12236	N	4500
Tensile Strength (MD/CMD)	EN ISO 10319	kN/m	25.0/28.0
Elongation at Break		%	> 50
Dynamic Cone Drop	EN ISO 13433	mm	8
Hydraulic Properties			
Flow Rate (Δh = 50 mm)	EN ISO 11058	$l/m^2/s$	42
Permeability (Δh = 50 mm)		m/s	0.04
Pore Size, O_{90W}	EN ISO 12956	µm	70
Roll Dimensions			
Widths	-	m	6.0
Length	-	m	100

Fibertex geotextiles are manufactured to ISO 9001:2008 quality management procedures.

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Figure B.5: Specification sheet for the Fibertex with mass per unit area of 400 g/m^2

FIBERTEX F600M SA

Fibertex SOUTH AFRICA

Fibertex geotextiles are used in building and construction works for separation, filtration, drainage, protection, stabilisation and reinforcement. Fibertex geotextiles are manufactured from virgin polypropylene fibres with added UV stabiliser. The basic strength of the Fibertex geotextiles is obtained by needle punching the polypropylene fibres, which provides strong elastic bonding. Fibertex is highly durable and resistant to all natural occurring soil alkalis and acids.

SPECIFICATIONS

Physical Properties			
Unit Weight (Mass)	EN ISO 9864	g/m^2	600
Thickness at 2 kPa	EN ISO 9863	mm	4.7
UV Stability	ASTM D7238	%	70
	EN 12224	To be covered within 1 month after installation.	
Life Expectancy	Predicted to be durable for a service life of 100 years in natural soils with $4 \leq pH \leq 9$ and soil temperatures ≤ 25°C on the basis of the results of test method EN ISO 13438 procedure A		
Mechanical Properties			
Static Puncture Strength (CBR Test)	EN ISO 12236	N	7300
Tensile Strength (MD/CMD)	EN ISO 10319	kN/m	45.0/45.0
Elongation at Break		%	> 50
Dynamic Cone Drop	EN ISO 13433	mm	≤ 1
Hydraulic Properties			
Flow Rate (Δh = 50 mm)	EN ISO 11058	$l/m^2/s$	27
Permeability (Δh = 50 mm)		m/s	0.02
Pore Size, O_{90W}	EN ISO 12956	µm	70
Roll Dimensions			
Widths	-	m	6.0
Length	-	m	50

Fibertex geotextiles are manufactured to ISO 9001:2008 quality management procedures.

The above values represent typical values based on current production test results.

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Figure B.6: Specification sheet for the Fibertex with mass per unit area of 600 g/m^2

Appendix C: Reinforcements used in the tests

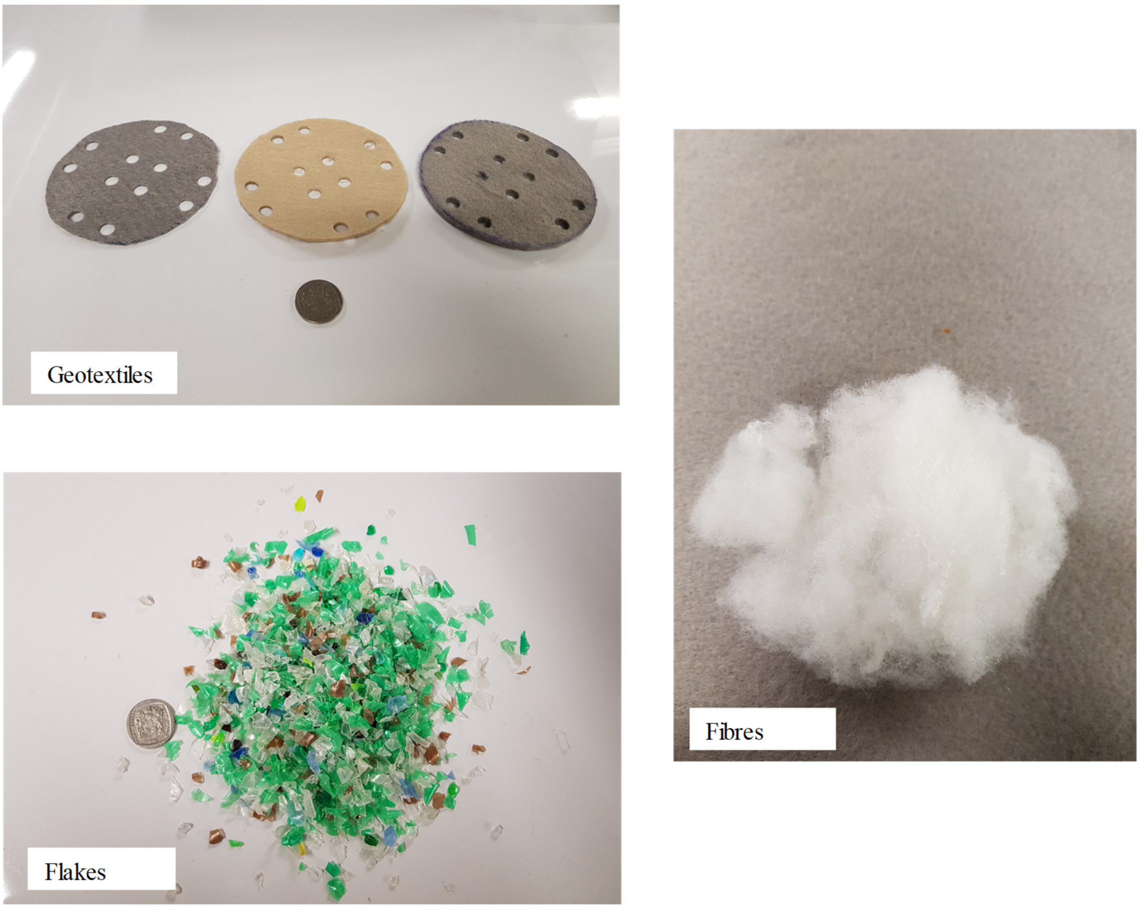

Figure C.1: The different types of reinforcements in the forms that they were used in the tests (not to scale)

Appendix D: Additional pictures for the process of physically modelling the deformation of the columns

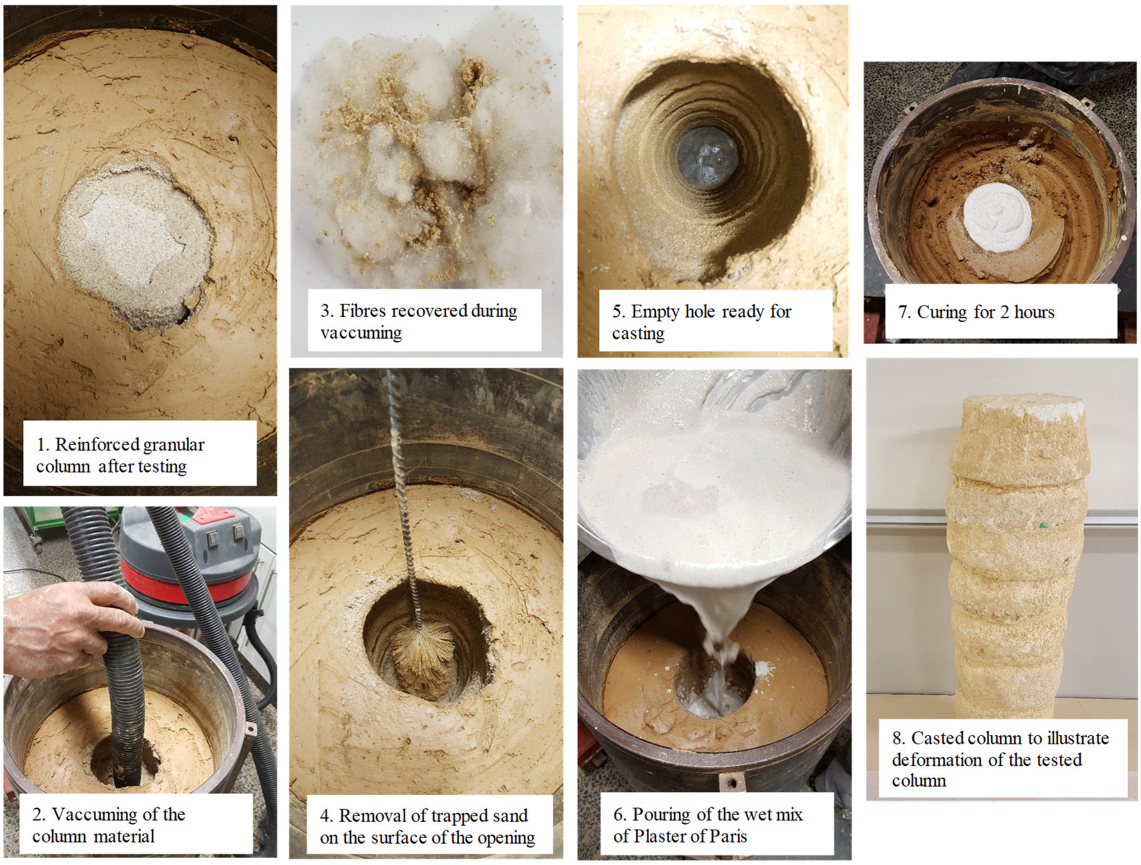

Figure D.1: Typical stages involved in the physical modelling of the deformation of a column after an experiment