STUDY OF THE STRENGTH OF SOFT CLAY IN REINFORCED WITH GROUP ENCAPSULATED BOTTOM ASH COLUMNS

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SUPERVISOR'S DECLARATION

"I hereby declare that I have read this final year project report and in my opinion this final year project is sufficient in terms of scope and quality for award of the degree of Bachelor of Civil Engineering"

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STUDENT'S DECLARATION

"I hereby declare that the work in this thesis in my own except for quotation and summaries which have been duly acknowledged. The thesis has not been accepted for any degree and it is not concurrently submitted for award of other degree"

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To my beloved mother, father, along, angah, kak Nana, kak Faz and my lovely Qisha, Aufa and Aira

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ABSTRAK

Peningkatan permintaan ke atas tanah yang sesuai untuk pembinaan pada masa kini membawa industri pembinaan untuk mengeksploitasi kawasan yang sebelum ini dianggap sebagai tapak yang tidak ekonomi dan tidak sesuai untuk pembagunan seperti tanah liat lembut. Dengan kemajuan dan kecanggihan teknologi yang membuktikan bahawa terdapat alternatif untuk menggunakan tanah liat lembut dalam pembinaan yang dikenali sebagai pembaikan tanah atau teknik pengubahsuaian. Kaedah pembaikan tanah meningkatkan kekuatan ricih, mengurangkan kebolehtelapan dan kebolehmampatan tanah lembut. Kaedah ruangan batu adalah yang paling digemari dan digunakan secara meluas dalam pembinaan. Kajian ini adalah menggunakan bahan-bahan buangan yang dikenali sebagai abu bawah sebagai gantian kepada penggunaan bahan-bahan semula jadi dalam ruangan batu itu. Oleh itu, dengan mengguna semula sisa abu bawah, kos pembinaan boleh dikurangkan dan dengan itu, kawasan pelupusan bahan buangan juga berkurangan. Kajian ini bertujuan untuk menentukan kekuatan tanah liat lembut di diperkuatkan dengan kumpulan kapsul abu bawah. Abu bawah dalam kapsul dengan geotekstil digunakan sebagai tiang batu dalam tanah liat kaolin. Pada bahagian pertama kajian ini, sifat-sifat fizikal dan mekanikal tanah liat kaolin dan abu bawah ditentukan. Terbukti bahawa tanah liat kaolin boleh diklasifikasikan sebagai tanah berkelodak sementara, abu bahagian bawah mempunyai ciri relatif dengan bahan-bahan berbutiran kasar. Pada bahagian kedua kajian, sampel tanah liat lembut di diperkukuhkan dengan kumpulan tiga terkapsul tiang abu bawah diuji di bawah ujian tidak tepu dengan perbezaan pelbagai diameter dan ketinggian tiang abu bawah. Dapat disimpulkan bahawa kehadiran kapsul ruangan abu bawah, semakin bertambah kekuatan parameter ricih tanah liat lembut.

ABSTRACT

The increasing of the demand and restrictions on the suitable land for construction in recent time led the construction industries to exploit sites that were previously considered as the uneconomical site to develop such as soft clay soil. Luckily, the advance of the technology prove that there is an alternative to using soft clay in construction by ground improvement or modification technique. The ground improvement method are increasing the shear strength, reduces the permeability and the compressibility of the soft soil. The stone column method is the most preferable and widely used in construction. This study are used the waste materials known as bottom ash as a replacement to the usage of natural materials in the stone column. Hence, by reutilize the waste of bottom ash, the construction cost can be reduce and thus, the disposal area also decreasing. This research is to determine the strength of the soft clay in reinforced with group encapsulated bottom ash columns. The bottom ash in encapsulated with geotextile are used as the stone column in kaolin clay. At the first part of the study, the physical and mechanical properties of the kaolin clay and bottom ash are determine. There is proven that the kaolin clay can be classified as silty soils while, the bottom ash have the relative characteristic with granular materials. At the second part of study, the sample of soft clay in reinforced with group of three encapsulated bottom ash columns are tested under Unconsolidated Undrained Test with difference various of diameter and height of bottom ash column. It can be concluded that the presence of encapsulated bottom ash column, are increasing the shear strength parameter of the soft clay.

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LIST OF SYMBOLS

A_c	-	Area of Bottom Ash Column
A_s	-	Area of Sample
C_c	-	Coefficient of Curvature
C_u	-	Coefficient Uniformity
D_c	-	Diameter of Bottom Ash Column
H_c	-	Height of Bottom Ash Column
H_s	-	Height of Sample
C	-	Cohesion
G_s	-	Specific Gravity
Kn	-	Kilo Newton
kPa	-	Kilo Pascal
Mg	-	Mega Gram
MN	-	Mega Newton
m/s	-	Metre per Second
Mm	-	Milimetre
μm	-	Micrometer
q_{max}	-	Maximum deviator stress
s_u	-	Undrained Shear Strength
W	-	Moisture Content
w_{opt}	-	Optimum Moisture Content
ρ_d	-	Dry Density
$\rho_{d(max)}$	-	Maximum Dry Density
Φ	-	Friction Angle

LIST OF ABBREVIATIONS

ACAA	American Coal Ash Association
ASSHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
BA	Bottom Ash
BS	British Standard
LL	Liquid Limit
PI	Plasticity Index
PL	Plastic Limit
SL	Shrinkage Limit
US	United States
USCS	Unified Soil Classification System
UU	Unconsolidated Undrained

CHAPTER 1

INTRODUCTION

1.1 Background

Soil is a one of the very importance component in human life because of it properties and characteristic that multi usage and natural materials that can be found everywhere around our surrounding. There are so many type of soil that can be found but basically the soil are divided into three type; clay soil, sandy soil and silt soil. Usually, in development area, clay soil are widely used compare to sandy soil and silt soil, because of their characteristic. Discovery of history proven that since the prehistoric era, the clay soil are used as the medium to support in the construction of building, houses, residential, walls and many more (Sa' adon, 2009). Since then, human start to study and develop more about the clay soil besides the usage in development are increasing gradually.

Nowadays, the numbers of the community are rapidly increasing and Malaysia is one of the country that experiencing the population growth. In February 2014, Department of Statistic Malaysia is stated that the population in Malaysia was reach 30 million people and this number are still increasing horary. The increasing of the population are required the increasing of the construction industry because of every people need the dwelling place. To fulfill the human needed, developer in construction industries need to explore and find the solution in the usage of the soil to develop.

Malaysia is one of the luckiest country in the world that richest with the difference type of soil and have the multipurpose usage. Unfortunately, developments throughout the industrialized sectors are cause for high usage of the suitable site. The uncontrollable use of the site for construction was led to exploit to the other type of soil. The increasing of the demand and restrictions on the suitable land for construction in recent time led the construction industries to exploit sites that were previously considered as the uneconomical site to develop (Eied *et al.,* 2014).

Clay are widely used in construction and development and clay are divided into two type; hard clay and soft clay. Usually, the clay that used in construction is hard clay type but presently, the researcher are started to find the solution to use the soft clay in construction. Soft clay is known as the unsuitable and uneconomical type of soil to be used in construction because Eied *et al.,* (2014) started that soft clay have low of shear strength and high compressibility that will cause the troubles during and after construction. By using the piling, building or structure can be construct, but the cost of the construction is higher and it can led to the uneconomical project.

Through the characteristic of the soft clay, any construction works that will constructed are believed to face more problem compared to other type of soils (Sa'adon, 2009). Luckily, the advance of the technology prove that there is an alternative to using soft clay in construction by ground improvement or modification technique. Other than more economical method, ground improvement is the technique that suit the construction requirement which change and improve the properties of the soil. The properties of ground improvement are; increases the shear strength, reduces the permeability and reduces the compressibility (How, 2011). There technique of ground improvement can be done either by; mechanical compaction, dynamic compaction, deep vibratory, stone column, preloading, soil stabilization by use of admixture, use of geotextiles and many more.

Although the ground improvement technique improve the properties of the soil, but not all the technique are economical, suitable and preferable. Out of several technique, stone column is the one of the ground improvement method that most preferable, economical and widely used in construction. Many researcher have developed theoretical solutions for estimating the bearing capacity and settlement of foundation reinforced with stone column (Priebe, 1995). The main advantage of the stone column lies in improving the soil properties below a structure (raft and depth) and following the reduction of an irregular settlement (Pivarc, 2011). Stone column is the method which consist of the granular materials such as crushed rock or gravel is replaced into the soft soil at regular intervals throughout the area of the land where the soil bearing capacity is to be improved.

Usually, the granular materials which used for stone column are crushed rock and sand, but important in view that the fact of the sources of the natural materials are getting depleted gradually. An alternative are needed to prevent uncontrollable usage of natural material and the possibility of the extinction of natural materials. One of the alternative that preferable to use are the recycle materials. We are known and very familiar with the recycle of paper, plastic, aluminum and glasses, but there are so many other type of materials that can be recycle. One of the example is the recycle of coal.

Coal are known as a largest source of the energy for the generation of electricity and throughout history, coal has been used as an energy resource, primarily burned for the production of electricity and heat, and is also used for industrial purposes, such as refining metal. One of the famous coal power plant in Malaysia is Tanjung Bin Power Plant, Johor. The coal was produced the combustion waste, especially ash and due to the increasing of the demand of the coal gradually, the waste are cause the harmful to the environment and led to the increasing the number of disposal area. The ash is the combustion waste that produced from the process of biomass combustion can be divided into two; bottom ash and fly ash (Carrasco *et al.,* 2012) and normally, fly ash are widely used as a cement replacement, whereas bottom ash have a minor usage and usually it will be disposed.

Carrasco *et al.* (2012) stated that bottom ash the waste materials that produced on the grate in the first combustion chamber of the boiler and the portion of bottom ash is often mixed with other materials such as sand and stone. Bottom ash have a similar characteristic with the granular materials, therefore it is appropriate if it is used to replace the utilization of natural materials in stone column. Ordinarily, bottom ash will be disposed at the disposal area, recently the waste of bottom ash are increasing day by day and it also increasing the number of disposal area. If bottom ash are widely used in the stone column, other than help to reduce the disposal area, it also one of the usage economical friendly materials.

This study is presents the determination of the strength parameters of soft clay in reinforced with group encapsulated bottom ash columns. The location of the construction site in the soft clay area is not a favorite choice to the engineer, because the weakness of the properties of the soil. Luckily, there is solution to solve this kind of problem, and the stone column are one of the best alternative used. The recycle of the usage of the waste material are help in save the environment and reduce the disposal area. Bottom ash are used in stone column to improve the properties of soft clay other than application of the recycle materials.

1.2 Problem Statement

As an engineer, to construct any building or any structure, the first thing first that need to consider is the base for the structure that it will stand firm. Hence, before any construction work are begin, the site investigation to study and analysis of the type and properties of the soil will be done first. It is important to analysis because soil have a difference type and properties that can cause a huge failure such as collapse, settlement and others if the making of decision are wrong in a beginning stage of construction works. Specifically, soft clay soil is believed to experience more problematic and failure during and after the construction due to the weaknesses of its properties.

Soft clay are known as a problematic soil type in engineering because of the low shear strength and high compressibility that failure to support the huge loads from the structure. Sa'adon (2009) have stated that the most emergence problem happened in structure of the building is the foundation settlement, in addition many commercial and residential building have distressed due to the settlement. In facts, the ground improvement is the best alternatives to the weaknesses properties problem of soft clay that it can improve and change the properties of the soft clay.

Certainly, from the various method of ground improvement, stone column is more preferable and economical alternatives other than it is widely used for soft clay. Unfortunately, the facts that the natural materials such as sand and rocks that have been used in stone column are getting depleted gradually. The best alternatives that can be applied is by using the recycle materials from the waste. Besides reused the waste materials, recycle the material that not contain any of chemical can also help in save the environment.

Presently, the environment issues is getting serious because of the increasing of the waste materials and limitation of the disposal area. For instance, the waste from the coal ash are getting increase, besides there is no sign that the utilization of the coal will be reduced. The two types of coal ash are fly ash and bottom ash. In addition, this type of waste cannot be thrown all over the place, but it have to be thrown in the land field. The increasing number of the waste of coal ash will led to the increasing of the number of disposal area. Recently, the fly ash are widely used as a replacement for the cement but there are not much usage for the bottom ash.

Alternatives way to improve the soft clay soil properties is by using the stone column ground improvement method. Besides use the natural materials, reused the waste material is more preferable because other than reduce the utilization of natural material, it also help to save the environment.

1.3 Objectives

The purpose of this project is to study the strength of soft clay in reinforced with group encapsulated bottom ash columns.

The objectives of this study are:

i. To determine physical and mechanical properties of soft clay and bottom ash.
ii. To determine the strength parameter of soft clay reinforced with group encapsulated of bottom ash columns.

1.4 Scope of Study

The main purpose of this study is to investigate the strength of soft clay in reinforced with group encapsulated bottom ash columns. All the experiment and testing of the sample are run and analyzed in the Soil Mechanic and Geotechnical Laboratory in Faculty of Civil Engineering and Earth Resources (FKASA), University Malaysia Pahang (UMP). This study are discuss the result of the improvement of the strength parameters of the soft clay.

A batches of kaolin grade S300 as a samples of soft clay are prepared in laboratory to study its compressibility parameter for reinforced with grouping encapsulated bottom ash. Each batch consisted of samples with partially penetrating bottom ash column and the sample with fully penetrating for grouping of three bottom ash column. The diameter for kaolin specimen is 36 mm whereas the height for the kaolin specimen is 76 mm. The physical and mechanical properties of bottom ash, were determined from the following laboratory tests:

a. Atterberg Limit Test
b. Specific Gravity Test
c. Particle Size Distribution
d. Falling Head Permeability Test
e. Standard Proctor Test

Furthermore, other material that used in this study is bottom ash and it is get from Tanjung Bin Power Plant, Johor. The group of encapsulated bottom ash are inserted into kaolin specimen to determine it strength of the soft clay. The bottom ash were used in this study are 6 mm and 8 mm diameter where the height is 38 mm, 57 mm and 76 mm. The test for soft clay specimen reinforced with the group of encapsulated bottom ash were tested by Unconsolidation Undrained (UU) Test. The physical and mechanical properties of bottom ash were determined from the following laboratory tests:

a. Particle Size Distribution
b. Specific Gravity Test
c. Standard Proctor Test
d. Constant Head Permeability Test
e. Relative Density Test
f. Direct Shear Test

The shear strength parameter of the soft clay reinforced with the group encapsulated bottom ash columns, had been determined from the Unconsolidated Undrained Triaxial Test.

1.5 **Significant of Research**

In engineering field, before any structure are constructed, the major geotechnical problem that the geotechnical engineer need to concern is the properties of the soil at the construction site. Soft clay is soils that have the major problematic type of soil because of the weakness of it properties. The ground improvement is the method or the alternatives that can resist to this kind of soil problem. Unfortunately, ground improvement have so many method that can be applied and each of the method have their own advantages. Stone column is the method of ground improvement that most economical and preferable method to be used to improve the properties of the soft clay soil.

Stone column is the ground improvement method that need the granular materials in its application. The coal burn also have a disposal waste that have the similar characteristic with the granular materials such as gravel. To avoid the consumption of natural material that depleted gradually, the use of recycle material from waste of coal, such as bottom ash is the excellent decision. Bottom ash is the waste materials that cannot be disposal, moreover the unused of bottom ash can cause the increasing number of disposal area.

The increasing of the number for disposal area are not a good choice because it give the harmful not only to the people, but also to the animals, tree and our surrounding environment. Thus, the utilization of the recycle material, bottom ash in the stone column method help in improving the soil properties other than help in the environmental issues that getting serious lately. This research are help in decreasing the soft clay problem, environmental problem and waste material problem by using bottom ash material.

CHAPTER 2

LITERATURE REVIEW

2.1 Clay

2.1.1 Background

Soil deposits can be divided into two group, which are residual soil and transported soil. The residual soil is the soil that created and formed from a weathering process of rock and remain at it origin location however the transported soil is the soil that moved from their place of origin (McCarthy, 2007). Soil are separated into three broad categories which are cohesion less, cohesive and organic soil (Sa'adon, 2009). The cohesion less soil is a type of soil particle that not tend to stick together, for example are gravel, sand and silt. Next, the organic soil is described as soil containing a sufficient amount of organic matter to affect its engineering properties. While cohesive soils have a very small particle size characteristic where the chemical predominate or in other words, the particle tend to stick to another. The texture of soil classification is shown in Figure 2.1. The classification of the soil is based on the texture and the three main soil classification is clay, sandy and silt. The most commonly type of cohesive soil is clay, which are clay are divided into two type; hard clay and soft clay.

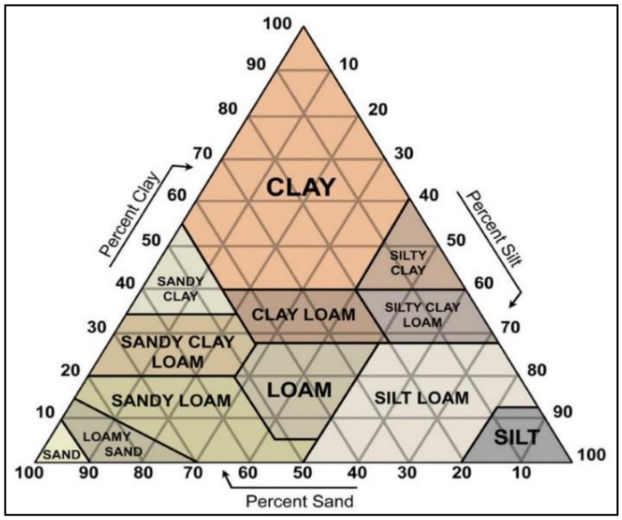

Figure 2.1: Texture of soil classification (Das, 2010)

2.1.2 Characteristic of Clay

There are three major mineral consist in clay mineral; which are kaolinite, illite and montmorillonite. The montmorillonite minerals shows the ability to swell and cation exchange capability (How, 2011). According to How, (2011), dispersive clay soils can be yellow, red, brown, grey or various combinations of those colors. Most of the fine-grained soils tested, known to be derived from in situ weathering of metamorphic rocks and igneous rocks, and limestone have been no dispersive. When dry, clay becomes firm and when fired in a kiln, clay are permanently change its physical and chemical properties. The Table 2.1 shown the clay characteristic of kaolinite, illite and montmorillonite. Typical thickness of the koilinite is 50 to 2000 mm, illite is 30 mm and montmorillonite is 3 mm. While for diameter, illite have the greater diameter 10000 mm while kaolinite and montmorillonite is 300 to 4000 mm and 100 to 1000 mm. The swell potential of montmorillonite is high, illite is medium whereas kaolinite have the low swell potential.

Table 2.1: General of clay characteristic (How, 2011)

Mineral	Kaolinite	Illite	Montmorillonite
Typical thickness (mm)	50 - 2000	30	3
Typical diameter (nm)	300 - 4000	10000	100 - 1000
Specific surface (m2/g)	10 – 30	50 - 100	100 - 800
Cation exchange capacity (meq/100g)	3	25	100
Activity	0.3 – 0.5	0.5 – 1.3	1.5 – 7
Swell potential	Low	Medium	High

2.1.3 Properties of Clay

The difference properties of silt and clay varies by discipline. Guggenheim (1995) stated that geologist and soil scientists usually consider the separation to occur at a particle size of 2µm which clay are being finer than the silts. In addition, sedimentologist often use 4-5 µm whereas the collois chemists use 1 µm. According to McCarthy (2007), the particle of clay soil sizes is less than about 0.005 mm. The particle of the soft clay are too fine that it cannot be separated by sieve analysis because there are no practical sieve can be made with the very small opening. Clay is a fine-grained mineral that generally plastic at appropriate water content and will harden with dried and fired (Guggenheim, 1995). Sa'adon (2009) stated that the soft clay soil is subjected to high plasticity when the optimum amount of water are mixed together. Table 2.2 shown the classification of plasticity index in quantitative manner. The plastic limit is increase with the increase of the plasticity.

Table 2.2: Classify of plasticity index in quantitative manner by Burmister. (1949)

Plastic Limit	**Description**
0	Non-plastic
1-5	Slightly plastic
5-10	Low plasticity
10-20	Medium plasticity
20-40	High plasticity
>40	Very high plasticity

Any construction project for structure built are subjected to the settlement of the soil. In Table 2.3, the order of soil suitability with the type of soil is shown. Bed rock has the best of order of soil suitability while sand and gravel, and medium to hard clay is very good and good. Meanwhile, the silts and soft clay are proved that have the poor in soil suitability. Soft clay is the type of soil that have the greater chances of settlement to happen because of the Eied *et al*. (2014) stated that soft clay have low of shear strength and high compressibility. While, according to Craig (2004), excessive settlement is tipped to be a big problem as it often exceeds the permissible limits. Settlement will affect the stability of the structure and this can cause the failure to the structure such as cracking, collapse and others. In addition, Table 2.4 shown the properties of void ratio, moisture content and dry unit weight of the soil. From the various type of the soil, soft clay have the greater value of void ratio 0.9 to 1.4, the higher moisture content that 30 to 50% in saturated stated and medium value of dry unit weight 12 to 15 kN/m^3.

Table 2.3: Order of soil suitability for foundation support

Type of soil	Order of soil suitability
Bed rock	Best
Sand and gravel	Very good
Medium to hard clay	Good
Silts and soft clay	Poor
Organic silts and organic clay	Undesirable
Peat	Unsuitable

Table 2.4: Void ratio, moisture content and dry unit weight for some typical soils in a natural state (Das *et al*., 2010)

Type of Soil	Void Ratio, *e*	Natural moisture content in saturated stated, w (%)	Dry unit weight, γ_d (kN/m^3)
Loose uniform sand	0.8	30	14.5
Dense uniform sand	0.45	16	18
Loose angular-grained silty sand	0.65	25	16
Dense angular-grained silty sand	0.4	15	9
Stiff clay	0.6	21	17
Soft clay	0.9 – 1.4	30 – 50	11.5 – 14.5
Loess	0.9	25	13.5

2.2 Ground Improvement

2.2.1 Background

To construct a structure such as building, highway, railway and others, the site investigation for soil at the site is important. The site investigation is the test to get the properties and type of the soil before construction work are begin. In geotechnical engineering field, major geotechnical problem that engineers always concerned is the properties of the original soil materials at construction sites which are unable to reach the specification requirement. Major Geotechnical problem cause by the soft soils such as clay are a large settlement during construction and differential settlement after construction completed. Ground improvement or modification technique is one of the popular method to improve the properties of the soils. Ground improvement techniques are given the utmost important in present days to adapt week ground or soil into the appropriate competent stable ground for different civil engineering applications (Tiwari and Kumawat, 2014). According to Raju (2010), after giving consideration to the nature of the ground being improved and the type and sensitivity of the structures being built, ground improvement often reduces direct costs and saves time.

2.2.2 Ground Improvement Techniques

Ground improvement techniques are recommended in difficult ground conditions as mechanical properties are not adequate to bear the superimposed load of infrastructure to be built, swelling and shrinkage property more pronounced, collapsible soils, soft soils, organic soils and peaty soils, karst deposits with sinkhole formations, foundations on dumps and sanitary landfills, handling dredged materials for foundation beds, handling hazardous materials in contact with soils, using of old mine pits as site for proposed infrastructure (Tiwari and Kumawat, 2014). The approaches incorporating ground improvement processes can generally divided into four categories by the technique or method by which improvement are achieved. There are;

i. Mechanical modification,
ii. Hydraulic modification,
iii. Physical and chemical modification and
iv. Modification by inclusion and confinement.

Firstly, Tiwari and Kumawat, (2014) noted the mechanical improvement technique is the method to increase the soil density by mechanical force, including compaction of surface layers by static vibratory such as compact roller and plate vibrators. This technique is further classifies as Dynamic Compaction, Vibro-Compaction, Compaction Grouting, Pre loading and Pre-fabricated Vertical Drains and Blast densification. Next, the hydraulic modification is the method modification of soil properties are achieved by forcing the free water out of soil via drains or wells. Some of the hydraulic modification method are; preloading using fill, preloading using fill with vertical drain, vacuum preloading with vertical drained and lastly the combined fill and vacuum preloading (Tiwari and Kumawat, 2014).

According to Tiwari and Kumarat (2014), the physical and chemical modification is the technique where the surface layers or column of soil are achieved by adhesives physical mixing. The adhesive includes natural soils industrials by products or waste materials or cementations or other chemicals which react with each other and the ground. Some of the physical and chemical modification methods are grouting, electro-osmosis, soil cement, heating, freezing and vitrification. Lastly, the modification by inclusion and confinement is the method of soil properties are achieved using reinforcement by means of fibers, strips bars meshes and fabrics imparts tensile strength to a constructed soil mass (Tiwari and

Kumawat, 2014). The methods are including a vibro replacement or stone column, dynamic replacement, sand compaction piles, geotextile confined columns, geosynthetic reinforced column or pile supported embankment and others.

2.2.3 Stone Column

The matters of time and cost is the key in construction planning, moreover the stone column technique is the most effective, economical and preferable method to improve the soil properties from the various technique of ground improvement. The stone column technique was adopted in European countries in the early 1960's (Pivarc, 2011). Based on Juran *et al*., (1991) the stone column method has been increasing used in the construction industry since the last two decades. Stone column is compressive load fail in two main difference modes: bulging (Hughes and Withers, 1974) and general shear failure (Barksdale and Banchus, 1983). The stone column technique, also known as vertical granular column is a ground improvement process where vertical columns of compacted aggregate are formed through the soils to be improved such as soft clay (Zahmatkesh and Choobbasti, 2010).

According to Pivarc (2011) the stone column method is the ground improvement technique has been successfully used to increased bearing capacity and reduce the settlement of construction such as storage tank, earthen embankments, raft foundations and other. Based on Frikha *et al*. (2014), stone column are usually designed to improve bearing capacity and to reduce settlement of soft soils and this technology is well suited for the improvement of soft soils such as silty sand, silts and clays. Hasan *et al*. (2011) stated stone column usually installed in soft cohesive soil to improve the bearing capacity, reduced settlement and accelerate the dissipation of pore water pressure. In addition, this ground improvement technique has successfully increased load bearing capacity and stiffness of soil and consequently, reduced settlement for foundation of structure, for instance liquid storage tanks, earthen embankments and raft foundation (Young, 2012).

2.2.3.1 Case Studies

Previous researchers are carried out the investigation of the various method of test for stone column improve the soil properties such as shear strength, consolidation, settlement and many more. Firstly, the study the improvement in strength of soft clay when inserted with singular and group bottom ash column by Hasan *et al*. (2011). Figure 2.2 is the photograph of clay specimen reinforced with singular and group of four bottom ash column. According to Hasan *et al.* (2011), in this study the samples without bottom ash column, samples with partially penetrating bottom ash column and samples with fully penetrating for singular and group of bottom ash column are prepared and tested. The singular column are installed at the center of the clay specimen and for the group bottom ash are arranged in square pattern. The detail arrangement for the soft clay specimen reinforced with singular stone column are shown in the Figure 2.3 and for soft clay specimen reinforced with group stone column in Figure 2.4.

Figure 2.2: Clay specimen reinforced with singular and group bottom ash column (Hasan *et al*., 2011)

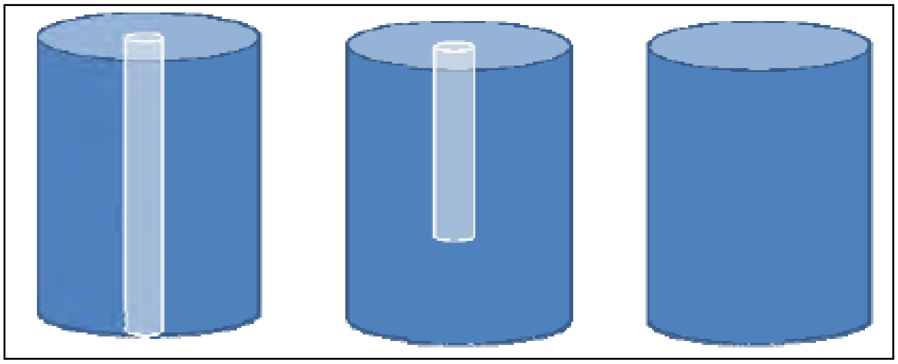

Figure 2.3: Detail arrangement for soft clay specimen reinforced with singular bottom ash column (Hasan *et al*., 2011)

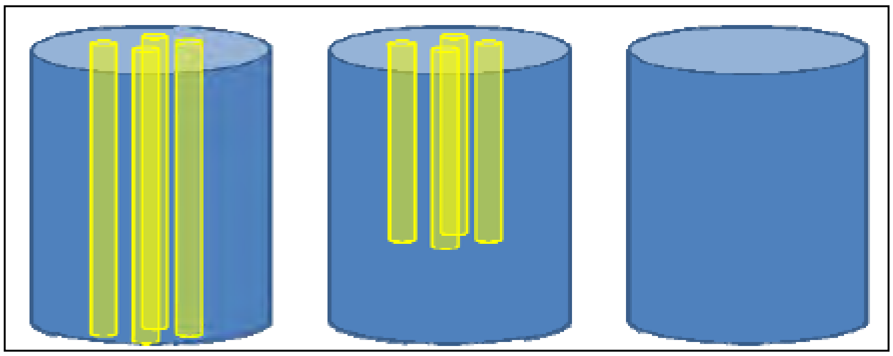

Figure 2.4: Detail arrangement for soft clay specimen reinforced with group bottom ash column (Hasan *et al*., 2011)

Table 2.5: The improvement of shear strength

Arrangement	**The improvement of shear strength (%)**	
	Partially penetrating	**Fully penetrating**
Singular	13.33 – 25.62	1.65 – 14.90
Group (square pattern)	20.25 – 27.03	9.22 – 65.54

The table 2.5 above shown the summary of test result of the improvement rate of shear strength for soft clay reinforced with singular and group bottom ash. The improvement rate for partially penetrating singular column is 13 to 26% while for fully penetrating of singular column is 2 to 15%. However, the group in square pattern of bottom ash, the improvement rate for partially penetrating is 20 to 27% while for fully penetrating is 9 to 66%. It can be concluded that the shear strength can be improved by the installation of bottom ash column (Hasan *et al.*, 2011). In addition, the partially penetrating for singular column showed more significant improvement compare to the fully penetrating and for group bottom ash column, the shear strength of soft clay increased as the height of column increased. Based on Hasan *et al.* (2011), due to the fact that the area and volume of soil replaced by bottom ash for group column is much higher than singular column, the improvement of shear strength for group column is in line with the increase of height of bottom ash column.

Secondly, Kousik *et al*. (2007) have study the behavior of geosynthetic-reinforced granular fill over soft soil improved with stone column. The non –linear behavior of the granular fill and the soft soil is considered, other than the effect of consolidation of the soft soil due to inclusion of the stone column has also been included in the model. Figure 2.5 is illustrate the Geosynthetic-reinforced granular fill-soft soil system with stone columns and Figure 2.6 is the proposed foundation model. In this paper, the development of a mechanical foundation model is reported for geosynthetic- reinforced granular fill over soft soil with stone column inclusions, which incorporates the nonlinear behavior of the granular fill and soft soil as well as the effect of consolidation of the soft soil (Kousik *et al*., 2007).

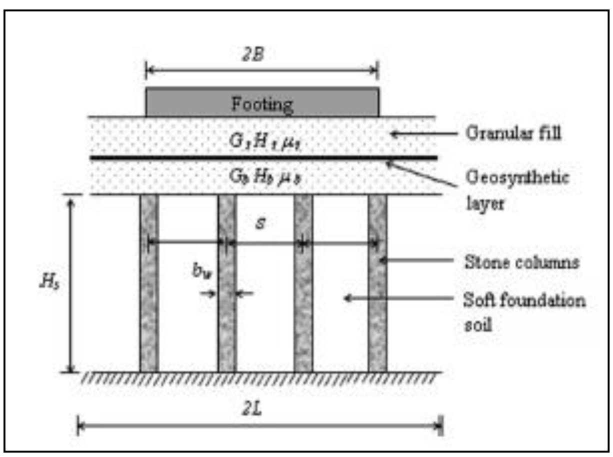

Figure 2.5: Geosynthetic-reinforced granular fill-soft soil system with stone column (Kousik *et al.*, 2007)

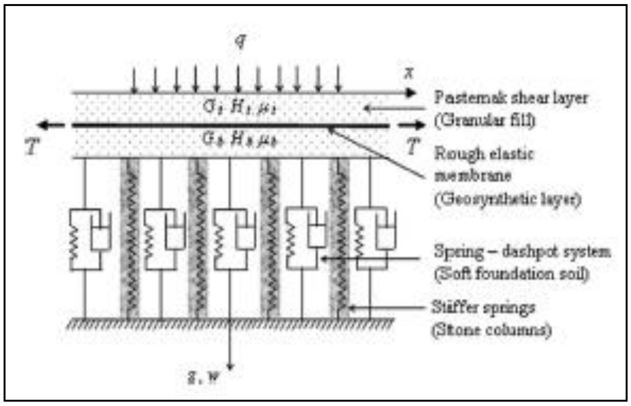

Figure 2.6: The proposed foundation model (Kousik *et al.*, 2007)

Figure 2.7 and Figure 2.8 shows the effect of modular ratio on the maximum settlement and the effect of the modular ratio on the differential settlement at the ground surface. This study also showed that the mobilized tension in the geosynthetic increases as stiffness of the stone columns is increase and the rate of increase is more at higher load intensity. According to Kousik *et al.* (2007), the use of geosynthetic reinforcement transfers the stress from the soil to stone columns due to stiffness difference between the stone columns and soil, and this may prevent large displacement due to the intermediate support provided by the stone column. As the conclusion, the geosynthetic layer is effectively reduces the settlement of the soft soil.

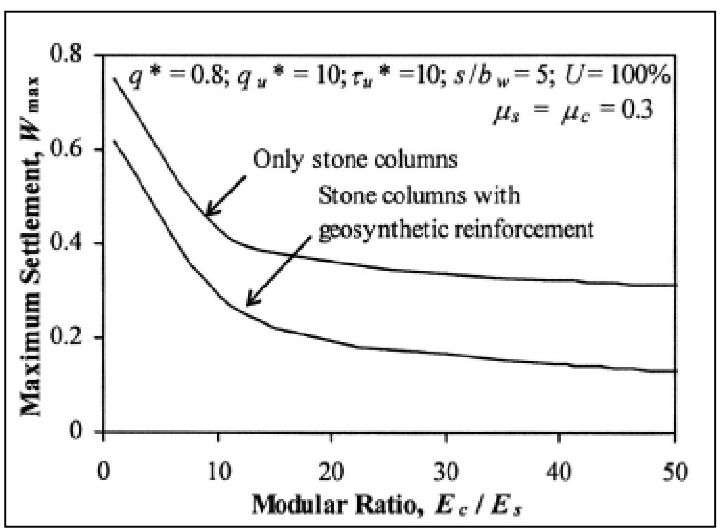

Figure 2.7: Effect of modular ratio on maximum settlements (Kousik *et al.*, 2007)

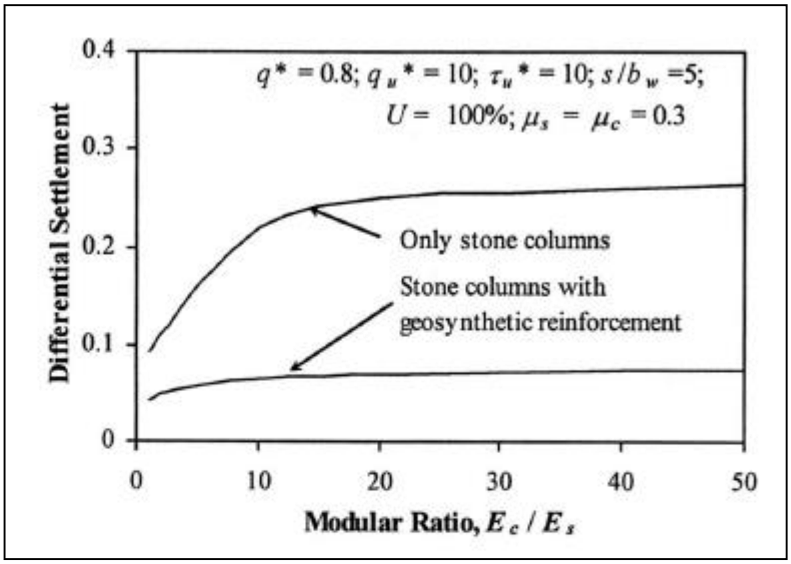

Figure 2.8: Effect of modular ratio on differential settlement (Kousik *et al*., 2007)

Next, the study of the soil improvement by reinforced stone column based on experimental work by Hamed *et al*. (2011). The purpose of this paper is to provide a review on ground improvement by using reinforced stone column in geotechnical engineering project and based on previous result, the critical value were discussed and recommended. Soil is stronger in compression than in tension but geosynthetics can be improve the tension strength in soils (Hamed *et al*., 2011). According to Hamed *et al.* (2011), reinforcement of the soil by compacted granular columns or stone columns is accomplished by the top feed method, thus the aggregates are the allowed to take place of the displaced soil which exerts a pressure in the surrounding soil hence helping to improve the soil's load bearing capacity.

Sharma *et al.* (2004) performed a series of laboratory analysis to investigate the effect of geogrid on the load bearing capacity and bulging reduction on granular column.

Figure 2.9 is illustrate of the experimental setup. Based on Hamed *et al.* (2011), from this analysis, it was found that the geogrid has effectively improved the load carrying capacity of the granular column and reduce the bulging diameter and bulging length of the granular column. The improvement factors increased with the increase of numbers of geogrid and decrease of geogrid spacing. The stress to induce a settlement of 3mm increased 80% comparing to the unreinforced granular column. For 5 numbers of geogrid with a spacing of 10mm, the bulge was negligible at 1.04 times of the column diameter. Meanwhile, the bulge length was 1.33 times of the column diameter. However, the effect of mesh size and strength of the geogrid was not investigated.

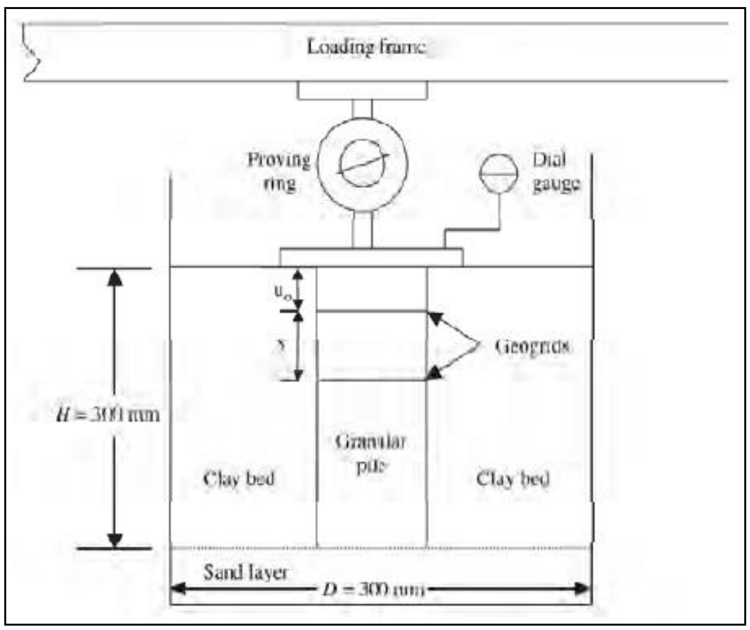

Figure 2.9: Experimental setup by Sharma *et al.* (2004)

Besides, Gniel and Bouazza (2009) conducted a series of small scale model tests on the geogrid encased column to investigate the geogrid encasement length on the strain reduction and the bulging prevention. The laboratory tests were carried using enlarged oedometer with 143mm internal diameter which was designed based on the unit cell idealization concept. From this analysis, Hamed *et al.* (2011) stated it was found that the encasement of the stone column using geogrid can effectively increase the stiffness and reduce the strain of the stone column. For a fully encased stone column in a column group the strain can be reduced up to 80%. Meanwhile for the isolated column which was loaded at the column area, the load carrying capacity increase with the increase of encasement length; however the strain at failure was remained quite consistent. For the fully encased stone column, bulging was observed at the base of the encasement. For the partially encased isolated stone column and stone column group, bulging was observed along the full length of the non-encased column in the column groups and confined to a length of about 2 column diameters respectively. However, in this analysis, the stone column was prepared by using frozen method which cannot represent the actual construction process of stone column at site as the confining pressure of the soil was low at site thus the quality of the stone column might be lower.

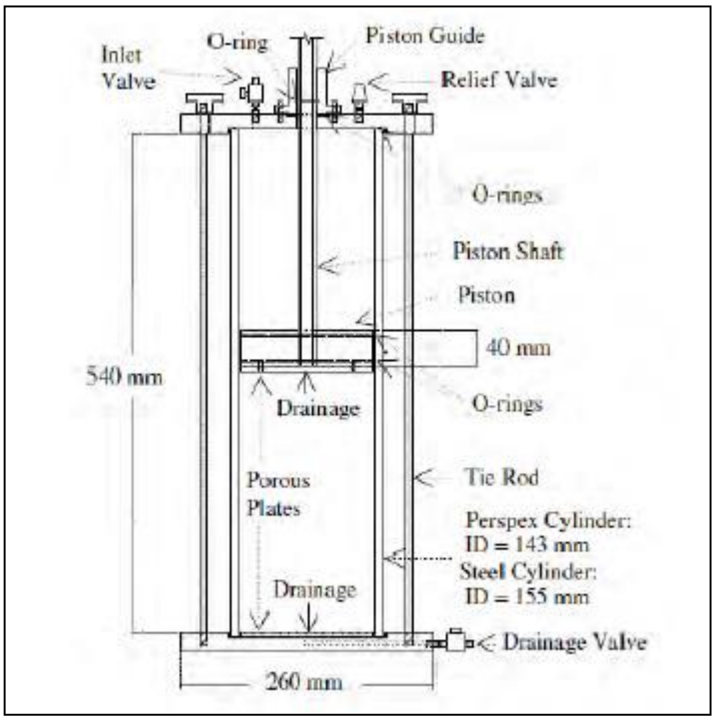

Figure 2.10: Sketch of enlarged consolidation cell by Gniel and Bouazza (2009)

Figure 2.11: Photograph of enlarged consolidation cells in operation (Gniel and Bouazza, 2009)

Other than that, Gniel and Bouazza (2010) were conducted a series of small and medium scale tests to investigate an alternative method of geogrid encasement construction which is “method of overlap”. Also, the effects of the geogrid properties and column aggregate sizes on the encased stone column performance were also investigated. According to Hamed *et al*. (2011) from the investigation, it was found that the overlap method is suitable for biaxial geogrids. The 20/50 mm rubble, which is a typical conventional stone column backfill material, can provide the greatest interlocking to the stone column and geogrid. However, the cutting might reduce the strength of geogrid. The cutting of geogrid can be reduced by incorporate a higher strength of geogrid. However, the temporary fixing of the encasement sleeves is needed to be refined and the minimum number of junctions required in the section of overlap should be investigated. Other than that the method of overlap is not suitable for geotextile encasement and geotextile or geogrid composites.

Figure 2.12: Welded geogrid encasement by Gniel and Bouazza (2010)

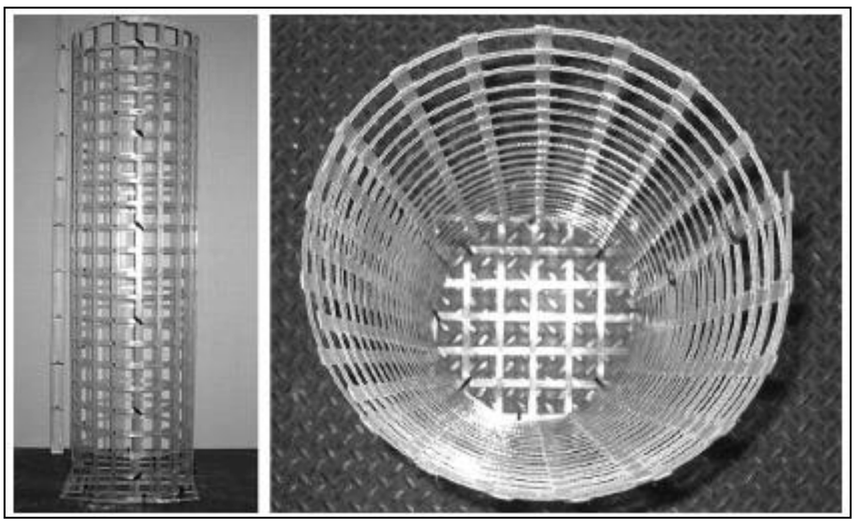

Figure 2.13: Typical encasement sleeve used for medium-scale testing (Gniel and Bouazza, 2010)

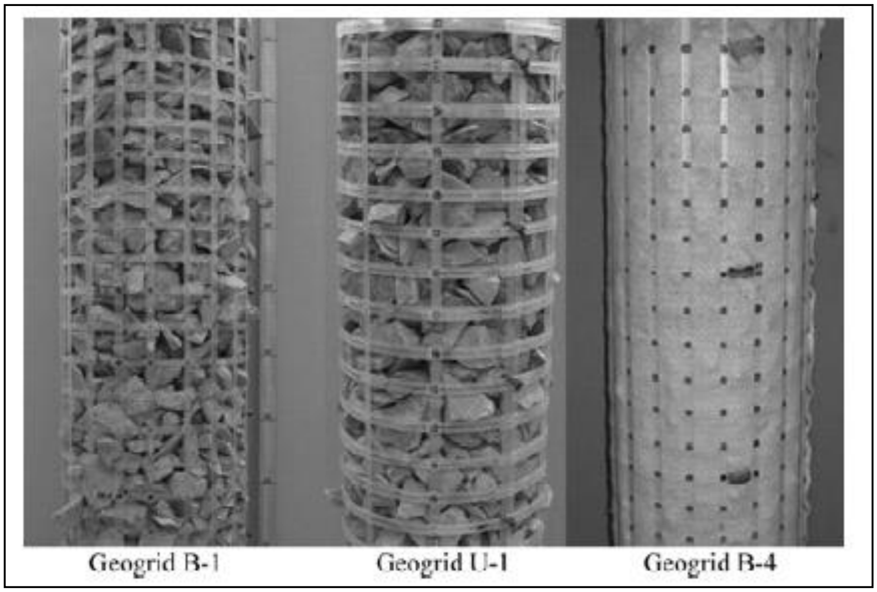

Figure 2.14: Columns prepared for testing with encasement constructed from different geogrids by Gniel and Bouazza (2010)

Figure 2.15: Column being loaded in unconfined compression (Gniel and Bouazza, 2010)

Next, Murugesan and Rajagopal (2007) conducted a series of laboratory tests on the geosynthetic encased stone columns to investigate the influence of the stiffness of encasement, depth of encasement and column diameter on the performance of stone column. From the investigation, it was found that the geosynthetic encasement increase the stiffness of the stone column. The stiffness of the stone column increased with the stiffness of geosynthetic encasement. However, the strain levels were smaller for the stone column with smaller diameter. The confinement effects reduced with the increase of stone column diameter. The test results also showed that geosynthetic encasement is only needed for the part where the bulging occurred. However, strain softening will occur in partially encased stone column when it is loaded beyond a particular stress. The geosynthetic encasement also prevents the contamination of stone column and thus will not reduce the friction between the stone aggregates and clay bed. However, the results should be further verified by conducting field testing.

Lastly, Wu and Hong (2008) conducted a series of laboratory tests on the granular columns with the horizontally laminated reinforcing sheets to verify the analytical procedure proposed to analyses the column expansion four layers of geotextile layer were installed at an equal spacing which is the double distance from the end of the column as shown in Figure 2.16. The effect of the reinforcement stiffness, reinforcement strength, granular column radius and spacing of the reinforcing sheets were investigated. Hamed *et al.* (2011) stated that from the investigation, it was found that the increase of the inclusions stiffness can lower the axial strain of the granular column. Smaller spacing of the reinforced granular column increases the stiffness granular columns for the same radius/spacing ratio. However, the tests were conducted only for the sand as the granular column materials. The usage of the stone as the column materials should be further investigated.

Figure 2.16: Initial and deformed triaxial specimen with four-layer reinforcement: (a) initial shape of the reinforced triaxial specimen and (b) deformed shape of the reinforced triaxial specimen (26% axial strain)

2.2.3.2 Shear Strength

In the previous study, Maakaroun *et al.* (2009) stated that, the area replacement ratio for singular column increases, shear strength of soft clay also increases. This correlation is strongly supported by previous researcher as shown in Table 2.6.

Table 2.6: Effect of area replacement ratio on undrained shear strength

Researcher	**Area Replacement Ratio, A_c/A_s (%)**	**Undrained Shear Strength Increment (%)**	**Remarks**
Najjar *et al.* (2010)	7.90	19.50	Singular Column
	17.8	75.00	Singular Column
Black *et al.* (2007)	10.0	33.00	Singular Column
	12.0	55.00	Group Column
Ali (2011)	4.0	50.00	Singular Column
	9.0	58.14	Singular Column
Fadzil (2011)	16.0	45.00	Group Column
	36.0	-33.00	Group Column

Based on the study of Black *et al*. (2007), the deviator stress failure for unreinforced sample was 56 kPa. The presence of fully penetrating single column (diameter = 32mm) in the sample increased the deviator stress to 75 kPa. This shows the increase of approximately 33% in deviator stress for an increased area ratio of 10%. Figure 2.20 shows the column arrangement designed by Black *et al*, (2007). On the other hand, the presence of a group of three 20 mm diameter columns increased the deviator stress from 56 kPa to 70 kPa (for $H_c/H_s = 0.6$) and 87 kPa (for $H_c/H_s = 1$) respectively. The increase in the deviator stress in the case of fully penetrating columns ($H_c/H_s = 1$) was 55% for an increased area ratio of 12%. Figure 2.20 depicts the column arrangement for both of the “single 32 mm diameter column” and “a 24 group of three 20 mm diameter columns”. However, study of Fadzil (2011) was in contradiction with previous study. The decrease performance of shear strength was due to the unsuitable of area replacement ratio which was the soil replacement too much. Besides, studies of Tandel *et al.* (2012) and Murugesan *et al.* (2010) were in contradiction with the previous study too, in which the single of the column made less significant in load-carrying capacity of the reinforced clay. The reduced performance was explained that the due to mobilization of higher confining stresses in smaller bottom ash column. The higher confining stresses in the column leads to higher stiffness of smaller diameter.

In the previous study, researchers such as Narashima *et al*. (1992), Muir *et al*. (2000), McKelvey *et al*. (2004), found that the increase in undrained shear strength of soft clay does not depends only on total area replacement granular material in the soft clay, but also on column penetration ratio, which is the ratio between the height of column and the height of sample (H_c/H_s). However, the idea is not support by Black *et al*. (2007). They found that the relative increment in strength due to sand columns is independent of the column configuration and is only dependent on the area replacement of the reinforcement. Figure 2.17 and 2.18 shows the results done by Black *et al*. (2007) and Najjar *et al*. (2010).

The studies done by Najjar *et al.* (2010) had proven that the increase in shear strength depends on the column(s) penetration ratio. This correlation has been studied by other researchers for example Narasimha *et al*. (1992), Muir *et al*. (2000), McKelvey *et al*. (2004) and it is suggested that the “critical column length” falls between four to eight times the column diameters. Beyond the “critical length”, the penetration ratio may no longer participate in increasing the load carrying capacity of soft cohesive clays. A long stone column having a length greater than its critical length fails by bulging irrespective to whether it is end bearing or floating (Ambily and Ghandi, 2007). McKelvey *et al.* (2004) conducted a research of a group of five (5) stone columns and observed that the central column bulged uniformly, whereas the edge columns bulged away from the neighboring columns.

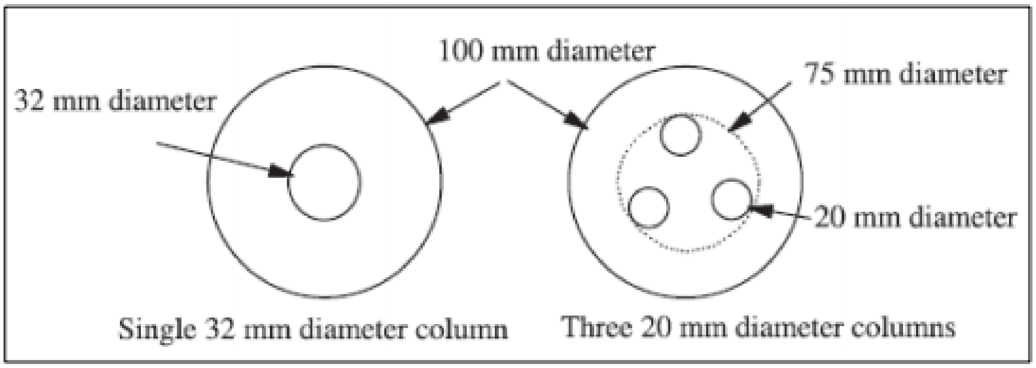

Figure 2.17: Column arrangement (Black et al., 2007)

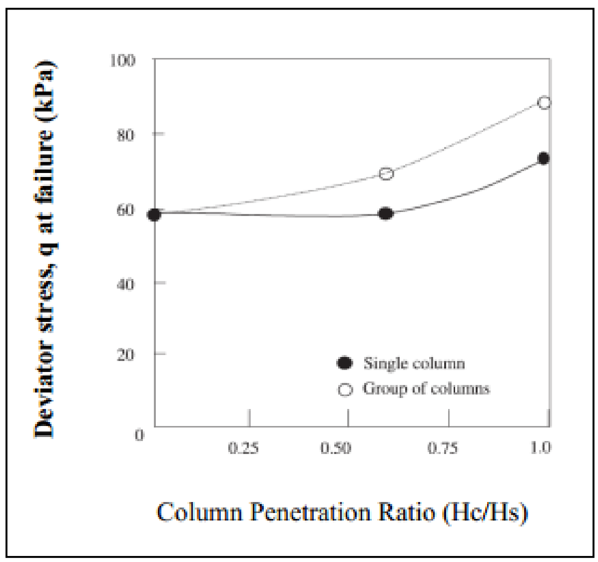

Figure 2.18: Deviator stress at failure for various column penetration ratio (Black et al., 2007)

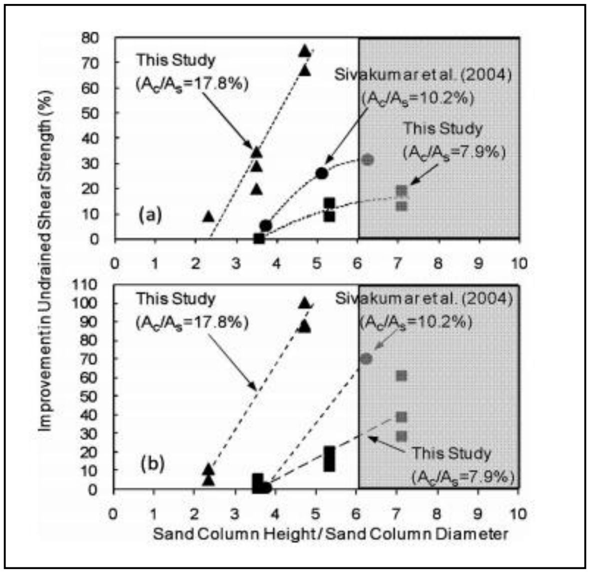

Figure 2.19: Effect of ratio of column height to diameter (Najjar *et al*., 2010)

2.3 Bottom ash

2.3.1 Background

Presently, the highly increasing the number of community are lead to the increasing of the usage of the natural materials in our daily life. The increasing usage of the natural materials are cause the decreasing of the natural materials sources. There is a possibility of the extinction of natural materials if there is no prevention alternative to control the usage of natural materials. Therefore, the Reduce, Reuse and Recycle (3R) program are introduced and are widely use all over the world and the familiar materials are paper, aluminums, plastic, bottle and glasses. Besides that, there are so many natural materials that can be recycle such as the coal ash.

Coal are known as a largest source of the energy for the generation of electricity and throughout history, coal has been used as an energy resource, primarily burned for the production of electricity and heat, and also used for industrial purposes, such as refining metal. When the coal is combusted for the production of electricity at coal-fired power plants, significant amounts of combustion residues remain and required proper disposal or reuse. According to Benson *et al.* (2011), the majority of coal bottom ash are produced at coal-fired electric utility generation stations, with some coming from coal-fired boilers or independent coal-burning electric generation facilities. The ash is the combustion waste that produced from the process of biomass combustion and can be divided into two types; bottom ash and fly ash (Carrasco *et al.*, 2012). About 40% is beneficially used in a variety of application and about 60% are managed in storage and disposal area. One of the famous coal power plant in Malaysia is Tanjung Bin Power Plant, Johor, Malaysia.

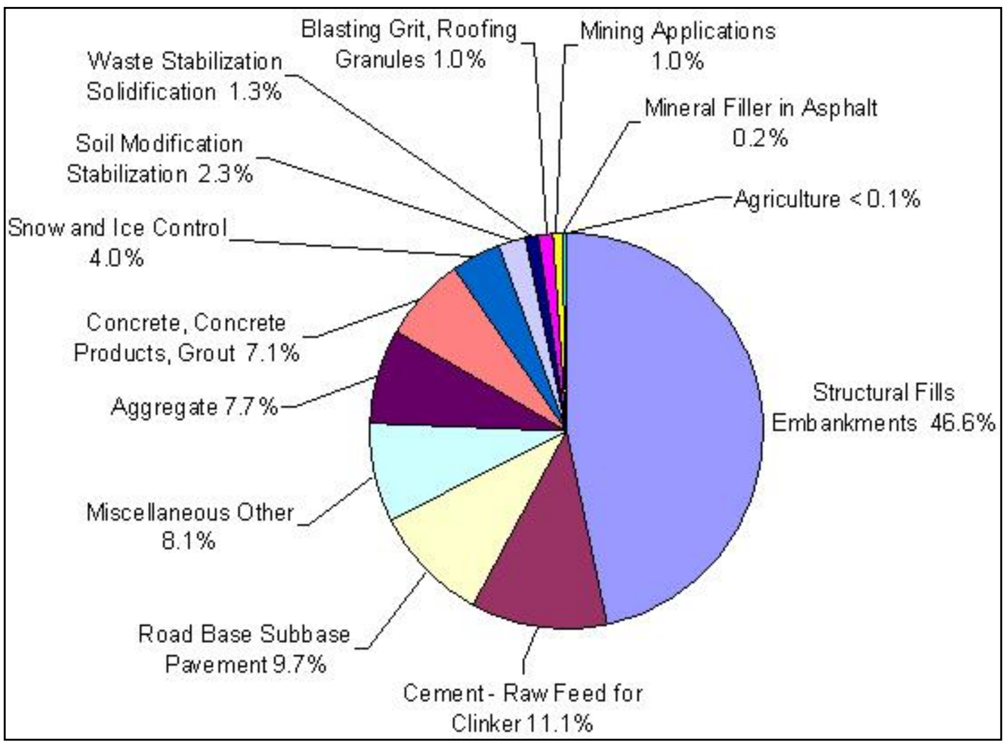

Figure 2.20: Illustrates the common applications of coal bottom ash (Steam, 1998)

Figure 2.20 shows the application of the bottom ash. According to statistic in 2006 on bottom ash usage, only over 45 percent of all bottom ash produced is being use, and the mainly application in transportation such as road material and structural fill (Steam, 1978). From 45% of the bottom ash usage, 47% are used in structural fills embankments and 11% is used in cement-raw feed for clinker. In the figure shown that the most of the bottom ash is used in construction industries and as the substitute to the granular materials. Only less than 0.1% of bottom ash are use in agriculture industries.

Figure 2.21: Tanjung Bin Power Plant (Abubakar *et al.*, 2012)

2.3.2 Properties of Bottom Ash

Bottom ash is produced as a result of burning coal in a dry bottom pulverized coal boiler and the unburned material from the dry bottom boiler consist of 20 percent of bottom ash (Benson *et al.*, 2011). According to Steam (1978), bottom ash is a porous, glassy, dark gray material with a grain size that similar with the sand or gravelly sand Benson *et al*. (2011) also stated that bottom ash have angular particles with very porous surface texture. Previous researcher have stated that although similar to the natural fine aggregates, bottom ash is lighter and more brittle and has a greater resemblance to cement clinker.

. The ash particles range in size from a fine gravel to a fine sand with very low percentages of silt-clay sized particles. Bottom ash is predominantly sand-sized, usually with 50 to 90 percent passing a 4.75 mm (No.4) sieve and 0 to 10 percent passing a 0.075 mm (No. 200) sieve. The largest bottom ash particle sizes typically range from 19 mm (3/4 in) to 38.1 mm (1½ in). Bottom ash is usually a well graded material although variations in particle size distribution may be encountered in ash from the same power plant (Benson *et al.*, 2011). According to Abubakar *et al*. (2012), the bottom ash from Tanjung Bin Power Plant are well graded size distribution ranging from fine gravel to fine sand sizes and the majority of the sizes occurred in the range of 0.075mm and 20mm.

Figure 2.22: Bottom Ash from Tanjung Bin Power Plant

The result from the previous researcher are shown in Table 2.7 and Table 2.8. From the research of Benson *et al*. (2011), the specific gravity for bottom ash is 2.1 to 1.7 and from the research of Alto *et al*. (2009) the specific gravity is 2.3 to 3.0. Besides that Benson *et al.* (2011) finding that the dry unit weight of bottom ash is 7 to 16 kN/m^3 and no plasticity recorded. While Alto *et al*. (2009) are finding the bulk density is 65 to 110 and the optimum moisture content is 12 to 26%. The finding for porosity is 0.25 to 0.40 and angle of friction of stone column is 35° to 45°. Figure 2.26 is the photograph of the comparison of physical properties of the bottom ash, fly ash, boiler slag and FGD.

Table 2.7: Typical physical properties of bottom ash (Benson *et al.*, 2011)

Property	Bottom Ash	Test Method
Specific Gravity, Gs	2.1 – 1.7	ASTM D854-06
Dry Unit Weight (kN/m^3)	7.07 – 15.72	
Plasticity	None	ASTM D4318-05 AASHTO T 090
Absorption (%)	0.8 – 2.0	ASTM C128-07a

Table 2.8: Typical ranges for geotechnical properties of bottom ash (Alto *et al.*, 2009)

Property	Bottom Ash
Specific Gravity	2.3 – 3.0
Bulk Density	65 – 110
Optimum Moisture Content (%)	12 – 26
Hydraulic conductivity (cm/s)	$10^{-1} – 10^{3}$
Porosity	0.25 – 0.40
Angle of Internal Friction (◦)	35 - 45

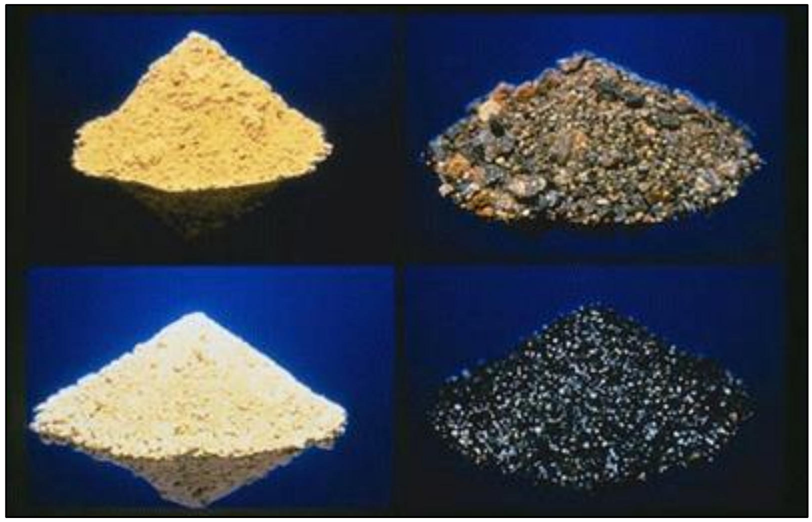

Figure 2.23: Comparison of bottom ash with fly ash, boiler slag and FGD material.

In the study of Muhardi *et al.* (2010), bottom ash of Tanjung Bin Power Plant is found in the range of 0.075mm – 20mm. Based on the BS1377:1975, this bottom ash is classified as coarse grained soil. The average coefficient of uniformity, C_u for bottom ash is approximately 16.56 while the average coefficient of curvature, C_c is approximately 1.01. From Unified Soil Classification System (USCS), bottom ash is classified as well graded sand while from AASHTO system, bottom ash fall in the A-1 group and classified as A-1-a. The result done by Muhardi *et al.* (2010) is shown in Figure 2.24.

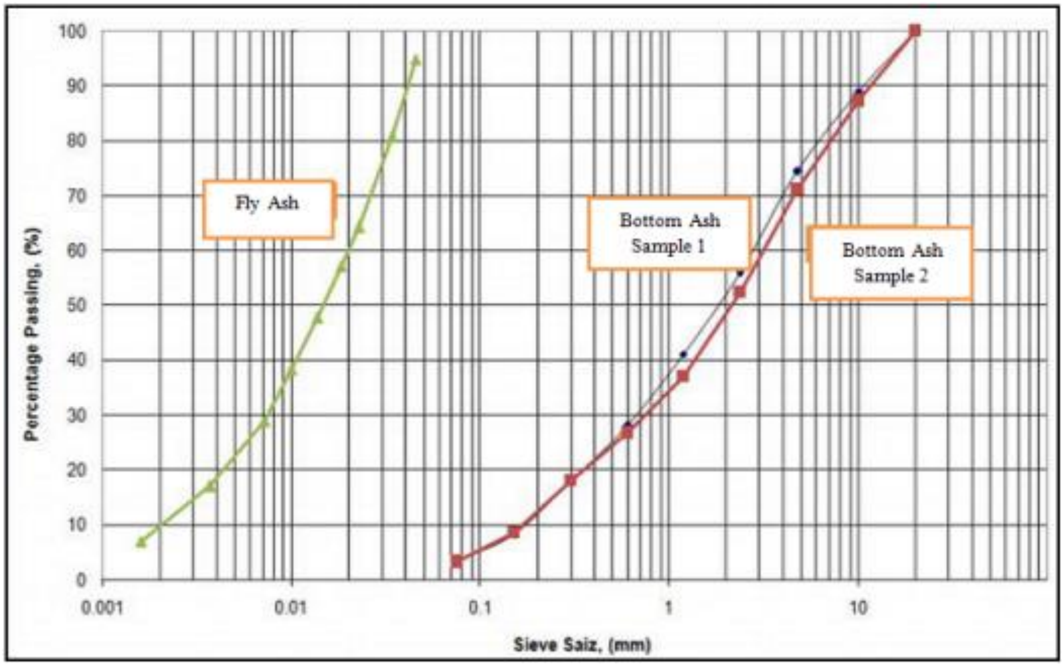

Figure 2.24: Particle size distribution of fly ash and bottom ash (Muhardi *et al.*, 2010)

2.4 Kaolin

2.4.1 Background

Kaolin are white raw materials, their essential constituent being fine grained white clay, which are amenable for beneficiation that make them ideal for an assortment of industrial application (Prasad *et al*., 1990). Kaolinite or kaolin clay occurs in abundance in soils that have formed from the chemical weathering of rocks in hot, moist climates such as in tropical rainforest areas. According Prasad *et al.* (1990), comparing soils along a gradient towards progressively cooler or dried climates, the proportion of kaolinite decreases, while the proportion of other clay minerals such as illite (in cooler climates) or smectite (in drier climates) increases. The differences climatically-related clay mineral content are often used to infer changes in climates in the geological past, where ancient soils have been buried and preserved. One layer of the mineral consists of an alumina octahedral sheet and a silica tetrahedral sheet that share a common plane of oxygen atoms and repeating layers of the mineral are hydrogen bonded together.

There are two basically difference process to refine kaolin and remove the major impurities. The first is the simplest process is called as air flotation or dry process. According to the Murray (2010), the properties of the finished product depend on a large extent on those properties inherent in the crude kaolin. In dry operation process, a deposit must be chosen with desirable properties of color and relatively low content of grit. The crude kaolin is transported to the mill where the large chunks are reduced to about egg size by roll crushers. The crushed kaolin is fed into rotary driers and then into air floating equipment. The latter usually consists of a pulverizing unit and an air separator. The fine particles are transported to collecting chambers and the coarse particles are fed back into pulverizer. The second process used to produce kaolin is much more complex and is called wet process. The kaolin is dispersed in water after it is mined. The first step after dispersion is to removal of the coarse grit by settling procedures and vibrating screens. The resultant degritted slurry is fed into centrifuges to separate the kaolin into fine, intermediate and coarse particle size fractions. The kaolin is then dewatered through a filtration process, dried in either rotary, apron or spray driers and prepared for shipment. This process is used to produce highly refined kaolin having controlled properties (Murray, 2010). One of the supplier for kaolin in Malaysia is Kaolin (M) Sdn. Bhd.

Figure 2.25: Kaolin (M) Sdn. Bhd

Figure 2.26: Mountain of White kaolin clay

2.4.2 Properties of the Kaolin

Kaolin is the type of clay is generally used as fillers or raw materials in ceramic, paints, plastics, paper, rubber and many more. Murray *et al*. (2005) noted some specific physical and chemical properties of kaolin are dependent on the environment of deposition, geological origin, geographic source and the method of processing. The kaolinite structure possesses great advantages in many process due to its high chemical stability and low expansion coefficient (Murray *et al*., 2005). Murray *et al*. (2005) also stated the changes in the mechanical and chemical properties of the clay are discussed as the interactions of the heavy metal cations with the kaolinite could affect the structure of the kaolin and influence properties such as swelling capability and the double-layer behaviors.

According to Murray *et al*. (2005) kaolin as a consequence of well-packed structure, kaolin particle are not easily broken down and the kaolin layers are not easily separated. Therefore most sorption activity occurs along the edge and surfaces of the structure and kaolin can form barrier that is not easily degraded and naturally occurring sediments and deposits containing an abundance of kaolin interspersed with other minerals are effective in controlling the migration of dissolved species (Bloodworth *et al.*, 1996). Kaolin non-expanding and as a result of its high molecular stability, isomorphous substitution is limited or non-existent (Mitchell, 1993). Kaolin is the least reactive clay (Prasad, 1990) however, Mitchell (1993) are noted the high pH dependency enhances or inhabits the adsorption of metals according to the pH of the environment.

2.5 Sample Preparation

In order to create a model after in situ condition at a construction site, the strength of in situ materials and the column materials need to be duplicated so the test can be carried out on the sample. The surrounding soil played an important part to provide the lateral support to the stone column. To create the same effects created at real site to the small scale laboratory test is by doing a centrifuge modeling. The option to centrifuge modeling are very expensive and posed more problem since there are many tests that need to be conducted. Some of the researcher have come up with the idea to use reconstituted samples by pre-consolidating the samples using one dimensional consolidation at single gravity.

Based on the research from Maakaroun *et al*. (2009), kaolin slurry was prepared by mixing the kaolin clay with water at a water content of 100% (1.8 times the liquid limit of 55.7%). From the resulting of homogeneous mix, the slurry was transferred into each of four custom- fabricated 1- dimensional consolidometer. Figure 2.27 shows the setting up of the custom fabricated 1-dimensional consolidometers. Primary consolidation required a period of about seven days to be completed. The consolidated specimen were then removed from the PVC pipes by splitting the two halves of the pipes. The clay specimen from the PVC pipes are shown in Figure 2.28. The height of the specimen after the consolidation was about 18 cm, therefore the sample had to be trimmed to a final height of 14.2 cm.

Figure 2.27: Custom fabricated 1- dimensional consolidometers (Maakaroun *et al*., 2009)

Figure 2.28: Clay specimen after 1-D consolidation (Maakaroun *et al*., 2009)

Next, to prepare homogeneous soft clay, the kaolin slurry is poured into the CBR mould and a metal loading plate with the same diameter of the internal diameter of CBR mould then put onto the slurry surface for two hours (Hasan *et al.,* 2011). Figure 2.29 shows the laboratory model test used to produce the homogenous kaolin sample. Vertical loads were then applied through the loading equipment to the loading plate incrementally with the load shown in Table 2.9. After the soft kaolin clay had been produced, three samples of 50 mm diameter each were pushed slowly inside the kaolin clay in CBR mould. Specimen were then been taken out from the sampler by using extruder and trimmed to the desired height of the column for further test.

Figure 2.29: Laboratory model test used to produce the homogenous kaolin samples (Hasan *et al.,* 2011)

Table 2.9: Loading application in the production of soft kaolin clay samples (Hasan *et al.*, 2011)

Applied Load (kgf/cm^2)	Duration (Hours)
0	24
0.5	24
1.0	24
1.5	24
2.0	24

According to the research done by Black et al. (2007), the kaolin powder was mixed at 1.5 times the liquid limit (the liquid limit was 70% and the plastic limit was 36%). The sample was then consolidated under a vertical pressure of 200 kPa in a one – dimensional loading chamber, which was 100 mm in diameter and 500 mm in height. After the consolidation was completed, the pressure in the cylinder was reduced to zero under undrained conditions.

While, the sample preparation from Murugesan and Rajagopal (2010), the clay bed for the tests was prepared in a large test tank of plan dimensions 1.2 m x 1.2 m and 0.85 in depth. The clay soil was obtained from the lake bed and was soaked in water for one month before using it in the tests. Initially the wet clay was mixed with water equal to 1.5 times the liquid limit of the soil by kneading thoroughly in a large tank form slurry free from any lumps and previous stress histories. Before the clay bed was installed with encased stone column, the slurry was filled in the test tank and allowed to consolidate under a pressure of 10 kPa in a tank by using dead weight. This procedure yielded clay beds of uniform moisture content and consistency.

2.6 Column Installation

There are two type of installation for the stone column inside the soft soil. The two option is neither by displacement or replacement method. Hasan *et al*. (2011) stated that through the displacement method, the column is pushed into the soil, while through the replacement method, the soft soil is removed and replaced by the column. According to Black *et al*. (2007), since it is very expansive to create a miniature vibrocat model, there are two suitable methods that can be used to applied on the installation of column in a small scale test which are the 'rain' and 'frozen' methods.

Based on study from Hasan *et al*. (2011), to avoid heave occurred at the surface of the kaolin clay specimen and also to minimize disturbance, replacement method was chosen to remove the clay to create hole for the bottom ash column to be installed. This is because, from pilot test using displacement method to create hole, the soil was pushed laterally causing heave, hence disturbed the specimen. Drill bits with desired diameter were used to create the hole for the bottom ash installation. Figure 2.30 shows the drilling process using drill bit to create hole for installation of bottom ash column. Since every batch of soft kaolin clay from the CBR mould produced three specimen of 50 mm in diameter and 100 mm in height. One of the batch are used as the "control specimen" with no bottom ash column installed in the specimen. Whereas, another batch of specimen was installed with partially penetrating columns of 60 mm height of bottom ash columns and third specimen was installed with fully penetrating column of 100 mm height of bottom ash columns.

Figure 2.30: Drilling process using drill bit to create hole for installation of bottom ash column (Hasan *et al.*, 2011)

Further, the bottom ash had been densified by pouring it into the pre- drilled hole by free fall from a predetermined height. To avoid any major void created between the bottom ashes, smooth surface from the backside of the drill bit used to drill the hole was used to gently compact the bottom ash. This process is done delicately to minimize the disturbance towards the specimen. Figure 2.31 shows the installation of singular bottom ash column. For the singular columns, the columns was installed at the center of the specimen, while for the group column, square pattern was chosen because it is much easier to maintain the location of the column that will be installed especially in terms of spacing in between the column. Figure 2.32 shows the kaolin clay specimen reinforced with singular and group bottom ash column.

Figure 2.31: Installation of singular bottom ash column (Hasan *et al.*, 2011)

Figure 2.32: Clay specimen reinforced with singular and group bottom ash columns (Hasan *et al.*, 2011)

The previous researcher, Maakaroun *et al.* (2009) and Black *et al.* (2007) has been using the frozen method to apply on their respective research on stone column. The frozen sand columns of difference diameters and lengths were installed in the pre-drilled holes. Figure 2.33 is the predrilled of 3 cm diameter hole for sand column. The sand columns were prepared (prior to freezing) by pouring three layers of dry Ottawa sand in circular geosynthetic fabrics which were inserted in a glass tube having the same inner diameter as the sand column and vibrated using electric vibrator. After vibration of the column, water was added to the sand column through the geosynthetic encasement to achieve the water content. The sand column was then allowed to freeze at -4° for 24 hours after which the geosynthetic fabric was cut and detached from the sand column. The frozen sand column was then inserted into the predrilled hole and allowed to thaw. Figure 2.34 shows the insertion of frozen sand column in clay.

Figure 2.33: Pre-drilled of 3 cm diameter hole (Maakaroun *et al.*, 2009)

Figure 2.34: Insertion of frozen sand column in clay (Maakaroun *et al.*, 2009)

CHAPTER 3

METHODOLOGY

3.1 Introduction

This chapter describes the laboratory tests that had been conducted in order to fulfill the objectives of the study. The flow of the works and activities is shown in Figure 3.1. The focus of the study was to study the strength of the soft clay in reinforced with group encapsulated bottom ash column. Firstly, it is important to identify the problem of the study, so that at the end of the study, the objectives are achieved. Next, the previous studies regarding the topic were reviewed carefully, to recognize the previous efforts from other researcher in dealing with the problems known.

After that, the project methodology is created in order to make a proper flow of the project. The methodology consists of various type of laboratory tests, to determine the physical and mechanical properties of kaolin and bottom ash, as well as to test the strength of the soft clay in reinforced with group encapsulated bottom ash column. The test for kaolin and bottom ash are included Atterberg Limit test, specific gravity test, hydrometer test, sieve analysis test, direct shear test, falling head and constant head permeability test, standard compaction test and relative density test. While, the laboratory work for main test is sample preparation and unconsolidated undrained (UU) test. Data collection is crucial for further analysis of the changes in the physical and mechanical properties. Prediction and conclusions are done based on the analysis of the data. The flow chart of the study are shown in Figure 3.1.

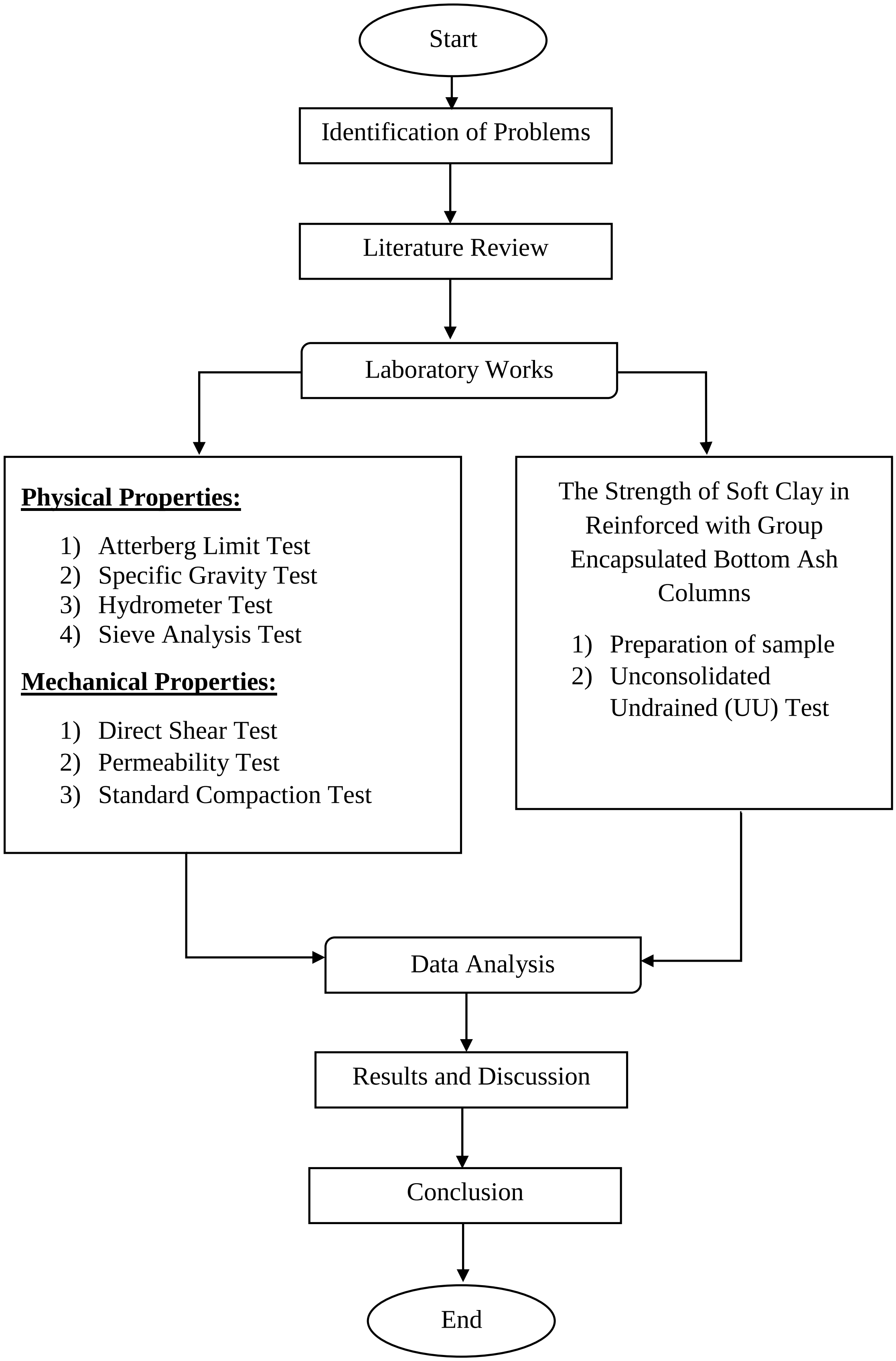

Figure 3.1: Flow Chart of the Project Methodology

3.2 Laboratory Test

The laboratory works have been conducted to determine the physical and mechanical properties of the kaolin and bottom ash. In addition, the strength properties of the soft clay in reinforced with group encapsulated bottom ash column have been identified through the laboratory test. Details of the testing materials, the testing methods and the experiment procedure are discussed in this chapter. The laboratory works and the method standard are shows in Table 3.1.

Table 3.1: Summary of Laboratory Testing

Material	Laboratory Test	Standard/ Reference
Kaolin	Atterberg Limit Test	BS 1377: Part 2: 1990: 4.3 & 5.3
	Hydrometer Test	BS 1377- Part 2:1990: 9.5
	Specific Gravity Test	BS 1377- Part 2:1990: 8.3
	Standard Proctor Test	BS 1377- Part 4:1990: 3.3
	Falling Head Permeability Test	ASTM D 2434
Bottom Ash	Specific Gravity Test	BS 1377- Part 2:1990: 8.3
	Standard Proctor Test	BS 1377- Part 4:1990: 3.3
	Sieve Analysis Test	BS 1377- Part 2:1990: 9.3
	Direct Shear Test	BS 1377- Part 4:1990
	Relative Density Test	ASTM D 4253
	Constant Head Permeability Test	ASTM D 2434
Soft Clay in Reinforced with Group Encapsulated Bottom Ash Column	Unconsolidated Undrained (UU) Test	ASTM D 2166

3.3 Preliminary Test of the Soil.

The preliminary test is the test that held to find the basic properties and mechanical properties of the kaolin clay and bottom ash. In this study, the kaolin powder grade S300 bought from Kaolin (M) Sdn. Bhd are been used as a sample of soft clay soil. While, for the bottom ash that used in this test are collected from Tanjung Bin Power Plant in Johor, Malaysia.

Figure 3.2: Kaolin Powder bought from Kaolin (M) Sdn. Bhd

To get the basic properties and mechanical properties for the sample, there are various test and experiment are held for the kaolin specimen and bottom ash which is being used in this research. The preliminary test are divided into two, which are the test to determination physical properties and the test to determine the mechanical properties.

3.3.1 Laboratory Test for Determination of Physical Properties

There are six (6) laboratory tests has been carried out to determine the physical properties of kaolin and bottom ash, including;

i. Three (3) tests for kaolin: Atterberg Limit : Liquid Limit (LL) test and Plastic Limit (PL) test, Specific Gravity test, Particle Size Distribution; Hydrometer test and;

ii. Three (3) tests for bottom ash: Specific Gravity test, Particle Size Distribution: Sieve Analysis test, and Relative Density test.

3.3.1.1 Specific Gravity Test

Based on the British Standard (BS) 1377: Part 2 1990, the principle of specific gravity is the specific gravity is the ratio of the mass unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. The specific gravity test conducted is to determine the average specific gravity (G_s) that used to get the weight-volume relationship. Besides, the specific gravity of soils is used in the phase relationship of air, water and solid in a given volume of the soils. To determine the specific gravity of the soil, the small pyknometer method are been used.

Specific gravity, $$Gs = \frac{w2 - w1}{(w4 - w1) - (w3 - w2)} \qquad \textbf{(3.1)}$$

Figure 3.3: Sample preparation for small Pyknometer test

The soft soil sample which is passing through a 2mm sieve were used for the test. The sample are inserted into the small pyknometer and the distilled water are added then the small pyknometer were placed in vacuum desiccator as in Figure 3.4 for approximately one hour. Average value of the measurement was obtained and the specific gravity was calculated using the formula shown. The same test had been carried out for bottom ash.

Figure 3.4: Small pyknometer in vacuum desiccator

3.3.1.2 Particle Size Distribution Test.

As defined in BS 1377: Part 2 1990, the method is used to determine the particle size distribution in order to check the adequate content of materials passing 63 µm. The particle distribution test is conducted by using two type of test which is sieve analysis test and hydrometer analysis test. Sieve analysis method is applicable for particle size larger than 0.075 mm in diameter where for the hydrometer test is for the particle that having sizes smaller than 0.075 mm in diameter. In this study, the sieve analysis test are conducted to find the particle distribution for bottom ash. Whereas, to determine the particle distribution for kaolin, the hydrometer test are carried out.

3.3.1.2.1 Sieve Analysis Test

The sieve analysis is the test to determine the relative proportions of different grain sizes as they are distributed among certain size ranges. Usually, sieve analysis test is used to determine the distribution of the larger grain size. The grain size distribution curve of soil samples is determines by passing the soil sample through a stack of sieve of decreasing mesh-opening sizes and measure the weight retained on each sieve. The size for the sieve used in this test are 10mm, 5mm, 2.36mm, 1.18mm, 0.6mm, 0.3mm, 0.15mm and the pan. The whole nest of sieve with receiving pan is placed in the shaker and the dried soil is placed in the top sieve. The top of the lid are fitted with the lid and the sieve are sieving by the mechanical shaker for 10 minutes. In Figure 3.5 shows the sieve analysis test held in the sieve shaker. The mass retained on the first sieve is subtracted from the initial mass to give the mass passing the first sieve. The mass retained on each subsequent sieve is subtracted from the mass passing the previous sieve to give the mass passing each sieve.

Figure 3.5: Sieve analysis test

3.3.1.2.2 Hydrometer Analysis Test

Fine grained soil is the test held to determine the grain size distribution of material passing the 63 µm, and the hydrometer analysis method are commonly used. The soil is mixed with the water and a dispersing agent, stirred vigorously and allowed to settle to the bottom of a measuring cylinder. As the soil particles settle out of suspension the specific gravity of the mixture reduces. A hydrometer is used to record the variation of specific gravity with time. By making used of Stoke's law, which relates the velocity of a free falling sphere t its diameter, the test data is reduced to provide particle diameter and the percentage by weight of the sample finer than a particular particle size.

Figure 3.6: The Hydrometer Test

The sedimentation cylinder is used and filled with 100ml dispersant solution and made up t 1000ml distilled water and then is placed in the constant-temperature bath set at 25 degree Celsius. The equivalent particle diameter, D are calculate from the equation;

$$D = 0.005531 \sqrt{\frac{\eta H}{(\rho S - 1)t}} \qquad \textbf{(3.2)}$$

Where:

η = the dynamic viscosity of water at the test temperature as shown in table below,

Table 3.2: The viscosity of water

Temperature, T (°C)	**Viscosity of water (mPa.s)**
10	1.304
15	1.137
20	1.002
25	0.891
30	0.798

H= the effective depth at which the density of the suspension is measured (mm)

ρ = the particle density (Mg/m^3)

t = the elapsed time (min)

0.005531 = the constant value

The true Hydrometer reading, R_h (mm) get from the equation;

$$R_h = R_h' + C_m \qquad \textbf{(3.3)}$$

Where:

C_m is the meniscus correction

R_h is the observed hydrometer reading

The modification hydrometer reading, R_d get from the equation;

$$R_d = R_h' - R_o' \qquad \textbf{(3.4)}$$

Where:

R_o is the hydrometer reading at the upper rim of the meniscus in the dispersion solution. The final result is from the calculation of percentage from the equation below and the particle distribution of soil graph are plotted.

$$K = \left[\frac{100\rho S}{m(\rho s-1)}\right]Rd \qquad \textbf{(3.5)}$$

3.3.1.3 Atterberg Limit Test

To describe the consistency of fine-grained soils which is clay with varying moisture content, a Swedish scientist name Atterberg are developed a method that named as Atterberg Limits. The soil have characteristic that when the moisture content are very low, it behaves like a solid, whereas when the moisture content are high, the soil and water can flow like a liquid. Therefore, the behavior of soils can divided into four basic state – solid, semisolid, plastic and liquid. To classify the behavior of the soil, the Atterberg limit test is conducted and this test consist of three test; plastic limit (PL) test, liquid limit (LL) test and shrinkage limit (SL) test.

3.3.1.3.1 Liquid Limit Test (LL)

One of the popular method to determine the liquid limit of the soil is fall cone method. In this test, the liquid limit is defined as the moisture content and expressed in term of percentage. The equipment for this test consist of a cylindrical cone with apex angle 30 degree and have the sharp and smooth polished surface. The sharp point of the cylindrical cone are penetrate perpendicular at the surface of the soil with the total mass of the cylindrical allowed to fall freely is 0.78N. The liquid limit of the soil is taken as the moisture content at a penetration of 20mm in 5 second when allowed to drop from a position its contact with the soil surface. A Figure 3.7 shown the fall cone test. Due to the difficulty in achieving the liquid limit from a single test, fur or more tests can be conducted at various moisture contents to determine the fall cone penetration, *d*.

Figure 3.7. Fall cone method of liquid limit test.

3.3.1.3.2 Plastic Limit (PL)

Plastic limit is the lower limit of the plastic stage of soil and it represent the moisture content at which soil changes from plastic to brittle state. The plastic limit often used together with the liquid limit to determine the plasticity index (PL) which when plotted against the liquid limit on the plasticity chart provides a means of classifying cohesive soils. The test is of plastic limit is simple and is performed by rolling a thread of 3.2mm ellipsoidal-sized soil mass by hand on a glass plate until its crumbles. The sample will reflects as wet side of the plastic limit if the thread can be rolled in diameter of below 3mm and the dry side if the thread breaks up and crumbles before it reaches 3mm diameter.

3.3.1.3.3 Plasticity Index.

The plasticity index is very important in classifying the fine-grained soil. The difference between plastic limit (PL) and liquid limit is calculated to get the plasticity index of the soils sample.

$$\text{Plasticity Index (PI)} = \text{Liquid Limit (LL)} - \text{Plastic Limit (PL)} \qquad \textbf{(3.6)}$$

If it is not possible to conduct the plastic limit test, the soil is reported as non-plastic. This also applied while plastic limit is equal to or greater than liquids limit.

3.3.1.4 Relative Density Test

Relative density test methods cover the determination of the maximum and minimum dry density of the bottom ash by using the vibrating table as shown in Figure 3.8. Four alternatives are provided by ASTM D 4253, which is Method 1A, 1B, 2A and 2B. Method 2A, which is using the oven dried soil and an eccentric or cam- driven, vertically vibrated table was selected to determine the relative density of bottom ash. The size of bottom ash used was in the range between 0.6 mm and 2.36 mm. the bottom ash was placed in typical still mold, followed by putting a surcharge plate on the bottom ash. Subsequently, the vibrator vibrated the bottom ash for 10 minutes with the frequency 50 Hz. Maximum and minimum index densities were first determined, followed by the relative density.

Figure 3.8: Relative density test equipment

3.3.2 Laboratory Test for Determination of Mechanical Properties

There are five (5) laboratory tests has been carried out to determine the mechanical properties of kaolin and bottom ash, including;

i. Three (3) tests for kaolin: Atterberg Limit : Liquid Limit (LL) test and Plastic Limit (PL) test, Specific Gravity test, Particle Size Distribution; Hydrometer test and;

ii. Three (3) tests for bottom ash: Specific Gravity test, Particle Size Distribution: Sieve Analysis test, and Relative Density test.

3.3.2.1 Standard Compaction Test

The solid particles are packed more closely together when the soil sample are compacted together with the mechanical, thus increasing the soil density while air is being removed. According to the BS 1377:1975, the size of the individual soil particles does not change, neither the water removed. The soil are fully saturated, when the percentage of the air voids is zero. Increasing the water content for a saturated soil results in a reduction in dry unit weight. The relation between the moisture content and dry unit weight for saturated soil is known as the zero air void line.

The standard proctor test is to determine the optimum water content and the maximum dry density that can be achieved with a certain compaction effort. From this test, the relationship between the moisture content and the density of the soil are obtained.

The standard proctor test are held for kaolin and bottom ash. For the bottom ash, the particle size was 0.6 mm to 2.36 mm, while for kaolin, the particle size is passing 2 mm was used for the test. The test was carried out by putting the kaolin in 1 liter of compaction mold in three layers. Each layer had been compacted by applying 25 blow of free fall of the 2.5kg rammer. The bottom ash also test by the same procedures. Figure 3.9 shows the typical apparatus of the standard compaction test.

Figure 3.9: Typical apparatus of standard compaction test

3.3.2.2 Permeability Test

Permeability of a soil is measure of its capacity to allow the flow of a fluid through the soil. There are two types of permeability test, known as the constant head permeability test and falling head permeability and falling head permeability test. Constant head permeability test was used to measure the coefficient of permeability of the bottom ash in which the particle size falls in the range of 0.6 mm to 2.36 mm. the purpose of conducting the constant head permeability cell was to test bottom ash which is compacted into a cell by using a specified compactive effort to achieve 1.34 Mg/m^3. The sample was prepared together with the permeameter cell, with water flowing through the sample. Quantity of water (liter) flown was collected and the flow rate was calculated. The permeability of the soil can be calculated by dividing the flow rate with the area of the sample.

Falling head permeability test was used to measured coefficient of permeability of kaolin. The procedure of this test ate referred to Head, (1992). The purpose of conducting the falling head permeability was to determine the kaolin clay which was compacted into the cell by using specified compactive effort to achieve certain dry density. The assembled cell was placed in the immersion tank for 24 hours in order to obtain a saturated sample. Next, the water flow was allowed to pass through the sample, and the time of the test was observed. The water level in standpipe was recorded after 10 minute of the test.

3.3.2.3 Direct Shear Test

Direct shear test is done to determine the relationship between the shear stress at failure and normal stress. Three samples of bottom ash had been tested by using 60 mm shear box method. Through this test, the angle of shear resistance was obtained. The bottom ash with particle size in the range of 0.6 mm to 2.36 mm was used for the test. Figure 3.10 shows the direct shear box test equipment.

Figure 3.10: Direct shear test equipment

3.4 Reinforcing Soft Clay with a Group Encapsulated Bottom Ash Columns

3.4.1 Preparation of Sample

The sample of kaolin clay reinforced with a group encapsulated bottom ash columns need to be prepared before the main test are conducted. To prepare the sample, kaolin was air dried first, after that mixed with 20% of water, which the optimum moisture content of kaolin is obtained from the standard compaction test. After the water and kaolin are uniformly mixed, the kaolin are become soft and wet. About 152 g of the wet kaolin was required to fill the customized mould to create one test specimen. The specimen were prepared by applied compaction method using customized mould as shown in Figure 3.11. The kaolin are divided into three part equally to pour into the customized mould in three layers. Each layer was compacted with five free fall blows by the customized steel extruder. The customized mould was designed so that the amount of kaolin clay used inside the mould will be compressed into a same diameter and height which is 38 mm and 76 mm. By the uniformity of the kaolin mass and the volume of the mould, the volume and dimension of each kaolin specimen could be maintained.

Figure 3.11: Customized mould for 38 mm diameter and 76 mm height of specimen

3.4.2 Installation of a Group Encapsulated Bottom Ash Columns

There are three specimen are prepared for each batch of the kaolin with the same diameter and height, which are 38 mm and 76 mm. Each batch of kaolin contains difference diameter and height of bottom ash columns. Unconsolidated Undrained test was applied to test every batch with the difference pressure for three specimen in each batch.

In order to prepare for the installation of encapsulated bottom ash columns, the kaolin specimen need a hole. Therefore, there are three hole was drilled using drill bit of respective diameter for each of the kaolin specimen when the specimen still inside the mould to prevent the soft clay to hardly distributed and expansion as shown in Figure 3.12. Then, the geotextile fabric are inserted into the hole before bottom ash are placed. Through the result of several pilot tests, it was decided that the raining method was the best way to create the homogeneous bottom ash column in clay specimen.

Figure 3.12: Specimen in the mould was being drilled

Each batch of the specimen have difference diameter and height of bottom ash column which are effect the height penetrating and area penetrating ratio of each batch. Figure 3.13 shows the detail arrangement of the group column with difference area replacement ratio. While, Figure 3.14 shows the difference height of column arrangement. Therefore, the mass of bottom ash inserted into the encapsulated column, the mass of bottom ash are difference. Table 3.3 are shown the variables of bottom ash installation.

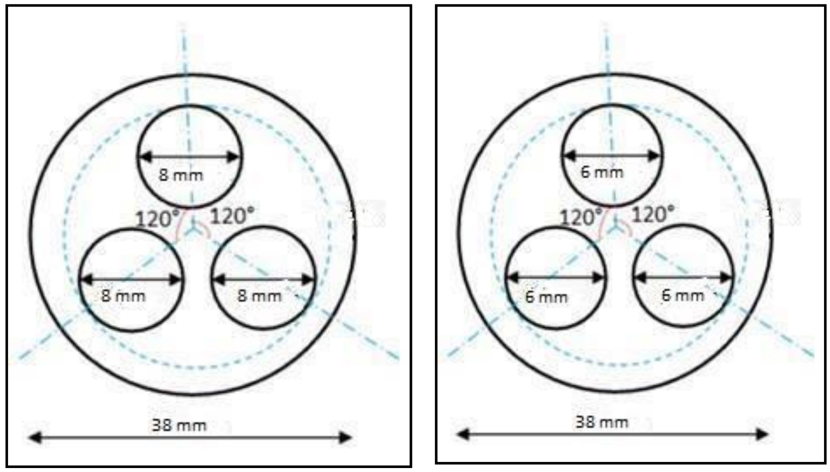

Figure 3.13: Detail arrangement of group column with difference area replacement ratio

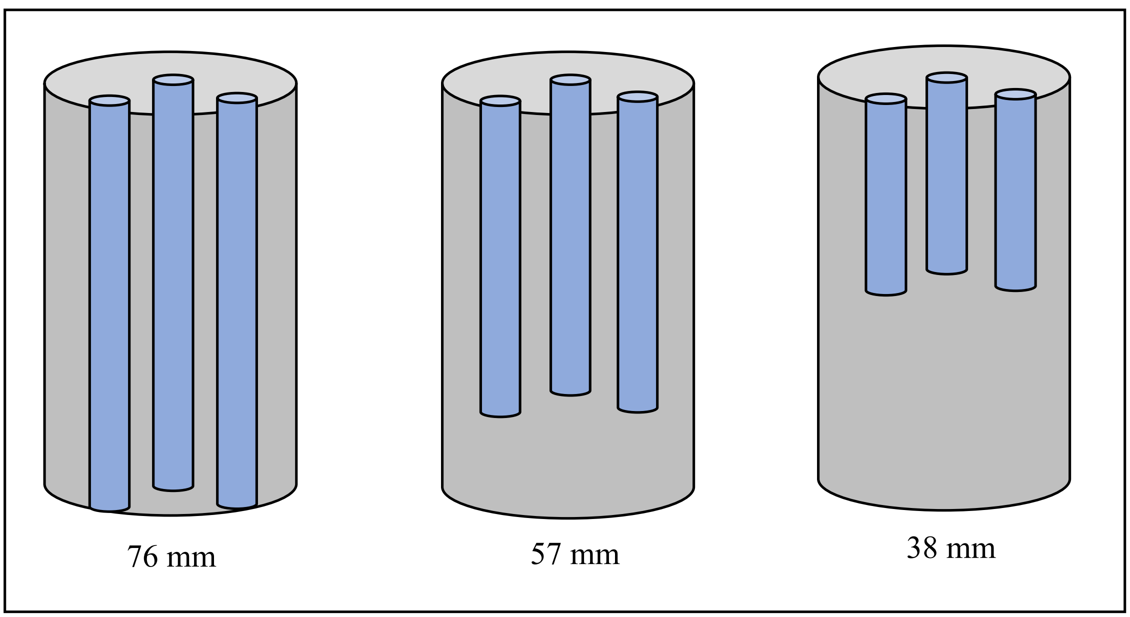

Figure 3.14: Detail arrangement of group column with difference height penetration ratio

Table 3.3: Sample with variables of bottom ash installation

BATCH	COLUMN DIAMETER, (mm)	COLUMN HEIGHT, (mm)	COLUMN HEIGHT PENETRATING RATIO, H_c/H_s	COLUMN AREA PENETRATING RATIO, A_c/A_s	MASS OF BOTTOM ASH (g)
1	6	38	0.5	7.5	0.38
2		57	0.75		0.49
3		76	1.0		0.89
4	8	38	0.5	13.29	1.00
5		57	0.75		1.56
6		76	1.0		1.90

3.4.3 Non- woven Geotextile

The Polyester Non-woven Geotextile Needlepunched Fabric (MTS 130) has been used to encase the kaolin clay reinforced with bottom ash columns. It is produced by Malaysia Company called MTS Fibromat under the brand Fibrotex. The main reason of the encasement using this type of non- woven fabric is because of its unique properties that helps to increases the soil shear strength by providing bonding mechanism of the geotextile soil system in order to improve the quality of soil and the structural stability. The properties of chosen geotextile was made from highest quality of virgin fibers which form a strong fabric designed. The non- woven fabric have a cushion or protection properties which allows permanently protection of synthetic sealing systems against any mechanical damage during the installation and after completion the construction. Figure 3.15 shows the non-woven geotextile with difference diameter and height that used in this study.

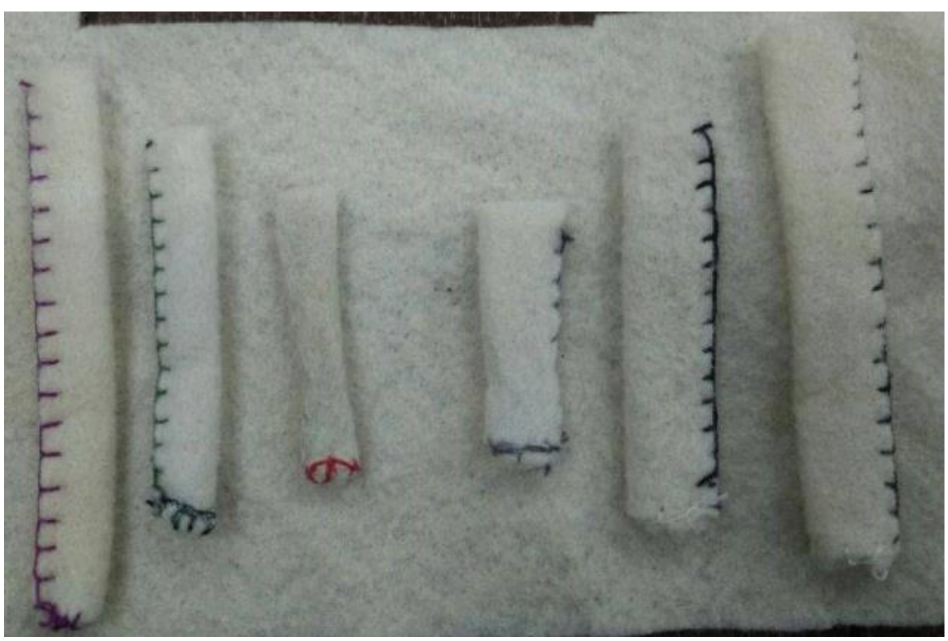

Figure 3.15: Non- woven geotextile with difference diameter and height

3.4.4 Unconsolidated Undrained Test

The Unconsolidated Undrained (UU) test is the method to determine the shear strength of the soil for cohesive soil. While, the specimen is shared at constant rate of axial deformation until failure occurs. The objective of the test are to determine the shear strength of cohesive soil and to observe the mode of failure of the soil specimen.

To determine the strength parameter of the soft clay reinforced with group encapsulated bottom ash column, six batch of sample are prepared. Each of the batch have difference height and area penetration ratio. Each batch consist of three sample that have the same diameter and height of bottom ash column. Three difference pressure are applied to the sample for each batch, which are 50 kPa, 100 kPa and 200 kPa. In Figure 3.16 shows the setting up equipment for UU test. To prevent the kaolin clay from the damages and minimize the disturbance, the test need to handle in high of caution and carefully. Figure 3.17 is the sample of kaolin clay reinforced with group encapsulated bottom ash columns are tested by UU test.

Figure 3.16: Setting up of Unconsolidated Undrained test

Figure 3.17: The tested of Kaolin clay in reinforced with group encapsulated bottom ash columns

CHAPTER 4

RESULT AND DISCUSSION

4.1 Introduction

This chapter discuss all the laboratory test result conducted on kaolin S300 taken from Kaolin Malaysia and bottom ash taken from Tanjung Bin power plan to fulfill the objective of the study. The laboratory tests carried out to identify the physical and mechanical properties of kaolin included Atterberg Limit test, Specific Gravity test, Hydrometer test, Compaction test, and Falling Head Permeability test. Meanwhile, the test conducted for bottom ash is Specific Gravity test, Particle Size Distribution test, Compaction test, Relative Density test, Direct Shear test and Constant Head Permeability test. Apart from the discussion of the engineering properties of the materials used in this study, the Unconfined- Undrained (UU) Triaxial test has been conducted on the six (6) batch of soft clay in reinforced with group encapsulated bottom ash column with two difference size of diameter and three difference penetration ratio in order to identified the increment of shear strength are discussed in this chapter as well.

4.2 Summary of Kaolin, Bottom Ash and Geotextile Properties

Table 4.1 and Table 4.2 shows the summary of the bottom ash and kaolin properties from the conducted test. Based on all the required test conducted on kaolin, it was proven that the kaolin clay had the similarities characteristic with the soft clay. Meanwhile, the Tanjung Bin bottom ash had proven that its characteristic are relatively similar with the sand and fine gravel. Therefore, the Tanjung Bin bottom ash has the huge potential to be used as the replacement material to natural materials such as sand for granular column. Another summary of non-woven geotextile was tabulated in the Table 4.3 shows the information on the properties of geotextile was produced by Fibrotex as the manufacturer of the geotextile.

Table 4.1: Summary of Tanjung Bin bottom ash properties

Properties	Result
Particle Size Range	2 mm to 0.063 mm
Soil Classification: • AASTHO	 A-1-a
Specific Gravity, Gs	2.33
Standard Compaction Characteristic: • Maximum Dry Density, $\rho_{d(max)}$ • Optimum Moisture Content, w_{opt}	 1.34 kg/m^3 21.75 %
Shear Strength (Direct Shear Test) : • Peak Friction Angle • Peak Cohesion	 23.93° 89.71 kPa
Constant Head Permeability	1.57 x 10^{-3} m/sec

Table 4.2: Summary of kaolin clay properties

Properties	Result
Liquid Limit	41.0 %
Plastic Limit	31.25 %
Plasticity Index	10.05 %
Specific Gravity, Gs	2.62
Standard Compaction Characteristic: • Maximum Dry Density, $\rho_{d(max)}$ • Optimum Moisture Content, w_{opt}	 1.575 kg/m^3 20.00 %
Soil Classification: • AASTHO • USCS (Plasticity Chart)	 A-7-6[b] ML
Falling Head Permeability	1.124×10^{-9} m/sec

Table 4.3: Summary of Polyster Non-woven Geotextile Needlepunched properties (MTS 130)

Properties (Typical)	**MTS 130**
Material	Polyster
Unit Weight, γ	130 g/m^2
Thickness	1.08 mm
Mechanical Properties	**MTS 130**
Max. Tensile Strength, MD	10.0 kN/m
Max. Tensile Strength, CD	9.3 kN/m
Elongation at Max Tensile Strength, MD	56.0 %
Elongation at Max Tensile Strength, CD	84.0 %
CBR Puncture Strength	2.2 kN/m
Trapezoid Tearing Strength, MD	350 N
Trapezoid Tearing Strength, CD	280 N
Index Puncture Strength, MD	310.3 N
Apparent Opening Size	140 µm
Vertical Permeability	0.27 cm/s
Grab Tensile Strength, MD	620.2 N
Grab Tensile Strength, CD	668.0 N

4.3 Physical Properties

4.3.1 Atterberg Limit

Atterberg Limit test was carried out to determine the amount of water needed to achieve a range of states behavior of kaolin. It is known as Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PL). Figure 4.1 shows the graph of penetration versus moisture content. The graph shows that, at 20 mm penetration, the liquid limit is 41% and the plastic limit for kaolin is 31.25%. Through calculation, the plasticity index is 9.75%.

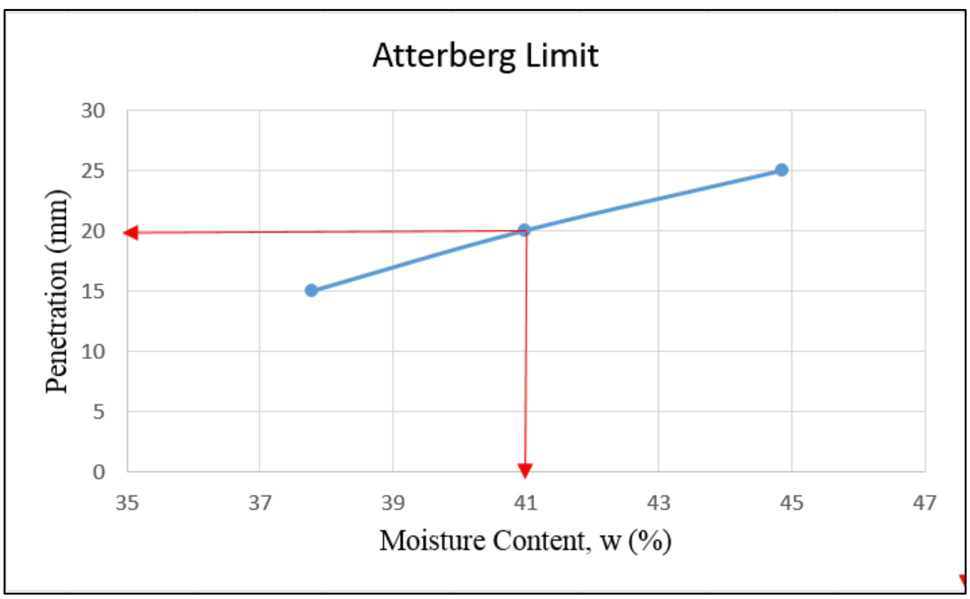

Figure 4.1: Graph of penetration versus moisture content

As depicted in plasticity chart shown in Figure 4.2, kaolin shows liquid limit of 41% and plasticity index of 9.75% which is low plasticity. The red point stated in the figure shows that the kaolin S300 that being used in this study is actually located below the “A line” which is classified as ML (low plasticity silt).

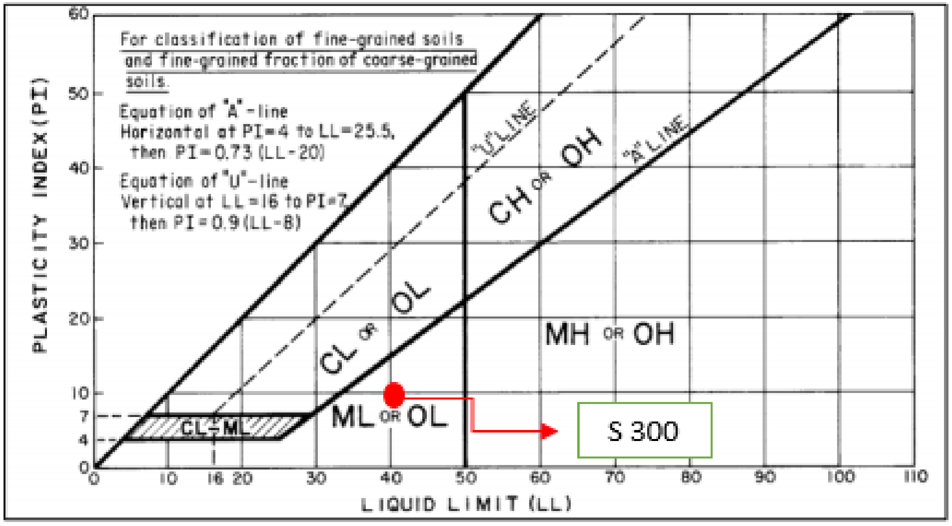

Figure 4.2: Plasticity chart (ASTM D2487)

4.3.2 Specific Gravity

Particle density is defined as the specific gravity of the sample of the soil. In this study, method used to determine the specific gravity of the kaolin ash bottom ash is by using small Pyknometer test. According to Head (1992), generally the specific gravity of particle of most soil lies in between 2.60 – 2.80. The specific gravity of kaolin for this study is found 2.62, which is in the range of particle density of most soil. The previous study by Hasan *et al*. (2011) the specific gravity of kaolinite is 2.65, which is indicated that kaolinite mineral is part of the composite of kaolin.

The specific gravity of bottom ash collected from Tanjung Bin power plant is determine as 2.33. The bottom ash with slighter quantities of porous and popcorn-like particles, it generally shows a higher specific gravity which is as high as 2.8, while a porous or hollow ash may present a specific gravity as low as or even lower than 1.6 (Muhardi *et al*., 2010). Besides that, the specific gravity of bottom ash was reported by Muhardi *et al*. (2010) was varies from 2.0 to 2.6 with an average is 2.35. According to Singh and Siddique (2012), the specific gravity of bottom ash varies from 1.39 to 2.33 depending on its chemical composition.

Table 4.4: Comparison of bottom ash specific gravity values

Researcher	Location	Specific Gravity Value
Current Study (2015)	Tanjung Bin	2.33
Hasan *et al*. (2012)	Tanjung Bin	2.35
Muhardi *et al*. (2010)	Tanjung Bin	1.99
Abdul Talib (2009)	Manjung	2.39
Abdul Talib (2009)	Kapar	2.00

Table 4.4 shows the comparison of specific gravity value of bottom ash collected from difference researcher and different coal power plant. The current study of sample bottom ash was taken from Tanjung Bin power plant located at Pontian, Johor. The result of specific gravity is 2.33 is approximately similar to the result conducted by Hasan *et al.* (2012) which is 2.35. While, the result is relatively higher compare to test conducted by Muhardi *et al.* (2010) with the specific gravity is 1.99.

4.3.3 Particle Size Distribution

The particle size distribution of the kaolin sample was determined by hydrometer test. The procedure of hydrometer analysis is used to measure the density of a soil in a water suspension at the various interval of time. Figure 4.3 shows the result of hydrometer test of kaolin clay. Based on AASTHO classification system, kaolin is classified in Group A-7-6^b which is clayey soils.

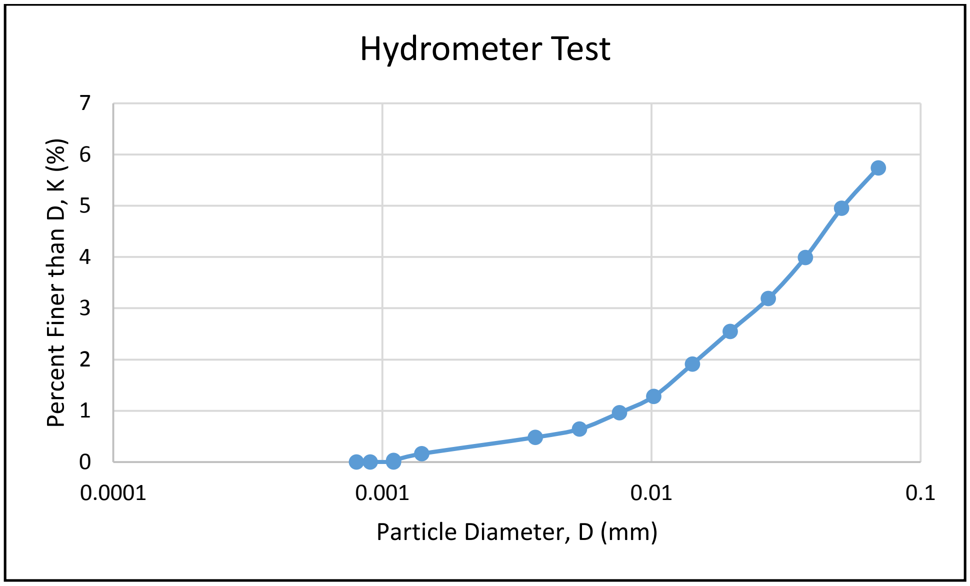

Figure 4.3: Hydrometer Test of Kaolin

Figure 4.4 shows the result of particle size distribution of bottom ash. Particle size distribution for bottom ash were performed using sieve analysis in accordance with dry sieving method. A significant friction of bottom ash sizes was found to be in a range between 10 mm to 0.063 mm, which are falls in the range of fine gravel to fine sand sizes. The shapes of gradation curve in Figure 4.4 indicated that the sample of bottom ash were well graded and relatively similar size distribution. According to Unified Soil Classification (USCS), the bottom ash is classified as well graded sand. While, based on classification by AASHTO system, bottom ash fall in the A-1 group and classified as A-1-a which consist of stone fragments with a well graded binder of fine material.

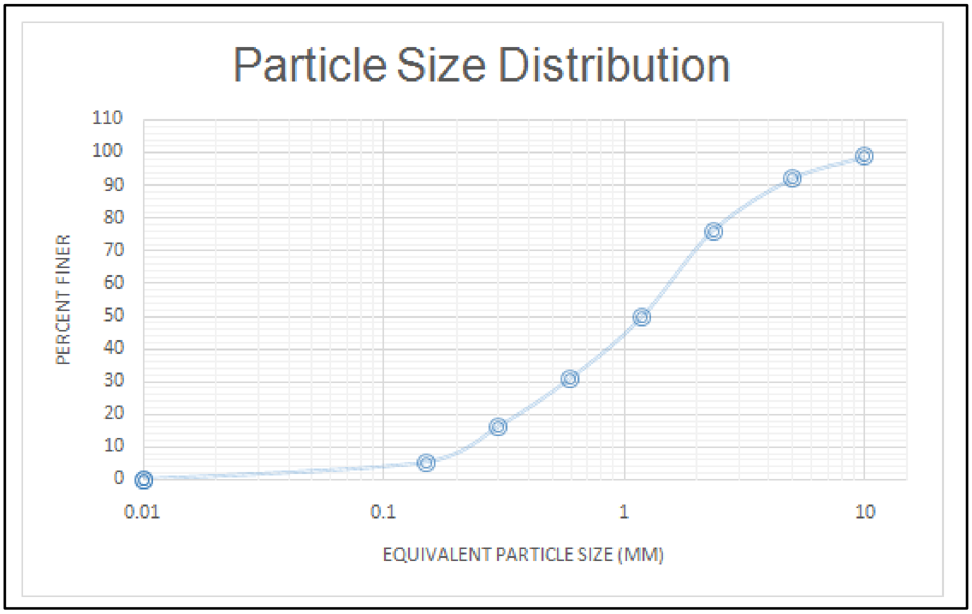

Figure 4.4: Particle Size Distribution of bottom ash from Tanjung Bin power plant

4.3.4 Relative Density

The density of Tanjung Bin bottom ash was determined by using a vibratory table. The minimum density, $\rho_{(min)}$ for bottom ash was 0.868 Mg/m^3, while the maximum density, $\rho_{(max)}$ for bottom ash was 1.004 Mg/m^3. The relative density for bottom ash from Tanjung Bin was 98%. According to Head (1992), the density used is falls in the range of very loose in compaction. Thus, apart from its major function in increasing the shear strength of the soft soil, it can be used as the vertical drainage as well.

4.4 Mechanical Properties

4.4.1 Standard Proctor

The standard proctor compaction test is to determine the optimum water content and the maximum dry density that can achieved with a certain compaction effort. In this study, the relationship between optimum water content with dry density of kaolin and bottom ash are shown in Figure 4.5 and Figure 4.6 respectively. The result for kaolin shows that the maximum dry density, $\rho_{d\ (max)}$ is 1.58 Mg/cm^3 (15.50 kN/m^3) with the optimum moisture content, $w_{(opt)}$ 20%. Whereas, for bottom ash from Tanjung Bin power plant, the maximum dry density, $\rho_{d\ (max)}$ is 1.34 Mg/cm^3 and the optimum water content, $w_{(opt)}$ is 21.75%. The compaction curve of bottom ash are generally in a flat shape, indicating that the properties of bottom ash insensitivity to water content.

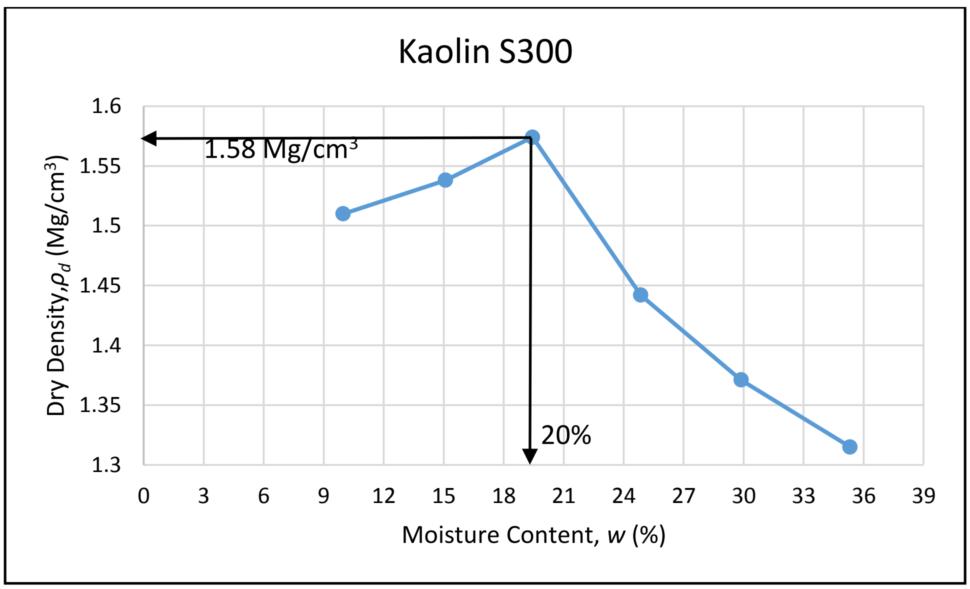

Figure 4.5: Standard proctor compaction graph of Kaolin S300

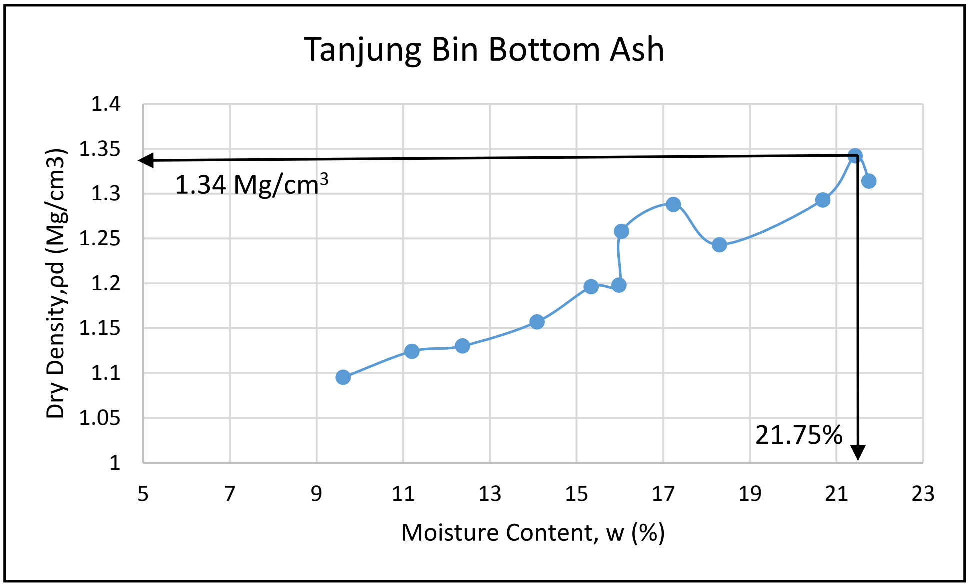

Figure 4.6: Standard proctor compaction graph of Tanjung Bin bottom ash

In the previous study, Muhardi *et al*. (2010) state the optimum moisture content, $w_{(opt)}$ and maximum dry density $\rho_{d\,(max)}$ of Tanjung Bin bottom ash is 21.5% and 1.31 Mg/m^3 (12.85 kN/m^3) respectively. According to previous researcher, the range of maximum dry density is 1.20 to 1.60 Mg/m^3 and the optimum moisture content, *w* is 12% to 24%. The variation of compaction characteristic is mainly due to different low in specific gravity and a high void content (Muhardi *et al*., 2010).

4.4.2 Permeability

The value of permeability coefficient of kaolin obtained from the falling head permeability test. The falling head permeability test is used to measure the permeability of soils of the intermediate and low permeability of soil. In this study, the value of permeability coefficient was 1.124 x 10^{-9} m/sec at the maximum dry density 1.58 Mg/m^3. The value obtained for kaolin shows the impermeable behavior of kaolin and indirectly indicated its poor drainage characteristic, which generally correspond to the clay soil (Head, 1992)

The permeability coefficient of Tanjung Bin bottom ash was measured by a constant head test using a permeameter. The measured of permeability coefficient was 1.57 x 10^{-3} m/sec at the maximum dry density is 1.34 Mg/m^3. The measured coefficient of permeability is indicated that the bottom ash exhibits the permeability approximately corresponding to the clean sand or gravel and it is comparable to those of well graded sand or gravel soils (Muhardi *et al*., 2010). Referring to Muhardi *et al*. 2010, the permeability coefficient of Tanjung Bin bottom ash is 1.72 x 10^{-4} at maximum dry density 1.31 Mg/m^3. The value of permeability coefficient is lower than current study due to the lower maximum dry density was being used. Since the gradation of bottom ash and sand are similar, they tend to exhibit similar permeability (Muhardi *et al*., 2010)

4.4.3 Direct Shear Strength

Figure 4.7 shows the graph of the maximum shear stress on normal stress applied, from the direct shear strength test. Based on the test, the internal friction angle and cohesion of Tanjung Bin bottom ash were 23.93° and 89.71 kPa respectively. The internal friction angle and cohesion are higher compared to the previous studies. Muhardi *et al.* (2010) stated that the internal friction angle of Tanjung Bin bottom ash was 32° while the cohesion was 3.8 kPa. Huang (1990) stated that the friction angle of bottom ash falls in the category of higher natural sandy soil. it is due to the shear strength of bottom ash is attributed to the angularity and rough surface texture that develop a high degree of the interlocking, which is tend to resist inter-particle sliding.

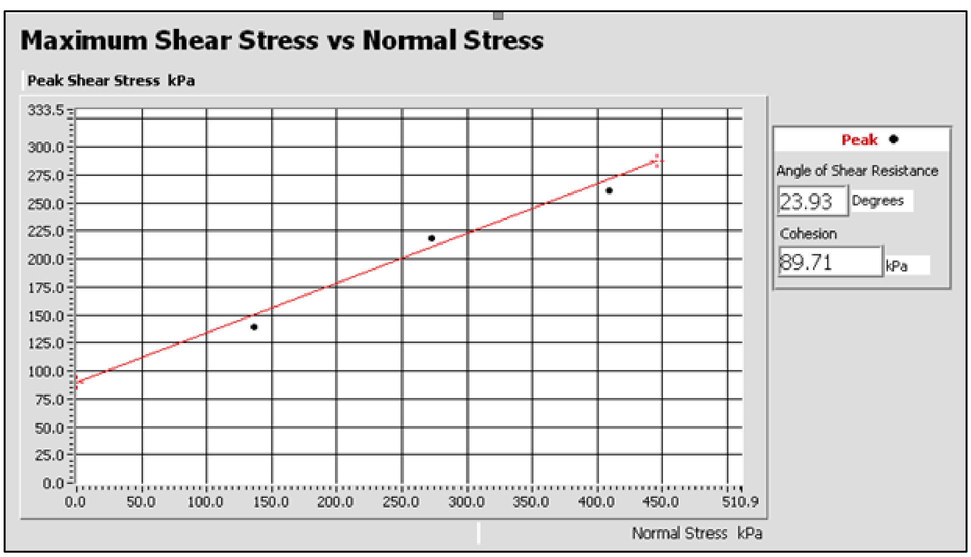

Figure 4.7: Graph of shear stress versus normal stress

4.5 Unconsolidated Undrained Triaxial Test

4.5.1 Shear Strength Parameter

Shear strength parameter of the soft clay in reinforced with the group encapsulated bottom ash columns was measured by conducting the Unconsolidated Undrained Traixial (UU) Test. A six (6) batch of the soft clay reinforced with the group encapsulated bottom ash are tested using UU test. All of the batch have two difference diameter and three difference height of bottom ash column.

Each of the test result are interpreted from the deviator stress on strain and the Mohr's circle. The deviator stress versus strain and the Mohr's circle for 6 mm diameter with the height 38 mm, 57 mm and 76 mm of bottom ash column are shown in Figure 4.8(a), Figure 4.9(a), Figure 4.10(a), and Figure 4.8(b), Figure 4.9(b), Figure 4.(10) respectively. Meanwhile, for 8 mm diameter with the height 38 mm, 57 mm and 76 mm of bottom ash column, the deviator stress versus strain and Mohr's circle are shown in the Figure 4.11(a), Figure 4.12(a), Figure 4.13(a) and Figure 4.11(b), Figure 4.12(b), 4.13(b) respectively. Each of the batch shows the deviator are increasing when the strain are increase. The Mohr's circle result are shows the cohesion and the friction angle for each of the batch sample.

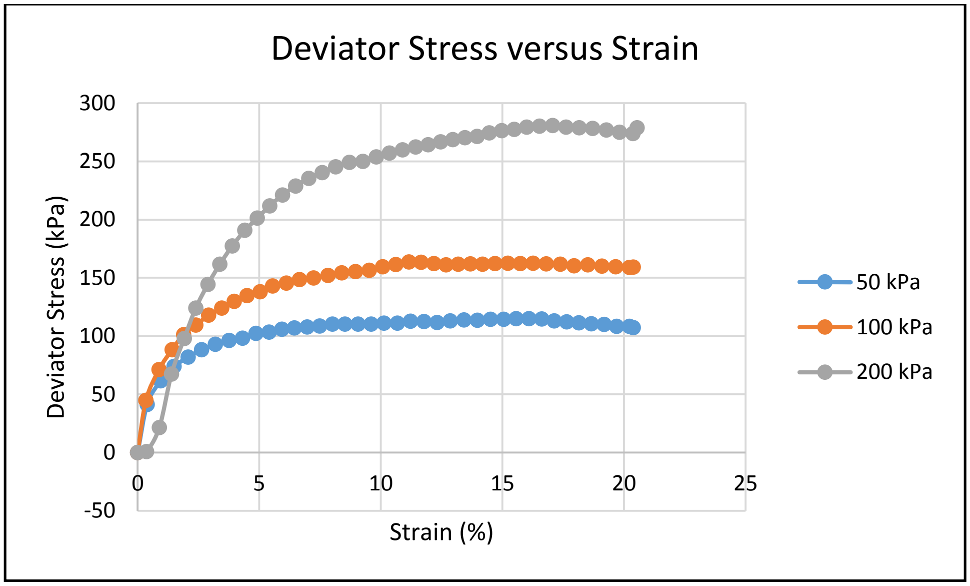

Figure 4.8(a): The deviator stress versus strain for bottom ash with height 38 mm and diameter 6 mm

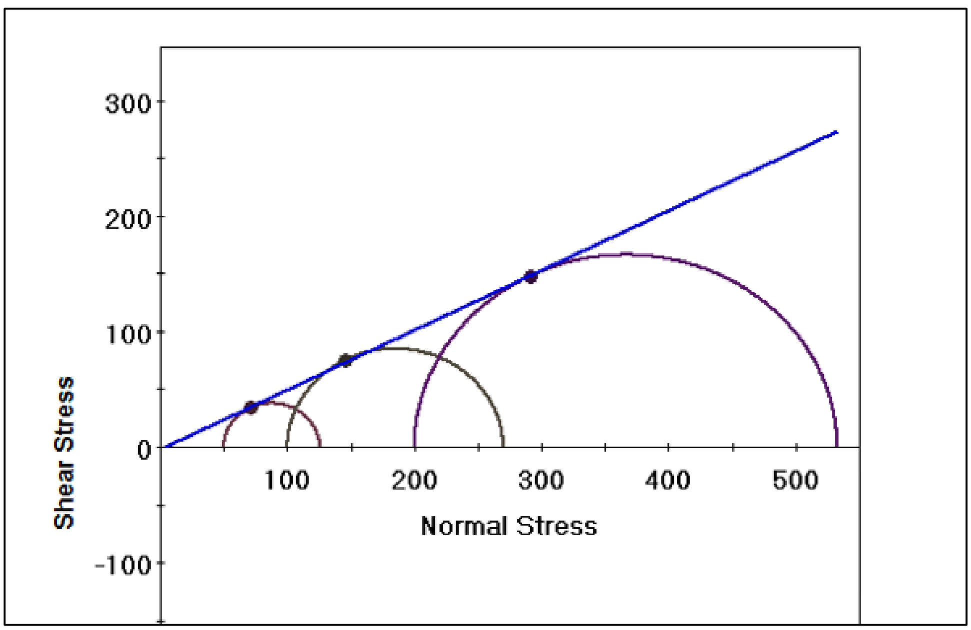

Figure 4.8(b): The Mohr's circle for bottom ash with height 38 mm and diameter 6 mm

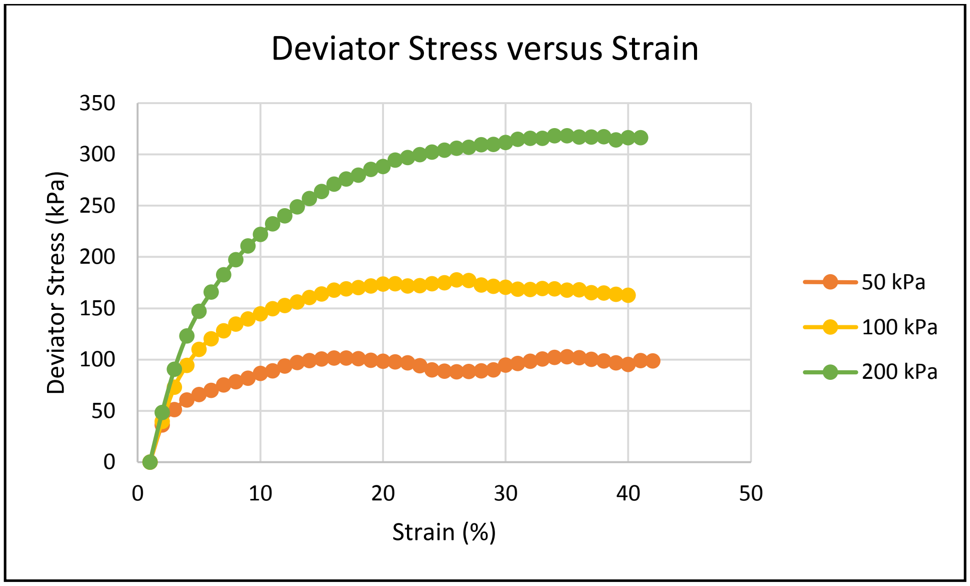

Figure 4.9(a): The deviator stress versus strain for bottom ash with height 57 mm and diameter 6 mm

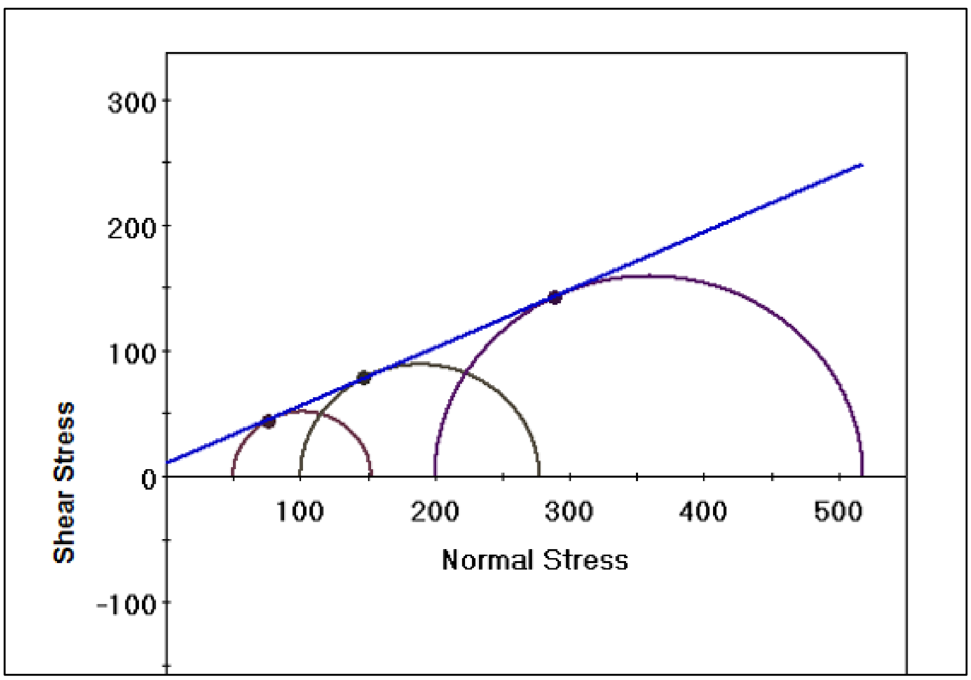

Figure 4.9(b): The Mohr's circle for bottom ash with height 57 mm and diameter 6 mm

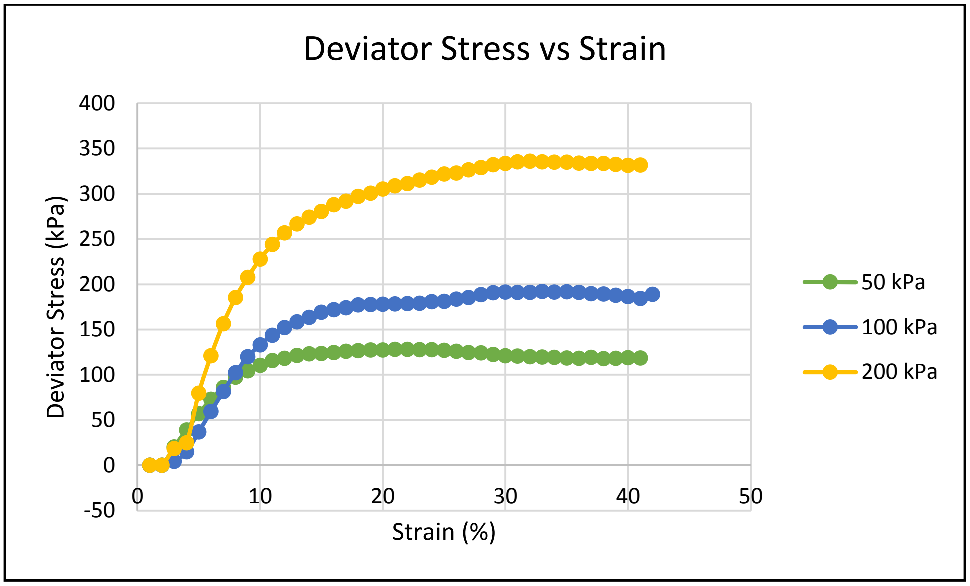

Figure 4.10(a): The deviator stress versus strain for bottom ash with height 76 mm and diameter 6 mm

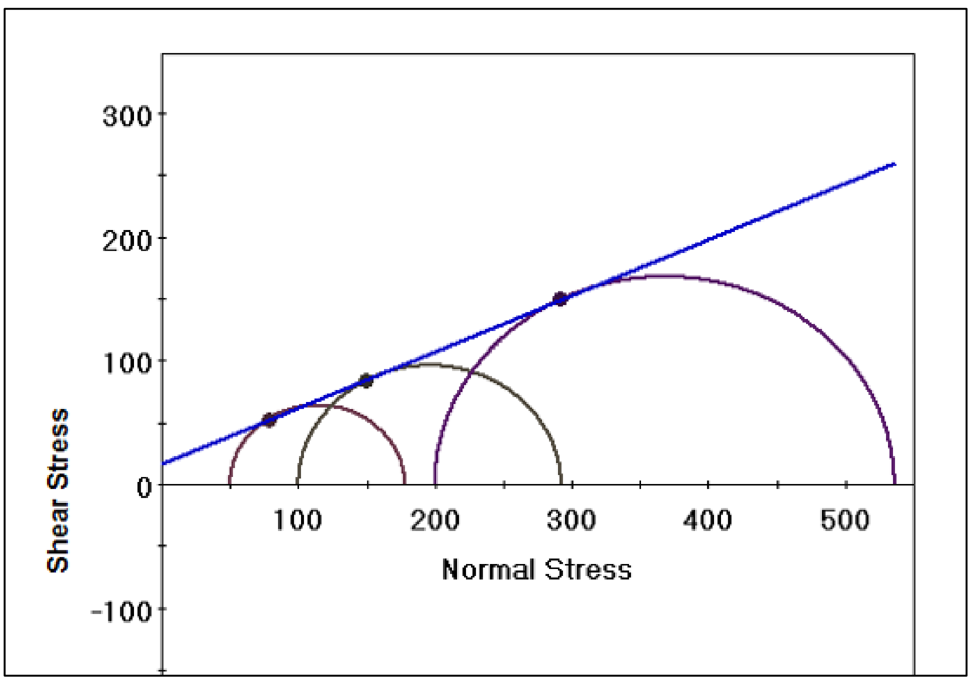

Figure 4.10(b): The Mohr's circle for bottom ash with height 76 mm and diameter 6 mm

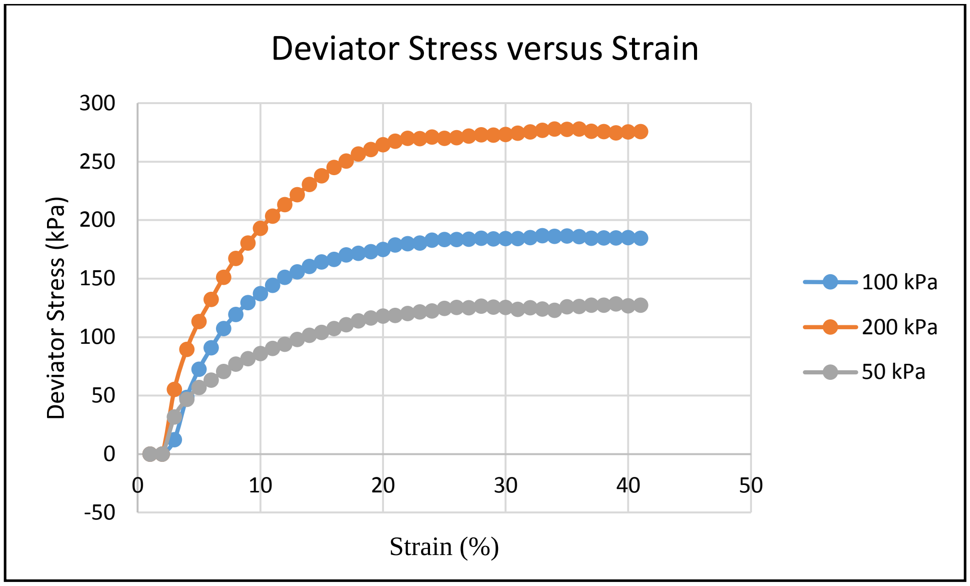

Figure 4.11(a): The deviator stress versus strain for bottom ash with height 38 mm and diameter 8 mm

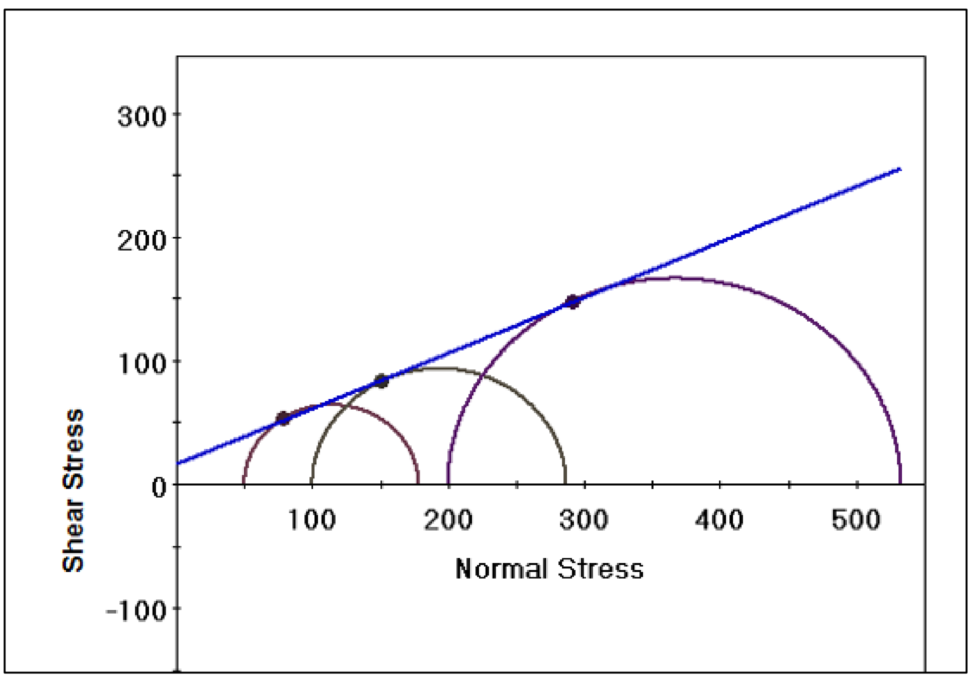

Figure 4.11(b): The Mohr's circle for bottom ash with height 38 mm and diameter 8 mm

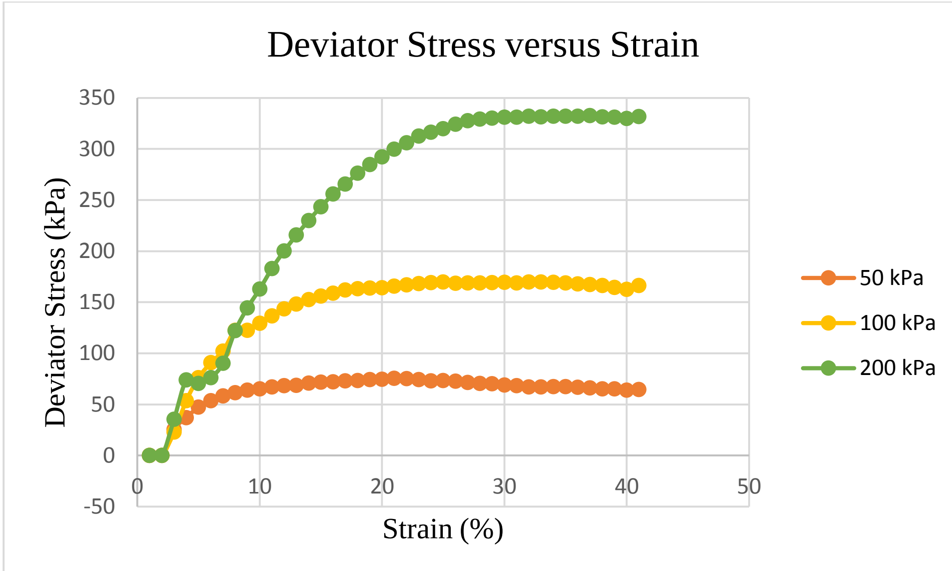

Figure 4.12(a): The deviator stress versus strain for bottom ash with height 57 mm and diameter 8 mm

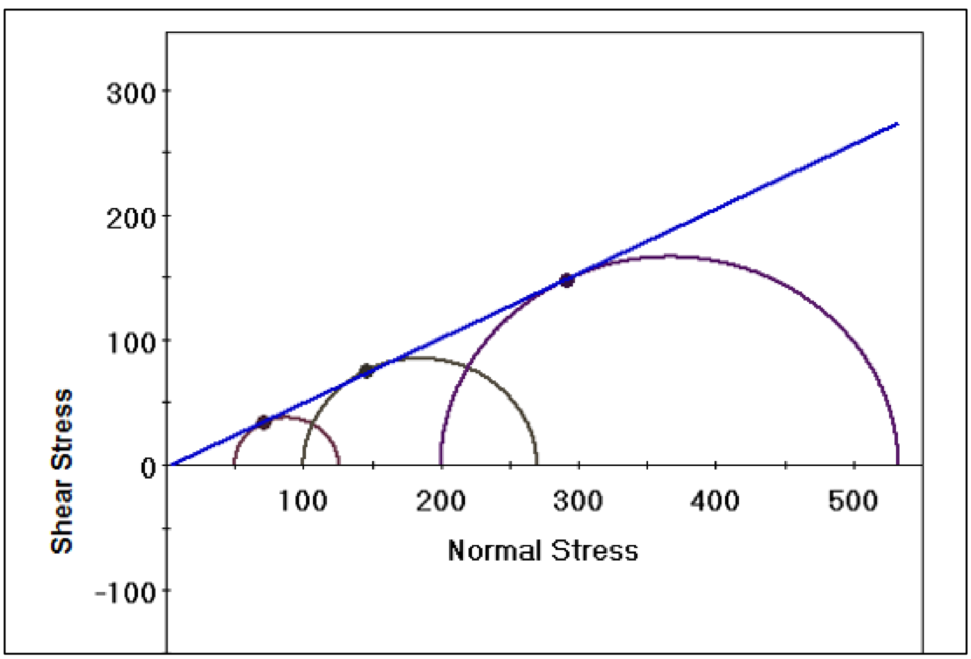

Figure 4.12(b): The Mohr's circle for bottom ash with height 57 mm and diameter 8 mm

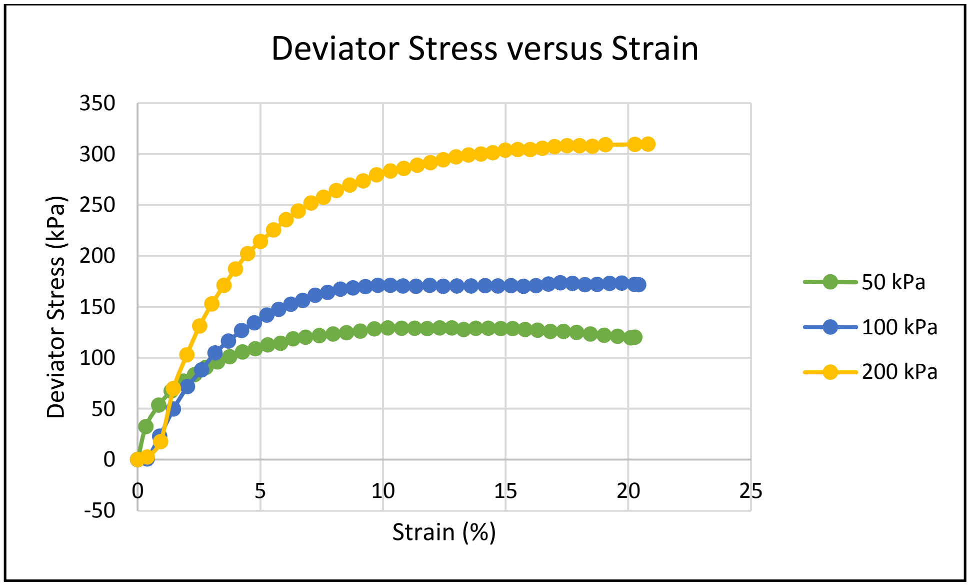

Figure 4.13(a): The deviator stress versus strain for bottom ash with height 76 mm and diameter 8 mm

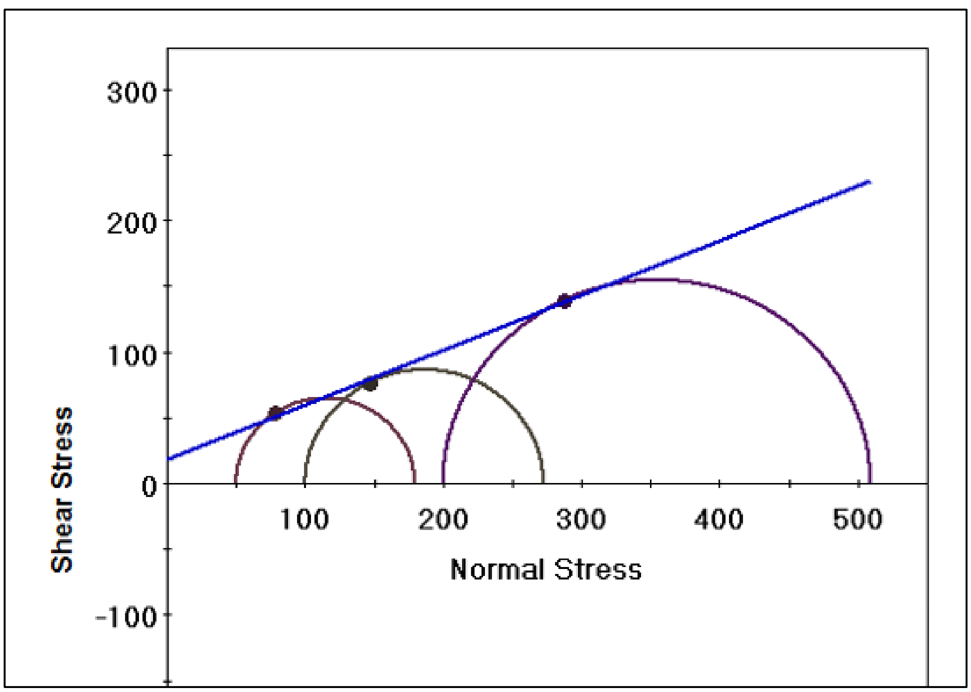

Figure 4.13(b): The Mohr's circle for bottom ash with height 76 mm and diameter 8 mm

The shear strength parameter of the soft clay in reinforced with the group encapsulated bottom ash column are tabulated in the Table 4.5. The result shows that for 6 mm diameter of bottom ash column, the shear strength of the soft clay are increase when the height of bottom ash column is increase. When the height of the bottom ash is 38 mm, the shear stress is 0. While when the height is 57 mm and 76 mm, the shear stress is 10 kN/m^2 and 15 kN/m^2 respectively. Whereas, for the bottom ash with diameter of 8 mm, the shear stress is 18 kN/m^2 when the height is 38 mm and 0 when the height is 57 mm. When the height is 76 mm, the shear stress is 20 kN/m^2. Meanwhile, the friction angle decrease when the height of the column are increase.

Table 4.5: Result of the shear strength parameter

Batch	Diameter, d (mm)	Height, H (mm)	Cohesion, c (kPa)	Friction Angle, ϕ (°)
1	6	38	0	27
2		57	10	26
3		76	15	24
4	8	38	18	24
5		57	0	27
6		76	20	22

4.5.2 Effect of Area Replacement Ratio

The graph of shear stress versus area replacement ratio are shown in Figure 4.14. For the bottom ash column diameter 6 mm, the area replacement ratio is 7.5%, while for bottom ash column with 8 mm diameter the area replacement ratio is 13.29%.The graph shows that the values of the shear stress for area replacement ratio 7.5% with the difference height penetrating ratio are 0, 10 kPa and 15 kPa. Meanwhile, value of the shear stress for area replacement ratio 13.29% are 18kPa, 0, and 20 kPa. The performance of 13.29% area replacement ratio in shear stress is greater compare to 7.5% area replacement ratio with the shear stress.

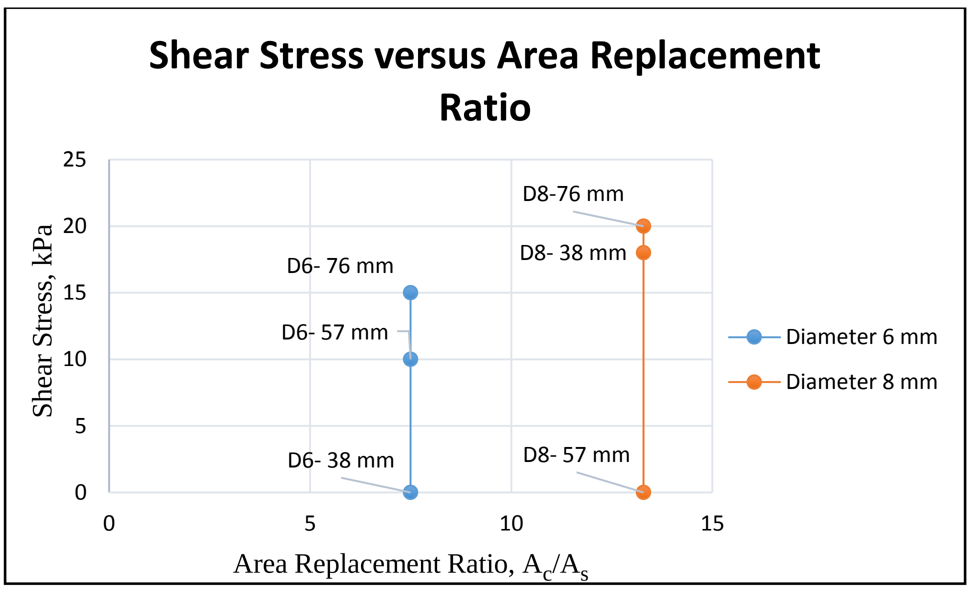

Figure 4.14: Shear stress versus area replacement ratio

The result is also similar with the study done by Najjar *et al*. (2010), the replacement of the portion of the soft and compressible clay by a strong and stiffer sand column could substantially increase the shear strength of the soft clay. The study are proven that the area replacement ratio 17.8% have the greater shear strength compare to the 7.9% area replacement ratio, with the shear strength value are 75% compare to 19.50% respectively. However, the study from Fadzil (2011) prove the contradiction from the current study. The 16% area replacement ratio shows 45% shear strength while 36% of area replacement ratio, the shear strength is -33%. The decrease performance of the shear strength was due to the unsuitable of area replacement ratio which was the soil replacement too much.

4.5.3 Effect of Column Penetrating Ratio

Figure 4.15 show the increment of the shear stress at different penetration ratio of the bottom ash column. The result from the graph shows the sample reinforced with group encapsulated bottom ash column at 0.5, 0.75 and 1.0 height penetrating ratio for 7.5 and 13.29 area replacement ratio.

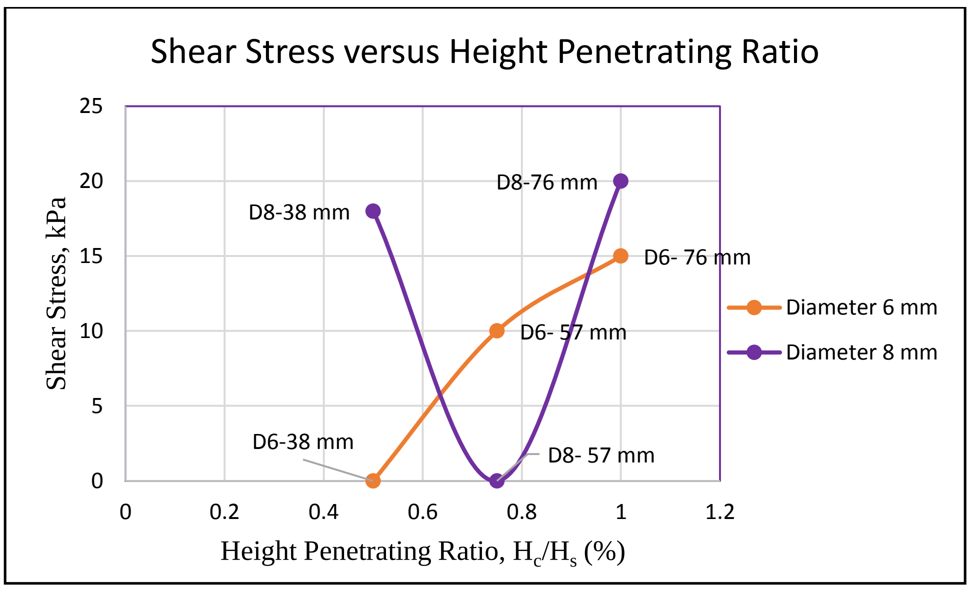

Figure 4.15: Shear stress versus height of penetrating ratio

The result from the graph shows the shear stress are increase when the height penetrating increase for 7.5% area replacement ratio. The fully penetrating, 1.0 have the greater value of shear stress compare to partially penetrating, 0.5 and 0.75. Meanwhile, the area replacement 13.29%, the shear stress for partially penetrating column are decrease from 0.5 to 0.75 height of penetrating ratio. While, fully penetrating, 1.0 the shear stress are increasing. In the 13.29% area replacement ratio of fully penetrating also shows that the shear stress are greater than partially penetrating. Both of the area replacement 7.5% and 13.29% are shows the fully penetrating column have the greater value of shear stress compare to partially penetrating column. The reason of the increase of increment is because of certain portion of soft soil is replaced by stiffer material which is bottom ash as the column. The result are similar to the study done by Hasan *et al*. (2011), where in group column the fully penetrating column gave much higher improvement than partially penetrating column. The improvement of shear strength of soft clay installed either bottom ash column or sand column does not merely depends on the area replacement ratio, but the penetration ratio as well (Marto *et al*., 2014)

4.5.4 Effect of Height over Diameter

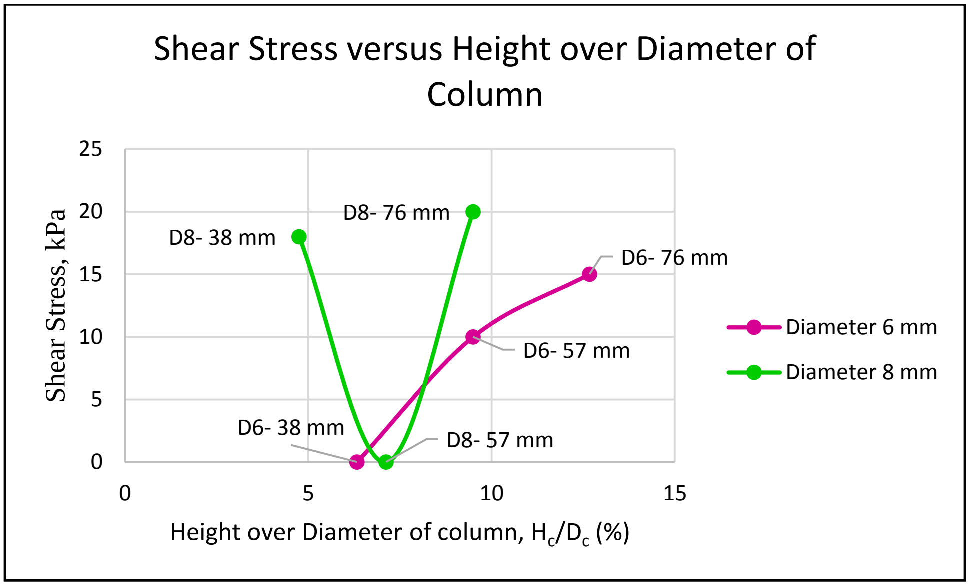

Figure 4.16: Shear stress versus height over diameter of column

The shear stress versus height over diameter of group encapsulated bottom ash column graph are shown in Figure 4.16. The result from the graph shows the shear stress increase when the height over diameter are increase for the column with 6 mm diameter. Meanwhile, for the column with 8 mm diameter of column, the shear stress shows a similar behavior except for the column at height 57 mm, which the shear stress are lower than height at 38 mm. According to Najjar *et al*. (2010), on his study on encasement of sand column, some disproportionate increase in strength indicated that the improvement in undrained shear strength may not only be a function of the column penetration ratio, H_c/H_s, but also of the ratio of the column height over the column diameter.

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

5.1 Introduction

In this chapter, the conclusion of the study and the recommendation to improve the study in the future are discussed. The study has been successfully conducted in order to fulfill all of the objective that has been outlined in Chapter 1 of this study. The strength of soft clay in reinforced with group encapsulated bottom ash column difference size of diameter and various height penetrating ratio were analyzed.

5.2 Conclusions

The major focus of this study is to determine the properties of kaolin clay and bottom ash and to determine the strength of soft clay in reinforced with group encapsulated bottom ash column under Unconsolidated Undrained Triaxial test. Based on the laboratory tests, the conclusion drawn are as below:

1. Based to the AASHTO classification system, kaolin lies in Group A-7-6[b], which means kaolin is proven as clayey soils. Meanwhile, according to the Unified Soil Classification System (USCS), kaolin can be characterized as ML. Based on its liquid limit, 41.3% and plasticity index 10.05%, which indicated that kaolin is low in plasticity silts. Besides, specific gravity of kaolin is determined to be 2.62, proven that the substantial amount of kaolinite mineral are contains in the kaolin composition. Furthermore, from compaction test conducted, the maximum dry density is 1.575 kg/m^3 and the optimum moisture content is 20%. In addition, the permeability coefficient of kaolin is 1.124 x 10^{-9} m/sec.

2. The Tanjung Bin bottom ash has the relatively similar characteristic to those typical sand and fine gravel. According to Unified Soil Classification System (USCS), bottom ash is classified as SW, which means graded sand. Based on AASTHO classification system, bottom ash is in Group A-1 which is classified as fragment, gravel and sand. The specific gravity is 2.33 while the permeability coefficient is 1.57 x 10^{-3} m/sec, which indicated to the clean sand, implying a god drainage characteristic of bottom ash as a column. the value of internal friction angle from the direct shear test of bottom ash is 23.93°, which proven that the value is slightly similar to the natural sandy soil since the shear strength of bottom ash was contributed by angularity and rough surface texture that develop high degree of interlocking, hence resist inter-particle sliding.

3. The presence of the encapsulated bottom ash column in group of three (3) has greatly increased the shear stress of the kaolin clay. The increment of the shear stress are depending on the area replacement ratio, height penetrating ratio and the height over diameter ratio of the column. For the area replacement ratio 7.5% (6 mm column diameter), the increase in shear stress are 0, 10kPa and 15kPa with the height penetrating 0.5, 0.75 and 1.0 respectively. Meanwhile, for the area replacement ratio 13.29% (8 mm column diameter), the shear stress are 18kPa, 0, and 20kPa with the height penetrating same with the previous one. For both of the area replacement ratio, the fully penetrating column have the greater value of shear stress compare to the partially penetrating column. In addition, the shear stress for 13.29% area replacement ratio is higher than the shear stress of 7.5% area replacement. Thus, the greater the diameter of the column, the higher the value of the shear stress.

4. The encapsulated bottom ash column has greatly increased the shear stress of the kaolin clay and the increment in shear stress are dependent on the area replacement ratio and height penetrating ratio of bottom ash column. Meanwhile, for the area replacement ratio 13.29% at the height penetrating 0.75, the shear stress is 0. The value are not increasing with the increasing of the height penetrating ratio. This may happen because of the sample are in small size. The sensitivity of the small sample are higher compare to the bigger size of sample. Thus, the tendency of the sample to hardly disturb are higher before and during the sample are set up into the machine. The distribution of the sample may affect the final result of the test.

5. Excessive area replacement can affect the performance of the shear stress of the sample in the group encapsulated columns. This is because of the remaining width of the soil sample is too thin to hold the column when the vertical load is distributed in the column. The area replacement of the column is important so that can evasive the overlapping of influence zone.

5.3 Recommendations

In this research, there have been several limitations of work discovered that would affect the accuracy of the final result. Hence, here are several of the recommendation and suggestion for the future work to improve the quality of the result in the study:

1. A larger diameter and height of the sample as well as the bottom ash column should be formed. The larger size of sample will ensure that the soft soil can behave naturally as in real site case, and the bottom ash column can act like one solid mass of column which is stronger in terms of shear strength. Other than that, with the larger size of the sample, the tendency of the sample to hardly disturb can be reduces. It is because, the larger size are less sensitivity compare with the sample with the small size.

2. Due to the limitation of the result got in term of shear strength in this study, further study should investigate the sample under both, Unconsolidated Undrained Traixial (UU) test as well as Unconfined Compression test (UCT). The test is recommended because from the UCT, the shear strength of the sample can be analyzed compare to the sample only tested using UU test. There will improve the result of the study because there are many parameter that be tested.

3. Other than comparison the shear strength parameter between two differences of the column diameter, further study can do the comparison of the shear strength as well as shear strength parameter between singular and group encapsulated bottom ash column. Besides, the test of the control sample, that the sample without encapsulated bottom ash column, also can be compare with the group encapsulated

bottom ash column. To improve the result of the study, the comparison between difference numbers of group bottom ash, such as between three bottom ash column compare with four bottom ash column should be studied.

4. To prevent the sample from the distribution during the installation and set up the sample into the machine, the sample should be handle more carefully. It is because, the sample are form from the soft clay that have the higher sensitivity. A small distribution to the sample can affect the result from the analysis. The test have to redo and retested if there are any distribution to the sample.

5. Further study should investigate the suitable diameter a group of bottom ash column to be installed into the soft clay. In addition, the spacing between the columns should be determine to prevent influence zone around the bottom ash column overlap and decrease the performance of the samples in load-carrying.

REFERENCES

Abdul Talib. N. R, (2009). University Teknologi Malaysia, Johor. *Engineering Characteristics of Bottom Ash from Power Plants in Malaysia.*

Ali, M. H. (2011). Universiti Teknologi Malaysia, Johor. *Soft Soil Reinforced Singular Bottom Ash Column.*

Ambily, A. P., Gandhi, S. R., (2007). Behavior of stone columns based on experimental and FEM analysis. *Journal of Geotechnical and Geoenvironment Engineering@ ASCE*: 133, 405-415

Ammar, A.M., Liu, X., Lin, H., & Ren, A. J. (2009). Enlarged Base Stone Columns to Improve Soft Clay Soil. *International Conference on Transportation Engineering 2009,* 4240-4246.

Awang, A. M. (2011). Geotehnical properties of Tanjung Bin coal ash mixtures for backfill materials in embankment construction. *EJGE Vol. 16,* 1515-1531.

Barksdale, R.D., Bachus, R.C., (1983). Design and Construction of Stone Column - Volume I - Final Report, FHWA/RD-83/026. U.S. *Department of Transportation,Federal Highway Administration.*

Benson, Craig, H., Bradshaw, S., (2011). Bottom Ash- Boiler Slag – Recycle Materials Resources Centre. *University of Wisconsin-Madison USA.*

Black, J., Sivakumar, V. and McKinley j. d., (2007). *Performance of Clay Sample Reinforced with Vertical Granular Columns.* Canadian Geotechnical Journal, 44:89-95.

Bloodworth, A.J., Highley D.E. and Mitchell, C.J., 1993. Kaolin, Industrial Mineral Laboratory Manual, Tech. Report WG/93/1, *Natural Environment Research Council, Bristish Geological Survey,*

British Standard Institution (1990). *Methods of Test for – Soils for Civil Engineering Purposes*, BS 1377-1990: Part 2, 4, 5.

Carrasco, B., Cruz, N., Terrados, J., Corpas, F.A., Perez, L (2012). An evaluation of bottom ash from plant biomass as a replacement for cement in building blocks. *Fuel 118 (2014) 272 – 280*

Craig, H., Benson, Bradshaw, S., (2011). User guideline for coal bottom ash and boiler slag in green infrastructure construction. *University of Wisconsin-Madison USA.*

Craig, R. F., (2004). Craig‟s Soil Mechanics. Spon Press Taylor & Francis Group, London and New York.

Das, S., & Yudbhir. (2006). Geotechnical Properties of Low Calcium and High Calcium Fly. *Journal of Geotechnical and Geological Engineering,* Vol. 24, p 249- 263.

Eied, M. M., Helmy, H. M.M., El-Kaissouny M. G., (2014). Numerical Analysis of Grond Improvement of Group of Ordinary and Encased Stone Column. *Journal of American Science.*

Environmentally sound use of bottom ash. *Confederation of European Waste-to-Energy Plants.*

Fadzil, M. M. (2011). Universiti Teknologi Malaysia. *Soft Clay Reinforced with Group Bottom Ash Column.*

Frikha, W., Bouassida, M., Canou, J., (2014). Parametric Study of a Clayey Specimen Reinforced by a Granular Column. *International Journal of Geomembranics @ ASCE.*

Gneil, J. R., & Bouazza, A. (2008). Model tests in geogrid encased columns. *In Proceeding of the Fourth European Geosynthetics Conference, Ed. N. Dixon, UK Chapter of the International Geosynthetics Society, Edinburgh Scotland UK,* 1-8.

Guggenheim, S., and Martin, R. (1995). Definition of clay and clay mineral: joint report of the AIPEA nomenclature and CMS nomenclature committees. *Clay and clay minerals, 43(2), 255-256. [Long Island City, NY]: Pergamon Press 1968.*

Hamed, N., Kassim, K. A., Chong, S. Y., (2011). Soil Improvement by Reinforced Stone Columns Based on Experiment Work. *Electronic Journal of Geotechnical Engineering, Vol. 16 (2011), Bund. L*

Hasan, M., Marto, A., Hyodo, M., and Makhtar, A. M. (2011). The Strength of Soft Clay Reinforced with Singular and Group Bottom Ash Columns. *Electronic Journal of Geotechnical Engineering, Vol. 16 (M), pp*. 1215-1227.

Hasan, M., Marto, A., Hyodo, M., & Makhtar, A. (2014). Shear Strength Parameters and Consolidation of Clay Reinforced. *Arabian Journal for Science and Engineering @ AJSE.*

Head K. H. (1992). *Manual of Soil Laboratory Testing. Volume 1: Soil Classification and Compaction Tests (Second Edition).*Pentech Press, London.

Head K. H. (1992). *Manual of Soil Laboratory Testing. Volume 2: Permeability, Shear Strength and Compressibility Test.* Pentech Press, London.

Ho, M. H., & Chan, C. M. (2010). The Potential of Using Rubberchips as a Soft Clay Stabilizer Enhancing Agent. *Modern Applied Science Vol. Canadian Center of Science and Education, Vol. 4, No. 10,* 122 -131.

How, C. G., (2011). The Properties of Soil. *Universiti Teknologi Malaysia*.

Huang W. H., (1990). *The Use of Bottom Ash in Highway Embankments, Subgrade and Subbases.* Joint Highway Research Project, Final Report, FHWA/IN/JHRP-90/4 Purdue University, West Lafayette, Indiana.

Hughes, J. M., Withers, N. J., (1974). Reinforcing of soft cohesive soils with stone columns. *Slovak Journal of Civil Engineering*: 2011/3 Pg 17-21

Ibrahim, M. H. (2012). Waste Management of Coal Fired Power Plant Potential Utilization of Bottom Ash. *Malakoff, 10th Annual Waste Management Conference & Exhibition.*

Jamal, A., Raihan T. , Jimjali A. , Azmi, A. L. , Azmi , J., and Jamilah, J., (1997). *Prediction and Determination of Undrained Shear Strength of Soft Clay at Bukit Raja.* Pertanika Journal of Science and Technology, 5 (1): 111 -126

Juran, I., and Riccobono,O. (1991). *Reinforcing Soft Soils with Artificially Cemented Compacted-Sand Columns.* Journal of Geotechnical Engineering, 117 (7), ASCE.

Kempfert H. G. and Gebreselassie B. (2006). *Excavations and Foundations in Soft Soil* Springer, New York.

Kousik, D., Basundhar, P. K., Chandra, S., (2007). Generalized Model for Geosynthetic Reinforced Granular Fill- Soft Soil with Stone Columns. *International Journal of Geomechanics @ ASCE.*

Kumar S. and Stewart J. (2003*). Evaluation of Illinois Pulverized Coal Combustion Dry Bottom Ash for Use in Geotechnical Engineering Applications*. Journal of Energy Engineering @ ASCE.

Kumar, S. & Steward, J. (2013). Evaluation of Illinios Pulverized Coal Combustion Dry Ash for Use in Geotechnical Engineering Applications. *Journal of Energy Engineering,* 42- 54

Maakaroun, T., Najjar, S. S., & Sadek, S. (2009). Effect of Sand Column on the Undrained Load Response of Soft Clays. *Journal of Geotechnical and Geoenvironmental Engineering (ASCE)*

Malaysia Environment Quality Report. (2009). *Department of Environment, Ministry of Natural Resources and Environment Malaysia.*

Marto, A., Hasan, M. A., Makhtar, A, M., & Othman, B. A. (2013). Shear Strength Improvement of Soft Clay Mixed with Tanjung Bin Coal Ash. *ICESD 2013: January 19-20, Dubai, UAE.*

McCarthy, G. J., Solem, J. K., Manz, O. E., Hassett, D. J., (2007). *Use of a Database of Chemical, Mineralogical and Physical Properties of North American Fly Ash to Study the Nature of FlyAsh and Its Utilization as a Mineral Admixture in Concrete*; Western Fly Ash Research, Development and Data Center Topical Report;Energy & Environmental Research Center: Grand Forks, ND, Sept 1990.

McKelvey, D., Sivakumar, V., Bell, A., and Graham, J. (2004) "ModellingVibrated Stone Columns in Soft Clay," Geotechnical Engineering, 157, 137 -149.

Mitchell, J. K., & Katti, R. K. (1993). Soil Improvement‖ - State of the Art Report. *10th ICSMFE, Stockholm, 4*: 509-565.

Muhardi, Marto, A., Kassim, K. A., Makhtar, A. M, Lee, F. W. and Yap. S Y. (2010). *Engineering Characteristics of Tanjung Bin Coal Ash*. Electronic Journal of Geotechnical Engineering.

Muir, W. D., Hu, W., Nash, D.F.T., 2000. *Group effects in stone column foundations: model tests.* Geotechnique 50, 689e698.

Murray, H. H., (2010). *Industrial Application of Kaolin.* Georgia Kaolin Company, Elizabeth, New Jersey

Murugesan, S. and Rajagopal, K., 2006). *Geosynthetic- encased Stone Columns in Soft Clay: A Numerical Study.* Geotexiles and Geomembranes, 24: 349-358.

Najjar, S. S., Sadek, S. and Maakaroun, T., (2010). Effect of Sand Columns on the Undrained Load Response of Soft Clays, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 136, No. 9, ASCE.

Narasimha Rao, S., Prasad, Y. V. S. N., and Hanumanta Rao, V. (1992). Use of Stone Columns in Soft Marine Clays. *Proc. 45th Canadian Geotechnical Conference, Canadian Geotechnical Society,* September 1 – 7, 1992. Toronto, Canada.

Pivarc, J., (2011). Stone Column- Determination of the Soil Improvement Factor. *Slovak Journal of Civil Engineering.*

Prasad, M. S., Reid, K. J., and Murry, H. H., (1990). *Kaolin: processing, properties and applications.* Applied Clay Science, 6(2), 87-119.

Priebe, H. J., (1995). The design of vibro replacement. *Grouding Engineering December:* 31 – 46

Sa'adon, M. F. (2009). Universiti Malaysia Pahang. *Determination of Basic Soil Properties and Shear Strength*.

Sharma, R. S., Kumar, P., and Nagendra, G., (2004). *Compressive Load Response of Granular Piles Reinforced with Geogrids.* Canada Geotech. J, 41:187-192

Singh, M., and Siddique, R. (2012). Effect of coal bottom ash as partial replacement of sand on properties of concrete. *Resources, Conservation and Recycling.*

Sivakumar, V., McKelvey, D., Graham, J., and Hughus. (2004). Triaxial tests on model sand columns in clay. *Can. Geotech. J.*

Steam. (1978). Its generation and use. 39th ed. *Babcock & Wilcox Company.*

Tandel, Y. K., Solanki, C. H., and Desai, A. K. (2012). Reinforced granular column for deep soil stabilization. *International Journal of Civil and Structural Engineering, Volume 2, No. 3, 2012.*

Tiwari, S. K., and Kumawat, N. K. (2014). *Recent Development in Ground Improvement Technique*. International Journal of Recent Development in Engineering and Technology.

Wu, C. S., & Hong, Y. S. (2008). Laboratory tests on geosynthetic- encapsulated sand column. *Geotextiles and Geomembranes, Elsevier, 107 – 120.*

Young, M. Y., (2012). Shear Strength of Soft Soil Reinforced with Singular and a Group of Bottom ash Column. *University Teknologi Malaysia.*

APPENDIX A

ATTERBERG LIMIT TEST

LIQUID LIMIT

Test Number	**1**		**2**		**3**	
Cone penetration (mm)	14.8	14.6	19.2	19.1	25.5	26.0
Average Penetration (mm)	14.70		19.15		25.75	
Container No.	A1	A2	B3	B4	C5	C6
Container Weight (g)	10.34	10.71	10.08	10.06	9.78	10.70
Wet Soil + Container (g)	14.37	15.89	13.35	15.08	14.60	14.67
Wet Soil, W_w (g)	4.03	5.18	3.27	5.02	4.82	3.97
Dry Soil + Container (g)	13.28	14.45	12.4	13.62	13.12	13.43
Dry Soil, W_d (g)	2.94	3.74	2.32	3.56	3.34	2.73
Moisture Loss, W_w - W_d (g)	1.09	1.44	0.95	1.46	1.48	1.24
Moisture Content (%)	37.075	38.503	40.948	41.011	44.311	45.421
Average Moisture Content (%)	**37.789**		**40.980**		**44.866**	

PLASTIC LIMIT

Container Number	D1	D2
Container Weight	10.79	9.64
Wet Soil + Container (g)	18.74	18.76
Wet Soil, W_w (g)	7.95	9.12
Dry Soil + Container (g)	16.82	16.62
Dry Soil, W_d (g)	6.03	6.98
Moisture Loss, W_w - W_d (g)	1.92	2.14
Moisture Content (%)	31.841	30.659
Average Moisture Content (%)	**31.250**	

APPENDIX B

SPECIFIC GRAVITY TEST

Sample: Kaolin S300
Test Method: BS1377-2:1990: Clause 8.3

Date Test: 11.02.2015

TEST NO.	1	2	3	4
Density bottle No	1	12	13	18
weight of density bottle ,g	29.84	32.41	14.25	30.69
Weight of bottle + Stopper (W1) , g	34.56	37	37	35.31
Weight of bottle + Stopper + Dry soil (W2) , g	44.5	46.93	46.9	45.27
Weight of bottle + Stopper + Soil + Water (W3), g	141.49	142.13	142.16	141.15
Weight of bottle + Stopper + Water (W4), g	135.32	135.99	136.03	135
Weight of dry soil (W2-W1), g	9.94	9.93	9.92	9.96
Weight of water (W4-W1), g	100.76	98.99	99.65	99.69
Weight of soil + water (W3-W2), g	96.99	95.2	95.26	95.88
Specific gravity	2.64	2.62	2.61	2.61
Average specific gravity	2.62			

Sample: Tanjung Bin Bottom Ash

Test Method: BS1377-2:1990: Clause 8.3

Date Test: 11.02.2015

SPECIMEN REFERENCE		**1**	**2**	**3**
Density Bottle No.		50	51	53
Mass of Bottle		23.950	23.67	24.640
Mass of Bottle + Stopper, m_1	g	28.560	28.070	29.020
Mass of Bottle + Stopper + Dry Soil, m_2	g	37.050	37.250	37.340
Mass of Bottle + Stooper + Soil + Water, m_3	g	82.880	83.400	84.010
Mass of Bottle + Stopper + Water, m_4	g	78.050	78.190	79.240
Mass of Dry Soil, $(m_2 - m_1)$	g	**8.490**	**9.180**	**8.320**
Mass of Water In Full Bottle, $(m_4 - m_1)$	g	**49.490**	**50.120**	**50.220**
Mass of Water Used, $(m_3 - m_2)$	g	**45.830**	**46.150**	**46.670**
Particle Density, ρ_s	Mg/m^3	**2.32**	**2.31**	**2.34**
AVERAGE PARTICLE DENSITY, ρ_s	**Mg/m^3**	**2.33**		

APPENDIX C

SIEVE ANALYSIS TEST

Sample: Bottom Ash from Tanjung Bin Power Plant

Test Method: BS1377-2:1990: Clause 9.3

Dated Test: 13 February 2015

Sieve Size (mm)	Mass of Sieve (g)	Mass retained on Sieve + Sieve (g)	Percent Retained (%)	Percent Passing (%)
10	599.99	606.79	1.35	98.65
5	524.75	558.63	8.07	91.93
2.36	548.61	629.23	24.09	75.91
1.18	427.77	560.55	50.48	49.52
0.6	484.23	577.65	69.05	30.95
0.3	431.11	504.58	83.65	16.35
0.15	422.67	477.93	94.63	5.37
Pan	366.69	393.69	100	0

APPENDIX D

DETERMINATION OF MAXIMUM AND MINIMUM DRY DENSITIES FOR GRANULAR SOILS

Test Method: ASTM D4253 : 2000 &
ASTM D4254: 2000

DateTested: 4/3/2015

MOULD INFORMATION:

Mass of Empty Mould + Base Plate : g	7,390.00	g			
Diameter :	151.54	mm	Height :	125	mm
Area, *Ac* :	18,036.18	mm^2	Volume, *Vc* :	2.25E+06	mm^3

LOOSE SPECIMENS:

TEST		1	2	3	4
Mass of Mould + Base Plate + Loose Specimen	g	9,350.00	9,370.00	9,320.00	9,350.00
Mass of Loose Specimen, *M1*	g	1,960.00	1,980.00	1,930.00	1,960.00
$\rho_{d\text{-}min} = M1 / Vc$	g/cm^3	0.869	0.878	0.856	0.869
AVERAGE $\rho_{d\text{-}min}$	**g/cm^3**	**0.868**			

DENSE SPECIMENS:

TEST		1	2	3	4
Mass of Mould + Base Plate + Specimen (Before Vibration) #w/out collar	g	9,700.00	9,850.00	9,800.00	9,850.00
Mass of Mould + Base Plate + Specimen (After Vibration) #w/out collar	g	9,640.00	9,650.00	9,650.00	9,670.00
Mass of Dense Specimen (After Vibration), *M2*	g	2,250.00	2,260.00	2,260.00	2,280.00
$\rho_{d\text{-}max} = M2 / V$	g/cm^3	0.998	1.002	1.002	1.011
AVERAGE $\rho_{d\text{-}max}$	**g/cm^3**	1.004			

APPENDIX E

COMPACTION TEST RESULT

Sample: Kaolin S300

Water Content (%)	10%		15%		20%		25%		30%		35%	
Mass of mould + Base (m_1), g	4080		4080		4080		4080		4080		4080	
Mass of mould + Base + Compacted Specimen (m_2), g	5740		5850		5960		5880		5860		5860	
Mass of Compacted Specimen (m_2-m_1), g	1660		1770		1880		1800		1780		1780	
Bulk Density, $\rho = \frac{m_2 - m_1}{V}$ g/cm^3	1.66		1.77		1.88		1.80		1.78		1.78	
Container No.	68C	18D	117C	59D	4C	70C	38C	56D	91C	92C	3C	21D
Container weight (gm)	9.27	11.25	9.16	9.56	10.13	10.75	10.66	10.18	9.77	10.66	9.26	10.56
Wet Soil + container (gm)	15.05	17.60	21.47	22.04	24.33	21.83	31.11	32.85	29.09	28.04	35.40	29.03
Wet soil (gm), Ww	5.78	6.35	12.31	12.48	14.20	11.08	20.45	22.67	19.32	17.38	26.14	18.47
Dry soil + container (gm)	14.53	17.02	19.88	20.38	22.04	20.01	27.03	28.35	24.66	24.03	28.59	24.20
Dry soil (gm), Wd	5.26	5.77	10.72	10.82	11.91	9.26	16.37	18.17	14.89	13.37	19.33	13.64
Moisture loss (gm), Ww-Wd	0.52	0.58	1.59	1.66	2.29	1.82	4.08	4.50	4.43	4.01	6.81	4.83
Moisture content (%), (Ww-Wd)/Wd	9.89	10.05	14.83	15.34	19.23	19.65	24.92	24.77	29.75	29.99	35.23	35.41
Average moisture content (%)	9.97		15.09		19.44		24.85		29.87		35.32	
Dry Density g/cm^3,	1.51		1.54		1.57		1.44		1.371		1.315	
Dry unit Weight, kN/m^3	14.81		15.09		15.44		14.15		13.45		12.90	

Sample: Tanjung Bin Bottom Ash

Water Content (%)	10%		12%		14%		16%		18%		20%	
Mass of mould + Base (m_1), g	4080		4080		4080		4080		4080		4080	
Mass of mould + Base + Compacted Specimen (m_2), g	5280		5330		5350		5400		5460		5470	
Mass of Compacted Specimen (m_2-m_1), g	1200		1250		1270		1320		1380		1390	
Bulk Density, $\rho = \frac{m_2-m_1}{V}$ g/cm^3	1.20		1.25		1.27		1.32		1.38		1.39	
Container No.	45D	38C	84C	70D	30C	3G8	34D	30D	12C	53D	100C	117C
Container weight (gm)	9.90	10.71	10.79	10.75	10.50	10.35	10.37	10.95	9.62	9.92	10.72	9.17
Wet Soil + container (gm)	15.96	17.30	17.52	16.46	20.35	18.13	19.08	20.63	16.96	15.67	17.56	16.02
Wet soil (gm), Ww	6.06	6.59	6.73	5.71	9.85	7.78	8.71	9.68	7.34	5.75	6.84	6.85
Dry soil + container (gm)	15.43	16.72	16.86	15.87	19.27	17.27	18.00	19.44	15.94	14.94	16.63	15.06
Dry soil (gm), Wd	5.53	6.01	6.07	5.12	8.77	6.92	7.63	8.49	6.32	5.02	5.91	5.89
Moisture loss (gm), Ww-Wd	0.53	0.58	0.66	0.59	1.08	0.86	1.08	1.19	1.02	0.73	0.93	0.96
Moisture content (%), (Ww-Wd)/Wd	9.58	9.65	10.87	11.52	12.31	12.43	14.15	14.02	16.14	14.54	15.74	16.23
Average moisture content (%)	9.62		11.20		12.37		14.09		15.34		15.98	
Dry Density g/cm^3,	1.10		1.12		1.13		1.16		1.20		1.20	
Dry unit Weight, kN/m^3	10.74		11.03		11.09		11.35		11.73		11.75	

Water Content (%)	22%		24%		26%		28%		30%		32%	
Mass of mould + Base (m_1), g	4080		4080		4080		4080		4080		4080	
Mass of mould + Base + Compacted Specimen (m_2), g	5540		5850		5550		5640		5710		5680	
Mass of Compacted Specimen (m_2-m_1), g	1460		1510		1470		1560		1630		1600	
Bulk Density, $\rho = \frac{m_2 - m_1}{V}$ g/cm^3	1.46		1.51		1.47		1.56		1.63		1.60	
Container No.	37C	32D	42D	3C	56D	713C	45C	6C	116C	101C	46D	38D
Container weight (gm)	9.86	10.20	11.05	9.27	10.18	10.18	9.59	10.07	9.76	10.01	10.43	10.56
Wet Soil + container (gm)	16.30	15.17	17.20	16.68	17.90	18.55	15.81	18.91	18.14	18.86	20.20	20.72
Wet soil (gm), Ww	6.44	4.97	6.15	7.41	7.72	8.37	6.22	8.84	8.38	8.85	9.77	10.16
Dry soil + container (gm)	14.35	14.53	16.33	15.55	16.72	17.24	14.85	17.25	16.65	17.31	18.45	18.91
Dry soil (gm), Wd	5.49	4.33	5.28	6.28	6.54	7.06	5.26	7.18	6.89	7.30	8.02	8.35
Moisture loss (gm), Ww-Wd	0.95	0.64	0.87	1.13	1.18	1.31	0.96	1.66	1.49	1.55	1.75	1.81
Moisture content (%), (Ww-Wd)/Wd	17.30	14.78	16.48	18.00	18.04	18.56	18.25	23.12	21.63	21.23	21.82	21.68
Average moisture content (%)	16.04		17.24		18.30		20.69		21.43		21.75	
Dry Density g/cm^3,	1.26		1.29		1.24		1.29		1.34		1.31	
Dry unit Weight, kN/m^3	12.34		12.64		12.19		12.68		13.17		12.90	

APPENDIX F

FALLING HEAD PERMEABILITY TEST

Permeameter Cell Dimension

Diameter, = 100mm

Length, L = 130 mm

Manometer Tube	Diameter, (m)	Start Level, h_1 (m)	End Level, h_2 (m)	Time, t (sec)
T_1	0.073	1.0	0.696	600
T_2	0.081	1.0	0.790	600
T_3	0.616	1.0	0.968	600

Manometer Tube	h1/h2	Log h1/h2	Time, t (sec)	Radius Manometer Tube, r (m)	Area Manometer Tube, a (m^2)	Area, A (m^2)	A X t
T_1	1.437	0.157	600	0.0365	0.24	0.14	84
T_2	1.266	0.102	600	0.0407	0.27	0.14	84
T_3	1.033	0.014	600	0.0808	0.55	0.14	84

Based on Manual Of Soil Testing Vol. 2 Page 424. Figure 10.14

Drainage Characteristic = Practically Impervious

Permeability Classification = Practically Impermeable

General Type = Intact Clay

APPENDIX G

CONTANT HEAD PERMEABILITY TEST

Constant head cell: 1924.5 g

Constant head cell + sample: 2698.76 g

Mass of bottom ash: 800 g

Mass cylinder: 507.96 g

Test No.	1	2	3
Time to accumulate water(sec)	10	20	30
Mass of water (g)	71	68	72

Total head at the top manometer (cm): 51.5 cm

Total head at the bottom manometer (cm): 48.5 cm

Vertical distance between manometers (cm): 15 cm

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NUR AQILAH BINTI MOHD DRUS

Report submitted in fulfillment of the requirements for the award of the degree of

Bachelor of Engineering (Hons) in Civil Engineering

Faculty of civil Engineering and Earth Resources

UNIVERSITI MALAYSIA PAHANG

JUNE 2015

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