ENVIRONMENT-FRIENDLY GROUND IMPROVEMENT TECHNIQUE USING WASTE SHELL HUSK

A dissertation submitted in partial fulfillment of the requirements of the degree of Doctor of Philosophy in the Department of Environmental Science and Technology of Mie University

GRADUATE SCHOOL OF BIORESOURCES

MIE UNIVERSITY

By

SITI HANGGITA RACHMAWATI

September, 2018

FOR MY BELOVED FAMILY

ENVIRONMENT-FRIENDLY GROUND IMPROVEMENT TECHNIQUE USING WASTE SHELL HUSK

A Dissertation Submitted to the Department of Environmental and Science, Mie University, Japan, in Fulfillment of the Requirements for the Doctoral Degree

By

Siti Hanggita Rachmawati

Examined and approved as to style and contents by:

Prof. Dr. Zakaria Hossain

Chairman of the Examination Committee

Prof. Dr. Toshinori Sakai

Co-Chair of the Committee

Prof. Dr. Takamitsu Kajisa

Co-Chair of the Committee

Department of Environmental Science and Technology

Mie University, Japan

August, 2018

ABSTRACT

Every year huge amount of Seashell-By-Products (SBP) are produced in all over the world. According to the Japanese Ministry of Forestry, Fisheries and Agriculture, total amount of abandon shell husk is about 151,000 tons/year which generated of industry and household consumption. It nearly 32 million US$/year are being used for disposal cost which still unexpected event for developed country like Japan. In developing country, illegal dumping is most common way to treat this waste due to a high cost of waste treatment. In a certain time, if this waste is untreated, the air pollution and other environmental problems are occurred. It is because abandon shell husk without properly treatment becoming a source of unpleasant smells due to either the decay of remnant flesh attached to oyster shells or microbial decomposition of salts into gases such as NH_3, H_2S and amine. The development of technologies that using abandoned shell husk are needed according to the consequences for human being.

In the present study, the abandon shell husks are being used as the recycle aggregate in ground improvement technique. Recycling aggregate utilization has advantage to protect limited resources of natural aggregates, increasing the abandon waste value and problem solving of waste storage. Ground improvement techniques are contributed to increase the soil strength and performance under the load, also reducing compressibility of soil. These techniques are often used to improve the properties of soil in terms of their bearing capacity, shear strength, settlement characteristic and drainage that widely used in a large scope of construction such as industrial, commercial, housing projects and infrastructure construction for dams, tunnels, ports, roadways and embankments. There are many different types of ground improvement techniques, which can be tailored to the natural condition of soil and economical aspect in order to achieve its effectiveness and efficiency. Recently recycle aggregate has recently been used in all over the world to reduce project budget and protect environment. The shell husk has potency as recycle aggregate due to composition which consist of mainly 95-99% (by weight) of CaCO3 that potentially convert to CaO for reinforcing the soil or binding the material. Furthermore, the utilize of shell husk are purposed to improve agriculture land which need light weight material. Effective ground improvement technique is normally needed in improving agriculture land condition. This is to accommodate agriculture activities which have operational loadings due to tractor or rice transplanter. To enhance the soil-shell husk material properties, then shell husks are mixed with cement. Cement is a soil stabilizing agents which used widely, due to its quick process.

It does not need mellowing time and provides a non-leaching platform to stabilize soils. Application soil-cement with nominal dosage of cement also has significant contribution to environment and it is cost-effectiveness. In Japan, many terrace land uses cement treated soil to prepare new cultivation paddy fields from unused land.

Then, to evaluate this concept, in this study was prepared several specimens which are control (only soil), soil-shell husk, soil-cement, soil-cement-shell husk. Specimen which has cement percentages was cured for seven days before laboratory testing. The laboratory testings are included direct shear test, CBR (California Bearing Ratio), UCS (Unconfined Compressive Strength), triaxial test. In this study, the properties of soil and shell husk also had been clarified. The parameters were shear strength, angle of internal friction (φ), cohesion (c), dilatancy behavior, bearing capacity, stress, strain, moduli deformation, axial strain (Ɛ a), and principal stress difference (σa-σr).

Result of the direct shear test showed that both the and angle of internal friction cohesion of the soil-shell husk specimen increased by increasing of shell husk percentage. It also was observed that there is a decrease in the dilatancy behavior of soil which mixed with shell as compared to control specimens indicating ground improvement. From this testing was found the shear strength of soil increased by increasing the shell husk-cement percentage. The CBR results showed that the addition of shell husk in soil leads to improvement in the CBR values of the ground as compared to control specimens. It was further revealed that by increasing the percentage of shell husk in the soil-shell husk mixture, the CBR value was improved significantly. The addition of shell husk and cement increased the CBR value of all types of subbase layers. From UCS testing was observed that increasing of shell husk and cement percentage, increased the compressive strength of the soil. There was very little variation in the estimation of the moduli deformation for testing results. The triaxial test obtained that combination of shell husk and cement addition increase the shear strength than specimen which using shell husk and cement separately.

The outcome of this research indicated that shell husk has capability as the recycle aggregate in ground improvement technique. Moreover, combination of shell husk-cement are expected to improve agricultural earth structures.

CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

The shell of surf clam has strong construction due to its habit which like to burrow in rocks. It is known surf or trough clam which has a smooth surface with concentric growth lines and covered by thin periostracum (Vaughan, 2001). The clams are harvested then processed by shellfish industry which using manual or mechanical method to extract the meat. Shell husks are left as by-product material which usually disposed on landfill. At normal temperature, organic material such as remaining of flesh and blood will decompose into gas compounds (NH_3 and H_2S) then causing odor and harmful for the environment. Shell stacking has leachate and moisture which suitable as breeding place for mice, flies and mosquitoes. It is crucial to find the solution to reduce environmental problems, especially in those countries where shellfish product industries are existed (Agustini et al. 2011). Fisheries industries while doing their business has obligation to operate optimally and environment-friendly according to CCRF regulation (Code of Conduct for Responsible Fisheries). Not only industry, people usually throwing the shells away once the eatable part have been removed. This may contribute to the increase amount of food waste and thus the total amount of the waste at the landfill. Handling of shell husk as a solid waste of shellfish product industry require serious effort to get the benefit and reduce negative impact to human and environment.

Figure 1. 1 Shell husk waste

Huge amount of abandoned Mactridae which also known as surf or trough clam shell husk is produced in Japan. Mactridae is a family of order Veneroida and marine bivalve class. According to Japanese Ministry of Forestry, Fisheries and Agriculture, around 151.000 tons per year abandon shell husk is generated in Japan. Among the numbers, 9627 tons per year are produced in Mie prefecture, 715 tons per year are produced in Tsu City including 407 tons

Mactridae. Tsu city spend about 18 yen per kilogram cost disposal which means nearly 151.411US$ in total and nearly 86.188US$ for Mactridae cost disposal only. Furthermore, Mie prefecture itself requires nearly 2 million US$ and around 32 million US$ is needed in Japan, for cost disposal of shell husk. Shell husk has been listed worldwide as one of the worst environmental problems as it is difficult to its dispose. The processing of shell husk waste has a major technological problem due to generally insoluble and resistant to natural biodegradation.

As solution of environmental problem caused by abandon shell husk waste, then in present study shell husk had been used as recycle aggregate for ground improvement. Now-a-days, recycle aggregates are gradually used for ground reinforcement. The aim of recycle aggregate utilization is one way to apply sustainable development on construction industry including earthwork construction (Li et al. 2010; Ortiz et al. 2009; Tam and Tam, 2006). Recycling principle has benefits which are protecting limited natural resources of aggregates, increasing the abandon waste value and problem solving of waste storage (Hossain, 2013; Tam and Tam, 2006). Many researchers used organic and inorganic materials as recycled aggregates in their investigations. The comparisons between soil without reinforcement and soil with reinforcement have different trends. Previous studied showed that the application of recycle aggregates as ground reinforcement have produced positive results (Baumgartl and Horn, 1991; Hossain, 2013; Malkawi et al. 1999). Shell husk is one kind of aggregate which has been investigated widely as reinforcing ground. Recycle aggregate utilization has purpose to realize sustainable development on construction industry including earthwork construction (Li et al. 2010; Ortiz et al. 2009; Tam and Tam, 2006). Furthermore, the utilize of recycle aggregate has benefits to protect limited natural resources of aggregates, increasing the abandon waste value and solving waste storage problem (Hossain, 2013a; Tam and Tam, 2006). Previous studies showed application of recycle aggregates to reinforce soil or earth fill structure have positive results (Baumgartl and Horn, 1991; Hossain, 2013b; Malkawi et al. 1999).

Ground improvement techniques has purposed to increase the soil strength, reducing compressibility and enhance the performance under the load and improve the properties of soil in terms of their bearing capacity, shear strength, settlement characteristic and drainage (Hirkane et al. 2014). These techniques are widely used in a large scope of construction such as industrial, commercial, housing projects and infrastructure construction for dams, tunnels, ports, roadways and embankments (Hirkane et al. 2014). It is crucial that the stability of most

earth fill structures has to be maintained under any circumstances. Natural phenomena such as weathering, erosion, earthquake, and drought could reduce the stability then lead to the failure. To solve these problems, ground improvement that could enhance the stability of earth fill structures is desirable (Yoonet et al. 2009). Ground improvement techniques are provided to increase the soil strength, reducing compressibility and enhance the performance under the load. Ground improvement by reinforcing the soil with various types of geosynthetic material is widely used (Mishra, 2016). The interaction between soil and reinforcement could be complicated depend on nature and properties of the reinforcement and the soil. It could be classified into two categories; sliding of soil over the reinforcement and pullout behavior of reinforcement from the soil. The direct shear test and pullout test are generally used to analyze these interaction mechanisms (Hossain et al. 2012).

1.2 SCOPE OF THE STUDY

Recycle aggregates in ground improvement techniques are known as one of wide variety of geomaterial. Geomaterial is expected to improve soil properties and could be used in geotechnical application. Improved geomaterial are often the end products of ground improvement, therefore they are not problematic to geotechnical application (Han, 2015). In this study, some laboratory tests are performed to evaluate shell husk as geomaterial. The evaluation has seen from shell husk percentage in the ratio of mass. To increase effectiveness of soil-shell husk as geomaterial, several testing using specimen by cement addition. The utilize of cement in certain percentage could improve properties of soil and environmental friendly (Hossain and Sakai, 2008). The addition of cement has purpose to modified and stabilized soil properties. Term modification used for improvement in workability and compaction characteristics while the term stabilization means improving mechanical behavior of cement treated soil (Sarriosseiri and Muhunthan, 2009).

Major factors in this study are bearing capacity and shear strength of soil. CBR (California Bearing Ratio) tests are performed to evaluated bearing capacity of soil. Mostly, the CBR values are used in mechanistic design and as indicator of strength and bearing capacity of subgrade soil, subbase and base course material for pavement and foundation design (Yildgrim and Guynadin, 2011 and Hazirbaba and Gullu, 2010). CBR test is economic and simple in comparison to other tests such as triaxial, simple shear and direct shear tests. The way CBR test can be adjusted and simulated based on the specific conditions which needed. In previous studies, the CBR test was used to evaluate the reinforcing soil with various

recycle aggregate and cement percentages (Choudhary et al. 2010, Basha et al. 2005).

Triaxial test and direct shear test are conducted to evaluate shear strength of soil. A knowledge of shear strength is required in the solution of problems concerning the stability of soil masses. If at a point on any plane within a soil mass the shear stress becomes equal to the shear strength of the soil, failure will occur at that point. The shear strength (τ_f) of a soil at a point on a particular plane was originally expressed by Coulomb as a linear function of the normal stress (σ_f) on the plane at the same point where, c and φ are the shear strength parameters, now described as the cohesion intercept (or the apparent cohesion) and the angle of shearing resistance, respectively (Craig, 1974). Another test is UCS (Unconfined Compressive Strength) test which widely used to evaluate pavement and soil stabilization application. It is also used as an index to evaluate soil improvement after treatment (Sariosseiri et al. 2009).

1.3 OBJECTIVES

The main idea behind this study was to perform experimental investigation for the evaluation of the interaction between soil and shell husk as geomaterial which is relevant to the ground improvement process. The main objectives of this study can be summarized as follows:

- To investigate the effect of waste shell husk on the ground improvement
- To investigate the optimum percentage of shell husk which mixed with soil.
- To investigate optimum percentage combination soil-cement-shell husk.

1.4 OUTLINE OF THIS THESIS

Chapter 1 contain of background, scope of the study, objectives and outline of thesis

Chapter 2 presents brief explanation about shell husk waste problem, clam shell (source) description, shell husk as recycle aggregate, shell husk composition, recycling seashell waste in concrete, recycling seashell waste in mortar, recycling seashell waste in ground improvement

Chapter 3 discuss about ground improvement technique, factors to choose ground improvement technique, recent advances for future developments, geo-material, stabilization with cement, stabilization by cement and geomaterial (fiber), stabilization by cement and geomaterial (waste recycled)

Chapter 4 describes sample preparation experimental procedure of CBR (California Bearing Ratio), direct shear test, UCS (Unconfined Compressive Strength) and triaxial consolidated-drained tests (CD tests)

Chapter 5 Result and discussion of experimental procedure

Chapter 6 includes the summary, conclusion and recommendation for future work.

CHAPTER 2: WASTE SHELL HUSKS

2.1 OVERVIEW

Shellfish shell has been listed worldwide as one of the worst environment problems due to disposal process. The abandon shell husk without proper treatment could be a source of unpleasant smells due to either the decay of remnant flesh attached to oyster shells or microbial decomposition of salts into gases such as NH_3, H_2S and amine (Yoon et al., 2010). Shellfish have important part in Japanese people dietary and every year in Japan, then large quantity of shell husk waste is generated. According to the Japanese Ministry of Forestry, Fisheries and Agriculture, total amount of abandon shell husk is about 151,000 tons/year and nearly 32 million US$ are being used for disposal cost. In Mie prefecture, 9627 tons/year are produced and 715 tons/year are produced in Tsu city along with 407 tons/year of mactrachinensis (mactridaes) in Tsu city. These mactridaes, which are not only thrown away without any commercial return and took a lot of money for being disposal but also causing pollution and environmental problems. By rapid increasing of balancing requirement between natural phenomena and ecology in bio environment, there is significant need to continuous development of new technologies that using these abandon shell husk (Hossain, 2013). In China, which is the largest producer of shellfish in the world, about 10 million tons of waste seashells are disposed of in landfills annually. This amount of seashell waste primarily consisted of oyster, clam, scallop and mussel shell most of which are landfilled with only a small fraction re-used for other purposes, such as fertilizers and handicrafts. The re-use is limited due to the restriction on the amount that can be used, the problem of soil solidification and economic problems (Mo et al. 2018). In addition, there are problems with illegal dumping of these waste seashells into public waters and reclaimed land. At normal temperature, microbial decomposition of the organic matter produces NH3, H2S and harmful hydrocarbon gases with a significant odor and toxicity which are hazardous to human; on the other hand, discarded shellfish stacking is the breeding places of mosquitoes, flies, mice and insects because the waste of shellfish stacking place with its leachate and moisture is the location of adult mosquito for spawning and larvae breeding habitat (Li et al. 2012)

Many researches are conducted that use shell husk as main material on their research. Li et al. (2012) observed the ability of shellfish shell granule as an alternative filler for polypropylene (PP) by comparing the filler effects of shellfish shell and calcium carbonate on mechanical properties of filled PP and the crystalline structure of polypropylene in the composites.

Shellfish sea powders itself prepared from shell of *mytillus edulis* which is one kind of large number of shellfish breeding in China. Another research is modification of waste shell derived CaO via thermal hydration-dehydration technique is proposed in order to improve the catalyst's characteristic such as textural properties, basicity, transesterification activity and reusability due to high amount of clams (*Meretrix meretrix*) in Malaysia (Asikin-Mijan et al. 2015). In engineering structures and construction area, waste shell husk has been used as aggregate for concrete and ground improvement. In order to reduce the dependency on virgin materials of construction, efforts have been made to incorporate by-products and wastes from different industries as alternatives in concrete. Originating from the fishery industry, seashell waste, such as oyster shells, mussel shells and scallop shells, among others is available in huge quantities in certain regions and is usually dumped or landfilled without any re-use value. Another potential waste material that is available in abundance is waste seashells. There are many different types of waste seashell available, such as oyster shells, mussel shells, scallop shells, periwinkle shells and cockle shells (Mo et al. 2018). Most of the researches which used shell husk as aggregate in concrete recommended the utilization in lightweight concrete type (Hossain, 2013; Yusof et al. 2011; Barbachi et al. 2017). Yoon et al. (2010) found that oyster shells could be a good source as an alternative material to sand for soft ground improvement such as in sand compaction piles.

Now-a-days, recycle aggregates are gradually used for ground reinforcement. The aim of recycle aggregate utilization is one way to apply sustainable development on construction industry including earthwork construction (Li et al. 2010; Ortiz et al. 2009; Tam and Tam, 2006). Recycling principle has benefits which are protecting limited natural resources of aggregates, increasing the abandon waste value and problem solving of waste storage (Hossain, 2013; Tam and Tam, 2006). Many researchers used organic and inorganic materials as recycled aggregates in their investigations. The comparisons between soil without reinforcement and soil with reinforcement have different trends.

In this section will be reviewed the literature of source of shell husk that using in this research. Then the previous researches which had been using shell husk as recycle aggregate were reviewed also in this section.

2.2 SHELL FISH (CLAMS)

Clams are parts of a wide group of mollusks which classified as Bivalvia. There are 7000 species of this class, however less species has economic value. Older classification had referred Lamellibranchia and Pelecypoda as Bivalvia for this class. Two valves are hinged dorsally and jointed by an elastic ligament as the physical shape of Bivalves's shell. The shell composition is consisted of an organic matrix and crystalline calcium carbonate (aragonite or a mixture of aragonite and calcite). The periostracum is outer layer of the shell which secreted by extreme margin of the mantle and has role as the substrate for the deposit of calcium carbonate and the organic matrix of the shell. The ephitelial cells of the mantle secrete the shell materials. At the points of attachment of the pallial, adductor, and pedal muscles, the ephitelial cells are joined to the shell at the outer fold of the mantle. The mantle secreted the organic matrix and calcium carbonate into the space between the mantle and the shell. The extra pallial fluid to form new shell material accumulate the shell constituents. The valves of the shell are held together by two more or less equal-sized adductor muscles to maintain the soft tissue of the clam. These muscles are consisted of smooth and striated fibers so that they able to extend the length contraction as well as rapid movement. Muscle fibers along a semi-circular line which is short distance from the margin of the shell attach the mantle to the shell. In the shell, the pallial line shows the scars of this muscular attachment. The mantle merges to form the inhalant and exhalant siphons at the posterior end of the clam. In the shell, their location is determined by scar of the siphon retractor muscles which known as the pallial sinus. The paired gills or ctenidia and the visceral mass attached within the mantle cavity. The organs of digestion and excretion (stomach, crystalline style, digestive diverticula, nephridia) are existed inside the visceral mass. The gonads are located in the outer layer of the visceral mass. From the visceral mass and protrudes through the shell gape, the muscular foot of clams enlarges anteriorly and ventrally for burrowing into the substrate. Combination of muscular action and engorgement with blood moves the foot that able to make huge extension in some species (Peirson, 2000).

Many methods are used to harvest shellfish such as oysters, clam mussels and scallops are sessile (not free-moving) marine animals. For examples shovels, tongs and rakes as popular simple devices are used for taking these shellfish. Gear or dredge which known as more sophisticated technique, has been used in oyster, clam, and scallop fisheries.

Several types of rakes or dredge are used to take clams and mussels. A regular clam rake has similarity to common steel garden rake excluding the weight which is heavier and longer size,

also it has sharper teeth with distinct around an inch apart and the curved specifically upward. Clam rakes with different widths and handle lengths are constructed for shallow-water digging. On the other hand, a basket rake has modified with longer handle than the regular clam rake approximately 35 feet (11m) long for digging clams in deep water with. A basket or wire or netting is connected to the back of the rake to hold the clams the edge of the basket-handle is suited with a crosspiece to assisted dragging across the clam bed. The bull rake as a third rake which is used in harvesting clams, has a long handle like the basket rake but without mesh basket attached. The bull rake is constructed much similar the regular clam rake, except the size wider and has more teeth. The regular clam rake can be handled from shore and generally clam rakes operated from small boats. In different case, the teeth are operated into the sand or mud of the bottom, then the rake is pulled in and lifted out of the water.

Clams dredges could be operated with or without hydraulic device. Due to its effectiveness in extracting these mud-burrowing clams, the hydraulic or jet dredge is most regularly used for surf clams and ocean quahogs. Throughout operation hydraulic pump on board the vessel supplies pressurized water which is pumped through jets located in front of the toothed bar. The jets of water allow the clam to be scooped more efficiently by losing the bottom. The clams are accumulated in the metal ring bag or storage directly on the deck through conveyor by the hydraulic dredge (Collete, 2000).

2.2.1 Surf clam

The shape of the surf clam shell is oval by varying colors from cream to tan, with the umbo midway between the anterior and posterior edges. The shells are coated with a shiny periostracum and the valves gape slightly. Two partially-joined cardinal teeth are existed on right valve. The siphons can be fully retracted into the shell which is unified all the way to their tips. Most clams are completely mature in their second year age however surf clams are sexually mature after the size reach 45 mm on a year age. The force preventing the tendency of the hinge to open the shell is reduced while surf clams are out of the substrate (which many are when at high densities), which demanding more effort of the adductor muscles to control the shell closed. Sometimes the predators could get the chance when the muscles exhausted then exposes the soft body. To prevent predator, clams usually use sensory cues from the siphons. As function of photoreceptors which allow withdrawal as a response to shadows, the siphons have sensory papillae that give instruction to close or withdrawal when stimulated. When get out from substrate, clams show as quick as possible lengthening the

foot to do evasive jump. Around 300 mm of height could be reached by a large clam touch the ground about a meter away (Pierson, 2000).

The shell of surf clam is quite strongly constructed due to its habit which like to burrow in rocks. It is known surf or trough clam has a smooth surface, with concentric growth lines and covered by thin periostracum (Vaughan, 2001). The clams are stored inside metal mesh cages after harvesting. Next process is included unloaded and storage inside processing area. These clams are put on the conveyor belt which moves through a gas flame, steam or hot water bath to open the shell. Then meats are extracted by using the hand, mechanical shakers or eviscerator after the shell opened. The surf clam meats are packed and frizzed, then the sale and shipped for further processing (Downey et al. 2012). The whole process left the shell husk as the by-product which necessary to get proper treatment. The shell husks are contained organic material then decomposing into gas compounds which caused odor and harmful for the environment, unfortunately most of the shell husk is usually disposed of in landfills. Furthermore, shell stacking becomes the place of insects breeding and habitat of other pest types. Based on that condition, it is essential to find the solution in purpose reducing those environmental problems, especially in those countries where shellfish product industry is well established (Li et al. 2012). According to CCRF regulation (Code of Conduct for Responsible Fisheries), fisheries processing has to operate optimally and environment-friendly. Handling of shell husk as a solid waste of shellfish product industry require serious effort to get the benefit and reduce negative impact to human and environment (Agustini et al. 2011).

2.2.2 Composition of seashell waste

Generally, basic composition of seashells such as oyster shells, scallop shells, mussel shells, cockle shells and clam shells, which have been used in earlier investigation are calcium carbonate ($CaCO_3$) and calcite. Calcium carbonate (C_aCO_3) is the highest part of shells chemical composition approximately more than 90%, it was observed similar with the limestone powder or dust-like stone powder to compose Portland cement (Lettawarnuk et al. 2012). Mo et al. (2018) mention that CaO (Calcium Oxide) compound of oyster shells are vary between 48.0% and 86.8% with a high loss on ignition (LOI) ranging between 23.2% and 51.0%. with a high loss on ignition (LOI) ranging between 23.2% and 51.0%. LOI is happened due to decomposition of calcite to form CaO and CO_2 (carbon dioxide), the calcite could be observed from the XRD existence pattern of the material.

To substitute ordinary aggregate in concrete seashell waste are used as coarse and fine aggregate. Generally crushing of seashell waste with sizes <5 mm are used as fine aggregate. In compare to sand shape, crushed seashell fine has a flakier and elongated particle shape which observed flakiness index of 96.9% for crushed scallop shell aggregate. Mo et al. (2018) found that the oyster shell fine aggregate particles were found have similar fineness compared to normal sand, which is reflected by the fineness modulus of between 2.0 and 2.75 which is sieving below 5 mm, however by using minimum sieve size of around 1 mm the fineness modulus was found approximately to 4.0, which was coarser than normal sand. In plain concrete, coarse aggregate which generated from seashell waste for example periwinkle shells and mussel shells are usually used in the uncrushed form with a maximum size of between 16 and 25 mm. On the other hand, for smaller sizes of crushing seashells has maximum size between 4 and 9.5 mm which integrated in previous concrete as coarse aggregate.

2.3 RECYCLING SEASHELL WASTE IN CONCRETE

Recently sustainability of natural resources due to depletion of natural source materials of concrete and its consequences of environmental impact become the big issue which give impact to the present condition of concrete engineering, the trend in concrete engineering has been changed. Therefore, the utilization of recycle waste materials as replace for ordinary materials in concrete due to enhancement awareness of concrete production sustainability. To reach sustainability purposes, many studies have been performed which using waste of vary sources for example construction and demolition waste. Waste materials usage in concrete are expected to resolve the disproportion consumption of conventional materials and in the same time decreasing number of waste generated (Mo et al. 2018). Several waste by-product materials such as fly ash, quarry dust, and silica fumes which could increase the properties of concrete such as strength, stiffness, shrinkage, density, creep, durability and permeability of hardened concrete are suggested by some scientist. The characteristics of waste are determined which one proper material that could be used as aggregate. According their source such as agricultural, by-product, industrial and mineral, waste materials and by-products are defined into four main groups (Yusof et al. 2011).

Waste seashells is one of potential source material which existed in enormous amount. There are many different types of waste seashell available, such as oyster shell, mussel shell, scallop shells, periwinkle shells and cockle shells. China is biggest shellfish producer in the

world, approximately 10 million per a year waste seashells are generated then disposed in the landfill. Majority of seashell waste contain of oyster, clam, scallop and mussel shell, the shell small percentages of shell waste that landfilled are re-used for other utilize such as fertilizers and handicrafts. There are boundaries to re-use this material such as limited amount that can be used, soil solidification, and economic problems. Another problem is illegal dumping activity, untreated abandon shell waste in certain period of time lead air pollution such as odors due to microbial which decompose remaining flesh in the shells into gases, such as H_2S, NH_3 and amines. These problems could be decrease the sanitation quality of the people who live nearby then become environmental pollution issues (Mo et al. 2018).

There are many studies which investigate effect of ground SBP_s (Seashell by products) as a substitute material. For example, the utilization of cockle shell enhances the compressive strength of cement-sand mixtures. Generally, the conclusion showed that cockle shell which replace the ratio of cement decreases the compressive strength when comparing to the mixtures which has higher cement content. On the other hand, for workability and plasticity, the compressive strength is not main point for rendering and plastering utilizations (Motamedi et al. 2015). Therefore, mortars and concrete materials which composed of the SBP_s could be used for application that demand lower strength material.

The application of waste materials to produce concrete in the concrete engineering and technology field have been reach extraordinary achievement then improve expectation of waste management and concrete. The achievements have focused to identify and optimize varying types of cement substitute materials as well as different options aggregate in concrete (Yusof, 2011).

Investigation the utilization of ground seashells as a stone-like replacement material to produce concentrate and mortar has been studied earlier. For instance, a freshwater snail, genus *Viviparus*, has been applied as an alternative material aggregate in concrete producing. The compressive and tensile strengths and workability of the concrete decline when the replacement amount of freshwater snail in the concrete mix increasing. A study of ground oyster shells found that the shell is mainly composed of calcium carbonate and small organic compounds (Lertwattanaruk et al. 2012).

Hossain (2013) studied the general effect of the mechanical properties of concrete or cementitious composites with varying percentage of abandoned mactridae shell husk as coarse aggregate which are 0%, 10%, 0, 30%, 40% and 50% in the ratio of mass. Various percentages of abandoned mactridae shell husk are tested under heavy environmental

conditions for a period 28, 88, 148, 208 and 268 days. In severe environmental conditions several types of evaluations such as compression tests, permeability tests and durability tests on concrete containing different percentage of abandoned mactridae shell husk of the conditions and periods that mention above are conducted. Based on the results of those experimental tests, it showed possibility to use abandoned shell husk in concrete construction. The results of engineering properties such as compressive strength, Young`s modulus, tensile strength, unit weight, water absorption capacity and coefficient of hydraulic conductivity fitted with the ACI results. The type of concrete which reinforced by shell aggregates are suitable for light weight concrete construction where strength is not a major factor but the water absorption properties more important for example partition wall, slope surface protection, ridge between paddy fields, roads and embankments slopes and sea-shore protection structures.

Barbachi et al. (2017) investigated shells as aggregate from the coastal area to produce composite material called ecological concrete especially on feasibility area in on from marine coasts of the Souss Massa region in Agadir (Morocco). There are various types performed, initially, in order to identify and quantify the various types of waste mapping, it performed prospecting step on different spots of specific area. After the first step had been done, the shell husk get thermal purifying purpose then followed by crushing treatment. Next step evaluated physical characterization of varying aggregate along with shells, especially to specify the bulk density sizing, the apparent particle density and compactness of shells. It also conducted a geometric specification by means particle size analysis. The sand dune and quarry sand were the standard for those parameters which found. The physical characterization of sea shell waste aggregates originally collected from Agadir area in order to enhance concrete formulation is the objective of this study. Scientists in this scheme investigated the attribute of varying conventional ingredients of concrete (two types of sand) and those of shell husk crushing. Previously, scientists performed a survey in the Souss Massa area at location which known as Cap Ghir. Then this study decides shells of mussels due to analysis results of this site which showed that dominating of these shells. Finally the scientists withdrawn the conclusion based on several tests such as particle size analysis, bulk volume and absolute mass, that the shells of crushed mussels are potential as ingredient of lightweight concrete. The results showed that the crushed shells have lower compactness then the sands in this study by calculating the percentage of vacuum showed.

Yusof et al. (2011) studied the possibility of using the local clam (*lokan*) which adding in the

concrete as beach retaining wall. In order to enhance the maximum stress of the concrete and, the scientist taking the chance to decide the potency of *lokan* shell powder as the alternative fine aggregate since the shell is known as abandon product and normally thrown out in open area. Local clam shell which is usually known as *lokan* among local people and has the scientific name polymesoda expansa is selected as an aggregate in the concrete. It has habitat in stiff mud of the mangrove swamp with huge and heavy bivalve. When preparing the dish, the shells has been thrown away after the edible past taken out. The waste at the landfill and amount of food waste has developed due to that activity. The study about the *lokan* shell characterizations of the *lokan* are limited. On the other hand, a study about the blood cockle (known as another close bivalve) is contained calcium carbonate C_aCO_3 which construct approximately 98.7% of the total mineral composition of the shell. It is because plants and animals on water absorb calcium carbonate which majority dissolved form of calcium hydrogen carbonate $C_a(HCO_3)$ [The European Calcium Carbonate]. Based on this fact, it could be assumed that the shell of *lokan* contained of large portion of calcium carbonate. Mechanical properties determination of the concrete which contained fine aggregate 10%, 20% and 30% clam shell of sand total mass sand is the primary objective of this research. In concrete production, fine aggregate could be determined as the particle size up to 5 mm while the coarse aggregate if from 20 to 79 of an inch (20 mm). There are three specimens as repetition for each sample which contained fine aggregate from the clam shell, then to take final results average value is calculated of those specimens. The investigation also prepare specimen which has not contain any percentage of *lokan* powder is known as reference concrete (control), in order to compare the effect of *lokan* powder percentages in terms of compressive strength of the concrete. The conclusion of this study showed that the utilization of clam shell powder in the concrete could enhance compressive strength of concrete. As the filler in the concrete, the clam shell powder could reduce the number of voids, then improve the compressive strength of the concrete. In initial stage of study showed that fine aggregate in the concrete which produced from clam shell which known as abandon material has high potential as excellent filler. From this study is obtained that specimen which contained 30% *lokan* powder of total mass give highest compressive strength, and could become potential candidate as concrete that might be applied in the beach retaining wall construction.

2.4 RECYCLING SEASHELL WASTE IN MORTAR

The government improves new approach of waste-utilization programs due to awareness environmental condition. The characteristic of concrete which was added Seashell-By-Products (SBP) showing that material gave positive results based on studies of many scientists. The Calcium Carbonate (C_aCO_3) is the main component of SBPs materials, approximately more than 90% of the total weight based on the result of previous study (Motamedi et al. 2015). Then C_aCO_3 is expected as economic substitute material to replace sand and cement in concrete or mortar comparing to other type materials due to most dominant results of those studies.

To skimp budget production, ground seashells also are used as material to replace cement or sand. It is interesting that green mussel and cockle shells have higher strengths and densities compared to limestone powder due to the crystal structures which largely composed of aragonite and calcite. Particles of calcium carbonate crystallizes in the form of calcite have rounder and lighter shape than sand particles. The compressive strength of mortar which using ground oyster shells to substitute sand shows none significantly decreasing results. The decreasing of fineness modulus, give the declining result of concrete workability because there is no reaction between oyster shell and Portland cement. The development of compressive strength and the lower the elasticity modulus are affected by the higher substitution levels of ground shell oyster in sand (Lertwattanaruk et al. 2012). The limestone has round shape while ground particles mussel shell known slim needle shaped which give affect the internal morphology of mortar mixed with mussel shells has a structured mesh and smaller pores. Mortar which is mixed with limestone powder has lower compressive and bending strengths compared to mussel mortar which added shells (Ballester et al. 2007). The investigation of Motamedi et al. (2015) show the ground SBP_s (i.e. cockle shells and mussel shells) achieve higher compressive strength compared to the limestone powder, due to existence Aroganite (a form of C_aCO_3) and Calcite (the most stable polymorph of C_aCO_3).

A new filler material from limestone which used as ingredient in mortar has gathered from mussel shell waste of the cannery industry (Ballester et al. 2007). The main source of limestone has been managed by industry which performed the reuse and recycling process. Annually more than 80.000 tons of limestone are produced in Galicia, a northwestern region where the cannery industry is essential. Aggregates from quarts-limestone-dolomite mixtures are usually used. The workability and mechanical properties of mortar has been changed by the binder/aggregate ratio and aggregate characteristic (*viz*, grain size distribution, chemical composition and particle shape). New formulations for better performance mortars could be

achieved by an appropriate choice of aggregates. A specific particle morphology separating from that typical of quarry limestone is possessed by the limestone which used in this work. The physico-mechanical properties of the mortar contained substitution material from mussel shells has increased comparing to ordinary limestone which concluded based on the experimental results.

The shrinkage of mortar is truly influenced by difference types of powder materials. For example in mortar, some varying types of fly ash give affect enlargement development and slightly affected shrinkage. The utilization of silica fume enhanced density of concrete then reduced drying shrinkage. The properties of masonry and plastering cement could be developed by limestone powder. Limestone powder is relatively inert and grouped as variety of aggregate which the drying shrinkage could be reduced by the mixing limestone powder in mortar (Lertwattanaruk et al. 2012).

Mostly, there are three components inside dry mortars which namely as binder, an aggregate and additives. Mechanical properties such as strength, durability, rheological behavior or workability are increased by the integration of new components (additives material dominated) in recently mortar formulations. It could decrease production costs in some works. Substitution material is interesting method to decrease production costs due to ability to maintain or increase mechanical properties then expected replacing cement as binder element in mortar, which known as most expensive components (Ballester et al. 2007).

Lertwattanaruk et al. (2012) studied substitution materials in the production of plastering cement which properly for general application, based on report in 2006 of Department of fisheries, from the utilization 4 types of waste seashells; short-necked clam, green mussel, oyster and cockle are the most popular shellfish in Thailand. Portland cement and four types of ground seashells were incorporated to produce the mortar. Every type of the 4 seashells received comparable properties in the mortars containing seashells which had purpose for plastering and masonry construction. The compressive strengths of mortars containing ground seashells show decreasing result compare to control mortar. Substitution material of ground green mussel in Portland cement give lower compressive strengths than those gathered from other 3 types of ground seashells. Nevertheless, the compressive strengths of ground seashells mortars were suitable and higher than those demanded by standards for plastering. Afterwards drying shrinkages of mortars which using replacement material from ground short-necked clam and ground oyster in Portland cement, compared with the control mortars show decreasing result. Mortar which using ground oyster material show higher

shrinkage than mortar mixed with ground short-necked clam. Drying shrinkages of the mortars which using substitute material from ground green mussel and ground cockle show increasing results compare to control mortar. The control mortar show higher thermal conductivities compared to mortars containing ground seashells. Substitution material of ground green mussel in Portland cement give lower thermal conductivities than those achieved from the other 3 types of ground seashells.

As an aggregate in mortar, limestone which obtained as a by-product from waste of the mussel cannery industry is an effective alternative which develop mechanical properties relative to quarry limestone (Ballester et al. 2007). The establishment of a reticulate network that is a more efficient host for the fine needles of calcium silicate hydrate formed during the setting of cement is facilitated by the existence of significant fraction of large elongated particles in the previous type limestone, rather than the more rounded particles of q-limestone. The existence of smaller pores in the mussel limestone-based mortars are persistent with more efficient filling of the pores and responsible for developing compressive and textural strength which showed by mercury porosimetry. Along with the increasing of the mussel limestone content in the mortar, an improved kinetics of cement hydration and portlandite carbonation give suitable thermo-gravimetric. In the end, mechanical properties developed by substitute cement with this limestone. It allows lower binder/aggregate ratios in formulation of new mortars. For that reason, the utilization of the limestone by-product of the cannery industry to produce mortar has the following opportune: (i) it provides a new alternative to manage this industrial waste; (ii) it decrease mortar production costs; and (iii) it has good impact for the environment as it reduces demand of new quarries exploitation.

Other investigation was performed by Motamedi et al. (2015) which evaluated the effect of cockle shell content on the Unconfined Compressive Strength (UCS) of the cement-sand moistures. In this study to justify exactly prediction UCS values for cockle shell-cement-sand mixture are estimated by using new soft computing techniques. It has mechanism to collect input/output data pairs and use these data to learn the proposed network as the basic idea behind the soft computing methodologies. Three SVR_s were investigated: (1) polynomial function (SVR with polynomial kernel), (2) radial basis function (SVR with RBF kernel) and (3) linear function (SVR with linear kernel). According to comparison between ANFIS resulted and the achievement of the SVR_s generated in forecasting system developing. To obtain the RMSE and the coefficient error for the UCS estimation, ANFIS could predict more accurate compared with the SVR_s results. In consequence, this research withdraws the

conclusion that ANFIS method gives more specific result for the UCS distribution. The following results compile the results of this research: first, cement has the highest sensitivity on contrary with cockle shell which has the smallest sensitivity on the output the variable based on screening analysis of input parameters on the output result. Second, ANFIS is suitable tool estimating the UCS according to the quantity of input parameters based on the recommendation of this study. Third, for the UCS forecasting, the ANFIS estimations give more accuracy results than SVR in terms of the RMSE and the coefficient error. Fourth, the soft computing methodologies performed excellent learning and prediction ability. In this case, the experiments showed that the estimation model accomplish the major inadequacy of the ANN without out-lining the network structure and trapping in the local optimum.

2.5 RECYCLING SEASHELL WASTE IN GROUND IMPROVEMENT

In many countries such as China, South Korea and Taiwan have similar problem on oyster shell waste. From the total abundance seashells waste in China are obtained around 370-700 g of waste shells for each kg of oyster shells, at the same time every year in Taiwan approximately 300,000 tons of oyster shells are existed and 160,000 tons of oyster shell waste being part of those shell waste (Mo et al. 2018). On the other hand, annually it is predicted 300.000 tons of waste oyster shells are produced in Korea which give severe problems from economic and environment aspects (Yoon et al. 2010).

To develop recycling of waste oyster shells, Yoon et al. (2010) studied the important characteristics of shear strength and deformation of crushed oyster shell-sand mixtures. The Standard Penetration Tests (SPT) and large-scale direct-shear tests were conducted by using varying kinds of dry unit weight and mixing rate of oyster shell-sand mixture. It give a chance to predict the in situ strength from SPT and the coefficient of volume compressibility from the confined direct-shear compression tests which has relation between *N*-value, dry unit weight and friction angle of mixtures which obtained from the results of experimental tests. For soft ground improvement application, those results give the possibility to calculate the adjustment of oyster shell-sand mixture. To investigate their characteristics under different conditions of initial dry unit weight and vertical stress in the tests, in this research were performed new experiments on oyster shells mixed 1:2 with sand. The standard penetration test and large-scale direct-shear compression test were conducted to obtain the shear strength and deformation characteristics of oyster shell-sand mixtures. The results from investigation could be pointed out as follows:

(1) It was similar with sandy soil, SPT *N*-value and their stress-strain behaviors get larger value by higher the confining pressure and the dry unit weight of the oyster shell-sand.
(2) The friction angle has ranged approximately 50^o to 60^o for the oyster shell-sand mixtures.
(3) On the same dry unit weight, the friction angle of the oyster shell-sand mixtures decline with increasing vertical stress. On the other hand, the friction angle relatively stable for vertical stress more than 150 kPa.
(4) Even though overestimation was obtained for initial dry unit weight of the mixtures more than 10.5 kN/m^3, experimental results and suggested establishment show good agreement. It is concluded that oyster shells could be a good resource as a preference material to sand for soft ground improvement such as in sand compaction piles.

Several investigation obtain that optimum level for aggregate substitution from seashell are existed in term compressive strength development. There are researchers found that 5% oyster shells and scallop shells which substitute sand an increase in the compressive due to the effective filling of voids, but a further increase in the replacement level to 20% and 60%, respectively, gave lower compressive strength. On other research showed that at the early age compressive strength of mortar was higher compared to that of mortar without oyster shells were contained up to 20% fine aggregate replacement which assumed cause the water absorption of the oyster shells, which effectively reduced the w/c ratio of the mortar (Mo et al. 2018). On the other hand, the compressive strength of the mortar with oyster shell aggregate was lower compared to the control mortar at later ages beyond 56 days which was associated to the build-u of stress concentration over time on the weaker oyster shell aggregate (Mo et al. 2018). Yusof et al. (2011) discovered a development in the compressive strength when addition up to 30% fine particles of calm shell (<0.5 mm) and cockle shell (<1.18 mm) aggregate were incorporated as sand substituent in concrete and cement-sand bricks, respectively.

Combination between pozzolanic material and seashell waste which is recognized to be an inert material expected give better result. However some researcher found contrary results in this case. When fine oyster shell powder was used as an addition up to 20% in cement-fly as brick mixtures, it was obtained that compressive strength developed (Mo et al. 2018). For case cement-fly ash brick mixtures, the existence of oyster shell powder which give progress the pozzolanic reaction with fly ash as the calcium hydroxide concentration. The mortar containing oyster shell fine aggregate showed a lower decreasing value in the compressive strength compared to the plain cement mortar with oyster shell fine aggregate when ground

granulated blast furnace slag was incorporated at 75% cement replacement level. Opposite effect on workability could be decreased, but it is necessary to give limitation amount of aggregate substitution. There is inadequate available literature to withdraw conclusion of seashell powder as a cement substitution for the workability effect. However, the decreasing cement content through substitution with more seashell powder could develop the workability as the degree of hydration is lower. On other research, for seashell powder which was used as a filler material, the fineness is usually higher than cement which could enhance the water requirement (Lertwattaranuk et al. 2012).

CHAPTER 3: GROUND IMPROVEMENT

3.1 SHORT DESCRIPTION

There are increasing requirements of the land utilization to improve living condition due to civilization and urbanization process. Many constructions are built and showing sustainable condition in future such as houses, commercial buildings, high-rise office buildings, highways, railways, tunnels, levees, and earth dams. It is necessary to use low profile of sites for construction causing the suitable construction sites with favorable geotechnical conditions become are limited. Recently increasing of geotechnical obstacles which becomes challenges, for example bearing failure, large total and differential settlements, instability, liquefaction, erosion, and water seepage are necessary to get big attention by engineers. Some solutions to solve geomaterials and geotechnical problems such as: (1) abandon the site, (2) make a plan to build suitable extraordinary structures (3) utilize better quality of geomaterial after dismiss and substitute unsuitable geomaterials (4) developing geomaterials properties and geotechnical conditions. Developing geomaterials and geotechnical conditions grow into important things for many projects.

For geotechnical practice, ground improvement turn into major factor. Many terminologies which have similar meaning with ground improvement such as soil improvement, soil stabilization, ground treatment, and ground modification. Most literature and practice are using the term “ground improvement” which adopted for book by Han (2015).

To accommodate better achievement under planning and/or operational loading conditions, ground improvement is the method which modifying the existing soils foundation. The new projects which accept lack subsurface condition of selected site

use ground improvement techniques to face the problem. Initially, this condition force to switch the project location or substitute with engineered fill due to poor soils were considered as economically unjustifiable or technically not feasible. In brief, ground improvement has purpose to develop the bearing capacity, decrease the magnitude of settlements and the time in which it happens, obstruct seepage, accelerate the rate at which drainage occurs, enhance the stability of slopes, mitigation of liquefaction potential, etc. (Hirkane et al. 2014).

Compare to other deep foundation methods, many ground improvement methods give much less environmental impact for illustration concrete piling. One project in the UK which known as the Dartford Park is the example of creative application of various ground improvement methods which give advantage to finished that project and others. Based on experience which achieved from this projects show that farther utilization of ground improvement technique give bigger chance to decrease the environmental impact compared with traditional method, in other words ground improvement suggest more sustain benefit. Sustainable methods had applied on general projects which showed by previous case studies. Some recommendation has proposed to solve the problem which obstruct bigger sustainability advance (Egan and Slocombe, 2010).

According to soil properties, a proper technique of ground improvement should be considered economic aspect during the time application. The application of ground improvement covers wide scope construction for industrial purpose, commercial and housing projects to infrastructure construction for dams, tunnels, ports, roadways and embankments (Hirkane et al. 2014).

Since the 1920s, modern ground improvement methods were established for example, the use of vertical sand drains to enhance consolidation of soft soil was first suggested

in 1925 and then patented in 1926 by Daniel D. Moran in the United States. In 1926, South Carolina Highway Department in the United States applied the cotton fabric as reinforcement for roadway construction. In Germany 1937, to solidify loose cohesion-less soil was used the vibro-flotation method. Walter Kjellman in Sweden 1947 expanded the first type of prefabricated vertical drains. In Italy 1952, the root pile method to under-pin existing foundations was developed and patented by Fernando Lizzi. There were several expansions of ground improvement techniques such as the steel reinforcement for retaining walls by Henri Vidal in France, dynamic compaction by Louis Menard in France, deep mixing in Japan and Sweden, and jet grouting in Japan in the 1960s. The expansion from geotextiles to geosynthetics is a innovation in geotechnical engineering which stated by J P. Giroud in 1986 (Han, 2015).

3.2 FACTORS FOR SELECTING GROUND IMPROVEMENT METHOD

Han (2015) state that ground improvement techniques application should consider the following conditions: (1) structural condition, (2) geotechnical conditions, (3) environmental barrier, (4) construction condition, and (5) reliability and durability.

1. *Structural conditions* Shape and dimension of structure and footing, flexibility and ductility of structural and footing elements, type, magnitude and distribution of loads and performance demands (e.g., total and differential settlements, lateral movement and minimum factor of safety) are contain of the structural conditions.
2. *Geotechnical Conditions* Geographic landscape, geologic formations, type, location and thickness of problematic geomaterials, possible end-bearing stratum, age, composition, distribution of fill and groundwater table were the factors of geotechnical conditions.
3. *Environmental constraints* Limited vibration, noise, traffic, water pollution,

deformation to existing structures, spoil and headspace were parts of the environmental constraints.

4. *Construction condition* (1) site condition, (2) allowed construction time, (3) availability of construction material, (4) availability of construction equipment and qualified contractor and (5) construction cost are the factors of construction condition that should considered during selection of a ground improvement method. The site should be accessible to its associated construction equipment when choose appropriate ground improvement method for example access road and headspace. One major factor of the ground improvement method is construction time.
5. *Reliability and durability* Several factors which included on reliability of a ground improvement were the level of establishment, variability of geotechnical and structural conditions, availability variance of construction material, quality of the contractor, quality of installation, and quality control and assurance.

3.3 RECENT ADVANCES AND TRENDS FOR FUTURE DEVELOPMENTS

Ground improvement techniques have been developed in many ways. Innovations and developments in the technology which provided consistently by manufacturers have made important contribution to these recent advances. Scientists also have important role to support improvement on design methods. Han (2015) mentions several points of the current advances as following:

- Geosynthetic-encased stone columns, controlled modulus (stiffness) columns, hollow concrete columns, multiple stepped columns, X-shape or Y-shape concrete columns, grouted stone columns, T-shaped DM columns; and composite columns are several varying types of column technologies.
- Column-supported embankments.

- The SHRP II R02 team cutter soil mixing method to construct trench walls horizontal twin-jet grouting method which improve online interactive technology selection system.
- Application of Intelligent compaction on unbound geomaterial.
- Utilization of recycle materials in ground improvement.
- Utilization of combination technology of two or more ground improvement techniques such as; combination of deep mixed columns with prefabricated vertical drains; combination of short and long columns; and combination of geosynthetic reinforcement and columns.
- Utilization sensor-enabled geosynthetics.
- Application of computer monitoring in ground improvement construction.
- Using biological treatment.

For future development on ground improvement methods are available several common trends which withdrawn as follow:

- In order to get technically and cost-effective are utilized mix technology
- To enhance efficiency and quality of ground improvement are used intelligent construction technologies with sensors and computer monitoring
- In purpose to get sustain ground improvement are used of recycled materials and other alternative materials
- Utilization of end-result or performance-based specifications
- Biological treatment on site application

3.4 GEOMATERIAL (RECYCLE AGGREGATE)

Egan and Slocombe (2010) explained that the environmental impact of any product or service can be minimized by application of the principles of reduce, reuse and recycle.

1. Reduce

Decreasing quantity of raw or processed material in a product has purpose to protect natural sources, decrease utilization of energy demand which expected decrease greenhouse gases released are several examples in reduce method. Application of ground improvement show the demand prevention for piles which affect fewer raw material utilization (stone aggregate) and energy during the construction process (e.g. to make cement and steel). The focus on research which conducted by Egan and Slocombe (2010) is about decreasing the carbon dioxide footprint (an indicator of embodied energy) of deep foundation solutions by using ground improvement technique as an alternative to piling.

2. Reuse

When the previous structure will be demolished or redeveloped, traditional steel and concrete piling creates artefacts which remain in the ground. In order to solve that problem, the reuse of piles is to be encouraged according to other research which conducted by Butcher et al. in 2006, however only a relatively few projects have been completed. Vibro application gives lower problems in the ground and none where dynamic compaction is applied. Those things verified potential reuse for future application of ground improvement on site. Many projects have applied redeveloping of vibroed sites and reusing of stone columns. For example in specific projects new stone columns have been installed next to or within the original stone column layout.

3. Recycle

Raw materials and energy are saved by recycle method. Utilization of recycle aggregate gives great chance to apply at stones columns which conducted by Serridge in 2005. On the other hand in comparison with virgin stone, such material may have other physical properties (Egan and Slocombe, 2010).

Han (2015), state that other materials are source of processed or manufactured geomaterials. For illustration, rock is source of crushed stone aggregates and recycled asphalt pavement (RAP) from aged asphalt pavements. For varying applications lightweight aggregates are produced by giving temperate for raw shale, clay or slate in a rotary kiln at high temperatures, causing the material to enhance, cooling, crushing, and screening it. Processed or manufactured geomaterials have a high diversity, ranging from granular fill, lightweight fill, uncontrolled fill, recycle material, fly ash, solid waste, and bio-based byproducts to dredged material which are mainly utilize for fill materials. Wide variation of fill materials are expected to develop soil properties (e.g., granular fill and fly ash) but several variance could become big problem for geotechnical application (e.g., uncontrolled fill and sludge). Uncontrolled fill or un-compacted fill has stabilization below its own weight due to loose and under-consolidated characterization.

The geomaterials are treated hydraulically, mechanically, chemically and biologically known as improved geomaterials. As illustrations, sand or clay could be combined with to develop fiber-reinforced soil. Other sample is lime or cement-stabilized soil which made by mixing soil with lime or cement. The utilization of biological aspect is denitrifying bacteria which inserted into soil to generate tiny, inert nitrogen gas bubbles to decrease the degree of saturation of sand. Then the outcome is liquefaction potential

sand decreasing. For geotechnical application, improved geomaterials are often the end products of ground improvement which has none problem (Han, 2015).

3.5 SOIL TREATED CEMENT

Sariosseiri and Muhunthan (2009), report that many documentations show the advantageous effects of cement treatment on the achievement of large range soil. Cement utilization has several specific benefits, other than stabilization agents. It could stabilize immediately, does not need mellowing time, and gives a non-leaching platform. The addition of cement to silt as coarse-grained materials gives the best results of cement application. Modification and stabilization are the purposes of cement utilization. The stabilization has meaning for mechanical behavior developing then modification to explain increasing in workability and compaction characteristics which both of terms are used in research of the term cement treated soils. Modification accelerates construction activity by increasing the drying level of the soil, its workability, compactability and wet strength.

1. Portland cement

Intergrinding clinker and gypsum are source of Portland cement which has characterization finely divided material. Clinker is a pyroprocessed hydraulic material composed of four major oxide phases: tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A) and tetracalcium aluminoferrite (C4AF) (in cement chemistry notation, C = Ca, S = SiO_2, A = Al_2O_3, and F = Fe_2O_3). According to soil stabilization purpose, the two calcium silicate phases are the most important parts. Over the hydration process, these two phases generate both calcium hydroxide, which provides available calcium for cation exchange and flocculation and agglomeration and calcium silicate hydrate (C-S-H), the offers strength and structure in the soil matrix:

$2C_3S + 6H \rightarrow C\text{-}S\text{-}H + 3Ca(OH)_2$ (1)

$2C_2S + 4H \rightarrow C\text{-}S\text{-}H + Ca(OH)_2$ (2)

Where $H = H_2O$ and $C\text{-}S\text{-}H = C_3S2H_3$.

Hydration is begun and calcium concentration in the soils developed in short time when portland cement mixed with water. When water and cement are mixed, in 12 minutes the solution becomes saturated with calcium hydroxide (1). Then at the time calcium ions (Ca^{2+}) are released in solution which existed to stabilize the clayey soil. The settlement of cement is associated with a drop in calcium content after 12 h where a significant amount of calcium and water are used to form C-S-H and Ca(OH)2 (2). In the beginning the promptly absorption of calcium by the clay is happened and then slowly due to development of diffusion dependent. Calcium hydroxide is product of these reactions; in such way calcium is replenished as the beginning supply cause by unhydrated cement which is reduced. In a cement-soil system, the calcium hydroxide crystals that formed are highly dispersed which remained in the very fine shape and highly reactive particles of pure “hydrated lime”. These crystals are illustrated “more reactive than ordinary lime” which describing by Herzog and Mitchell (3). To continue stabilization of the soil particles, for clay the calcium in the crystals and in the pore solution are available.

Typically, 75 percent part of cement is C_3S and C_2S, for type I and II cements. More than years, hydration processes are continuously happened at an ever-slowing pace, hence calcium hydroxide is generated during this time. In mortar and concrete system, it could help maintain high pH levels of about 12.5 which occurring of high pH important for long-term pozzolanic reactions, therefore maintaining a high pH in a soil-stabilizer system is important (Prusinski and Bhattacharja, 1999).

2. Cementius Hydration

The particular process which happened in cement but not lime is cementitious hydration. Equations 1 and 2 are shown cementitious hydration which generates cementitious material. Portland cement when hydrated, also generate calcium-aluminum-hydrate (C-A-H) and $Ca(OH)_2$, in addition to C-S-H where C-S-H and C-A-H develop a network and serve as the "glue" that gives structure and strength in a cement-modified clay. Through cementation process, the hydrates support stabilization of the flocculated clay particles. Between one day and one month, the fastest strength developed however for years, smaller gains in strength due to continued hydration and formation of cementitious material-continue happened.

The first month after mixing show the formation of a network of cementitious material provides significant increasing in strength of the cement-soil system. High-strength mass are established due to cement which enhances strong bonds between the hydrating cement and clay particles. By developing larger aggregates from fine-grained particles, it also raises the gradation of stabilized clay soil. Some results recommends that formation of cementitious bonds decreases the leaching potential of calcium hydroxide when the soil is exposed to seasonal wetting and drying cycles or when ground water moves through the stabilized soil (Prusinski and Bhattacharja, 1999).

Sarriosseiri and Muhunthan (2009) had obtained problem with high water content and low workability of local soils in the western side of Washington state have often caused difficulties for highway construction projects. Then they conducted investigation on the effects of the addition of cement on three different soils from the state of Washington on their solidification, plasticity limit, compaction characteristic, unconfined compressive strength and undrained triaxial shear behavior. The performance moves to better control

of workability during compaction and savings over removal and replacement of fill material in some projects by the additional low percentages of cement. The following results that obtained from this research:

- The drying rates of the soils are significantly developed by the addition of a low amount percentage of cement. At the initial stages is significantly high effected but modest after about half an hour of treatment.
- In the beginning the addition of cement developed plasticity index while higher percentage of cement cause decreasing on plasticity index. Thus, better workability of soil has achieved by cement addition.
- The addition of cement has effect to enhance optimum water content and reduce maximum dry unit weight of the soils.
- Generally, cement additions cause significant development in unconfined compressive strength and modulus of elasticity of the soils which has more significant effect in mechanical behaviors on Palouse loess than Aberdeen and Everett soils. Unconfined compressive strength results show more significant effect for Aberdeen soil soaked samples with 7.5 and 10% cement addition than un-soaked samples.
- Soil only show lower brittle behavior than soils which added by cement percentage.

Based on the results of undrained triaxial tests exhibited that cement treatment developed shear strength significantly but failure type behavior has wide variety. Non-treated, 5% and 10% cement treated soils showed ductile, planar and splitting type of failure, respectively. For soils which treated by 10% cement show zero effective confining pressure at failure due to pore pressure increased rapidly. Then the specimens

split vertically. The increasing in strength could be reached by cement addition but for field application of cement on high percentages need ultimate caution.

3.5.1 Stabilization by cement and geomaterial (fiber)

For many geotechnical engineering problems, such as road and railways constructions, technology of treated soil by cement injection has become an alternative and economical solution. Basically, to make soil more resistant the method that applied is mix cement with soil. Sort of variance binder agents such as lime and cement could stabilize the soil depend on soil type. However, combination between lime and coal fly ash for pozzolanic reaction in purpose to stabilize subgrade soil or granular base under pavements of foundations based on current research show interesting result. By cement addition, it is faster to achieve strength characteristic, however when used as a base course, soil which added cement more prone to shrinkage and cracking (Kumar and Gupta, 2016).

The effects of cement and fiber on the mechanical behavior of sandy soils are stated on number of studies. It was reported from one research that soil containing fiber achieving more shear strength and energy absorption by performed static and dynamic triaxial compression and extension tests on cemented sand. By using fiber and cement for sand reinforcement, another study performed consolidated drained triaxial tests which drawn results that fiber additional has increased peak and residual shear strength and decreased residual dilation. From this research are also obtained that ductility of cemented soil developed as fiber content increasing and noted that polyester and glass fibers slightly decrease the peak cohesion intercept and brittleness of the cemented composite.

Another study has conducted to obtain conclusion behavior of clayey soils which

reinforce by cement-fiber. Experimental which using consolidated drained triaxial tests also performed to obtain the effect of fiber reinforcement on the mechanical behavior of sand over a wide cementation range which using samples with a relative density of 70% then proposed empirical equations to decide peak and residual strength based on cement content, fiber content and confining pressure. High pressure isotropic compression tests and hydrostatic compression behavior on cement-fiber reinforced sand are tested in next research. Furthermore, the research on sand which stabilized by fiber cement have been conducted (Hamidi and Hooresfand, 2013).

In purpose to develop engineering properties of soil, integrating reinforcement inclusions within soil is also useful and reliable method. The application of randomly distributed fiber as reinforcement is obtained some benefits comparing with conventional geosynthetics (strips, geotextile, geogrid, etc.). In similar method as cement, lime, or other additives, initially the fibers are simply added and mixed randomly with soil. Then, the fibers which randomly distributed fibers limit potential planes of weakness that can develop parallel to oriented reinforcement. For that reason, it attracts attention in recent years. The addition of fiber-reinforcement has significant development in the strength and decreased the stiffness of the soil based on results of those studies. It needs to point out that fiber reinforced soil shows higher toughness and ductility and lower loss of post-peak strength comparing with soil only. According those results, the discrete fiber could be considered as an excellent geomaterial due to significant effect on modification and improvement in the engineering properties of soil. However, further research compares the influence of fiber addition on the mechanical behavior of cemented and un-cemented soils, specifically the interfacial interactions between fiber surface and reinforced soil matrix (Tang et al. 2007). The geotechnical

properties of cement-fiber reinforced sand using conventional triaxial compression test Hamidi and Hooresfand (2013). Some variables in this research are peak and residual shear strength, volume change, initial stiffness and also the effect of these variables. Form this new type of geomaterial, distinguished attention is given to the effects of relative density, confining pressure and fiber content on deviatoric stress-axial strain and volumetric strain-axial strain behavior and brittleness index and energy absorption potential.

Based on engineering properties and mechanical behavior of cement-fiber reinforced sand, in this research could be drawn the following results:

- The peak and residual shear strength develop, on the other hand the initial stiffness and brittleness index decrease by addition of polypropylene fibers to the cemented soil.
- The increasing of fiber content reduces residual dilation but opposite effect to compressive volumetric strain which developing.
- By addition 1% fiber content, the energy absorption of cemented soil enhances into 20-25%. Large energy is permeated into soil with greater relative density under higher confinement and the slope of the absorbed energy ratio curve versus axial strain flattens as fiber content developing.
- Fiber addition develops both the internal friction angle and cohesion intercept. The peak friction angle for 50% relative density enhances from 39^{o} to 41.5^{o} and for 70% relative density with 1% fiber enhances from 40^{o} to 44^{o}. Therefore, it is shown that higher relative densities of fiber content giving larger effect on shear strength of cemented soil.

The increasing of fiber content develops the principal stress ratio at failure, however the

effect of fiber content reduces as confining pressure enhancement.

Numbers of laboratory experiments was conducted to confirm the response of such materials under static compression loading which have purpose to provide information to help understand the overall behavior of fiber-reinforced cemented and un-cemented soils. By addition of randomly distributed fiber glass, then 12 drained triaxial compression tests were performed on either reinforced or non-reinforced samples.

Fiber reinforcement has effect on behavior of both cemented and un-cemented soils (Kumar and Gupta, 2016). Generally, maximum point and residual compressive strengths are enhanced and stiffness decreases. However, the most important benefit of fiber reinforcement decreases soil brittleness, especially when added to cemented soils. It might be useful for. Fiber reinforcement might be useful for some engineering applications of artificially cemented soils by considering improving characteristics of cemented soil resulting from fiber reinforcement.

Based on engineering properties and behavior of fiber-reinforced/non-reinforced cemented and un-cemented soils, the following investigation and results are drawn:

1. Stiffness and maximum point of strength develops by addition of cement to soil.
2. By increasing fiber reinforcement, both the peak and residual triaxial strengths reduce stiffness and changes brittle behavior of soil which treated by cement to a more ductile one. Un-cemented soil shows more effective result on the increasing triaxial peak strength due to fiber inclusion. For cement treated soil showed inclusion fiber more effective result on residual strength increasing.
3. Due to fiber inclusion, the peak friction angle is enhanced from 35^0 to 46^0 for the un-cemented soil. On the other hand, fiber inclusion showed slightly effect on the peak cohesion intercept as a primary function of cementations. Combination 3% of

fiber and 1% of cement decreased the brittleness index from 2.6 to 0.6 then changed the post-peak behavior into an increasingly ductile one.

Un-cemented soil and cemented soil are strengthened by randomly distributed short PP-fiber (12 mm long) which established strength and mechanical behavior. Soil samples with varying percentages of fiber cement addition were tested by a series of unconfined compression and direct shear tests. The microstructure and the behavior of interfaces between fiber surface and soil were evaluated to achieve a basic knowledge of the mechanism of fiber-reinforced soil by performing scanning electron microscopy tests (SEM). To find out effects of randomly distributed short PP-fiber reinforcement on the strength and mechanical behavior of un-cemented and cemented soil were conducted a series of examines. Several parameters to figure out effects of fiber and cement inclusions were UCS, shear strength parameters, stiffness and ductility of soil specimens. By utilize SEM analysis, the effect of fiber-reinforced un-cemented soil and cemented soil on the fiber surface morphologies and interactions at the interface and mechanical behavior are known (Tang et al. 2007).

Un-cemented and cemented soil which added by fiber reinforcement caused developing in the UCS, shear strength and axial strain at failure. Increasing of peak axial stress, reducing the stiffness and the loss of post-peak strength which decrease the brittle behavior of cemented soil are effect of increasing fiber content. The development strength by combine fiber and cement are more effective than the development by them individually. The further increasing of tension cracks and deformation of soil are accurately prevented by the “bridge” effect of fiber. Dominant mechanisms which control the reinforcement advantage is bond strength and friction at the interface. Interaction which happened at the interface between the fiber surface and the clay grains

has major effect in the mechanical behavior in fiber-reinforced un-cemented soil. On the hand, cemented soil which combined fiber, has affected by the interactions between the fiber surface and the hydrated products which has major contribution to the strength at the interface. The factors such as binding material properties in the soil, normal stress around the fiber body, effective contact area and fiber surface roughness, had affected the micro-mechanical behavior of the fiber/matrix interface. In reinforced soil systems are known that the interface roughness has major role. Even though the optimum degree of the surface damage or plastic deformation due to hard particles as the mixtures are being mixed and compacted is substantial subject, but has not decided yet. Both for progress methods of improving the interfacial strength and for utilization in engineering projects, those results are important. The conclusion of this research show that discrete fiber which combine with cement get the benefits of both fiber-reinforced soil and cement-stabilized soil which could be considered that the addition of fiber-cement to soil is an efficient method for ground improvement (Tang et al. 2007).

3.5.2 Stabilization by cement and geomaterial (waste recycled)

Generally, combination and optimization of properties in individual constituent materials which generate composite material is meaning of soil stabilization. To achieve geotechnical materials improving, well-established techniques of soil stabilization are usually used by added material into soil for example cementing agents as Portland cement, lime, asphalt, etc. Solid industrial by-product which could substitute natural soils, aggregates and cement with solid is highly desirable. However, in some cases, traditional earthen materials are more preferable than a by-product. It would become an interesting alternative if performance is satisfying by consider cost efficiency. On the

other hand, in some cases, a by-product may have better characteristics comparing with those of traditional earthen materials. In order to generate a material with well-controlled and extraordinary attributes usually selected materials are added to industrial by-products (Basha et al. 2005).

In all over the world, power plants which using coal generates waste such as huge quantities of fly ash and bottom ash. Usually the ash is disposed in ash ponds in the vicinity of plants for example in some countries such as India. Poor bearing capacity and very low density is the characterization of pond ash deposits which are considered inappropriate to support any structural load. Furthermore, when the ash ponds capacity is exceeding which resulted abandoning, it will create vast flat barren lands. Recently, in India, nearly 20.000 ha of land are covered up by millions of tons of pond ash which accumulated in such abandoned ash ponds. Moreover, surface water, groundwater bodies, and soils have been contaminated by the carry toxic elements and heavy metals through leachates which obtained from ash ponds. The inclusion of cement and fibers enhances the UCS of a pond ash-rice husk ash-soil mixture. The increasing in UCS due to the combined action of cement and fibers is either more than or nearly equal to the sum of the enhancement caused by them individually, is depended by the mixture type and curing period. By considering un-stabilized/unreinforced soil, strength for such mix is obtained 485% increasing (Kumar and Gupta, 2016).

From the research result of cement-RHA (Rice-Husk-Ash) stabilized soils are drawn conclusions (Basha et al. 2005):

1. The plasticity of residual soil is decreased by addition cement and RHA. In this case, cement-stabilized soils achieved a considerable reducing.
2. The increasing of cement content slightly reduced the MDD of cement-stabilized

residual. The OMC is significantly risen by adding RHA and cement.

3. The inclusion of RHA enhances the unconfined compressive strengths of cement-stabilized soils. As comparison to cement-stabilized soils, the inclusion of RHA obtained similar strength by using lower amount of cement. More excellent development is performed by the resistance to immersion. It has more economic value due to lower cost of RHA than cement which expected cost efficiency on construction.
4. Increasing of cement content has relation with enhancement of CBR value. The CBR values develop several times by added RHA into cement-treated residual soil. Mixture of 4% cement and 5% RHA give 60% CBR value as the highest CBR result.
5. In order to achieve the properties of soils improving, generally 6-8% of cement and 15-20% RHA show the optimum results. An improvement verified by decreasing in PI and development in strength and resistance to immersion.
6. The residual soil which mixed with cement or individually could be potentially stabilized by RHA. Especially in the rural area of developing countries, utilization of RHA as an alternative is expected to decrease construction cost.

Chen and Lin (2009) state that other researches are conducted to investigate utilization of sewage sludge as alternative geomaterial. For example, in Taiwan, big promotion on resource regeneration and utilization of sewage sludge are occurred. However, the investigations of utilization incinerated sewage sludge ash as a soil stabilizer to enhance the strength of soft soil are limited numbers. Furthermore, the study of incinerated sewage sludge ash as an alternative admixture to substitute fly ash as conventional soil stabilizer is performed. This study has purpose to evaluate the influences of incinerated

sewage sludge ash additions on the properties and strength of soft subgrade soil. Then the results which investigate the utilization of ISSA (Incinerated Sewage Sludge Ash)/cement as a soil stabilizer to improve the basic properties of soft subgrade soil can be concluded as follows:

1. The effects of ISSA/cement mixture which added to the soft subgrade soil are PI values reducing and the soil type changes from mid-to-low plastic soil (CL) to CH soil which indicating that the basic properties of soft subgrade soil are effectively improved.
2. Experimental results of swelling potential test show that ISSA/cement admixtures significantly improve soil swelling. Among all, 1/3 of the original swelling of the untreated soil is decreased by the swelling of untreated soft soil with 16% ISSA/cement. For that reason, the ISSA/cement admixture is effective in the volumetric stabilization of soft subgrade soil.
3. Admixture amount enhance the unconfined compressive strength of ISSA/cement soil specimens which indicates the ISSA/cement admixture could be applied as soil conditioner agent.
4. The ISSA/cement admixture could effectively improve the soft subgrade soil from poor to excellent based on the results of CBR test. In some studies, the level of soil improved is superior than excellent subgrade soil which determined by regulation. It could be concluded that the ISSA/cement admixture effectively develop the soil strength of soft subgrade soil.

CHAPTER 4: EXPERIMENTAL STUDY

4.1 MATERIAL

Soil, shell husk and cement are used as the material specimen in this research. The soil sample was taken nearby the Shiratsuka Port on Mie Prefecture Japan. Based on results of laboratory testing which using Unified Classification System, sand was the highest part of this soil with silt and clay as another part.

Table 4.1 Properties of soil and shell husk

Particles	Parameters	Values
Soil particle	Dry density (ρ_d)	1.76g/cm^3
	Optimum Water Content (W_{opt})	13.29%
	Specific gravity (ρ_s)	2.589
	Cohesion (c)	60.95
	Angle of internal friction (φ)	32.75
	Sand > 75 μm	55.56%
	Silt >5-75 μm	24.64%
	Clay <5 μm	19.80%
	Liquid limit	41.00%
	Plastic limit	34.72%
	Plasticity Index	6.28%
Shell Husk	Water absorption	7.28%
	Specific Gravity	1.75
	Unit Weight (g/cm^3)	1.57

Both of the properties of soil and shell husk waste are given in Table 4.1. The Mactridae shell husk waste (Figure 4.1) was collected from the seashore closed to Mie University, Tsu city, Mie Prefecture, Japan. Then the shell husks were graded by performing sieve analysis. The results of

sieving analysis showed that the fineness modulus and the maximum size of the abandon shell husks were 0.84 and 25.4 mm, respectively. The soil and shell size distribution curves are presented in Figure 4.2. The type of cement is Ordinary Portland cement (Type I), which commonly used and easy to find in local markets. The properties of this cement can be found elsewhere (Mouazen et al. 2002).

Figure 4.1 Shell husk waste

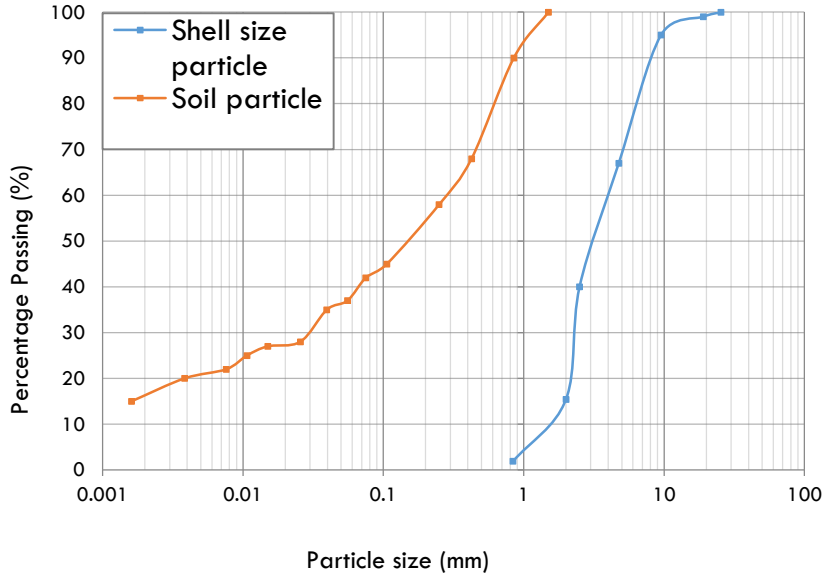

Figure 4.2 Particle size distribution curve

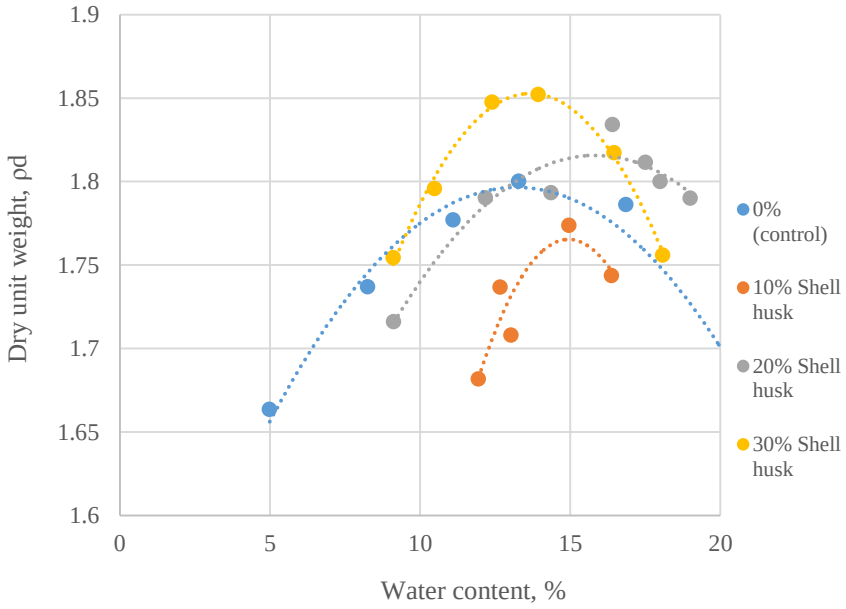

Figure 4.3 Optimum water content

Figure 4.3 show the optimum water content of soil which combined with shell husk percentage (10%, 20%, 30%). This curve also shows the optimum water content of control sample (soil only). The optimum water range of soil which combined with shell husk are 13, 29%-15,50%.

4.2 PREPARATION OF SAMPLES

4.2.1 Direct Shear Test Samples

This study used 0%, 10%, 20%, and 30 % of Mactridae shell husk as coarse aggregate in the ratio mass of clayey soil. Water was added gradually to the mixture shell husk and clayey soil with water content slightly less than that of the optimum water content of soil for every percentage (Figure 4.3). The reason is to maintain the water content in dry-side of optimum and also for field condition with unsaturated soil.

Then other types of sample are combination soil with 10% and 20% shell husk with 2%, 4% and 6% cement respectively. The mixtures of shell, soil and cement were filled in shear box in three layers. Each layer had the same compaction energy hence the density of soil-shell mixture was kept almost constant in every test. After completion of the compaction, the test specimens were cured for one week for strength development.

4.2.2 CBR (California Bearing Ratio) Samples

The CBR test specimens were prepared in steel molds with an internal diameter of 15 cm and height of 17.5 cm. To prepare the layers, soil and water (12% of the soil weight) were mixed homogenously. Then the soil was added into the mold after it had been assembled with the bottom plate, spacer disc, and mold extension. The soil was divided into three layers and then tamped 67 times per layer using an automatic rammer. The automatic rammer had diameter of 5.0 cm, mass of 4.5 kg, and falling height of 45.0 cm. The subgrade layers comprised soil mixed with shell husk 10%, 20%, and 30% additions of shell husks, respectively. The subgrade layer, shown in Figure 3.4, was flattened using a small rammer on the surface of the layer which had been tamped before to achieve a subgrade layer height of 1 cm. The height of the subgrade layers was based on ratio the between field application and laboratory scale, which was 1:5. After the sample had been set up in the mold, it was kept inside a plastic bag to maintain the moisture content for seven days. On the seventh day, the sample was taken out from the plastic and then measured using CBR testing apparatus.

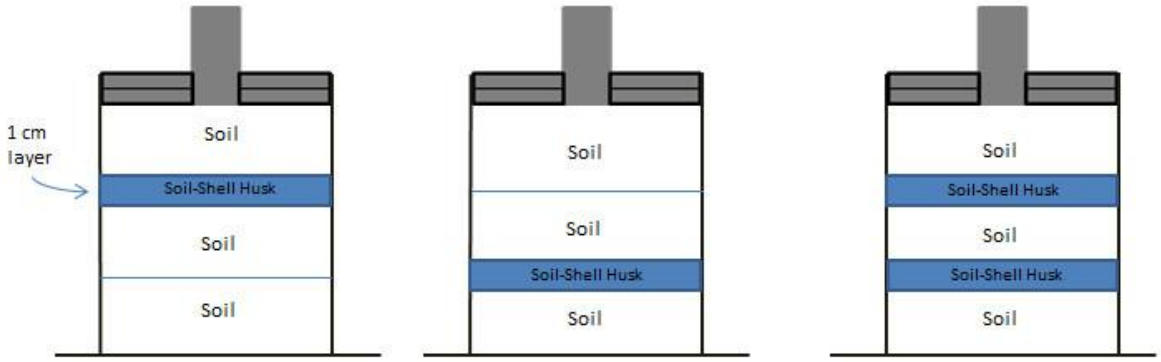

Figure 4.4 Soil-shell husk layer

Then for other type of CBR sample, inside the layers were built subgrade layers which contain of soil, shell husk (10% and 20%) and cement (2%, 4% and 6%). The subgrade layer positions were shown in Figure 4.5 which was flattened using small rammer on the surface of each layer, and the height was 1 cm. The height of subgrades layer is based on ratio between field application and the laboratory scale is 1:5. The mold had been assembled with bottom plate, spacer disc and mold extension, the soil samples were pour into it.

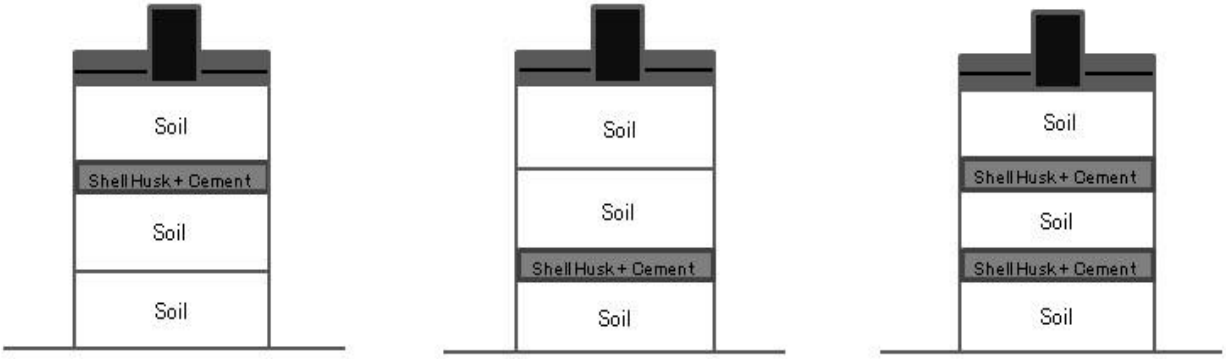

Figure 4.5 Soil-shell husk-cement layer

4.2.3 Unconfined Compressive Strength (UCS) Sample

This research used 0%, 10%, 20%, and 30 % of Mactridae shell husk as coarse aggregate in the ratio mass of clayey soil. Other types of sample were contained of soil, shell husk (10% and 20%) and cement (2%, 4% and 6%). The specimens were manually compacted inside the mold with 12.5 cm in height and 5.0 cm in diameter.

4.2.4 Triaxial Test Samples

All the test specimens were manually compacted inside the mould with 12.5 cm in height and 5.0 cm in diameter. The specimens were control (only soil), soil-cement percentages (2%, 4%, 6%), soil-shell husk percentages (10%, 20%, 30%) and combination soil-cement-shell husk by using both of similar percentages that mentioned before. From total 16 specimens, 12 specimens were contained cement percentage. Then soil was mixed with separately shell husk and cement percentages or both of materials which depend on composition. The specimens were compacted

in three layers using 4.9 cm diameter hand-rammer with rammer mass 1.0 kg and falling height of 30 cm. Each layer was compacted by 20 blows. The average water contents were 9%-12% which observed on the dry side of optimum water content. Specimens which contained cement were cured for seven days at room temperature. Then consolidated-drain (CD) test triaxial compression tests were conducted to evaluate specimens.

4.3 TEST PROCEDURE

4.3.1 Direct Shear Test

The earliest type of the shear testing apparatus is known as the direct shear test which had been found by Coulomb in 1776. An interpretation of the diagram of direct shear arrangement is shown on Figure 4.6. Actually, the shear soil specimen in half along a horizontal failure surface is the direct shear test intention. In Fig 4.6 could be seen that the direct shear has mechanism to implement a vertical load (also known as the normal force or normal stress) to the upper side of soil sample, and there is porous plate on both side top and bottom of soil sample which allow water movement in or out of soil sample. The direct shear box has two parts which balanced and fitted together with alignment pins. The upper part of the direct shear box has capability to be deformed laterally, but the lower part is fitted attached. The soil sample is sheared in half along a horizontal failure plane after implementing a horizontal force to the upper half part of the direct shear box. Both the vertical and horizontal displacement of the soil sample during the shear testing are measured by dial gauges. The apparatus is made of stainless steel, bronze, or aluminum to prevent corrosion process, however dissimilar metals are not allowable because they could lead to galvanic corrosive action (Day, 2001).

The apparatus allows both stress or strain controlled which depend on the apparatus. The shear force is applied in proportional increasing until the sample fails during stress-controlled tests. Along the plane of dividing part of the shear box, the failure is existed. Horizontal dial gauge measures the shear displacement of the half upper part of the box after applying of each increasing load. The examinations of a dial gauge which measure vertical movement of the loading on upper part could clarified the height deformation of the sample (and thus the volume change of the specimen) during the test (Day, 2001).

A stable rate of shear displacement is implemented to one-half of the box by a motor that worked through gears, in strain-controlled tests. Horizontal proving ring or load cell measure the stable rate of shear displacement. In similar mechanism of the stress-controlled tests, the volume change of the sample is also acquired during the test. As illustration for dense sand, the benefit of the strain-controlled test is failure which known as peak shear resistance and ultimate strength could be recognized and plotted. Failure occurs at a stress level at some place between the pre-failure load increasing and the failure load increasing which means the peak shear resistance in stress-controlled tests could be only approximate value. However, stress controlled tests show more real field condition compared with strain-controlled tests.

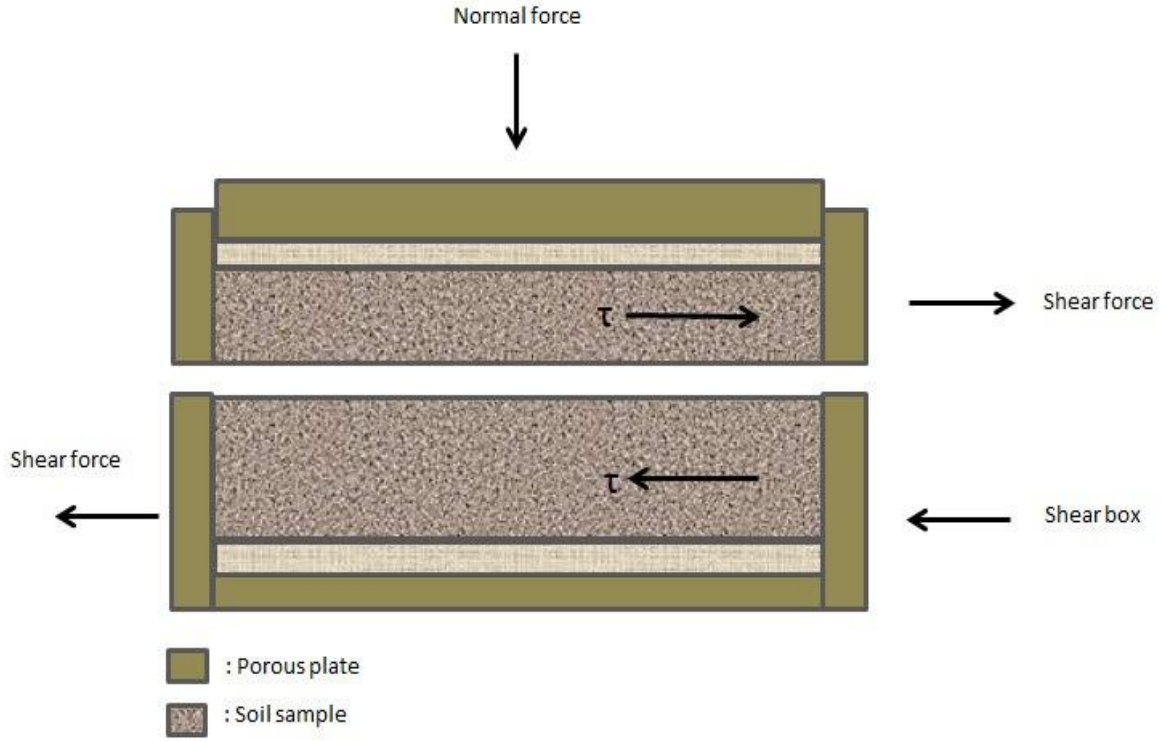

Figure 4.6 Diagram of direct shear test arrangement (Das and Sobhan, 2014)

For a given test, the normal stress could be determined as follow:

$$\sigma = \text{Normal stress} = \frac{\text{Normal force}}{\text{Cross-sectional area of the specimen}} \quad (1)$$

The resisting shear stress for any shear displacement could be determined as follow:

$$\tau = \text{Shear stress} = \frac{\text{Resisting shear force}}{\text{Cross-sectional are of the specimen}} \quad (2)$$

by concerning the variety of resisting shear stress with shear displacement the following generalizations can be developed:

1. Up to failure shear stress of τ_f has been achieved, the resisting shear stress keep increasing in case loose sand. Then for any further increment in the shear displacement the shear resistance is remained almost stable.
2. Shear displacement till achieves a failure stress of τ_f (which known as peak shear strength), the resisting shear stress keep increasing in case dense sand. The resisting shear stress after achieved failure stress gradually decline as shear displacement enhancement up to constant value (called the ultimate shear strength) (Das and Sobhan, 2014).

The apparatus picture of direct shear test is illustrated in Figure 4.7. It consists two box which known as fixed lower box (1) and a moving upper shear box (2), with similar size 50 mm in depth. During test four side walls along with its girder control the mobility of soil parallel to the shear surface which is especially constructed for this apparatus (3). Through a lower jack (4) below lower box, the normal stress is applied which balanced by the opposite stresses of the upper box (5). In the outside, the shear box with clamping system (6) has rectangular shape of size 150 mm in length, 100 mm in width and 100 mm in height. The vertical screws that have been set at both long side of upper box has purpose to remove the friction between upper box and lower box. In this test, computer software DCS-100 which connected with load cell (7), two dial gages (8,9) has been used to record data of shear force, horizontal and vertical displacement through one load cell, and two displacement transducers (one for shear displacement and other for vertical displacement measurement).

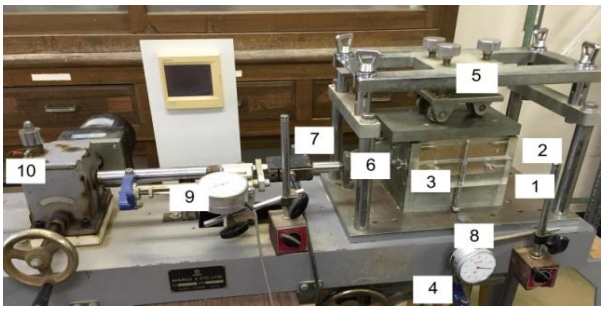

Figure 4.7 Direct shear testing apparatus

Inside shear box, the samples were filled on three layers. On each test, to keep density of soil-shell mixture consistent, every layer should get the same compaction energy. After the

consolidation by normal stresses has achieved then the shear load applied through screw jack (10). Screw jack has been worked using electrically operated constant pressure with a constant speed of 1.0 mm/min. During the test, normal stresses (vertical load) are kept stable. The specification of this apparatus is according on JIS and the Japanese Geotechnical Society (JGS: T941-199X) (Hossain, 2011).

4.3.2 California bearing ratio test

The Californian Division of Highways, considering the design of flexible pavements developed the California Bearing Ratio (CBR) test. The Corps of Engineers of the US Army adopted the basic procedure of this test which made certain modifications in the test procedure considered as the standard method of determining the CBR. This method was also adopted by the Bureau of Indian Standards (IS:2720-part 16, 1987). Design curves using CBR values to determine the required thickness of flexible pavements for airport runways and taxiways has developed by the Corps of Engineers (Raj, 2008).

In all over the world, this method has been widely used to evaluate the bearing capacity of soils and subgrades since its invention in 1930 by the California Division of highways, USA (Hossain and Sakai, 2008). The CBR is defined as the ratio of the resistance to sinking of a penetration piston having a velocity of 1.27 mm/min (0.05 in/min) into the soil to the resistance which determined as the CBR as shown by a standard crushed rock sample for the same depth of penetration (Yildrim and Gunaydin, 2011). The CBR value was calculated according to the following equation:

$$\text{CBR}\,\% = \frac{\text{Load Strength}}{\text{Standard Load strength}} \times 100 \qquad (3)$$

The basis of pavement design is formed by the CBR test which widely used in the past. Due to this reason, it is preferable with some as design and control method. In the laboratory where the CBR can be evaluated by variety of dry densities and also soaked or un-soaked, design testing is usually held according to the ground water or drainage conditions foreseen. To encourage the effect of construction thicknesses, surcharges could be included. After a design CBR has been developed, it can be controlled in the field either by sampling the placed fill and evaluating at the site laboratory using CBR apparatus, or by adopting in situ CBR procedures. In the last case, the outcome of test on granular soils could be affected by the absence of containment during

evaluating. In The Highways Agency Advisory Note HA 44/91, there is a useful discussion on the CBR test which related to pavement design. Similar with the MCV, the CBR test is empirical which directly related to undrained shear strength using equation as follow (Trenter, 2001):

$$C_u = 23 \text{ x CBR (kN/m}^2\text{)} \quad (4)$$

Where, notation C_u is the undrained shear strength. Through relationship evaluation, direct relationships between shear strength and dry density might be developed. Both of places, laboratory or the field, the California bearing ratio test could be performed. In ASTM D 1883-99 (2000), the laboratory CBR test is represented as "Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils". On the other hand, in ASTM 4429-93 (2000), the field CBR test is described as "Standard Test Method for CBR (California Bearing Ratio) of Soils in Place".

In the field, the CBR consists of a 2.0-in (5.08-cm) diameter piston which pushed into the soil however, in the laboratory, the CBR test is usually conducted on compacted soil samples. The CBR is evaluated as follows:

1. When the piston is being pushed into the soil, then the load (pounds) versus depth of penetration (inches) recorded.
2. Dividing the load by the area of the piston is calculated as the stress (psi) on the piston.
3. The stress (psi) versus the depth of penetration (inches) are plotted.
4. Usually the stress (psi) corresponding to a depth of penetration of the piston of 0.20 in. (2.54 mm) are known as the bearing value. A correction is applied, if the curve shows surface irregularities or if the curve is at beginning concave upward, in order to achieve the corrected stress for 0.10 in (2.54 –mm) penetration of the piston.
5. The penetration resistance of compacted crushed rock is represented by bearing value which converted to a ratio by dividing it by 1000 psi.
6. California Bearing Ratio is the bearing ratio, which is fraction then multiplied by 100. For dense sandy gravel, usually the CBR range valued from less than 5 up to 80.

The stress corresponding to 0.20 in. (5.08) of penetration should also be decided, and then the bearing value is changed to a ratio dividing by 1500 psi by considering above analysis. However, the test must be repeated, if bearing ratio of 0.20 in. (5.08) is bigger than the bearing ratio calculated for 0.10-in (2.54 mm) penetration. If the result is similar after test, then the ratio at

0.20 in. (5.08 mm) is used as the California Bearing Ratio. Lower result of CBR than originally measured probably due to interpreting the CBR data which might be affected by soil disturbance and loosening during the construction operations. It is also due to increasing of field moisture content, which could soften fine-grained soils and also could lead to a lower CBR than originally measured.

In this research, the soil mixtures were divided into three layers then tamped 67 times per layer with the 4.5 kg rammer. After the sample had set up into the mold, it was kept inside the plastic bag to maintain the moisture content for 7 days. On the seventh day, the sample was taken out from the plastic then measured by using CBR testing machine. The loads were recorded up to a penetration depth of 12.5 mm by using the apparatus (JIS-A-1211).

4.3.3 Unconfined Compression Test

Very simple type of test that consists of applying a vertical compressive pressure to a cylinder of laterally unconfined cohesive soil is recognized as the unconfined compression test which also known as simple compression test. Mostly, the unconfined compression test is conducted on cohesive soils in a saturated condition, for example soil collected from below the groundwater table. The soil specimen should be capable to hold its plasticity during the vertical pressure appliance because the soil specimen is laterally unconfined during testing (no lateral confining pressure). During the compression test, it should be noticed that the soil must not release water (known as bleed water). Due to that reasons, the unconfined compression test is usually conducted on saturated clays. Soil which has characterization such as crumble, fall apart, or bleed water during the application of the vertical pressure could not be evaluated.

The shear test data has been split into two different parts, as follows:

1. Axial deformation data. The axial deformation of the soil specimen which monitored by dial readings obtained during the shearing process. The differentiation between the recorded dial reading and the initial dial reading is equal with the change in height of the soil specimen which defined as ΔH during the shearing process. The change in dial reading and the initial height of the soil specimen is defined as the axial strain ε, or $\varepsilon = \Delta H/H_0$. The height of the soil specimen reduces as the result of soil specimen deformation during shearing test. However, the volume of the soil specimen does not change because it is an undrained triaxial

test. It is usually assumed that the soil specimen retains its cylindrical shape during shearing test.

2. Loading data. Either a load cell or a proving ring quantifies the axial load during shearing. The vertical total stress σ_v is equivalent with the major principal total stress σ_1. The vertical total stress σ_v is equal with the axial load P which divided by the corrected area of the specimen A_c, it is explained on equation as follow:

$$\sigma_1 = \sigma_v = \frac{P}{Ac} \quad (5)$$

Either the highest value or stress maximum value of the vertical total (σ_v) is known as analysis failure for a total stress. Unconfined compressive strength is the highest value of σ_v which usually designated as q_u.

Unconfined compression test is illustrated on a Mohr circle in terms of total stresses. The minor principal total stress σ_3 is equivalent with zero, since the lateral pressure is zero or no confining pressure detected. Maximum value of the vertical stress σ_v is the major principal total stress σ_1 at failure which designated as q_u. According to that explanation, the Mohr circle in terms of total stresses would have $\sigma_3 = 0$ and $\sigma_1 = q_u$. For the unconfined compression test, the maximum shear stress τ_{max} is equivalent with the shear strength which defined as the peak point of the Mohr circle. The maximum shear stress is equivalent with the radius of the Mohr circle, and thusly the undrained shear strength s_u is :

$$s_u = \tau_{max} = q_{u./2} \quad (6)$$

Where s_u = undrained shear strength of saturated cohesive soil, kPa or psi

τ_{max}= maximum shear stress, kPa or psi, that soil can withstand, which for this total stress analysis is assumed to be equivalent with undrained shear strength

q_u = unconfined compressive strength of soil, kPa or psi. the unconfined compressive strength of soil is equivalent with highest value of σ_v (Day, 2001).

In this research, the mixtures of soil-shell husk-cement were compacted in three layers using 4.9 cm diameter hand-rammer with rammer mass 1.0 kg and falling height of 30 cm. Each layer was compacted by 20 blows. UCS at a loading rate of 0.1 mm/min and recorded every 0.5 mm displacement which has been applied in this research. These tests were performed according to Japanese Industrial Standards (JIS-A-1211, 1980).

4.3.4 Triaxial Test

In the laboratory testing of cohesive soil, the triaxial test apparatus is widely used. In the triaxial test procedure, cylindrical specimen of cohesive soil is placed in the center of the triaxial apparatus, sealing the specimen with a rubber membrane, then soil applied to a confining fluid pressure, and at the last enhance the vertical pressure to shear the soil specimen. According to the soil specimen drainage conditions, the types of laboratory tests that demand the triaxial apparatus are categorized as follows:

1. Unconsolidated undrained triaxial compression test
2. Consolidated drained triaxial compression test
3. Consolidated undrained triaxial compression test
4. Consolidated undrained triaxial compression test with pore water pressure measurements (Das and Sobhan, 2014).

Figure 4.8 shows a picture of the triaxial equipment and the loading apparatus. From the picture could be seen that the soil specimen has been located in the center of the triaxial apparatus and sealing inside a rubber membrane. The essential parts of the triaxial apparatus are as follows:

1. Triaxial chamber. Top plate, baseplate, and chamber cylinder are parts of the triaxial chamber which showed in Figure 4.8. Both the top plate and baseplate have a round groove which made of metal material. Into each groove, a compressible O-ring and the chamber cylinder should be fitted snugly. The top plate, baseplate and chamber cylinder are clamped together using tie bars. The triaxial chamber is typically fitted with at least three tie bars. An axial load piston is used to apply a vertical load to the top of the soil specimen which equipped on the top plate. The top plate is assembled with ball bushings having pressure seals which purposed to decrease piston friction and the escape of camber fluid along the sides of piston. Usually, two ball bushings are applied to guide the piston, decreasing friction, and controlling vertical alignment. When the chamber fluid is filled with usually water, the top plate is also equipped with a vent valve which released out. One edge of a tube is linked to this vent valve with the other edge which linked to a chamber pressure control system. The soil sample is located on the baseplate which equipped with a central pedestal. The fluid is provided to the chamber through the inlet which belonging to the baseplate. When it is necessary, the baseplate has inlets leading to the soil specimen base and cap which allow saturation and drainage of the soil specimen.

2. Rubber membrane. Rubber membrane is used to encase the soil specimen which needed carefully inspection prior to use, and if any flaws or pinholes are found, then it should be rejected.
3. Chamber pressure control system, the cell pressure is also recognized as the chamber fluid pressure. The chamber pressure control system links to the triaxial chamber through a tube. The chamber pressure control system has purpose to apply and maintain a pressure to the fluid included within the triaxial chamber. Confining stress σ_c is applied to the soil specimen by using chamber fluid pressure. Self-compensating mercury pots, pneumatic pressure regulators, and combination pneumatic pressure and vacuum regulators are illustration of several varying types of chamber pressure control systems. Chamber pressure systems which have an air-water interface are not recommended by ASTM due to needed several days to complete a triaxial test. Electronic pressure transducer measure the chamber should be calibrated, recording the chamber pressure which exerted at the mid-height of the soil specimen.
4. Pore water pressure and drainage measurement system. The pore water pressure and drainage measurement system have three parts:
    a. Pore water pressure. During testing, the water pressure system has purpose to measure the pore water pressure within the soil sample which in consist of nonflexible components. Therefore, between the tubes on specimen should have small diameter. It is also occurred for the pore water pressure measuring device which required very stiff, and this is usually achieved by using a very stiff electronic pressure transducer. It should be noted, the pore water pressure transducer need to calibrate which records the pore water pressure at the mid-height of the soil specimen. Significant inaccuracies in the pore water pressure readings could be happened due to any air bubles that become trapped within the pore water pressure measuring system.
    b. The pore water pressure measurement system usually has capability to be detached from the drainage measurement system. This allows the drainage measurement system to be shut off, yet the pore water pressure within the soil specimen could still be controlled.
    c. Drainage measurement system. Either recording the volume of water that enters or exceeds the soil specimen is the purpose of the drainage measurement system. This

system is required for those soil specimens which consolidated or sheared in a drained state. To measure the volume of water that enters or exceeds the soil specimen is usually used a burette.

d. Backpressure system. The applying ability pores water pressure to the soils specimen is included on the triaxial equipment. The backpressure is pore water pressure which applied to the soil specimen. The chamber pressure control system is separately from the system which used to apply the backpressure. However, it is possible for the device which utilized to apply the backpressure has similar type of applying chamber pressure device.

   Typically, the backpressure system and the drainage measurement system are directly linked which allow exceed water from the soil specimen would flow through the drainage measurement system and into the backpressure system. For any change of water into or out of the soil specimen is able to be measured.

   Most of the system which included in the triaxial apparatus such as the pore water pressure measurement system, drainage measurement system, and backpressure system, usually have valves with function either open or close each individual system. When opened and closed, these valves must be of a type that produces minimum volume change. Moreover, the valves should have capability to resist the pore water pressure or backpressure without leaking.

5. Loading apparatus

   A screw jack driven by an electric motor acting through a geared transmission is illustration of the truly existing loading device could be consisted of weights applied to a hanger (controlled stress test) or a device used to control the displacement rate of the loading piston (controlled strain test). A load indicator device, for instance a load cell or proving ring determined load on the controlled strain test. The axial force up to within one percent of the axial force at failure is accurately measured by the load cell or proving ring.

During the triaxial compression test, the pressures which are applied to the soil specimens as follows:

1. Vertical total stress. The vertical total stress σ_v is equivalent with the major principal total stress $\sigma_{1.}$ The vertical total stress $\sigma_{v,}$ is equivalent with the chamber fluid pressure plus the load induced by the piston divided by the area of the specimen, or

$$\sigma_1 = \sigma_v = \sigma_c + \frac{P}{A} \quad (7)$$

where σ_1 = major principal total stress acting on soil specimen, kPa or psi

σ_v = vertical total stress acting on top of soil specimen, kPa or psi. It is equivalent with the sum of cell pressure σ_c and pressure exerted by loading piston

σ_c = chamber fluid pressure, also kown as cell pressure, kPa or psi

P = load applied by loading piston, kN or lb

A = area of soil specimen, m2 or in2. Because area of soil specimen changes as P is applied, an area correction is required.

2. Horizontal total stress. The minor principal total stress σ_3 is equivalent with the horizontal total stress σ_h. The horizontal total stress is induced on the soil specimen by the chamber fluid pressure σ_c

$$\sigma_3 = \sigma_h = \sigma_c \quad (8)$$

where σ_3 = minor principal total stress acting on soil specimen, kPa or psi

σ_h = horizontal total stress, kPa or psi, which is equal to cell pressure σ_c

σ_c = chamber fluid pressure, also known as cell pressure, kPa or psi

It is noted that for the triaxial compression test, the intermediate principal total stress σ_a is equal to the minor principal total stress (that is, $\sigma_2 = \sigma_3$).

3. Pore water pressure. Either it could be applied to the soil specimen or measured during testing, for the pore water pressure that exists within the soil specimen. As previously mentioned, the backpressure is known as an applied pore water pressure.

4. Deviatoric pressure. Deviatoric stress or stress difference is also recognized as the deviatoric pressure. When the axial load is given to the top of the soil specimen, the deviatoric pressure is increased during the shearing of the soil specimen. The deviatoric pressure is determined as the major principal total stress minus the minor principal total stress, or deviatoric pressure = $\sigma_1 - \sigma_3 = \sigma_v - \sigma_h$.

   By knowing the principal total stresses and the pore water pressure within the soil specimen, the principal effective stresses could be determined as follows:

$$\sigma'_1 = \sigma_1 - u \quad (9)$$

$$\sigma'_3 = \sigma_3 - u \quad (10)$$

where σ'_1 and σ'_3 are the major and minor principal effective stresses, respectively (Day, 2001).

In this research, the triaxial chamber (Figure 4.8) was consisted of soil specimen that encased using rubber membrane (1). In the baseplate groove (4), the chamber cylinder was installed (2), then the top plate was located on it (3). The three parts of the triaxial chamber (i.e., baseplate, chamber cylinder and top plate) were joined together using the tie ring (5). The soil specimen cap has a circular indention at the center, and the position of loading piston (6) should be alignment. By pushing down the loading piston and established precisely into the center of the specimen cap, the alignment could be achieved. Then from water channel, the chamber was filled with the fluid (7), while air release valve on top cap keep opened (8). Through the loading piston an axial stress ($\Delta\sigma$) was applied, and the dial gauge (11) which linked with piston and specimen recording vertical deformation during the process. The hydrostatic chamber pressure was achieved using the air channel (9) on the top cap while the air release valve on top cap closed. The confining pressures in this research were 50 kPa, 100 kPa, 150 kPa, 200 kPa. Water valve (10) was keep opened during the test, due to drained condition demand in CD test. When the maximum value of the principal stress difference (σ_a-σ_r) was obtained, then the test will be over. However, if a maximum value was not obtained then the peak value of the principal stress difference (σ_a-σ_r) determined at 15% axial strain (JGS 0524, 2001).

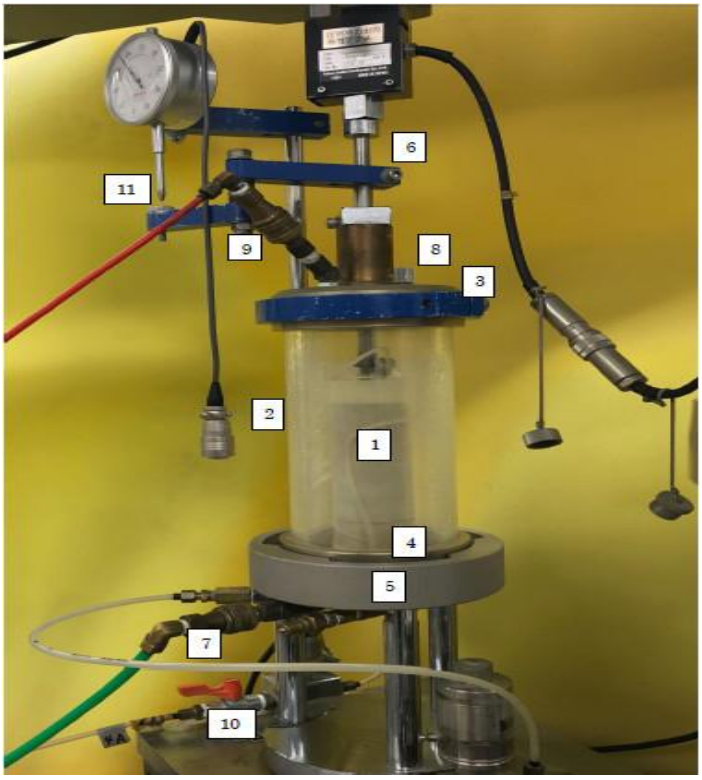

Figure 4.8 Triaxial apparatus

CHAPTER 5: RESULT AND DISCUSSION

5.1 Direct Shear Test

In the present study, abandon shell husk is used (Mactridae) in direct shear test under four normal stresses of 40, 60, 80 and 100 kPa. This test was performed to evaluate shear stress-displacement, shear strength, dilatancy behavior of clayey soil of Mie Prefecture, Japan which was added by three percentages (10%, 20%, 30%) abandon shell husk and also control. Pertinent discussion on the improvement of ground using different percentage of shell husk are made based on dilatancy behavior, shear stress displacement and shear strength. Then other types of sample are combination soil with 10% and 20% shell husk with 2%, 4% and 6% cement respectively. The normal stresses (40 kPa, 60 kPa and 80 kPa) respectively were applied to every specimen which using cement percentage.

5.1.1 Direct Shear Test (Soil and Shell Husk)

1. Soil with 0% shell husk

For 0% (control specimen) the shear stresses-displacement relationship is illustrated in Figure 5.1 and vertical-horizontal displacement in Figure 5.2. The trends of shear stress between normal stresses 40 and 60 kPa are almost similar, on 0-3 mm of displacement gradually increase than constant up to the end of test. However, shear stress trends between normal stresses 80 and 100 kPa gradually increase during the test. Similar trends between (normal stresses 40 and 60 kPa) and (normal stresses 80 and 100 kPa) are also shown in vertical-horizontal displacement. The vertical displacement of 40 and 60 kPa are slightly different on 0-4.7 mm of horizontal displacement then the range is start increasing until the end of test. On the other hand, the range of vertical

displacement between 80 and 100 kPa is slightly different after 4-6.7 mm. or normal stresses 40, 60, 80 and 100 kPa. For normal stresses of 40, 60, 80 and 100 kPa, ultimate shear stresses are 18.64, 28.33, 35.89, 45.41 kPa and the maximum vertical displacements are calculated as 6.25, 7.32, 10.07, 11.06 mm respectively.

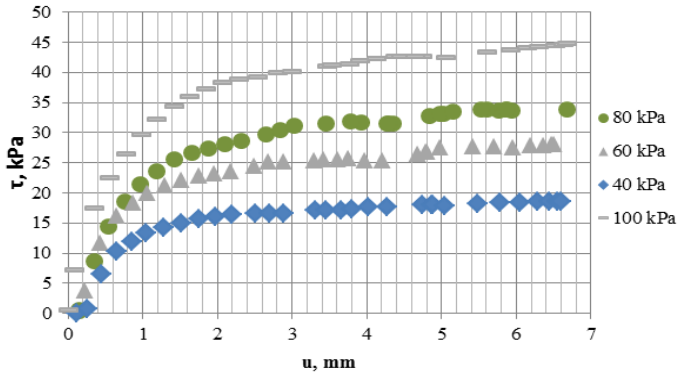

Figure 5.1 Shear stress-displacement of soil with 0% shell husk

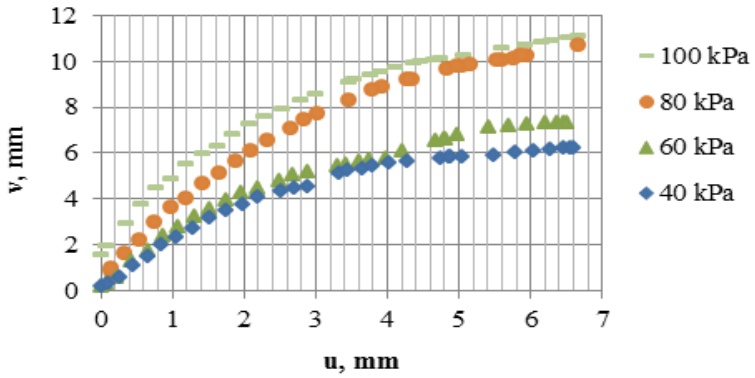

Figure 5.2 Dilatancy behavior of soil with 0 % shell husk

2. Soil with 10% shell husk

Figure 5.3 illustrates the shear stress-displacement of soil with 10% of shell husk and Figure 5.4 describes the relationship between horizontal-vertical displacement of that sample. For normal stress 40 kPa during 0-0.4 mm of displacement, the shear stress remains stable then gradually increase. In contrast for shear stresses on other normal

stresses which are rapidly increase on 0-0.6 mm of displacement. For normal stresses 60 and 80 kPa, on displacement 0-0.0.6mm are almost similar. This similar trend is also occurred on 0-2 mm horizontal-vertical displacement for normal stresses 60 and 80 kPa. The vertical displacement of 60 and 80 kPa for 0-2 mm horizontal displacement are almost similar and after that the vertical displacement of 80 kPa gradually increases higher than 60 kPa. On the contrary with vertical displacement of 40 and 60 kPa which has slightly difference on 5-6.7 mm horizontal displacement and not so much different in the end of the test. Vertical displacements of 80 and 100 kPa are gradually increase and have similar trends. For normal stresses 40, 60, 80 and 100 kPa maximum shear stress are 17.81, 28.69, 35.99, 44.05 kPa and the highest number of vertical displacements are noted as 6.57, 6.88, 8.01, 8.87 respectively.

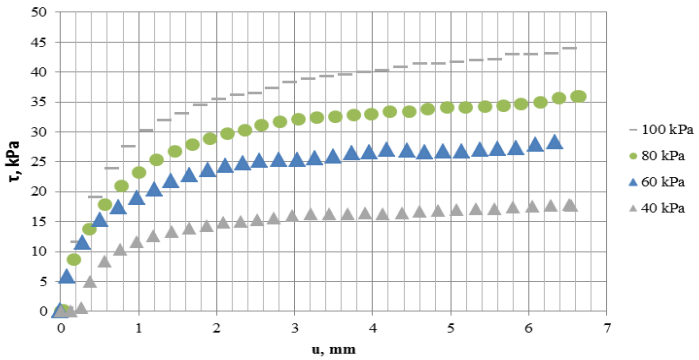

Figure 5.3 Shear stress-displacement of soil with 10% shell husk

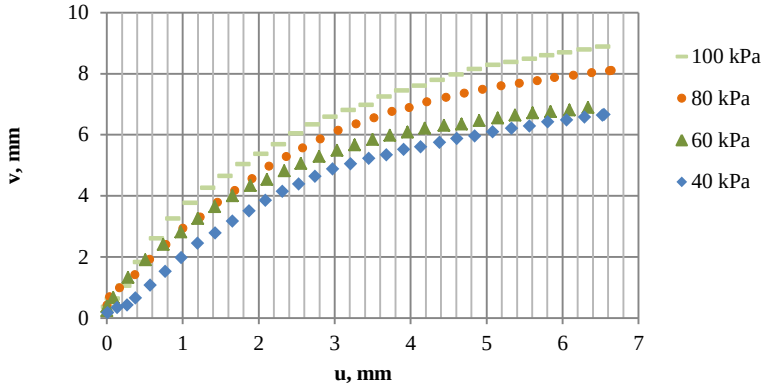

Figure 5.4 Dilatancy behavior of soil with 10% shell husk

3. Soil with 20% shell husk

The shear stresses of soil with 20% shell husk are described in Figure 5.5 and Figure 5.6, illustrates the vertical displacement versus horizontal (shear) displacement. In beginning of the test (0-0.4 mm), shear stress-displacement relationships for all normal stresses are similar. Then for normal stresses 80 and 100 kPa are keep similar until 0.6 mm. The shear stress trend for 40 kPa on 4-5 mm slightly decrease then increase moderately up to the end of the test. The shear stresses for other normal stresses are gradually increasing during the test. The dilatancy of 60, 80 and 100 kPa are slightly different on 0-1.5 mm shear (horizontal) displacement then it spreads gradually. As given above, similar trends are also happened in beginning of shear stress-displacement test. For 40 kPa, the vertical displacement gradually increased having moderately wide gap with other normal stresses. The trends between 80 and 100 kPa have similar patterns but quite different on numbers. The similar trends also happen between 40 and 60 kPa, however, the range is higher. In this case, there is not negative value of vertical displacement. For normal stresses 40, 60, 80 and 100 kPa, ultimate shear stresses are 19.83, 27.87, 36.53, 45.96 kPa and the peaks of vertical displacements are recorded 5.57, 7.67, 9.11, 10.86 mm respectively.

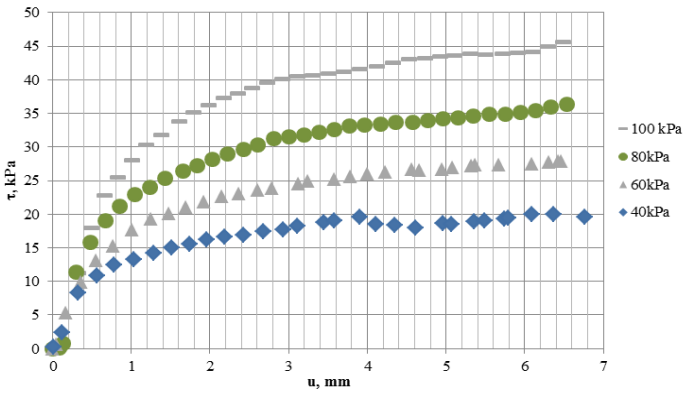

Figure 5.5 Shear stress-displacement of soil with 20% shell husk

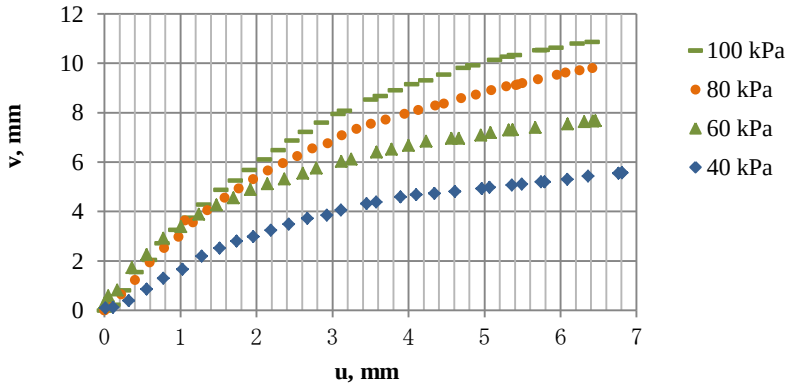

Fig 5.6 Dilatancy behavior of soil with 20% shell husk

4. Soil with 30% shell husk

The shear stress-displacement relationship of soil with 30% shell husk, are shown in Fig. 5.7 and Fig. 5.8 which also illustrates relationship between vertical displacement and horizontal displacement. Shear stresses for normal stresses 40 and 60 kPa are slightly different and have similar trends. On displacement 4-4.4mm, shear stress of normal stress 80 kPa slightly decrease then gradually increase and at this point also the horizontal-vertical displacement gradually increasing closed to horizontal-vertical displacement for 100 kPa. For normal stress of 100 kPa, shear stress extremely increased on 0-1 mm shear displacement. After that it increases moderately during the test. The vertical displacement curves of 60 and 80 kPa almost similar on 0-2 mm horizontal displacement. It is observed that the vertical displacement of 40 kPa and other normal stresses has large range. Initially, the dilatancy of 40 kPa has negative value and almost zero on 0-2 mm of shear displacement. The similar trends of vertical displacements for 60 and 80 kPa are occurred on 0-2 mm of shear displacement. By the end of test, the vertical displacements of 80 and 100 kPa have not much difference. For normal stresses 40, 60, 80 and 100 kPa, the maximum shear stresses are obtained as

21.33, 27.17, 38.42, 47.99 kPa and the maximum vertical displacement are recorded as 3.54, 8.51, 9.64, 10.10 mm, respectively.

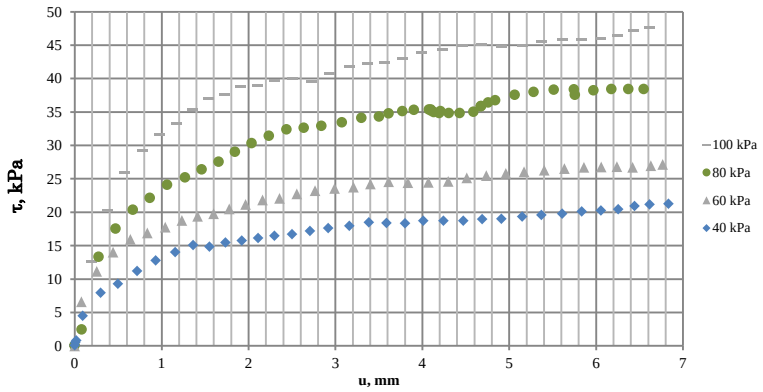

Figure 5.7 Shear stress-displacement of soil with 30% shell husk

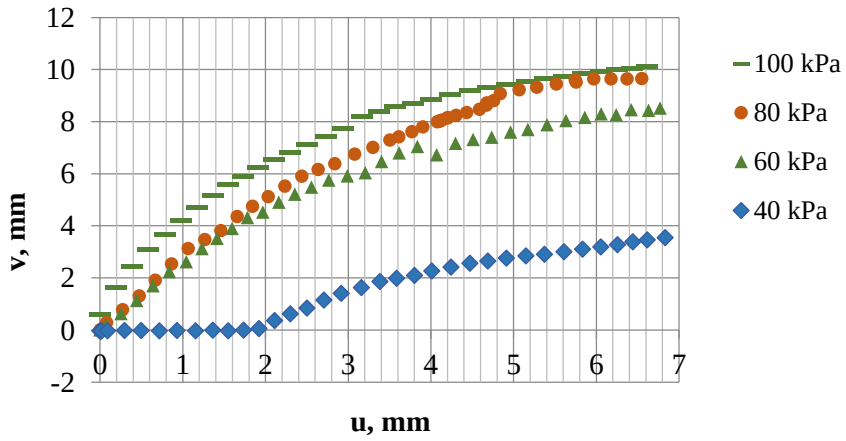

Figure 5.8 Dilatancy behavior of soil with 30% shell husk

A knowledge of shear strength is required in the solution of problems concerning the stability of soil masses. If at a point on any plane within a soil mass the shear stress becomes equal to the shear strength of the soil, failure will occur at that point. The shear strength (τ_f) of a soil at a point on a particular plane was originally expressed by Coulomb as a linear function of the normal stress (σ_f) on the plane at the same point :

$$\tau_f = c + \sigma_f \tan \varphi \qquad (1)$$

where, c and φ are the shear strength parameters, now described as the cohesion

intercept (or the apparent cohesion) and the angle of shearing resistance, respectively (Craig, 1974).

Figure 5.9 illustrates the cohesion of the shell-soil mixture with varying percentages of shell content. The cohesion for soil with 0% shell husk and soil with 10% shell husk is slightly different. Cohesion of soil with 20% shell husk is the highest than soil with other percentages of shell husk. Soil with 30% shell husk has lower cohesion than 20% shell husk but higher than soil with 0% shell husk and soil with 10% shell husk. The angle of internal friction (Figure 5.10) showed an increase in the internal friction of mixture soil with the increase in percentage of shell husk. Soil with 30% shell husk has the highest internal friction. Therefore, the cohesion of soil-shell husk are obtained as 1.52, 1.53, 2.08, 1.8kN/m2 and angle of friction are calculated as 23.23^{o}, 23.27^{o}, 23.52^{o} and 24.52^{o} for 0%, 10%, 20% and 30% soil-shell husk respectively. The cohesion values are assumed as results of over consolidation stress. On Table 5.1 cohesion values has converted according to normal consolidation stress.

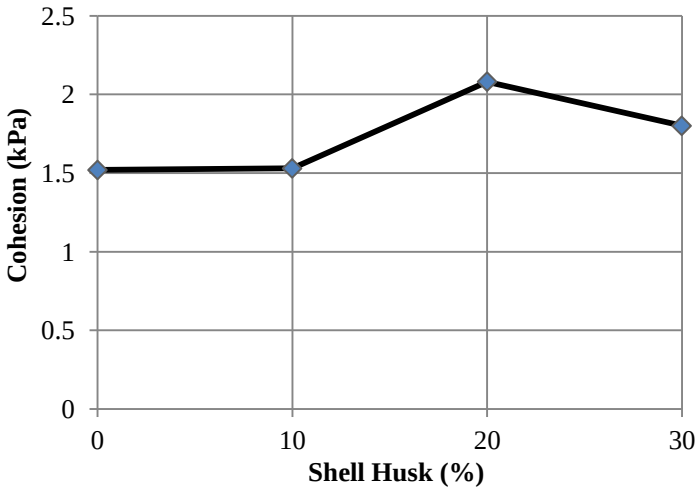

Figure 5.9 Cohesion of soil with shell husk percentage

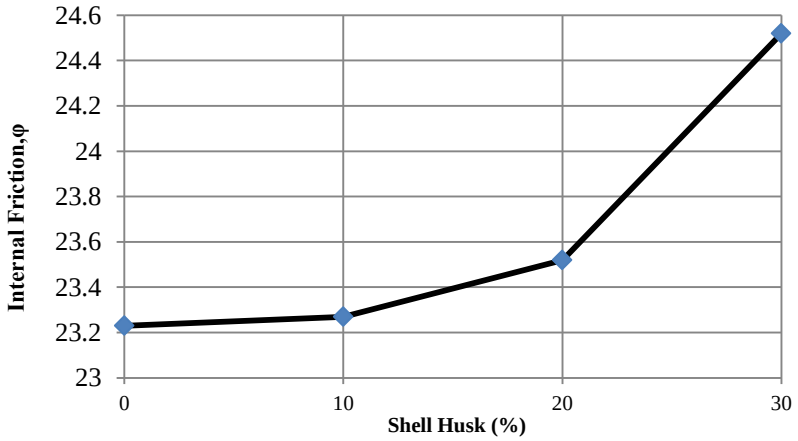

Figure 5.10 Internal Friction of soil with shell husk

Table 5.1 Conversion (normal consolidation) of cohesion soil with shell husk

Shell husk (%)	Cohesion (kPa)
Shell husk 0%	61.2
Shell husk 10%	63.2
Shell husk 20%	83.2
Shell husk 30%	72

As given above, the relationships shown in Figure 5.1 and 5.3 for soil with 0% shell husk and soil with 10% shell husk are slightly different. For normal stresses of 40 kPa and 100 kPa, soil with 0% shell husk has higher ultimate shear stress than soil with 10% shell husk. On the other hand, shear strength of soil with 10% shell husk is higher than soil with 0% shell husk for normal stresses of 60 kPa and 80 kPa. For normal stresses of 40 kPa, 80 kPa and 100 kPa, the ultimate shear strength of soil with 20% shell husk is higher than the soil with 10% shell husk. On contrary, the ultimate shear strength of soil with 20% shell husk (Figure 5.5) for normal stress 60 kPa is higher than soil with 30%

shell husk and vice versa for other normal stresses (Figure 5.7). Overall, for normal stresses of 40 kPa, 80 kPa and 100 kPa soil with 30% shell husk has the highest ultimate shear strength-displacement and soil with 10% shell husk has the highest ultimate shear strength for normal stress 60 kPa.

For all the tests were performed in this study, it was observed that the vertical displacements for all the cases are increasing along with the increase in horizontal displacement. These phenomena are related to volume change by means direct shear test for every type of percentage of shell (Arora, 1978). Over all data showed that the percentage of shell husk as ground improvement material has significant effect to decrease the vertical displacement than soil without shell. It was observed that the mixing of waste shell husk in soil resulted more stress-transfer ability of the ground thereby increased the resisting forces. These phenomena can be observed in others (Hossain, 2013). There are several negative value in the beginning of 30% of shell husk under 40 kPa normal stress which indicated the increase in the volume. It is shown that there is a decrease in the density of clayey soil with the increase of percentage of waste shell husk. This type of observation was also noted by Malkawi (1999).

The cohesion and angle of internal friction of soil-shell husk show different trends. Soil with 20% shell husk has the highest cohesion. This might be due to the water content of the soil-shell mixture. It is shown in Figure 4.3 that the optimum water content of soil with 20% shell husk has the highest number than others. Optimum water content affected the cohesion of clay soil and showed decreasing trend due to clay aggregate phenomenon that give rise to a granular feature to the soil mass at the 30% level. This type of trend was also observed in the past (Cokca et al., 2004). On the other hand, the angle of internal friction increase with the increasing in percentage shell husk. The

result is expected because of the increase in the granular percentage of mixture.

All the results of shear stress-displacement of soil-shell husk mixture including control (0% shell husk) show similar trends. The increasing of normal stress causing an increase in the shear stress-displacement of soil with shell husk even though the increasing rate varies depending on the shell husk percentage. The results showed that the soil with 0% shell husk and 10% shell husk are slightly different and this difference are getting more significant with the 20% shell husk and 30% shell husk addition.

For all the tests, generally the dilatancy (vertical displacement) were increased with the shear (horizontal) displacement. Soil without reinforcement (zero percent of shell husk) was reached the highest value of vertical displacement. The lowest vertical displacement was obtained by soil with 10% shell husk. Soil with 20% shell husk got the highest value of cohesion. Then soil with 30% shell husk has the highest ultimate shear stress displacement and angle of friction. It increases 5.6% compare to control. Over all the results compared to control show that shell husk could be considered as the good resource for ground improvement. Further research to increase the mechanical properties of shell husk as ground improvement could be performed such as combination between shell husk and other material construction.

5.1.2 Direct Shear Test (Soil, Shell Husk and Cement)

The relationship between maximum shear stress (τ) and normal stress (σ) of the soil-shell husk-cement are presented in Figure 5.11 and 5.12. It can be observed from this figure that soils with 20% shell husk and 6% cement have the highest maximum shear stress in comparison to other scenario. It is then followed by soils with 20% shell husk and 4% cement. Soil with other percentages of shell husk and cement additions are

not considered as a useful alternative.

The shear strength of soils is the most important factor to investigate due to its main contribution to the stability of soil under the load (Hossain et. al, 2006). Soil cohesion and internal friction are two factors that explained the soil shear strength as expressed in the Mohr-Coulomb failure criterion:

$$\tau_f = c + \sigma_n \tan\phi \quad (2)$$

Where, τ_f is the soil shear stress at failure, c is the cohesion, σ_n is the normal stress to the failure plane and ϕ is the internal friction angle (Mouazen et.al, 2002). Figure 5.13 illustrates the mechanism on a direct shear test where shell husk particle and cement particle resist shear force throughout horizontal shear plane.

Both cohesion (c) and internal friction (ϕ) of soil-shell husk with cement addition are presented in Table 5.1. Cohesion of both soils with 10% and 20% shell husk increases linearly as cement percentage is increased. This is due to cementitious hydration as shown in Figure 5.14. This process forms a network and serves as the glue that provides strong structure and finally stabilized the soil (Prusinski and Bhattachaja, 1998). Note that the internal friction angle of soil with 20% shell husk also increases as the cement percentage increases. For the internal friction of soil with 10% shell husk, it increases up to 4% cement and then decrease at 6% cement. Similar observation of results was also found in Hossain et al. (2006). Overall the direct shear test of soil which combined with shell husk and cement showed that by increasing the shell husk-cement percentage the shear strength of soil also increased. The largest cohesion and internal friction angle were achieved for the 20% shell husk with 6% cement.

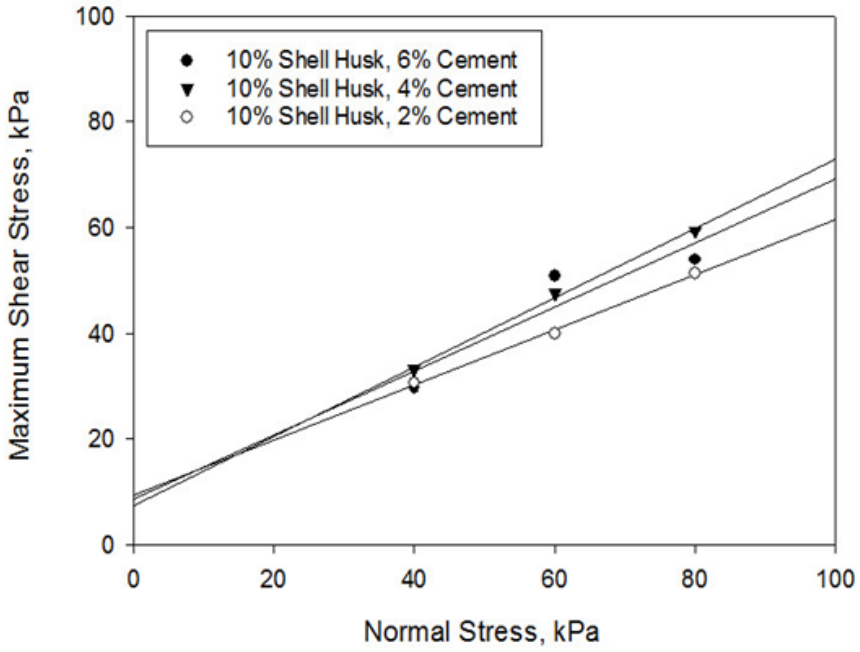

Figure 5.11 Normal stress vs maximum shear stress soil-shell husk-cement of 10% shell husk

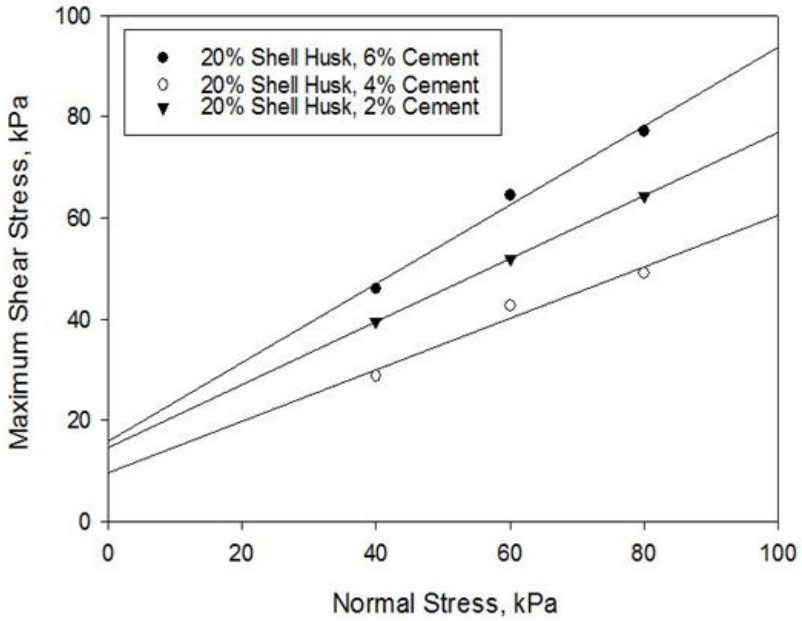

Figure 5.12 Normal stress vs shear strength soil-shell husk-cement of 20% shell husk

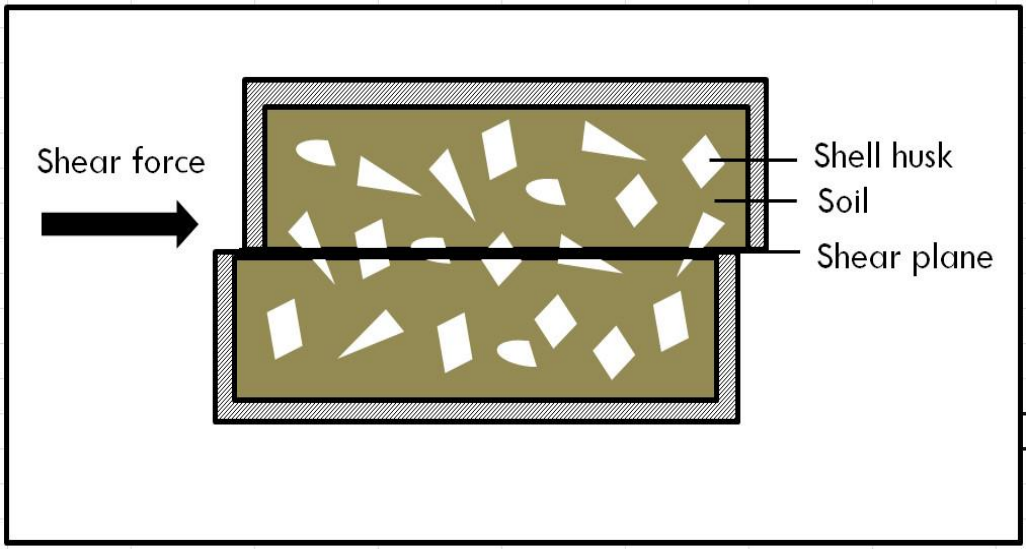

Figure 5.13 Shearing process of soil-shell husk-cement on direct shear test

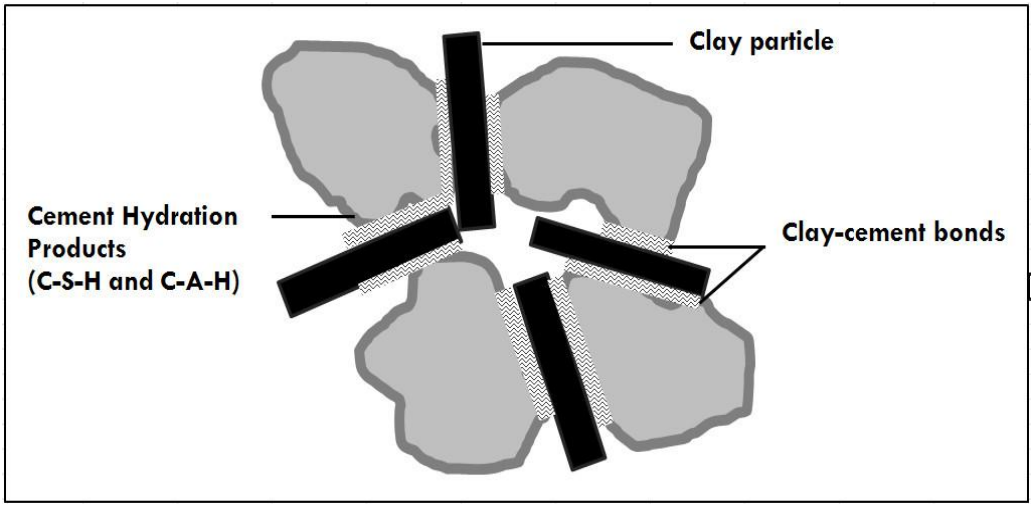

Figure 5.14 Hydration process

Table 5.2 Cohesion and internal friction angle of soil-shell husk-cement

Cement	Shell Husk 10%		Shell Husk 20%	
	Cohesion (*c*)	Internal Friction (φ)	Cohesion (*c*)	Internal Friction (φ)
2%	8.72	27.46	9.53	26.98
4%	8.90	32.40	14.62	31.88
6%	10.01	31.22	15.71	37.97

Table 5.3 Cohesion of soil-shell husk-cement (normal consolidation stress)

Cement	Shell husk 10%	Shell husk 20%
2%	87.2	95.3
4%	89.0	146.2
6%	100.1	157.1

Similar with the cohesion results of soil-shell husk samples, cohesion values of soil-shell husk-cement are assumed as results of over consolidation results. Table 5.3 show the conversion of soil-shell husk-cement cohesion related to normal consolidation results.

5. 2 CBR (California Bearing Ratio)

Earth fill structures play an essential part in agricultural performance (Figure 5.15). They offer various advantages and can be used in embankments, roads, and public facilities, depending on the purpose of the construction. Those agricultural structures must be stable under all static and dynamic loadings during construction and in service (Hossain, 2013). The subgrade soils are generally defined by their resistance to deformation under load. The strong subgrade soil can reduce the cost of the

embankment or road construction. By using strong or treated subgrade, the required thickness of a flexible pavement can be reduced as compared to untreated and weaker subgrades. It will therefore result in a significant cost saving advantage (Choudhary et al. 2010). Removing the soft soil and replacing it with stronger material such as crushed rock or other recycled materials is well known as a technique to improve subgrade soils (Choudhary et al. 2010 and Senol et al., 2006). In an investigation carried out in Spain, it was found that the utilization several types of stabilized materials for low volume roads had a significant on repair costs due to durability (Gallego et al. 2016). There are many types of stabilized materials one of which is recycled material, which has been recently used in subgrade soil to replace traditional earth material for the purpose of environmental sustainability. Sometimes recycled material is inferior compared to traditional earthen materials, but if its performance achieves the required level, it is nevertheless a very competitive material (Basha et al. 2005).

Shell husk has mechanical properties that are suitable to substitute for traditional earthen materials. Previous studies also showed that shell husk is a good source of calcium oxide (CaO) and calcium carbonate ($CaCO_3$), which provides the opportunity to reinforce the soil or bind the material construction (Park, 2014; Motamedi, 2015). The utilization of shell husk as recycled material has the aim of resolving several problems such as preservation of limited natural resources, saving disposal costs and environmental conservation. A huge amount of shell husk waste, especially in coastal areas could decrease the sanitation level of the people who live there. Furthermore, unhealthy living can become a trigger for social problems. Besides, budgeting is always a big problem; for example, Japan spent about US$ 32 million on disposing of shell husk waste and a large investment is required to treat this waste to give it value

(Hossain, 2013). In extreme cases, large amounts of shell husk can cause flooding by forming an embankment that blocks water flux.

Figure 5.15 Agricultural Structure

5.2.1 CBR of (Soil and Shell Husk)

In the present investigation, shell husks are used as material in the subgrade layer with the purpose of stabilizing the subgrade of soils. The stabilization of subgrade soil by using shell husk waste is expected to be used in infrastructure projects including those in the rural sector (roads, embankments) that require huge initial investments and after low rate of returns on investment (Satish, 2007). It is noted that shell husk, as an abandoned material, has a very low cost for infrastructure project investment. The performance of the subgrade is evaluated by using the CBR (California Bearing Ratio), which also becomes a key parameter in this study. The CBR test has been widely used to evaluate the strength of subgrade soils, sub-base, and base course material for the design of the thickness. A high CBR value indicates excellent quality of the material, although other relevant parameters may be necessary to re-confirm the material's performance (Ekeocha and Egesi, 2014).

In this study, three types of subgrade layers were tested in order to understand their performance responses based on CBR values. The types of layers are the upper layer, bottom layer, and combined layers (upper and bottom layers). The layers were cured for seven days. The purpose of this curing process to develop the relationship between the water content and strength (Senol et al. 2006). Three different percentages of shell husk that is 0 (control), 10, 20, and 30% were adopted for each type of layer. The results for the effects of different percentages of shell embedded at variable depths of soil as a subgrade layer are depicted and a pertinent discussion comparing the results with those of control specimens is presented.

1. Subgrade Upper Layer

The load-penetration curves obtained from the CBR tests of the subgrade upper layer containing 10, 20 and, 30% soil-shell husk mixture are shown in Figure 5.16. From this figure it can be seen that by increasing the shell husk percentage, the piston load at a given penetration also increases considerably which indicates that the obtained results are reasonable. All curves follow a typical trend of a CBR test.

The calculated CBR values for penetration depths of 5.0 and 2.5 mm in the upper layer subgrade are given in Table 5.2. The results show that the CBR values can be improved by increasing the amount of shell husk added to the soil-shell husk mixture. The CBR values for 5.0 mm penetration for soil mixtures with shell husk contents of 10, 20, and 30% are 9.88, 16.58, and 29.9% respectively. The CBR value of the control mixture (0% shell husk) in this research is 7.44%. This means that CBR values can approximately four times higher be achieved by increasing the soil-shell husk mixture's shell husk content to 30% in the subgrade layer compared to the CBR value of the control.

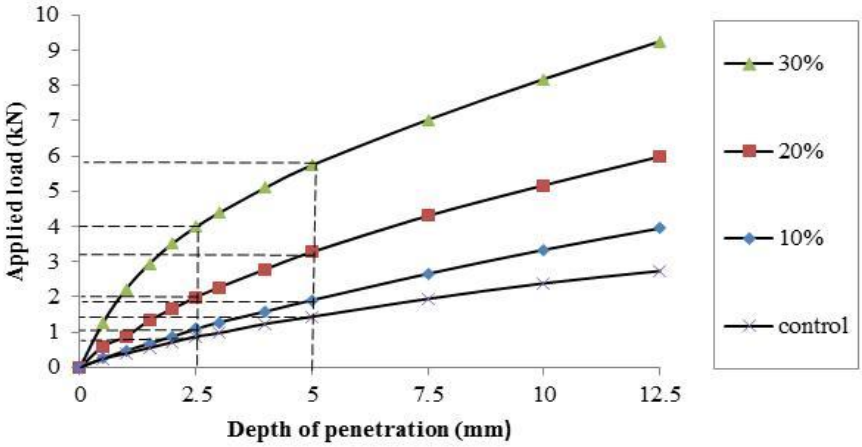

Figure 5.16 Load penetration curve of subgrade upper layer

2. Subgrade Bottom Layer

The CBR test results for the subgrade bottom layer containing soil-shell husk mixtures with shell husk contain of 10, 20 and, 30% are shown in Figure 5.17. It can be seen that the results of the control, and soil-shell husk mixtures with shell husk contain 10, 20, and 30% all display similar linear trends after penetration depth of 2.5 mm.

The CBR test values for penetration depths of 5.0 and 2.5 mm in the subgrade bottom layer are also given in Table 5.2. It can be observed that the CBR values increase with increasing shell husk percentage in the soil. CBR values for depths of 5.0 mm produce higher CBR values than those obtained for a depth of 2.5 mm. The CBR values of subgrade bottom layers containing soil-shell husk mixture with 10, 20 and 30 shell husk content are 10.55, 17.09, and 28.14% respectively.

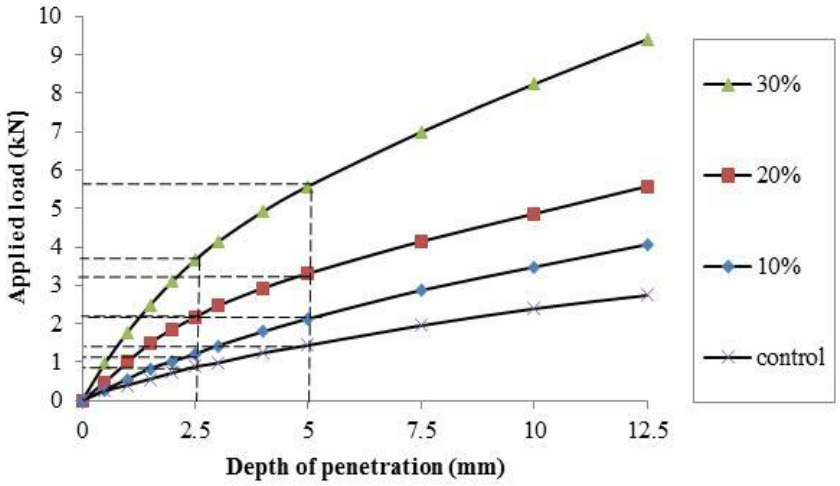

Figure 5.17 Load penetration curve of subgrade bottom layer

3. Subgrade Double Layers

Figure 5.18 describes the relationship of applied load versus the penetration depth in subgrade double layers containing soil-shell husk mixtures with 10, 20 and, 30% shell husk. The CBR curves for the double layer subgrade are less pronounced before 2.5 mm penetration in comparison to the previous two cases in the top and bottom layers. Again, in all cases, the results show similar linear trends after penetration depth of 2.5 mm.

The CBR values for penetration depth of 5.0 and 2.5 mm penetration are given in Table 5.2. In comparison to the previous two cases of the top and bottom layers, the current CBR results do not increase proportionally when the shell husk content in the soil-shell husk mixtures varies from 10 to 20%. Nevertheless, all the CBR test values show that the penetration depth of 5.0 mm produces higher CBR values than the penetration depth of 2.5 mm. The CBR values for a penetration depth of 5.0 mm for subgrade double layers containing soil-shell husk mixtures with shell husk contents of 10, 20 and, 30% are 15.08, 16.08 and, 24.37% respectively.

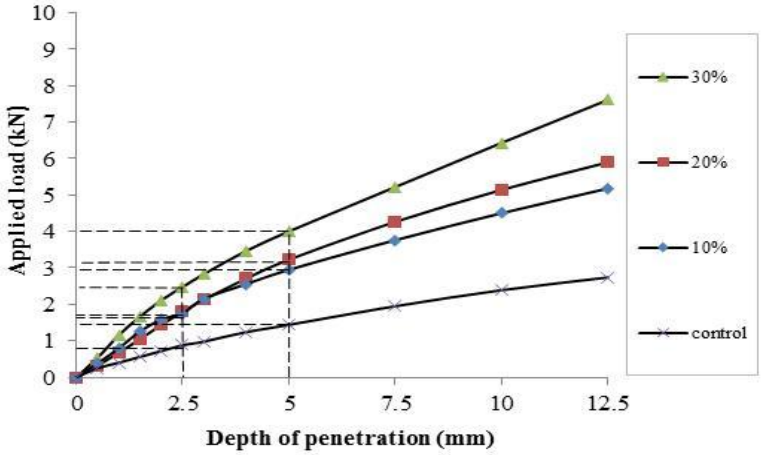

Figure 5.18 Load penetration curve of subgrade double layers

Table 5.2 summarizes the CBR values of the three types of subgrades with various percentages of shell husks. It shows that increasing the amount of shell husk in all three types of subgrade has a significant effect on improving the CBR value. The CBR value of upper layer of subgrade with 10% shell husk has the lowest CBR value, whilst the subgrade double layer has the highest one. On the other hand, with the addition of 20% shell husk in soil-shell husk mixture, the subgrade double layer shows the lowest CBR value and the subgrade bottom layer the highest. Further addition of shell husk in the subgrade layers alters the behavior of the ground conspicuously. It is observed that the 30% shell husk mixture in the subgrade upper layer gave the highest CBR value compared to the other two cases, whilst the subgrade double layer showed the lowest CBR value. Overall, for all types of subgrades, the 30% shell-husk mixture gave the highest CBR value as compared to other shell-husk percentages and the 10% shell husk mixture gave the lowest value. Interestingly, the increase in the double layer is less than those in the upper and bottom layers, indicating that there is no benefit in using a double layer in constructions with 30% shell husk.

Based on the experimental study of the three types of subgrade layers with various soil-shell husk mixtures (10, 20, and 30% shell husk), it was concluded that the CBR value increases by a factor approximately two to four as the addition of shell husk in the

Table 5.4 CBR values of three types subgrade

Subgrade Layer	**CBR standard**	**Shell Percentage (%)**		
		10	**20**	**30**
Upper	CBR 2.5	7.83	14.17	26.87
	CBR 5.0	9.88	16.58	29.9
Bottom	CBR 2.5	8.96	14.93	24.63
	CBR 5.0	10.55	17.09	28.14
Double	CBR 2.5	12.69	13.43	22.39
	CBR 5.0	15.08	16.08	24.37

mixture increases from the control (0%) to 30%. The highest CBR value was reached by the subgrade upper layer with 30% shell husk and the lowest CBR was obtained by the subgrade upper layer with 10% shell husk. It was also interesting to note that the double subgrade layer is not as beneficial as originally expected when compared to a single upper or bottom layer containing 30% shell husk. This conclusion has certain economic implications for optimizing design and construction using soil-shell husk mixtures. It is recommended that more research be done to outline the correlation between CBR values and shear strength parameters.

5.2.2 CBR of (Soil, Shell Husk and Cement)

Analysis in this part included the effect of shell husks percentages, subgrade layer types, and cement percentages. Figure 5.19 shows that by increasing the shell husk percentage,

the CBR value of samples is increased. Samples with 20% shell husk percentage have the highest CBR values. Based on the assumption of interlocking particle between shell husk and soil, 20% shell husks are better distributed when it is compared with 10% shell husk. This gives more resistance to soil layer. Evaluation of the CBR values based on the subgrade type layers showed the variation values. It can be seen from Figure 5.19 that CBR values between subgrade double and upper layer on soil with 20% shell husk (cement 6%) and 20% (cement 4%) are slightly different. From Figure 5.20 could be seen that upper layer and double layer are closer to the surface of sinking, indicating more resistance in comparison to bottom layer. Even though double layer gives the highest value, upper layers are more effective for field application due to materials supplied and economic reasons. It has benefit on budgeting aspect for design and construction when using this combination.

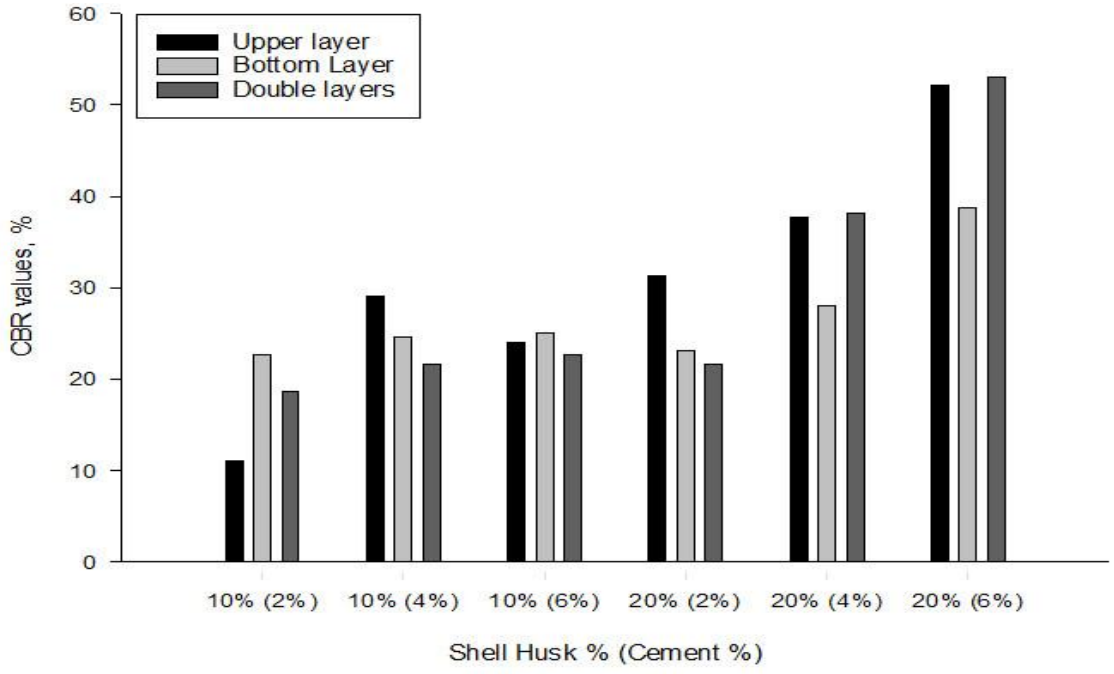

Figure 5.19 CBR value soil-shell husk-cement combination

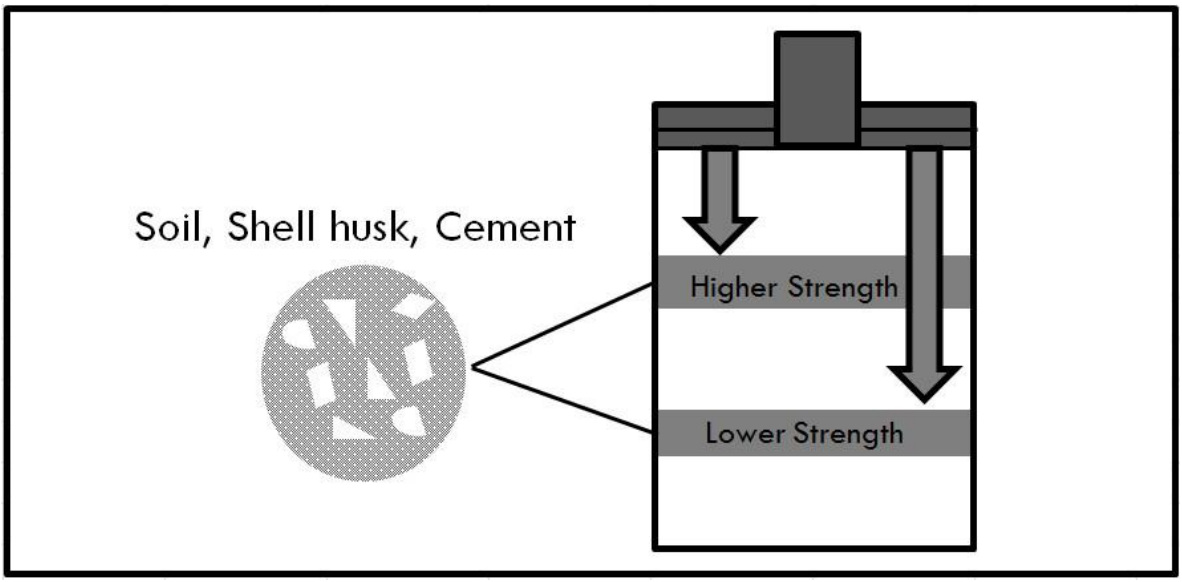

Figure 5.20 Mechanism strength of soil-shell husk-cement on CBR test

The figure also shows that the percentage of cement used does have significant effect to CBR values. Hossain and Sakai (2008) used SEM (Scanning Electron Micrographs) to explain the flocculation of soil particles where clay particles are brought together by cementing them to form a compound or secondary particle. This secondary particle is particularly responsible to strength development in cement treated soil even at nominal dosage rate of cement content. In summary, CBR of soils with 10% shell husk (cement 2%) for all type subgrade layers have lower values compare to others combination. On the other hand, combinations of soil with 20% shell husk (cement 6%) have higher CBR values for all subgrade layer type.

The addition of shell husk and cement increased the CBR value of all types of subbase layers. The highest CBR value was achieved by 20% shell husk with 6% cement. For practical and economic reasons, it is recommended to use the upper layer case in agriculture application.

5.3 Unconfined Compressive Strength (UCS) Tests

The UCS test is one laboratory test for pavement and soil stabilization application. It is also used as an index to evaluate soil improvement after treatment (Sariosseiri and Muhunthan, 2009). In order to understand the bearing capacity of the cement treated soils, the compression behavior of specimens under unconfined compressive test using the stress-strain curves (Hossain and Sakai, 2008).

5.3.1 Unconfined Compressive Strength (UCS) Tests (Soil and Shell Husk)

Test results on the unconfined compressive strength of the soil-shell husk with three different percentages of shell husk are provided in the figure 5.21. The stress-strain relationship of the controlled specimen (0% shell husk) is also illustrated in this graph.

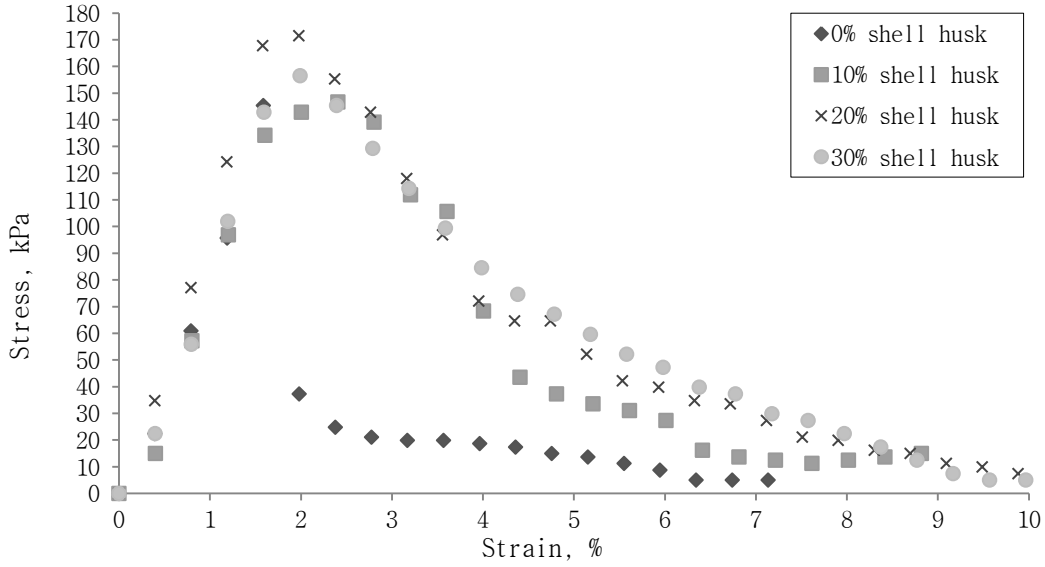

Figure 5.21 Stress-strain relationship of soil-shell husk

For each curve, there is an ultimate stress which known as compressive stress. It is observed that the compressive stress increases with increasing of displacement until the

peak value appeared depend on the percentage of shell husk. The compressive stress of soil-shell husk 0%, 10%, 20% and 30% are 145.36 kPa, 146.60 kPa, 171.45 kPa, 154.56 kPa respectively.

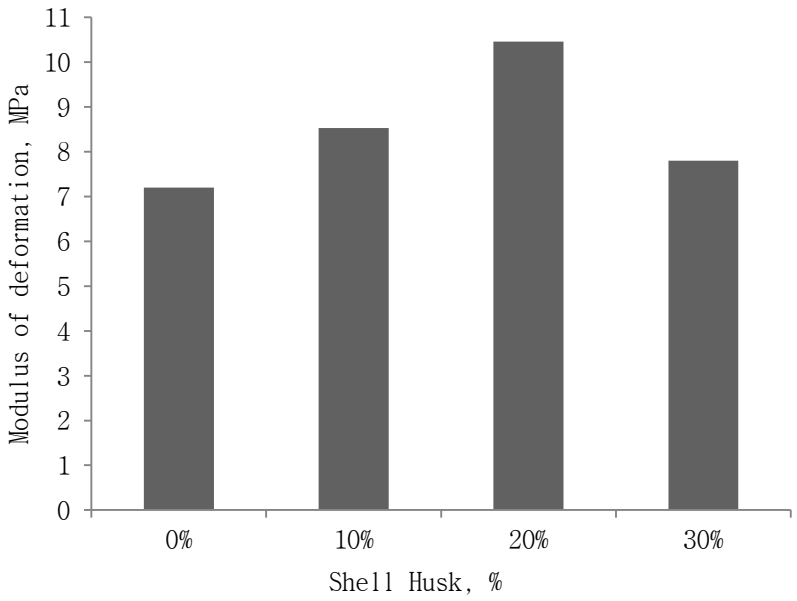

Figure 5.22 Modulus elasticity of soil-shell husk sample

The modulus of elasticity (E_{50}) of soil-shell husk sample are illustrated on figure 5.22. from this figure could be observed that modulus elasticity increasing up to 20% shell husk than decrease on 30% shell husk. From this figure, are known that addition of shell husk percentages giving higher modulus of deformation than soil without shell husk percentage (control). Modulus deformation of soil-shell husk 0%, 10%, 20% and 30% are 7.2 MPa, 8.52 MPa, 10, 45 MPa, 7.8 MPa respectively.

5.3.2 Unconfined Compressive Strength (UCS) Tests (Soil, Shell Husk and Cement)

Figures 5.23 and 5.24 present the stress-strain relationship of six compositions of soils, shell husk and cement. Initially the compression curves of the specimens are slightly

different and increment varies depending on the shell husk and cement percentage. Each curve shows the peak compressive stress at failure, and it then gradually decreases whilst showing the softening behavior. It is observed that soil with 20% shell husk has larger compressive strength than soil with 10% shell husk at same percentage of cement. As cement percentage increases so as the compressive strength of soil with shell husk.

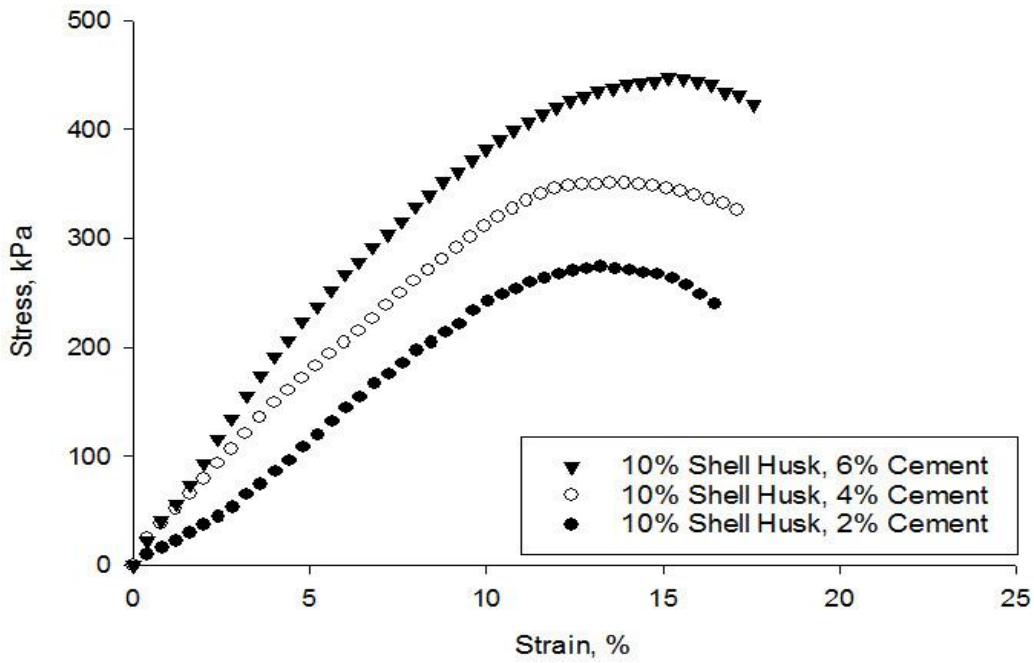

Figure 5. 23 Stress-strain relationship of soil-shell husk-cement under compression with 10% Shell husk

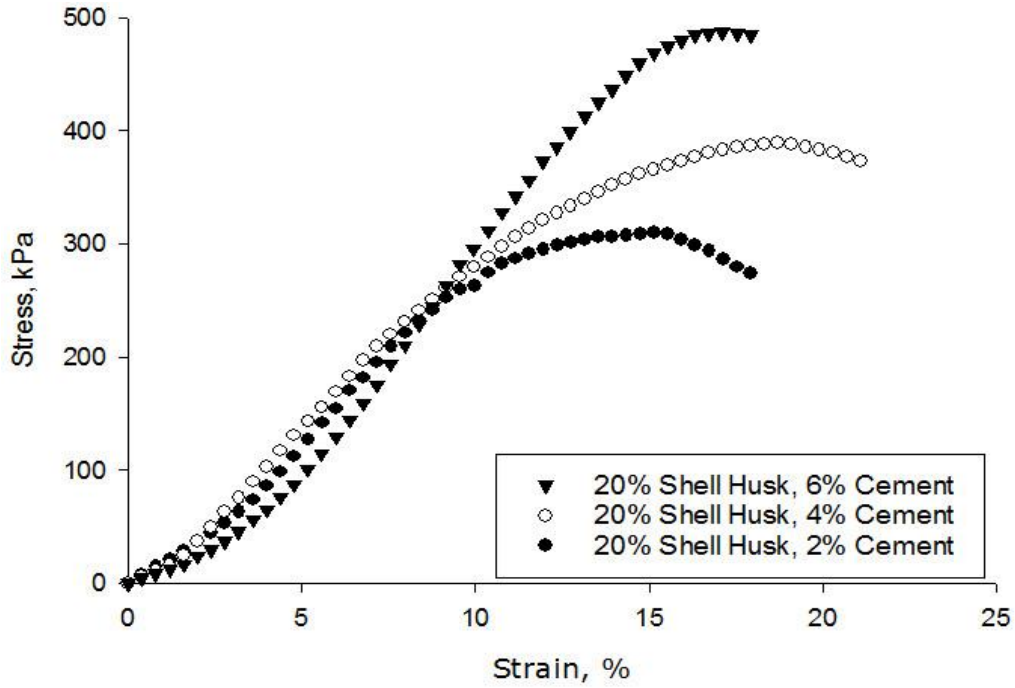

Figure 5.24 Stress-strain relationship of soil-shell husk-cement under compression with 20% Shell husk.

Figure 5. 25 Failure mode of control sample (left) and soil-shell husk-cement (right)

Figure 5.25 presents the failure modes of a control sample and a soil-shell husk-cement sample. It can be seen that the control has major diagonal failure pattern whereas soil with cement and shell husk addition has various slip lines showing the potential discontinuity due to the addition. It is known that both shell husk and cement particle would have altered the failure modes of soil.

Figure 5. 26 presents the moduli of elasticity (E_{50}) that was obtained using equation 2.

$$E_{50} = \frac{q_u/_2}{\varepsilon_{50}} \qquad (2)$$

In this equation, the 50% compressive strength is $q_u/2$ and $\mathcal{E}_{50}$ is the compressive strain when $\sigma = q_u/2$ in kPa. Note from the table that very little variation on the modulus of elasticity for all six cases were found. This is in contrast to the previous study by Hossain and Sakai (2008) which showed that, by using minimal dosage (<1%) of cement, both compressive strength and modulus of elasticity of clay soil are increased as the cement percentage increases. The main reason for this discrepancy is because of the soil – shell husk material used in this study. It can therefore be concluded that no direct benefit on the modulus of elasticity with the addition of cement to the current soil-shell husk material.

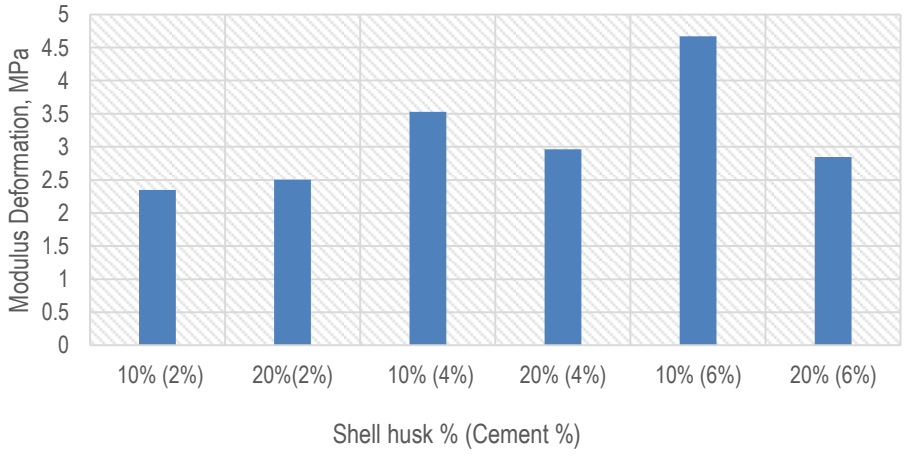

Figure 5.26 Modulus elasticity of soil-cement-shell husk sample

The increase of shell husk and cement percentage increased the compressive strength of the soil. There was very little variation in the estimation of the moduli deformation for all study.

5.4 Triaxial Test

The triaxial tests are performed to evaluate the shear strength of specimens which contained soil (only), soil-shell husks, soil-cement and soil-cement-shell husks. The utilize of cement has reason that additional 4 to 14% of cement could improve properties of soil (Hossain and Sakai, 2008). Furthermore, the addition of cement can be used for modified and stabilized purpose. Modified mean to improve workability and compaction characteristics while the term stabilized is encouraging to improve mechanical behavior of cement treated soil (Sarriosseiri and Muhunthan, 2009). In this study, all specimens are evaluated by the triaxial test. It is the most reliable methods for determining shear strength parameters under different drainage condition (Arora, 1978). The triaxial test provides information on the stress-strain behavior of the soil that the direct shear test does not. It also provides more uniform stress condition than the direct shear test with its stress concentration along the failure plane (Das, 2007).

5.4.1 Triaxial Test (Soil and Shell Husk)

Figure 5.27-5.30 show the relationship between axial strain (ε_a) and principal stress difference (σ_a-σ_r) of soil composite with 0%, 10%, 20% and 30% shell husk are illustrated. Most specimens under confining pressure 50 kPa and 100 kPa show the ultimate of that principal stress differences (σ_a-σ_r) then followed by softening behavior. On the other hand, under confining pressure 150 kPa and 200 kPa, the ultimate principal

stress for other specimens are defined on 15% axial strain because the peak value has not achieved. However, soil with shell husk 30% under confining pressure 200 kPa show peak value of principal stress difference (σ_a-σ_r) then followed by softening behavior.

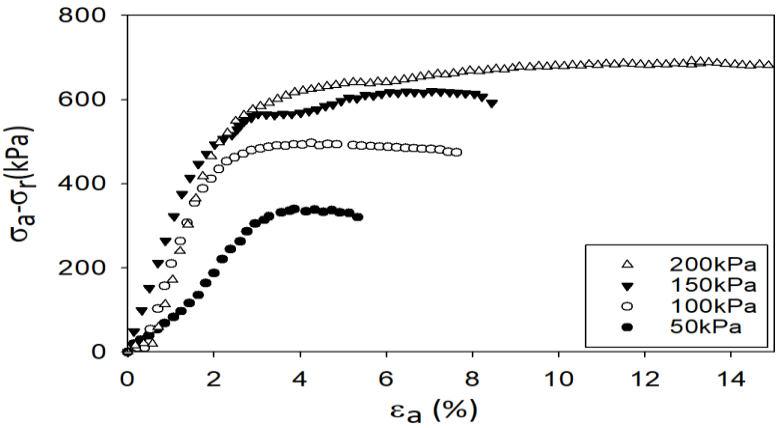

Figure 5.27 Curves of principal stress difference vs. axial strain of 0% shell husk

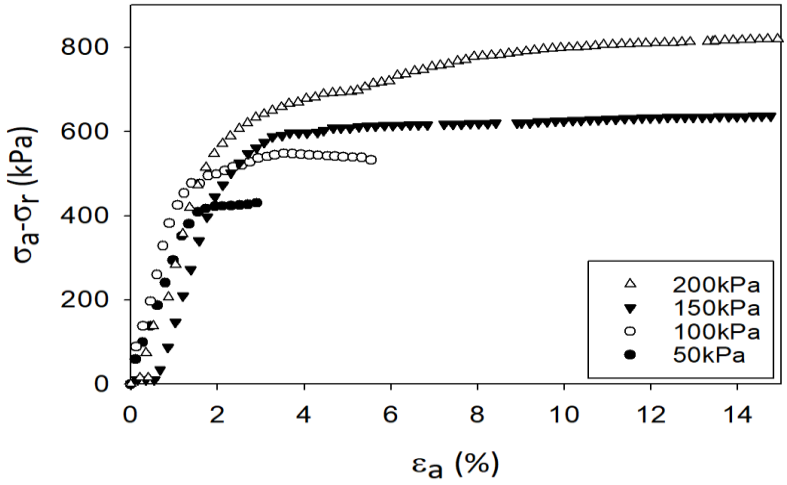

Figure 5.28 Curves of principal stress difference vs. axial strain of 10% shell husk

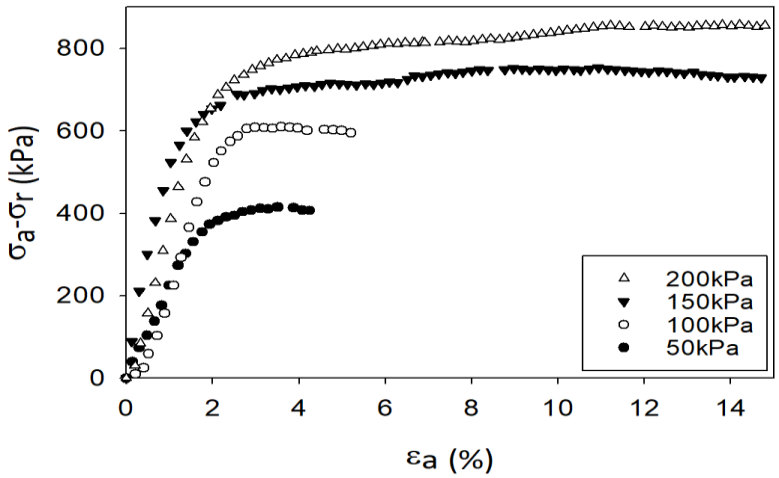

Figure 5.29 Curves of principal stress difference vs. axial strain of 20% shell husk

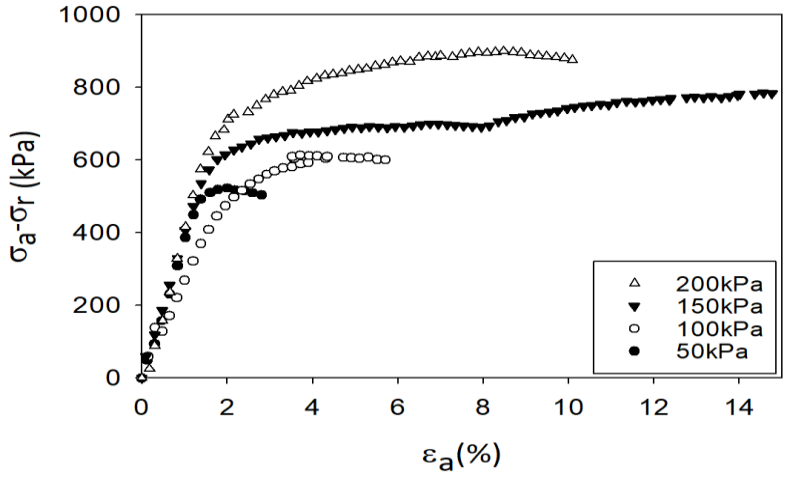

Figure 5.30 Curves of principal stress difference vs. axial strain of 30% shell husk

5.4.2 Triaxial Test (Soil, Shell Husk and Cement)

The relationship between axial strain (ε_a) and principal stress difference (σ_a-σ_r) of specimen which treated by 2%, 4% and 6% cement addition are given on Figure 5.31-5.33. The graphs show cement addition lead to decrease the axial strain of specimen. The increasing of cement percentage develops the principal stress difference (σ_a-σ_r) of specimens. Most of the maximal principal stress differences (σ_a-σ_r) are defined at 4% axial strain (ε_a). Furthermore, the specimens are treated by combination of cement and shell husk (Figure 5.34-5.42). The graphs show typically the combination

of shell husk-cement increase the axial strain (ε_a), compare to specimen which using cement only.

It was observed that overall specimen behaviour significantly affected by the addition of shell husk and cement percentage. Illustration of the relationship between axial strain (ε_a) and principal stress difference (σ_a-σ_r) of all specimens showed that peak strength and brittleness behaviour changed due to separate or combined effects of shell husk and cement percentages. From this figure is known that increasing of confining pressure enhance the principal stress differences (σ_a-σ_r). Most of the soil-shell husk specimens show the ultimate principal stress differences (σ_a-σ_r) which defined on 15% axial strain. On the other hand, soil with cement addition show peak strength then decrease axial strain. It was recognized that soil treated by cement exhibit much more stiffness and brittle behaviour than non-treated soil (Sarriosseiri and Muhunthan, 2009, Consoli et al. 1998). Figure 5.43 present failure pattern after triaxial test for (a) control, (b) soil-shell husk, (c) soil-cement, (d) soil-cement-shell husk respectively. As can be seen the failure patterns (c) and (d) show cracking which means brittle behavior due to cement addition.

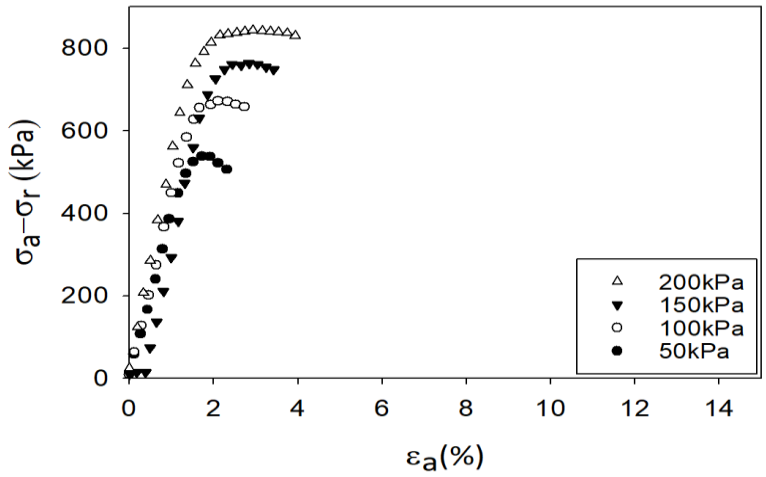

Figure 5.31 Curves of principal stress difference vs. axial strain of 2% cement

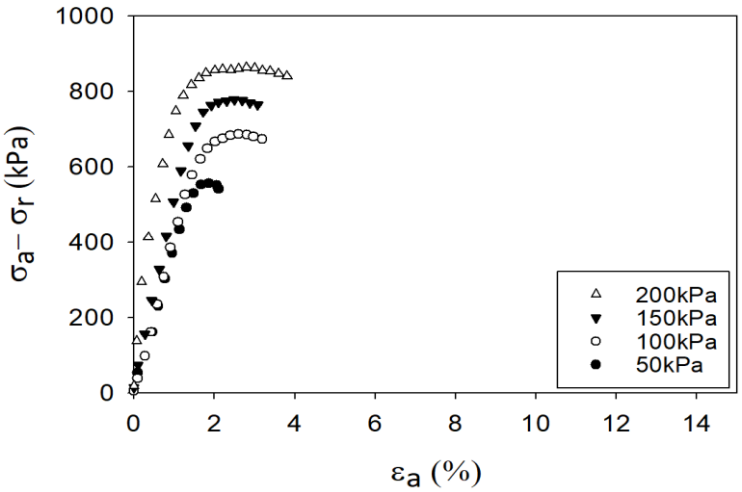

Figure 5.32 Curves of principal stress difference vs. axial strain of 4% cement

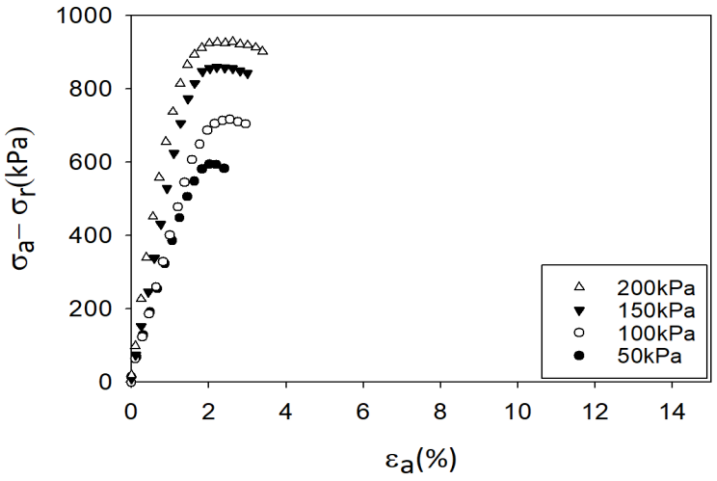

Figure 5.33 Curves of principal stress difference vs. axial strain of 6% cement

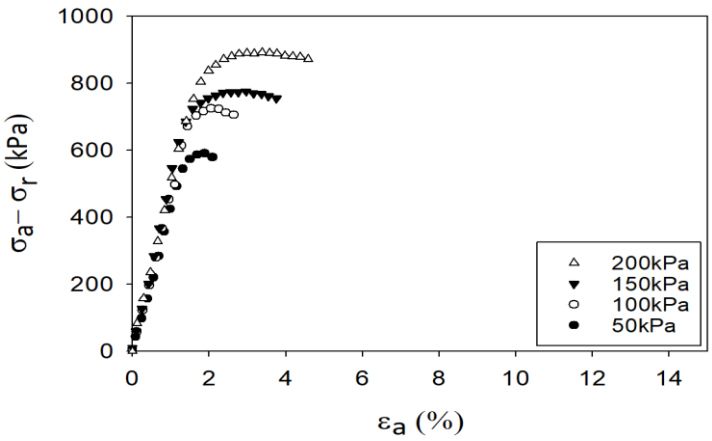

Figure 5.34 Curves of principal stress difference vs. axial strain of 10% shell husk (2% cement)

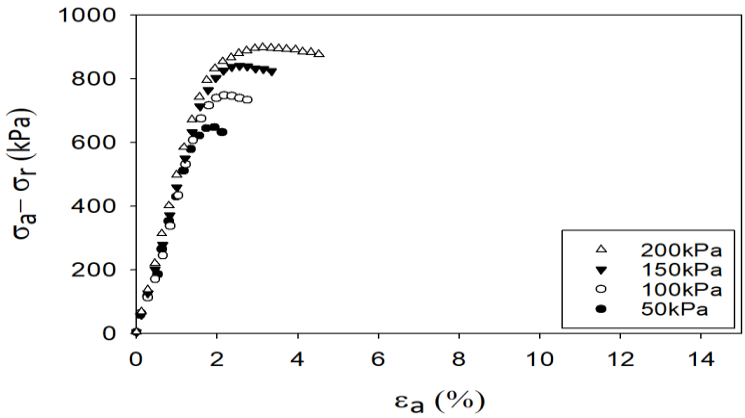

Figure 5.35 Curves of principal stress difference vs. axial strain of 10% shell husk (4% cement)

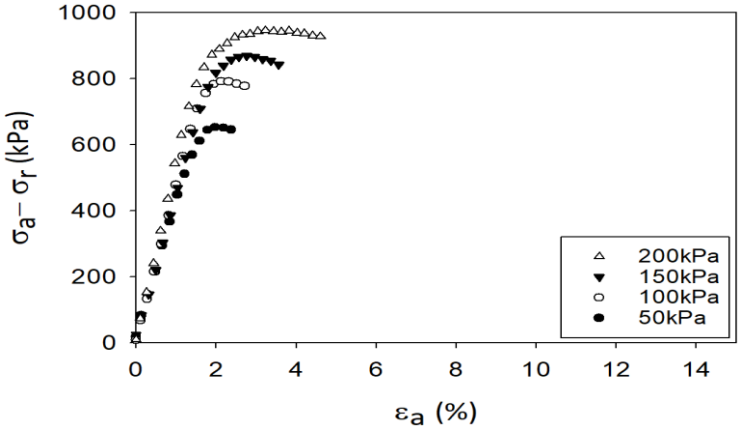

Figure 5.36 Curves of principal stress difference vs. axial strain of 10% shell husk (6% cement)

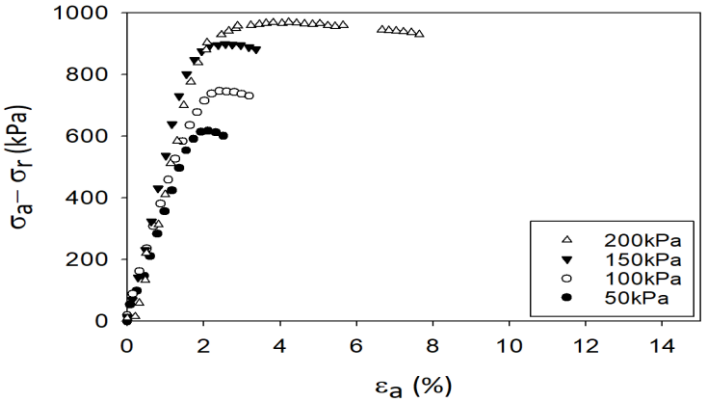

Figure 5.37 Curves of principal stress difference vs. axial strain of 20% shell husk (2% cement)

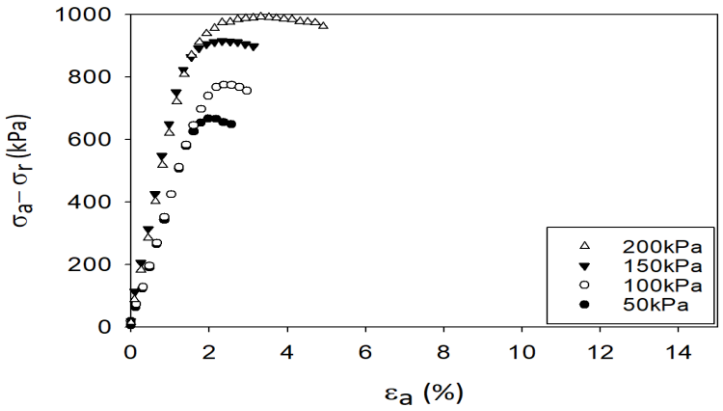

Figure 5.38 Curves of principal stress difference vs. axial strain of 20% shell husk (4% cement)

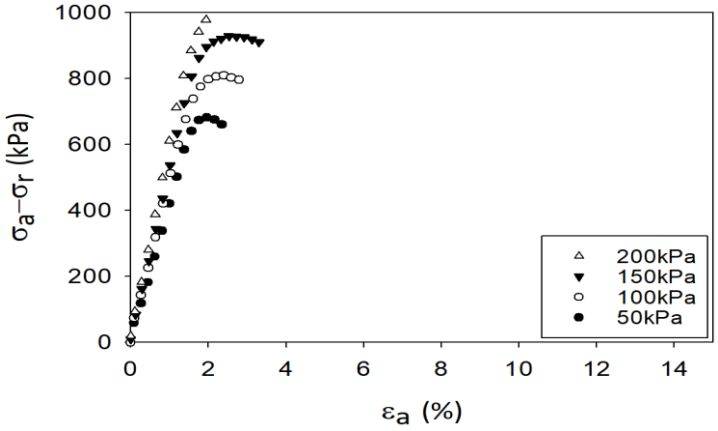

Figure 5.39 Curves of principal stress difference vs. axial strain of 20% shell husk (6% cement)

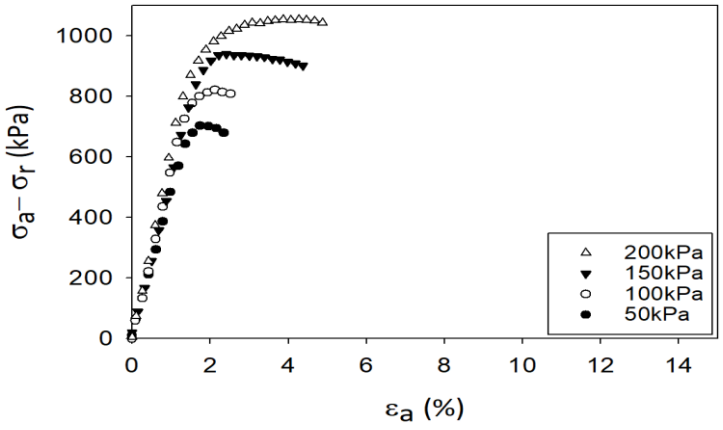

Figure 5.40 Curves of principal stress difference vs. axial strain of 30% shell husk (2% cement)

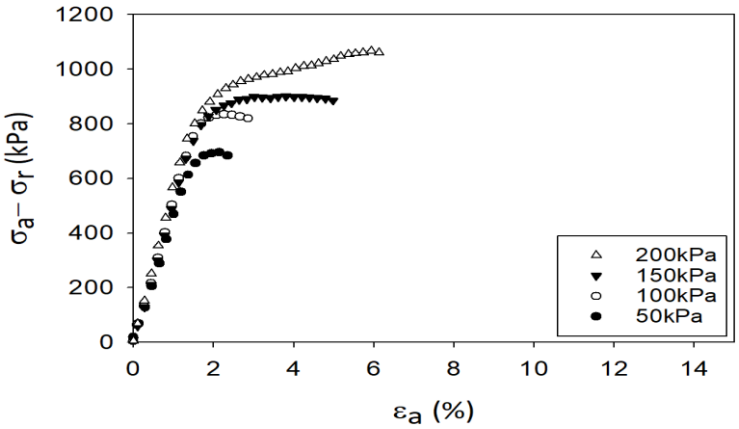

Figure 5.41 Curves of principal stress difference vs. axial strain of 30% shell husk (4% cement)

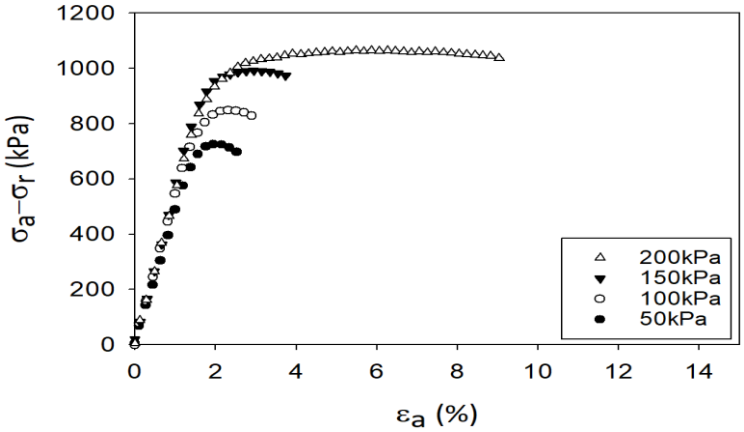

Figure 5.42 Curves of principal stress difference vs. axial strain of 30% shell husk (6% cement)

Figure 5.43 Failure pattern of samples

5.4.3 Shear Strength of Soil Composition with Shell Husk Percentages

The triaxial test has failure surface that reflected the real stress-strain characteristic of samples compared to direct shear test. In triaxial test, different conditions (drained, undrained, consolidation, and unconsolidated) can be simulated. The triaxial compression test has chosen as the accurate and reliable method by many researchers (Zhang et al. 2010). The calculations are obtained by Mohr-Coulomb criterion as a linear function of the normal stress (σ_f) on the plane at the same point which following by the equation (Mouazen et al. 2002):

$$\tau_f = c + \sigma_f \tan \varphi \quad (3)$$

An angle of internal friction (φ) is the measure of the shear strength of soils due to friction of soil and reinforcing materials (Zhang et al. 2010). On the other hand, cohesion (c) holds the particles of the soil together in a soil mass and independent of the normal stress (Arora, 1978). The results that obtained from triaxial test then calculating by using following equation to obtain (c) and (φ):

$$\sigma_a = \sigma_r \tan^2 (45 + \varphi/2) + 2c \tan (45 + \varphi/2) \quad (4)$$

The calculation is referred to equation 1, where σ_a and σ_r are the major and minor effective principal stresses, respectively (Das, 2007).

Indexes of angle of internal friction of soil-shell husk specimens are illustrated in figure 5.44. This table show that shell husk addition enhances angle of internal friction of soil-shell husk composite was assumed due to the irregular shape of shell husk particle which develops the frictional resistance between particles. Along with the result of direct shear test, cohesion which drawn on figure 5. 45, show increasing up to 20% of shell husk than decrease on 30% shell husk.

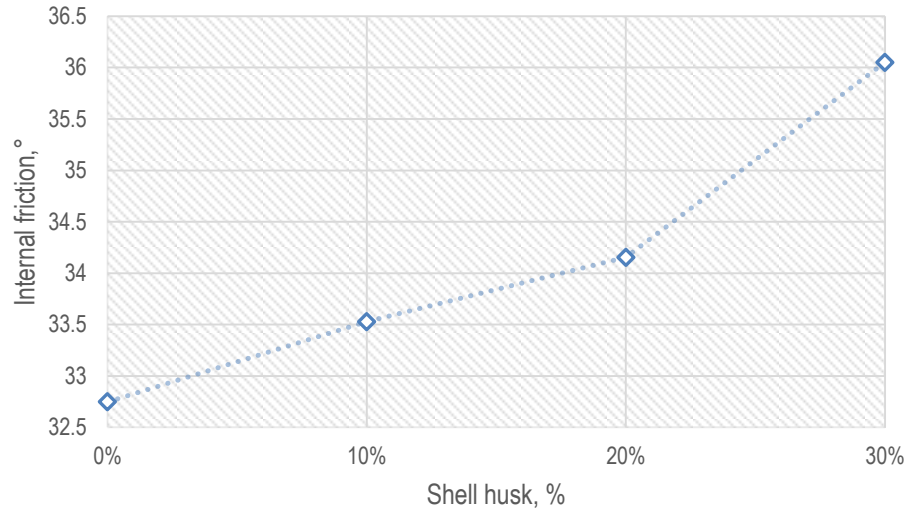

Figure 5.44 The angle of internal friction soil and shell husk

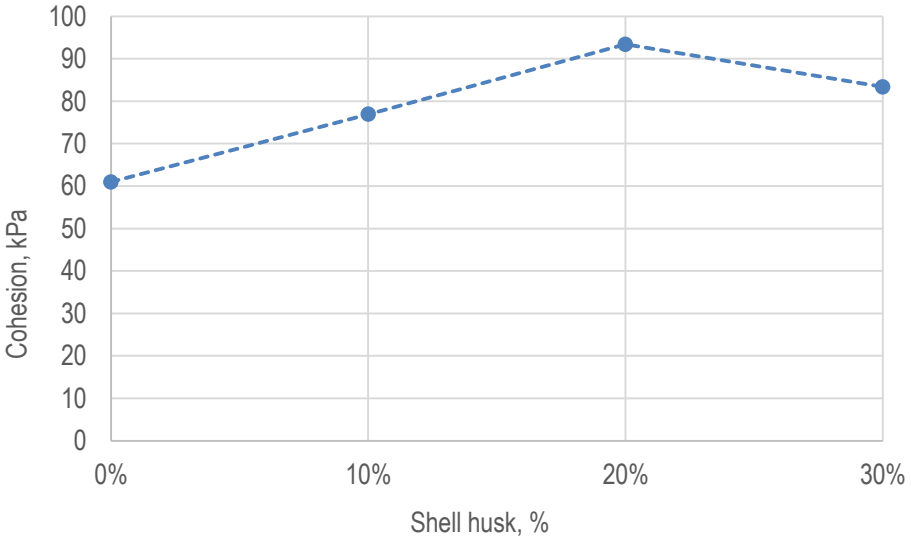

Figure 5.45 The cohesion of soil and shell husk

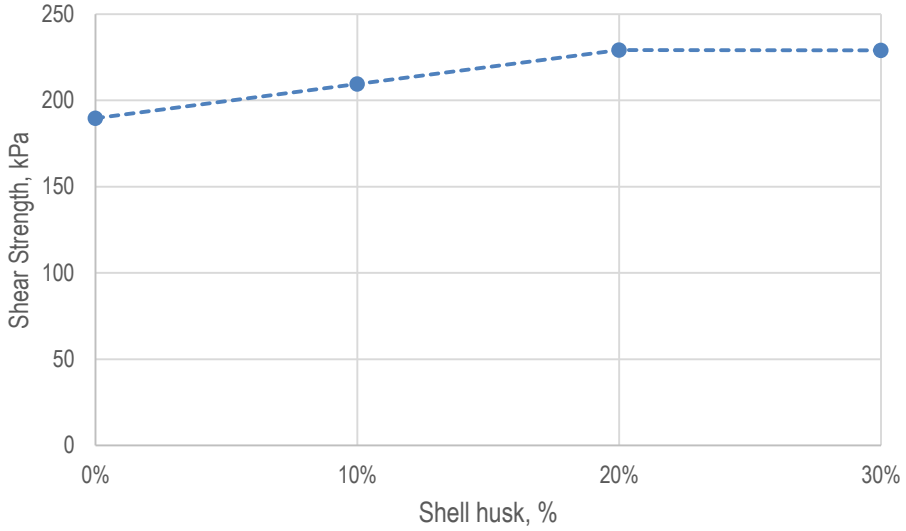

Figure 5.46 The shear strength of soil-shell husk (τ_f (200kPa))

For the specimen soil-cement-shell husk, shell husk percentage increasing also develop the angle of internal friction. However, soil with cement addition has a lower angle of internal friction compare to control and soil with shell husk only. It may due to anti-synergetic action between the angle of internal friction and cohesion (Figure 5.48) because specimen with cement addition has higher cohesion than other specimens (Hossain et al. 2006). The enhancement of cohesion is also known as the primary function of cementation process due to cementitious hydration. Shell husk also has an essential role to increase cohesion of specimens up to 20% shell husk. Shell husk has Ca^{2+} which attracted negative ion of soil which caused interlocking mechanism between soil and shell husks particles. However, the cohesion decrease at 30% shell husk for both specimen soil-shell husk and soil-cement-shell husk. It is realized that high percentages (>20%) of shell husk also increase brittle behaviour which has a consequence for the angle of internal friction and cohesion of specimen. The decreasing of soil cohesion after 20% shell husk addition occurred in previous research which

observed the shear strength of soil with shell husk reinforcement using direct shear test (Rachmawati and Zakaria, 2017).

Calculation of shear strength based on the angle of internal friction and cohesion are also available in Figure 5.49. The results were assuming the usual stress of 200 kPa, since the shear strength (τ_f) predicted by Mohr-Coulomb criterion which is proportional to the average stress (σ_f), conclusions based on Table 5.49 will be confirmed for any normal stress (Zhang et al. 2010). This table show that shell husk percentage give reinforcement to specimen. The reinforcement of shell husk percentage is illustrated in Figure 5.50. It is explained that shell husk percentage enhance shear strength of specimen by impede failure surface. The results show specimen which has shell husk and cement percentage combination has higher shear strength than those specimens which separately cement or shell husk addition only. Both angles of internal friction and cohesion may increase or

decrease by shell husk and cement percentage addition, but generally, the final results are an increase in shear strength.

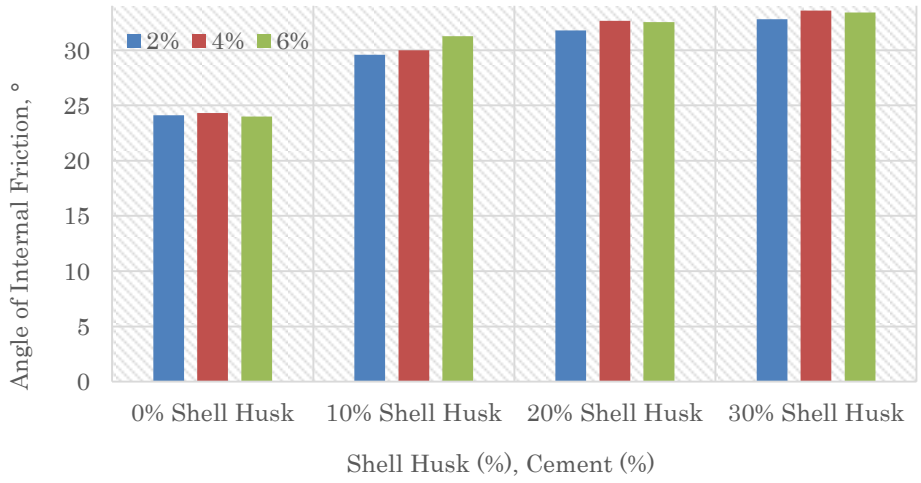

Figure 5.47 The angle of internal friction of soil-cement-shell husk

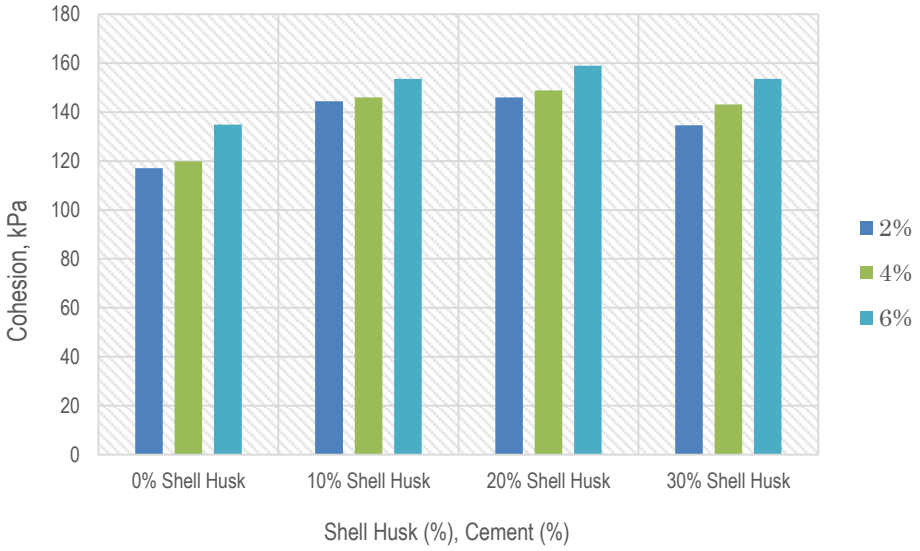

Figure 5.48 The cohesion of soil-cement-shell husk

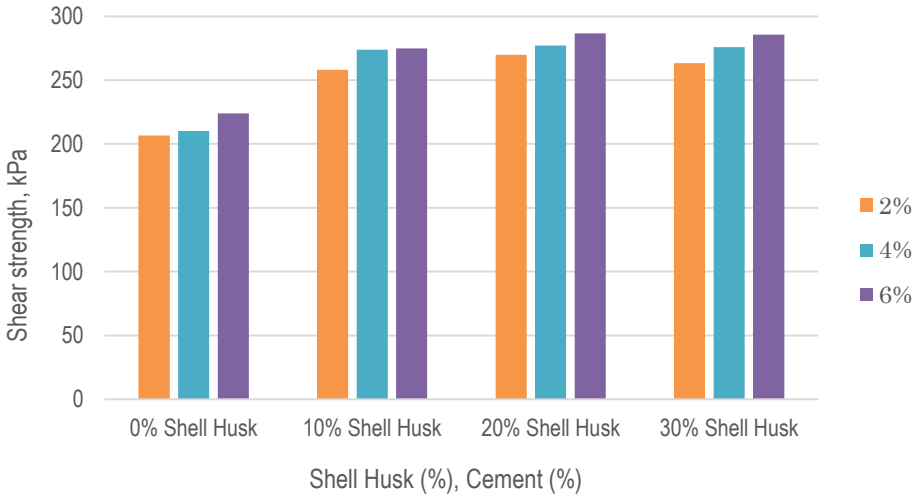

Figure 5.49 The shear strength of soil-cement-shell husk (τ_f (200kPa))

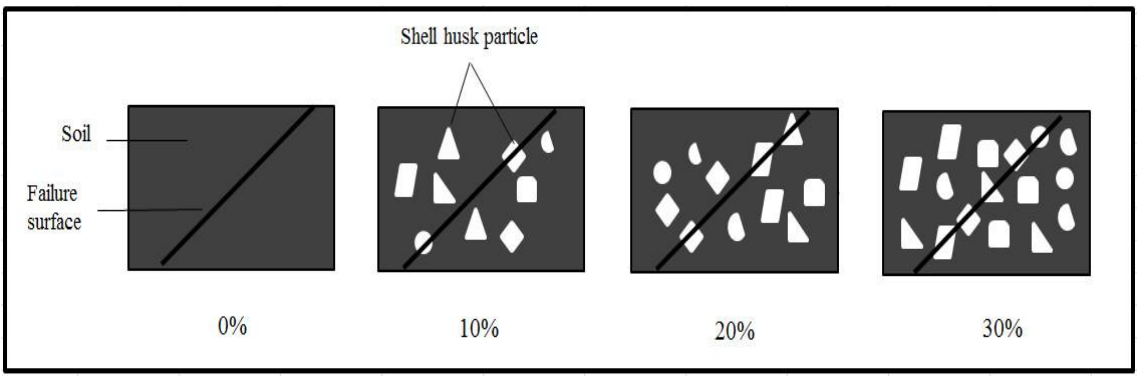

Figure 5.50 Failure surface of specimen soil-shell husk

5.5 Comparison of cohesion values among Direct shear test, UCS and Triaxial tests

Cohesion is one term to explain shear strength of soil. On this section are presented comparison cohesion values of two types sample and between three experimental methods (direct shear test, UCS and triaxial test).

Table 5.5 Comparison of cohesion soil with shell husk

Shell husk (%)	Direct shear test	UCS test	Triaxial test
Shell husk 0%	61.20 kPa	72.68 kPa	60.96 kPa
Shell husk 10%	63.20 kPa	73.30 kPa	76.91 kPa
Shell husk 20%	83.20 kPa	85.73 kPa	93.39 kPa
Shell husk 30%	72.00 kPa	77.30 kPa	83.34 kPa

Table 5.6 Comparison of cohesion soil-cement shell husk

Shell husk (%)	Direct shear test	UCS test	Triaxial test
Shell husk 10% cement 2%	87.20 kPa	136.66 kPa	144.42 kPa
Shell husk 10% cement 4%	89.00 kPa	175.18 kPa	159.86 kPa
Shell husk 10% cement 6%	100.10 kPa	223.63 kPa	153.53 kPa
Shell husk 20% cement 2%	95.30 kPa	154.86 kPa	145.90 kPa
Shell husk 20% cement 4%	146.20 kPa	194.43 kPa	148.84 kPa
Shell husk 20% cement 6%	157.10 kPa	243.51 kPa	158.96 kPa

CHAPTER 6: SUMMARY AND CONCLUSION

6.1 DIRECT SHEAR TEST

6.1.1 Soil-shell husk

1. Over all data showed that the percentage of shell husk as ground improvement material has significant effect to decrease the vertical displacement than soil without shell. It was observed that the mixing of waste shell husk in soil resulted more stress-transfer ability of the ground thereby increased the resisting forces. Soil without reinforcement (zero percent of shell husk) was reached the highest value of vertical displacement. The lowest vertical displacement was obtained by soil with 10% shell husk. The increasing of normal stress causing an increase in the shear stress-displacement of soil which showed that the soil with 0% shell husk and 10% shell husk are slightly different and this difference are getting more significant with the 20% shell husk and 30% shell husk addition.
2. Soil with 20% shell husk got the highest value of cohesion. Optimum water content affected the cohesion of clay soil and showed decreasing trend due to clay aggregate phenomenon that give rise to a granular feature to the soil mass at the 30% level. On the other hand, soil with 30% shell husk has the highest ultimate angle of friction which increased with the increase in percentage shell husk. It increases 5.6% compare to control. The result is expected because of the increase in the granular percentage of mixture.

6.1.2 Soil-cement-shell husk

1. It can be observed from this figure that soils with 20% shell husk and 6% cement have the highest maximum shear stress in comparison to other scenario. It is then followed by soils with 20% shell husk and 4% cement.
2. The direct shear test showed that by increasing the shell husk-cement percentage the shear strength of soil also increased. The largest cohesion and internal friction angle were achieved for the 20% shell husk with 6% cement. Cohesion of both soils with 10% and 20% shell husk increases linearly as cement percentage is increased due to cementitious hydration.

6.2 RESULTS CBR (California Bearing Capacity)

6.2.1 Soil-shell husk

1. The increasing amount of shell husk in all three types of subgrade has a significant effect on improving the CBR value. It was concluded that the CBR value increases by a factor approximately two to four as the addition of shell husk in the mixture increases from the control (0%) to 30%.
2. The highest CBR value was reached by the subgrade upper layer with 30% shell husk and the lowest CBR was obtained by the subgrade upper layer with 10% shell husk. It was also interesting to note that the double subgrade layer is not as beneficial as originally expected when compared to a single upper or bottom layer containing 30% shell husk. This conclusion has certain economic implications for optimizing design and construction using soil-shell husk mixtures.

6.2.2 Soil-cement-shell husk

1. The addition of shell husk and cement increased the CBR value of all types of subbase layers. The highest CBR value was achieved by 20% shell husk with 6% cement.
2. For practical and economic reasons, it is recommended to use the upper layer case in agriculture application. Even though double layer gives the highest value, upper layers are more effective for field application due to materials supplied and economic reasons. It has benefit on budgeting aspect for design and construction when using this combination.

6.3 RESULT OF UNCONFINED COMPRESSIVE STRENGTH (UCS) TESTS

6.3.1 Soil-shell husk

1. Soil with 20% shell husk has highest compressive stress than others shell husk percentages.
2. Modulus of deformation increase up to 20% shell husk then decrease on 30% shell husk.

6.3.2 Soil-cement-shell husk

1. The increase of shell husk and cement percentage increased the compressive strength of the soil. It is observed that soil with 20% shell husk has larger compressive strength than

soil with 10% shell husk at same percentage of cement. As cement percentage increases so as the compressive strength of soil with shell husk.

2. There was very little variation in the estimation of the moduli deformation for all study.

6.4 RESULT OF TRIAXIAL TEST

6.4.1 Soil-shell husk

1. Most specimens under confining pressure 50 kPa and 100 kPa show the ultimate of that principal stress differences (σ_a-σ_r) then followed by softening behavior. On the other hand, under confining pressure 150 kPa and 200 kPa, the ultimate principal stress for other specimens are defined on 15% axial strain because the peak value has not achieved.
2. The results show that cohesion increase up to 20% shell husk then decrease on 30% shell husk. On the other hand, internal frictions increase up to 30% shell husk.

6.4.2 Soil-cement

1. The graphs show cement addition lead to decrease the axial strain of specimen. The increasing of cement percentage develops the principal stress difference (σ_a-σ_r) of specimens. Most of the maximal principal stress differences (σ_a-σ_r) are defined at 4% axial strain (ε_a).
2. Most of internal friction of specimen soil-cement have lowest value compare to specimen soil-shell husk. However, cohesion have higher values than soil-shell husk specimen.

6.4.3 Soil-cement-shell husk

1. It was observed that angle of internal friction increased with shell husk percentage increasing, both of specimen soil-shell husk and soil-cement-shell husk, due to its shape which expand friction between particles. The cohesion both of specimens improve up to 20% shell husk addition then decrease on 30% shell husk addition. Variation values of angle internal friction and cohesion generated by cement and shell husk through anti-synergetic action.
2. Combination of shell husk and cement addition, generally increase the shear strength of specimen which using shell husk and cement separately. Shell husk has more effect to increase angle of internal friction while cement strengthening the cohesion of specimen.

REFERENCES

Agustini, T.W., Fahmi, A.S., Widowati, I., Sarwono, A. (2011). Utilization of Asian moon scallop (Amusium pleuronectes) shell waste on making of calcium-rich cookies (in Indonesian). Indonesian Fisheries Processing J, 1: 8-13.

Arora, K.R. Soil Mechanics and Foundation Engineering. (1978). Nai Sarak-Delhi: Standard Publishers Distributors.

Asikin-Mijan, N., Taufiq-Yap, Y.H., Lee, H.V. (2015). Synthesis of clamshell derived Ca $(OH)_2$ nano particles via simple surfactant-hydration treatment, 262, 1043-1051.

Baumgartl, Th., and Horn, R. Effect of aggregate stability on soil compaction. (1991). J. Soil and Tillage Research, 19, 203-213.

Barbachi, M., Imad, A., Jeffali, F., Boudjellal, K., Bouabaz, M. (2017). Physical characterization of sea shell for a concrete formulation. J. of materials and environmental sciences, 8 (1), 332-337.

Basha, E.A., Hashim, R., Mahmud, H.B., Muntohar, A.S. (2005). Stabilization of residual soil with rice husk ash and cement. J. Constr. Build. Mat, 19, 448-453.

Choudhary, A.K., Jha, J.N. and Gili, K.S. 2010. A Study on CBR behavior of waste plastic strip reinforced soil. Emi. J. Eng. Re, 15, 51-57.

Cokca, E., Erol, O., Armangil, F. (2004). Effects of compaction moisture content on the shear strength of an unsaturated clay. J. Geotechnical and Geological Engineering, 22, 285-297.

Collete, R.L. (2000). Marine and freshwater products handbook: Chapter Clams. A Technomic Publishing Company Book. Pennsylvania USA.

Consoli, N.C., Prietto, P.D.M., Ulbrich, L.A. (1998) Influence of fiber and cement addition on behavior of sandy soil. J. Geotech. Geoenviron. Eng, 124, 1211-1214.

Craig, R.F. Soil Mechanics. (1974). London: Van Nostrand Reinhold Company Ltd., ch. 4.

Das, M.B. (2007). Fundametals of geotechnical engineering. Chris Carson. USA.

Das, M.B., Sobhan, K. (2014). Principles of geotechnical engineering. Cencage Learning. USA.

Downey, R., Marsh, L., Flick G.J.F. Jr. (2012) The seafood industry: species, products, processing and safety-2nd ed. Balckwell Publishing. Ltd., 76-77.

Egan, D., Scolombe, B.C. (2010). Demonstrating environmental benefits of ground improvement. In. Proceeding of the Institution of Civil Engineers. Ground Improvement, 163, 63-69.

Ekeocha, N.E., Egesi, N. (2014). Evaluation of subgrade soils using California Bearing Ratio (CBR) in parts of rivers state. J. App. Sci. Env. Man, 18, 185-187.

Gallego, E., Moya M., Ayuga Tellez E., Garcia A.I., Ayuga F. (2016). The influence of management and construction methods in the repair costs of Spain's low-volume road network. J. Ag. Eng, 17, 537.

Han, J. (2015). Principles and practice of ground improvement. John Willey and Son. USA.

Hamidi, A., Hooresfand, M. (2013). Effect of fiber reinforcement on triaxial shear behavior of cement treated sand. Geotextiles and geomembranes, 36, 1-9.

Hazirbaba, K., Gullu, H. (2010). California Bearing Ratio improvement and freeze-thaw performance of fine-grained soils treated with geofiber and synthetic fluid. Journal of Cold Regions science and Technology, 63, 50-60.

Hirkane, P. S., Gore, N. G., Salunke P. J. (2014). Ground improvement techniques. International Journal of Intensive Engineering and Sciences, 2 (2), 11-13.

Hossain, M. B., Hossain, M. Z., and Sakai, T. (2012). Interaction properties of geosynthetic with different backfill soils. J. of Geosciences, 3, 1033-1039.

Hossain, M. Z., Narioka H., and Sakai, T. Effect of ordinary portland-cement on properties of clayey soil in Mie Prefecture. (2006). J. Jpn. Soc. Soil Phys,103, 31-38.

Hossain, M. Z. and Sakai, T. (2008). The Effectiveness of Nominal Dosage of Ordinary Cement on Strength and Permeability of Clayey Soil. Journal Japan Society Soil Physics, 110, 25-35.

Hossain, M. Z., Development of a Small-Scale Universal Apparatus for Evaluating Soil-Structure Interaction. (2011). J. of Civil Engineering and Architecture, 5 (4), 314-321.

Hossain, M. Z., and Inagaki, H. (2011). Dilatancy behavior of soil-structure interfaces for farm roads and embankments. J. of Agricultural Engineering, 2 (1),12-17.

Hossain, M. Z. (2013). Waste Shell Husks Concrete: Durability, Permeability and Mechanical Properties. J. of Building Construction and Planning Research, 1,

61-66.

Hossain, M. Z. (2013). Influence of Recycle Aggregate Composites on the Factor of Safety of Earthen Structures. J. of Geosciences, 4, 844-849.

Kumar, A., Gupta, D. (2016). Behavior of cement-stabilized fiber-reinforced pond ash, rice husk ash-soil mixtures. Geotextiles and geomembranes, 44, 466-474.

JGS 0524 (2001) Method for consolidated-drained triaxial compression test on soils, Japan Geotechnical Society. Dositsu Shiken Hou, Dositsu Kogakkai (in Japanese), 233-243.

Li, H.Y., Tan Y.Q., Zhang L., Zhang Y.X., Song Y.H., Ye Y., Xia M.S. (2012). Bio-filler from waste shellfish shell: preparation, characterization, and its effect on the mechanical properties on polypropylene composites. J. of Hazard Mater, 217-218, 256-262.

Li, X., Zhu, Y., and Zhang, Z. (2010). An LCA-based environmental impact assessment model for construction processes. J. Building and Environment, 45, 766-775.

Malkawi, A.I.H., Alawneh, A.S., and Abu-Safaqah O.T. (1999). Effects of organic matter on the physical and the physicochemical properties of an illitic soil. J. Applied Clay Science,14, 257-278.

Mishra, B. (2016). A study on ground improvement techniques and its applications. J. of innovative research in science, engineering and technology, 5, 72-86.

Mo, K.H., Alengaram, U.J., Jumaat, M.Z., Lee, S.C., Goh, W.I., Yuen, C.W. (2018). Recycling of seashell waste in concrete: A Review. Construction and Building Materials, 162, 751-764.

Motamedi, S., Shamshirband, S., Hashim, R., Petkovic, D., Roy, C. (2015). Estimating unconfined compressive strength of cockle shell-cement-sand mixtures using soft computing methodologies. Eng. Struc, 98, 49-58.

Mouazen, A.M., Ramon, H., and Baerdemaker, J.D. (2002). Effects of Bulk Density and Moisture Content on Selected Mechanical Properties of Sandy Loam Soil. Biosystem Engineering. Biosystems Engineering, 83 (2), 217-224.

Ortiz, O., Castells, F., and Sonnemann, G. (2009). Sustainability in the construction industry: A review of recent developments based on LCA. J. Construction and Building Materials, 23, 28-39.

Park K., Jun S., Kim D. (2014). Effect of strength enhancement of soil treated with environment-friendly calcium carbonate powder. The Sci. World J, 526491.

Peirson, W.M. (2000). Marine and freshwater products handbook: Chapter Harvesting. A Technomic Publishing Company Book. Pennsylvania USA.

Rachmawati, S. H., Hossain, Z. (2017). Utilization of waste shell husks for ground improvement. International Agriculture Engineering Journal, 26 (2), 11-18.

Rachmawati, S.H., Hossain, Z., Shiau, J. (2018). Ground Improvement Using Waste Shell for Farm Roads and Embankments, Journal of Agricultural Engineering, 49(1), 29-33.

Rachmawati, S.H., Hossain, Z., Shiau, J. (2018). Agriculture land development using shell husk as recycle aggregate. Agricultural Engineering International: CIGR Journal. (Accepted).

Raj, P.P. (2008). Soil mechanics and foundation engineering. Dorlingkindersley. India

Sariosseiri, F., Muhunthan, B. (2009). Effect of cement treatment on geotechnical properties of some Washington state soils. Journal of Engineering Geology, 104, 119-125.

Satish P. (2007). Rural infrastructure and growth: An Overview. Ind. Jn. Agri. Econ, 62, 32-51.

Senol A., Edil T.B., Sazzad Md., Acosta H.A., Benson C.H. (2006). Soft subgrades'stabilization by using various fly ashes. J. Re., Cons, Re. 46, 365-376.

Tam, V. W. Y., and Tam, C. M. (2006). A review on the viable technology for construction waste recycling. J. Resources Conservation and Recycling, 47, 209-221.

Tang, C., Shi, B., Gao, W., Chen, F., Cai, Y. (2007). Strength and mechanical behavior of short polypropelene fiber reinforced and cement stabilized clayey soil. Geotextiles and Geomembranes, 25, 194-202.

Vaughan. (2001). Gladys arched collection on Family: Mactridae (surf or trough clams, Mactras). Washington State University. http://shells.tricity.wsu.edu/ ArcherdShellCollection/ Bivalvia/Mactridae.html. Accessed 10 February 2018.

Yildrim B., Gunaydin O. (2011). Estimation of California Bearing Ratio by using soft computing systems. J. Exp. Sys. App, 38, 6381-6391.

Yoon, G.L., Yoon, Y.W., and Chae, K.S. (2009). Shear strength and compressibility of

oyster shell-sand mixtures. J. Environmental Earth Science, 60, 1701-1709.

Yusof, M., Ujai, S.J., Sahari, F., Taib, S.N.L., Noor Mohamed, N.H. (2011). Application of clam (lokan) shell as beach retaining wall. Proceeding of Engineering Conference 4th Kuching, Sarawak, Malaysia

Zhang C. B., Chen L. H., Liu Y. P., Ji X. D., Liu X. P. (2010). Triaxial compression test of soil-root composites to evaluate influence of roots on soil shear-strength. Ecological Engineering, 36, 19-26.