Chapter 1
Introduction

1.1 Introduction

For all the Earth's structure the foundation is very important, it should be strong enough to support the entire structure. The strong base, the ground around it plays a very important role. Therefore, to work with soil, we need to have known their characteristics and the factors influencing its behavior should have a good understanding. Soil stabilization process helps in providing the necessary justification, a necessary construction performance for the construction work. From the beginning of work in construction, we must strengthen the soil properties, it is revealed in Ancient Chinese civilization which used to improve the soil's resistance, the Romans and the Incas also work in stabilizing the soil in different ways, some of which are very effective, their buildings and roads still exist. In India, the beginning of the modern era in the soil stabilization was as early as in 1970s when there comes a general shortage of oil and aggregate, it becomes necessary for the engineers to look at how to improve soil other than to replace the poor soil from at building site. Use of soil stabilization lost favor, due to the lack of good technical methods and obsolete technique. But recently, with the an increase in the infrastructure, the demand for raw materials and fuel continues to increase; due to which soil stabilization began to take a new form. With better usability studies, materials and equipment, it is becoming a popular and effective method of soil improvement.

Expansive soils also called as swelling soils have the tendency to expand during rainy season or when there is an increase in their moisture content. The increase in moisture can be from floods, rains, sewer leaking or from the surface evaporation in an area which is covered with pavement or building get reduced. The most common example of expansive soil in India is the ‘black cotton soil’, which can be found in many parts of central, western and southern India which covers the state of Maharashtra, Madhya Pradesh , Karnataka and Andhra Pradesh. The expansive soil or Black cotton soil used in this project is taken from the district Shivpuri in Northern Madhya Pradesh, India. The southern part of the Shivpuri district is covered by the black cotton soils derived by the weathering of the Deccan trap formation. Depth of the soil varies from paper-thin to

15m. The soil is taken from a depth of 1m. The engineering structures made on these soils are adhered or get damaged as the structures are subjected to cracks due to the highly detrimental behavior of these soils. Large scale distresses which are produced due to the expansive nature of the soil can be prevented either by obstructing the soil movement or by improving the properties of the expansive soil.

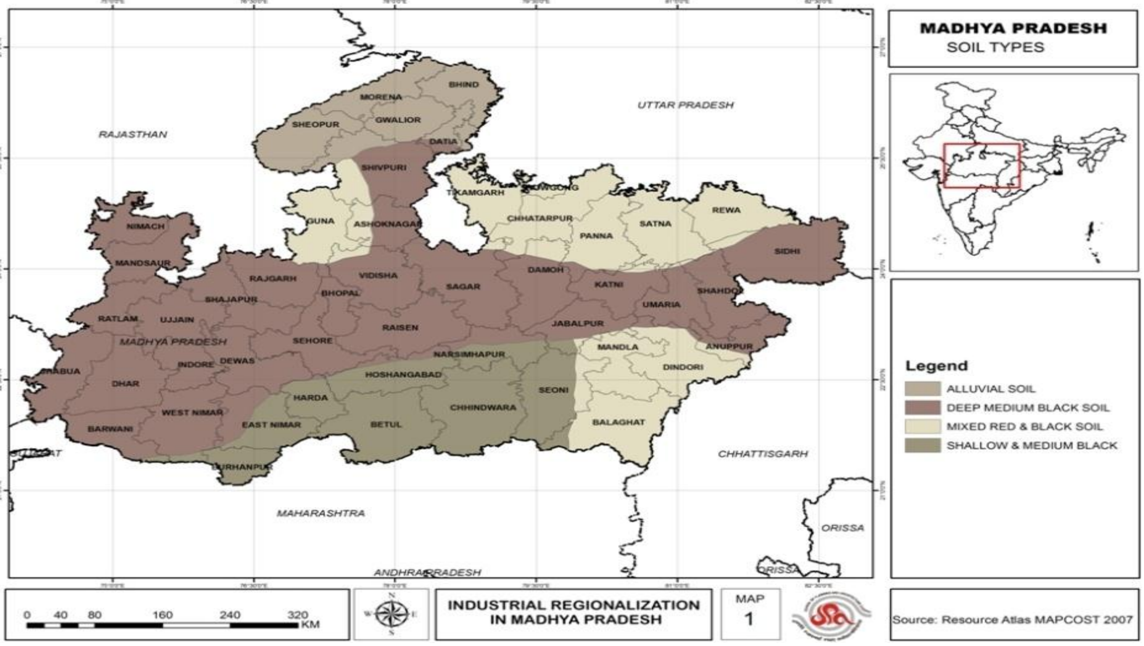

Figure1: Soil types in Madhya Pradesh(**Atlas MAPCOST 2007**)

Figure 2: Expansive soil at the site

As obstructing the soil movement cannot be achieved at every time and it is much costly so improvement of soil becomes a better option. Improvement of soil can be done either by modification or stabilization or by both. Soil modification uses modifiers such as cement; lime etc to change the index properties of the soil, while on the other hand soil stabilization is the treatment to enhance the strength and durability of the soil which can be used suitably for construction.

1.2 Soil stabilization: An overview

Soil Stabilization is the process of modifying certain properties of soil with different methods, such as mechanical or chemical for the production of an improve soil, which has all the desired physical and engineering properties.

The soils generally stabilized to increase strength and endurance and prevent the formation of dust and soil erosion. The main goal is the creation of a material or a ground system that suits the conditions of use and designs the life of the project design engineering. Soil properties vary considerably in different positions or in some cases even in a part; the success of the stabilization of soil depends on the soil test. Various methods are used to stabilize the soil and the procedure must be controlled in a laboratory soil material prior to application in the field.

1.2.1 Principles of Soil Stabilization:

- Evaluating soil properties of area under the consideration.
- Deciding the properties of the soil which needs to be changed to get the design value and to choose the effective and the economical method for the stabilization of soil.
- Designing composite stabilized soil mix samples and testing it in laboratory for the intended stability and durability values.

1.2.2 Needs & Advantages:

Soil properties vary greatly and the building structures depend largely on the bearing capacity of the soil, therefore, it should be stabilized to make it easier to predict the bearing capacity of the soil and even the improved bearing capacity. The soil classification is also very important to consider when working with soil variable characteristics. Well graded soil can be classified correctly is desirable because it has fewer gaps or uniform stable size sounds good having many gaps. Therefore, it is preferable to mix different types of soils and to improve the strength properties of the soil. It is very expensive to replace the entire subgrade soil and thus stabilizing the soils is what to look for in these cases.

- Improves soil resistance, which increases the capacity of the soil.
- It is more economical in terms of energy costs and increases the bearing capacity of the ground instead of going to deep or slab foundation.
- It is also used to ensure greater stability of soil slopes or elsewhere.
- Sometimes, soil stabilization is also used to prevent corrosion and dust, which is very useful especially in the time of dry and arid soil.
- The stabilization is also for sealing; that prevents water from entering the soil and thus helps the earth by the strength loss.
- Helps reduce soil volume change due to changes in temperature or humidity.
- Stabilization improves handling and durability of the plant.

1.2.3 Methods of soil stabilization:

1. Mechanical method of Stabilization

In this procedure, soils with different gradations are mixed together so to obtain the desired property and characteristics in the soil. This can be done at site or at some other place from where it can be easily transported. The final mixture prepared is then compacted by usual methods to get required density.

2. Additive method of stabilization

It refers to the addition of the manufactured products into the soil, which in the proper quantities can enhance the quality of the soil. Materials such as cement, bitumen, lime, fly ash etc. are used as the chemical additives. As in my project I have used microsilica and rice husk ash as an additive to stabilize the expansive soil.

1.3 Microsilica: An overview

Silica fume is a by-product of smelting process in silicon and ferrosilicon industry. The reduction of the high-purity quartz to form silicon at temperatures up to 2,000^{o} C produces SiO_2 vapors, which when oxidized and condensed in low temperature zone to form tiny particles which consists of non-crystalline silica. The by-products of production of the silicon metal and ferrosilicon alloys having silicon contents of around 75% or more contain 85–95% of non-crystalline silica. The by-product of production of ferrosilicon alloy which have 50% silicon has much lower content of silica and is characterized as less pozzolanic. Silica fume is also known as microsilica, volatilized silica or silica dust, and condensed silica fume. The American concrete institute (ACI) defines microsilica as a ‘‘very fine non-crystalline silica produced in electric arc furnaces as a by-product of production of the elemental silicon or alloys containing silicon’’. Its physical appearance is usually of a grey colored powder, somewhat similar to the Portland cement or some fly ashes. It can exhibits both pozzolanic and cementitious properties. Silica fume had been recognized as pozzolanic admixture that is effective in the enhancing of the mechanical properties to a great extent

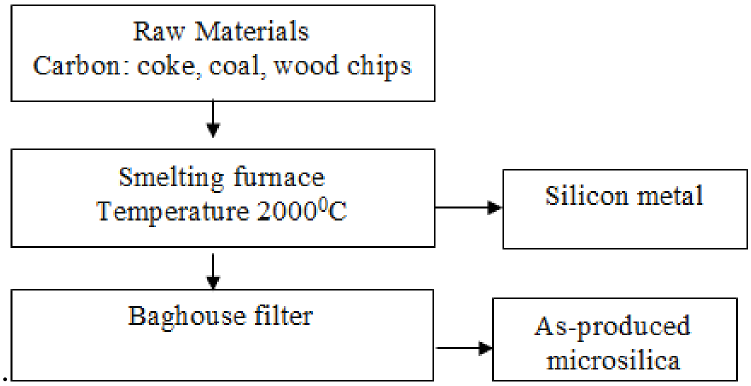

Figure 3: Schematic diagram of Microsilica production

1.3.1 Uses of microsilica

Microsilica can be used in following ways:

- Portland cement
- Dikes and engineering.
- Waste stabilization and solidification
- Mine reclamation
- Soft soil stabilization
- Road sub base.
- Blocks and bricks
- Aggregate
- Flow able filled
- Mineral filler in asphalt concrete
- Application in rivers icing
- Used as product designing in pavement for sub-base
- Other applications include concrete, geo-polymers, ceilings and tiles

1.4 Rice husk ash: An overview

India has a large agribusiness sector which has achieved notable successes in the past three and a half decades. Agricultural waste or waste consisting of organic compounds from organic sources such as straw, rice, palm oil empty fruit bunch, sugar cane bagasse, coconut, among others. Bark of rice is an example of alternative material which has a great potential. Rice husk is an important by-product of rice milling industry is one of the most common lingo cellulosic materials that can be converted into different types of fuels and chemical raw materials via a variety of thermo-chemical conversion processes. Rice Peel is abundant in rice producing countries in agricultural residues. The husk is produced during milling of rice. The rice for about 78% of the weight obtained as rice, bran and fragments. Balance 22% of the weight of the rice husk as obtained. The bark is used as fuel in the rice mills to generate the process of parboiling steam. This shell contains about 75% volatile organic compounds, and the remaining 25% of the weight of this layer is ashes used during the cooking process, which is known as rice husk ash. This RHA in turn contains about 85% - 90% of amorphous silicon dioxide. The moisture content ranges 8.68% – 10.44% and the bulk density ranged from 86-114 kg / m^3.

Rice husk ash is unusually high having 92-95% silica, highly porous and light, with a very high external surface. Absorbing and insulating properties of RHA are useful in many industrial applications, for example, as a strengthening agent in building materials applications and paddy processing in rectangular chipboard.

Construction is one of the fastest growing sectors in India. Rapid construction activity and the increased demand for housing had led to fall under the traditional building materials. Bricks, cement, sand and wood materials are becoming scarce. The demand for good quality materials for the replacement of traditional materials and the need for cost-effective and sustainable materials for the traditional material should be required by the researchers to develop.

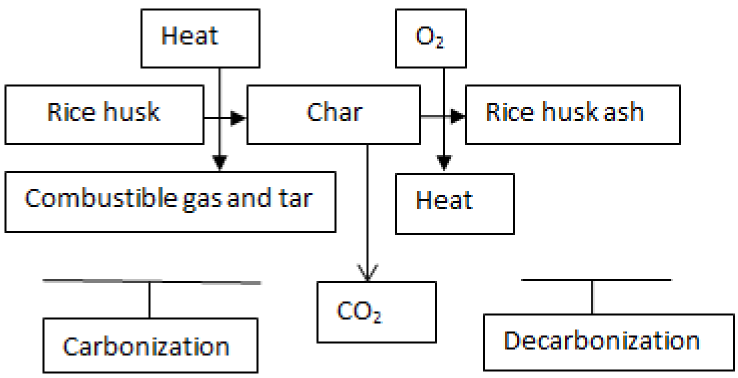

Figure 4: Thermal decomposition of Rice husk ash

A variety of new and innovative materials and specific requirements for housing materials in different geographical areas to reduce the risk of natural disasters and to protect against the elements to be cut also stressed the need for the development of lightweight, insulating material weight efficient construction, durable and ecological. Rice hulls can be used as building material, fertilizer, insulation material, or fuel. Using rice husk include aggregates and fillers for concrete and board production, efficient replacement for silica, absorbents for oils and chemicals, soil improvers, such as a silicon source, such as a steel powder insulation, so to name a few.

India is a major producer of rice husk which is usually used as fuel in the boilers for processing paddy, producing energy by burning and / or direct gasification during milling. Approximately 20 million tons produced annually RHA. This RHA is a major threat to the environment, causing damage to the land and the environment in which he left. Many ways are being thought to make a commercial use of this RHA rather than damping it.The annual production of rice husk in India is generally about 120 million tons. Rice Peel is generally not recommended as feed on cellulose and other sugars are low. Furfural and oil from rice husk is removed from rice husk. Industries use rice husk as fuel in boilers to generate electricity. Of the different types of biomass for gasification of rice husk a high ash content of 18 to 20%. Silicon dioxide is the main component of rice husk ash and the following table shows the normal composition of rice bran and rice husk ash. With such a high content of ash and silica content in the ash is advantageously extracted from the ash, which has a larger market, and also in order to ensure their removal. Number of rice producing countries is currently conducting research on the industrial uses of rice husk.

1.4.1 Uses of Rice husk ash

The uses of RHA are summarized below:

- Green concrete
- High performance concrete
- Refractory
- Ceramic glaze
- Insulator
- Roofing shingles
- Waterproofing chemicals
- Oil spill absorbent
- Speciality paints
- Flame retardants
- Carrier for pesticide
- Insecticide and bio fertilizers

1.5 Issues for the millennium

As India is on ninth number in the production of silicon, generating 68 thousands of tones, so silica fume which is a byproduct of smelting process in silicon and ferrosilicon industry having silicon content of 75% or more contains 85%-95% of non-crystalline silica. As silica fume, which otherwise be disposed of in landfills can be a threat to the environment producing toxicity. Disposal of the enormous amount of silica fume foils problem of huge land requirement, transportation, construction and maintenance. On the other hand, India which is an agrarian economy ranks second in the production of the rice in the world. Approximately generating 99000 metric tons of rice in which 23% of the total paddy weight that is 22,700 metric ton of rice husk ash is produced which if not disposed properly can respiratory and global warming problems. So using rice husk ash as a soil stabilizer, in clay brick and light weight concrete production can be beneficial. So now day uses of resource materials like silica fume and rice husk ash becomes a major area in search field. The past year sows a significant growth in technology level with respect to silica fume and rice husk ash and utilization in country and in next millennium silica fume and RHA in itself will be going to emerge as a major industry.

1.6 Need for the research

As many researchers have worked on the lime, cement, fly ash and other additives that can stabilize the soil, but use of microsilica and rice husk ash is limited. As India is at 9th position in producing silicon by which silica fume a by-product is extracted, on the other hand rice husk ash which is a by-product of rice hulls and in producing rice India is at 2nd position generating huge amount of rice hulls which has some calorific value which is being used in boilers for heating leaving behind a rice husk ash which might create some problems, which otherwise would be dumped and create many problems. So using these additives to stabilize the soils should be motivated and more and more research should be adopted to utilize these additives (microsilica and rice husk ash). Moreover India is a developing country, using these additives will put down some pressures on our economy. So in this report the microsilica and rice husk ash are used as an additive for the stabilization of the expansive soil.

1.7 Objective

The above goal was achieved with the following specific objectives:

1. Undertaking the literature review to establish the current position of the research relating to topic.
2. Investigating the engineering properties and characteristics of the Expansive soil.
3. Investigating the strength gain component of the soil when blended with microsilica.
4. Investigating the strength gain component of the soil when blended with rice husk ash.
5. Establishment of better Suitable additive i.e. microsilica and rice husk ash with expansive soil on the basis of the result.

1.8 Scope of thesis

1. Chapter 1 outlines the introduction of the report. It contains a figure of Madhya Pradesh showing the soil classification and an image of the soil used in this report. Need for the research and its objective is also given in this chapter.
2. Chapter 2 outlines the literature review of the report which includes the research done by the various scientists and result drawn by them.
3. Chapter 3 outlines the experimental work and methodology of the report which includes the investigation of the materials i.e. soil, microsilica and rice husk ash used in this report, their properties and specifications. This chapter also highlights the sample preparation and the experimental programs which are done in this report.
4. Chapter 4 outlines the results and discussion of the report which includes the index and engineering properties of soil, morphology of the materials i.e. soil, microsilica and rice husk ash. It also includes the result drawn when additives i.e. microsilica and rice husk ash are mixed individually in the soil, their engineering properties and discussion made on the results.
5. Chapter 5 outlines the comparison of results which includes the number of graphs showing the comparative results of additives i.e. microsilica and rice husk ash.
6. Chapter 6 outlines the conclusion of the report and the references used in the report.

Chapter 2
Literature Review

2.1 Introduction

Many researchers had contributed immense investigation in the field of stabilization of expansive soil with different additives like cement, lime, fly-ash etc. Numerous research papers were studied for the literature review in which additives using microsilica and rice husk ash to stabilize the soil are mentioned.

2.2 Expansive soil

Kariuki, P. C. (2004)[1] stated that “An expansive soil exists all over the world and can cause damages to the foundations and the associated Structures”.

Jones and Holtz et al.(1973)[2] had ascertained that the expansive clays which cause billions of dollars of damage every year in the USA, more than all the other natural hazards combined.

Chen, F. H(1988)[3] stated that the Geotechnical engineers did not recognize damages associated with the buildings on the expansive soils until late 1930s. The U.S. Bureau of reclamation made their first recorded observation about the soil heaving in 1938.The expansive soil usually swells and sticks when get wetted, and shrinks when dry developing wide cracks and a puffy appearance (desiccated clay). The swelling behavior can be usually attributed to intake of the water into montmorillonite, therefore expanding the lattice clay minerals in expansive soils. The montmorillonite which is made up of a central octahedral sheet, usually get occupied by aluminum or magnesium, and get sandwiched between the two sheets of the tetrahedral silicon sites so to give a 2 to 1 lattice structure.

Meehan and Karp(1994)[4] said that being apart from the increased research in the field of expansive soil, design of the shallow foundations to support the lightweight structures on the expansive soils is a potential problem than designing of foundations for heavy loads.

Day, R. W(1999)[5] discussed about the traditional designing criteria of considering the bearing capacity results in failure in the expansive soils. Problems of the expansive soils throughout the five continents results in a wide range of factors:

1. Shrinkage and swelling of the clayey soils resulting from moisture change
2. Type of clay size particles
3. Drainage– rise of the ground water or poor surface in drainage
4. Compression of the soil strata resulting from applied load
5. Pressure of the backfill soil
6. Soil softening
7. Weather
8. Vegetation and
9. The amount of aging

All the above factors considered to outcome out with the choice of the appropriate design criteria for the careful selection of proper type of the foundation and type of the structure and type of using construction materials. He discussed about the soils with high percentage of the swelling clay have a very high affinity for the water partly because of their small sized and partly because of their positive ions.

Patrick, M. D. and Snethen, D. R(1976)[6] discussed about the three layers clay mineral which have a structural configuration and a chemical makeup, which permits large amount of water to get adsorbed in the interlayer and in the peripheral positions on the clay crystalline, which results in remarkable swelling of soil. The presence of such various minerals such as the montmorillonite in expansive soil can be easily determined by use of X-ray diffraction method, among the other methods.

Ingles, O.G.et al(1972)[7] had given the reaction mechanism of stabilization of clayey soil. The figure 5 illustrates how the stabilization occurs in clay soil to form a gelatinous mass.

Al-Zoubi,M.S.(2008)[8] states that the swelling behavior of the soils can be influenced by many of the physical and environmental factors that cab contribute to expansive nature of a soil: these factors includes the type and amount of clay minerals, the physicochemical properties of the pore fluid, soil density, plasticity indices, water content, temperature, surcharge pressure and time.

Fattah et al.(2010)[9] studied the treatment of the expansive soil by using four types of additives i.e. cement, steel, gasoline fuel, fibers and injection by cement grout. The results shows that the treatment of expansive soils with a content of 5% cement or steel fibers or by the injection with cement grout reveals a better improvement while a content of 4% gasoline oil is sufficient resulting in the optimum treatment by this material and the angle of friction is not get affected by the above treatment while cohesion between the particles are slightly affected by the additives due to change in adhesion between additives and the soil particles.

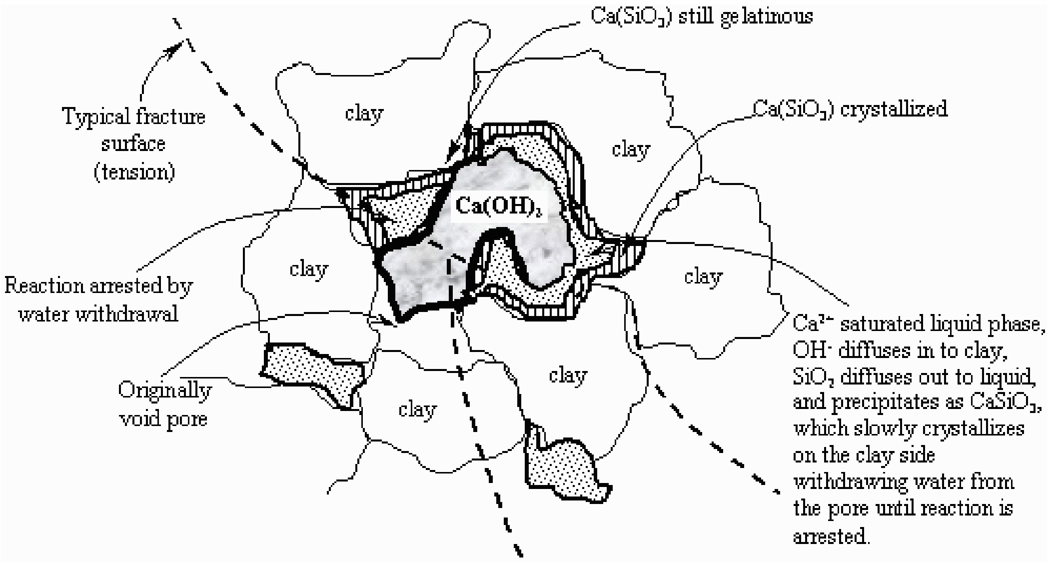

Figure 5: Reaction Mechanism of stabilization on Clay Soils **[7]**

Ali and Koranne (2011)[10] studied the effect of the stone dust and fly-ash combining at different percentage on expansive soil. The tests results such as an index property of the soil, Procter compaction, swelling pressure and the unconfined compressive strength obtained on the expansive clays when mixed at different proportions of the stone dust and fly-ash. From the results, it was observed that at the optimum percentages of 20% to 30% of admixture, it was found that swelling of the expansive clays was almost get controlled and there was a marked improvement in other properties of soil. The conclusion which can be drawn from this investigation was that the combination of the equal portion of stone dust and fly-ash is more effective than addition of the stone dust/fly-ash alone to expansive soil in the controlling the swelling nature.

Ramadas et al.(2011)[11] performed the laboratory test which was carried out on the three expansive soils when treated with lime and fly-ash to determine their effects on the geotechnical characteristics such as the Atterberg limits, compaction, the unconfined compression and the swelling properties. The experimental results noticed in a better improvement in properties of the expansive soil.

Abdullah and Alsharqi(2011)[12] discussed the rehabilitation of the medium expansive soil using the cement treatment with the various contents i.e. 1%, 2%, 3% and 4%. It was found that the 2% of cement content when cured for 28 days was sufficient for reducing the free swell percent of for medium expansive soil from as high as 7.4% 0.4%. The potential swell pressure gets reduced from a dramatically high value of 333kPa for untreated soil for a tolerable value of 20kPa for equal enhancement conditions.

Puppala and Musenda(2002)[13] discussed that the new methods should be continued to get researched so to increase strength properties and to reduce the swelling behavior of expansive soils.

2.3 Stabilization of soil with microsilica

Moayed, R.Z., et al(2009)[14] defined the Silica fume, which is also known as microsilica, is a fine –grained, thin and have a very high surface area of silica. Its sometimes get confused with the fumed silica which is also known as pyro-genic silica and the colloidal silica. This material possesses different technical characteristics, derivations and applications. He discussed that silica fume consist of very fine vitreous particles with surface area of an order of 20,000 m^2/kg or 215,280 ft^2/lb when measured by the nitrogen absorption techniques with particles approximately of 100 times smaller than average cement particle. So because of this extreme fineness and of high silica content, Silica Fume is highly effective pozzolanic material.

Makarchian, M. Et al.(2007)[15]

The history of the silica fume or microsilica is relatively was short, the first record testing of the silica fume was in Portland cement based concretes in 1952 and it was in early 1970 that concretes which contains the silica fume came into limited use. The biggest drawback of discovering that unique properties of the silica fume and its potential was lack of the silica fume to done experiment with. Early research which was made used an expensive additive which was called Fumed silica in a colloidal form of silica which was made by the combustion of the silicon tetrachloride in the hydrogen-oxygen furnaces. Silica fume on other hand is a by-product of a very fines pozzolanic material, which is composed mostly of amorphous silica produced by the electric arc furnaces in the production of elemental- silicon or the Ferro silicon alloys. In the late 1960s in Europe and in the mid 1970 s in the United States, the silica fume was simply went up in the stack as the smoke vented into atmosphere.

Abd El-Aziz, M.A.,et al(2004)[16] stated that the reaction mechanism of the pozzolanic materials like Lime reacts with any other fine pozzolanic component such as Silica Fume or microsilica to form the calcium-silicate cement with the soil particles .This reaction is water insoluble. The cementing agent which gets produced is exactly the same as that of ordinary Portland cement. The difference in that is the calcium silicate gel is formed from hydration of anhydrous calcium silicate i.e. cement, whereas with lime, the gel which is formed only by removal of the silica from clay minerals of soil. The pozzolanic process can be written as:

$$Ca\ (OH)_2 + SiO_2\ C\text{-}S\text{-}H_1 \ldots \textbf{(1)}$$

$$Ca\ (OH)_2 + Al2O_3\ C\text{-}S\text{-}H \ \textbf{...(2)}$$

Calcium Silicate Hydrate, C-S-H, produced is cemented material.

The silicate gel which get produced proceeds immediately to coat and to bind the clay lump in soil and blocking off the soil voids. In time, the gel gradually crystallizes into the well-defined calcium silicate which gets hydrates such as tobermorite and hillebrandite. The micro-crystals produced can also be mechanically interlocked. The reaction get ceases on drying and the soils which are very dry will not react with lime or with cement. The mechanism of this reaction can be represented as:

$$NAS_4H + CH\ NH + CAS_4H\ NS + \text{degradation product*}\textbf{..(3)}$$

$$NH + C_2SH^{**}\ (2CH)$$

Where: $S = SiO_2$, $H = H_2O$, $A = AL_2O_3$, $C= CaO$, $N = Na_2O$.

*As silica is progressively removed, the calcium aluminates and the alumina are formed residually. He also concluded by studying the effect of the lime silica fume stabilizer on the engineering properties of clay sub-grade. They have showed the result summarizing decrease in the plasticity index and swell potential and on the other side CBR was increased significantly. Also there is an improvement in shear strength parameter. He discussed the reason of decreasing of maximum moisture density which states that the compaction energy is less than the natural state. It is due to the microsilica fume which changes the particle size distribution and also the surface area. The other reason upholds the addition of the higher amount of microsilica fume with that of low density will fill the voids of composite samples.

Halstead (1986)[17] stated that “Pozzolans is defined as a siliceous or siliceous and aluminous material that itself possesses a little or no cementitious value but however, if be divided very finely in the presence of the water it will chemically react with the calcium hydroxide at the ordinary temperatures forming compounds which possess cementitious properties”.

Saranjeet Rajesh Soni et. al.[18] studied that solid waste disposal can improve the engineering performance of the black cotton soil which is an economical and effective method. The soil stability can be increased using fly ash and rice husk powder.

Azzawi et al.(2012)[19] studied the behavior of silty clayey soil on addition of the silica fume, and they have investigated that there is a significant variation on the swelling pressure and compressive strength on the samples prepared with silica fume. Their is an increase in permeability with the addition of silica fume and further they have observed that with the addition of silica fume the development of cracks on the surface of compacted clayey specimen reduced the crack width by 75%.

Venu Gopal N.[20] studied the properties of soil using silica fume as a stabilizer and compared the same with some other materials. His laboratory investigation indicate that the soil with low strength can be stabilized or treated with varying silica fume content from 5% to 20% weight of the dry soil. There is a significant increase in the strength characteristics using silica fume.

Biswas et al.(2012)[21] studied utilization of the rice husk ash with the lime in the sub-grade soil for rural road, they conclude that a very little amount of lime i.e. 3% when added to clayey soil with RHA can improve CBR value and the compaction characteristics to great extent.

Sabat and Nanda[22] studied the effect of marble dust with RHA on expansive soil has been studied by it has been reported that the CBR and UCS values increase substantially due to addition of these two materials with natural expansive soil.

Kalkan and Addulut (2004)[23] studied the effect of silica fume in the construction of hydraulic barrier in landfill and examined its suitability. They have showed in their results that clay when mixed with silica fume at different percentages results in high bearing strength , high compressive and low swelling potential.

M.Karimi and A.Ghorbani (2011)[24] studied the effect of the lime and microsilica admixtures on the silty sand soil, in the presence of sulphates. Results shows that addition of the microsilica in the silty sand soil increases the value of CBR which increase the strength and decreases the swelling characteristics, therefore the microsilica waste material can be used successively to enhance strength of the silty soil.

Ishraq Khudhair Abass(2013)[25] concluded that that the Lime and Silica Fume mixture was a effective stabilizer in improving the geotechnical properties of the clayey soil samples.

Taylor, et.al.(1996)[26] stated that microsilica fume is being used in the civil engineering works as binder material in the combination with the cement materials or as a individual for stabilizing the soil and outcomes of the results are great.

Das (1999)[27] discussed that the “flocculation and agglomeration produce change in texture of the soft clayey soils, in which clay particles tend to clumps together to form a larger particles, these reaction results in decrease of liquid limits , increase plastic limits, and decrease the plasticity index”.

National lime association (2006)[28] stated that the OMC increases with addition lime and microsilica fume due to the change in the surface area of the soil mixture. Also it was studied that the flocculation and agglomeration produce the change in texture of clayey soil, therefore the clay particles tend to clump with each other to form larger particles, these reaction tends to decrease the MDD and increase the OMC of the soil.

2.4 Stabilization of soil with rice husk ash

Beagle (1978)[29] assumes that husk to paddy ratio is about 20%.

Velupillai et.al (1997)[30] assumes a ash to husk ratio of 18% results 24.4 million tons of RHA worldwide.

Mehta, 1986[31] "The RHA is defined as a pozzolanic material because of its high amorphous silica content". RHA is obtained from burning of the rice husk. The husk is a by-product of rice milling industry. By weight only 10% of rice grain is formulated rice husk. On burning rice husk, about 20% becomes rice husk ash (RHA)**.**

Ramakrishna and Kumar, 2008[32] stated that in India, the annual production of paddy is about 100 million tones, thereby generating more than 4 million tons of RHA.

Payá, J., et. al[33] states that rice husk ash (RHA) chemically consists of 82 – 87 % silica which exceeds silica content of that of fly ash. Materials which contain high reactive silica i.e.SiO_2 are suitable to be used as the lime-Pozzolana mixes and can be a substitution of Portland cement. This high percentage of siliceous materials in RHA makes it an excellent material for the stabilization.

Della et al. 2002[34] states that the reactivity of rice husk ash (RHA) depends on its surface area and that surface area of rice husk ash (RHA) depends on environment and the temperature of combustion chamber.

Ganesan et al. 2008[35] stated the cement concrete paste hardens because of the reaction between the water and cementitious compounds so as to form several types of calcium silicate and calcium aluminate hydrates. The byproduct of initial reaction is the calcium hydroxide, which in the solution react slowly with the pozzolanic materials such as RHA. The product generated in this reaction is same type and characteristics of initial reaction, thus the additional bonding product will get available and the additional strength will develop.

Rahman 1987[36] discussed that the rice husk ash (RHA) which contains high amount of silicate reacts with calcium hydroxide to produce bonding substances; therefore rice husk ash (RHA) can be used as alternate to clay in the brick production.

Gromko (1974)[37] observed that the addition of rice husk ash (RHA) shows less reduction than the lime. However, rice husk ash (RHA) and lime mixture achieved a considerable reduction in the plasticity index, approximately by 90%, from 41.25% down to 4.74%. He reviewed that the plasticity index and, especially the shrinkage index are the early indicators of the potential expansion because in most soil the expansion occurs at the water contents between these indices.

Qijun Yu. Et.al (1999)[38] discussed that at the ordinary temperature and in presence of water, rice husk ash (RHA) can also react with Ca(OH)2, forming Ca1.5SiO3.5·xH2O. This reaction between SiO2 in rice husk ash (RHA) and Ca(OH)2 can yield the bonded gel i.e. C-S-H. It can cause agglomeration of the fine particles to become bigger particles.

Balasubramaniam et.al (1999)[39] concluded that the addition of 10 percent of lime shows the brittle behavior in soils. Adding RHA can make it more ductile, though the level attained in the strength is not that high.

Chapman, H.D.(1965)[40] stated that the chemical reaction that occurs when the fly-ash is mixed with the clay that includes the pozzolanic reactions, carbonation, cation-exchange and cementation. When fly-ash is mixed with clay it results in agglomeration in the large size particles. This causes increase in the compressive strength of the soil. Low cohesion makes rice husk ash (RHA) a poor cushioning and a construction material. However, after stabilizing it with fly-ash and curing it for 28 days, the rice husk ash (RHA) can acquire better cushioning properties and hence it can be used as construction material between sub-grade and the foundations. The result shows that at 15% fly-ash, for 28 day in curing period, the unconfined compressive strength comes to be 94 KPa**.**

Kate, J.M. and Katti, R.K.,(1980)[41], qualifies RHA as a cushioning material at a content of 15% fly-ash.

Ito et.al (2008)[42] conducted a field experiment in pineapple plantation at the Lampung Province of Indonesia to investigate the effect of the rice husk and the tapioca wastes i.e. cassava bagasse and cassava peel to be used as organic amendments on the soil physical and the biological properties. This treatment includes the control of rice husk mulch, cassava bagasse mulch, cassava peel-soil mixture, cassava peel mulch and black polyethylene film mulch. The organic material was applied manually. The soils physical and the biological properties at the initial and the final stages of the experiment was measured and compared. The result shows that moderate rate of the rice husk's decomposition process can slightly increased soils organic matter in a surface layer that can led to somewhat decrease in particle density and in the available moisture content enhancement. On other hand, the cassava bagasse mulch get decomposed within a very short period of time after the application and thus the role of cassava bagasse especially in the soil physical properties were no more in notice in months after its application. Due to the slow decomposition rate, the months were probably very short for cassava peel to get contribute in changing the soil physical properties.

Brooks (2009)[43] made trial to upgrade the expansive soil as construction material by using rice husk ash and the fly ash, which are waste materials. Remolded expansive clay has been blended with rice husk ash (RHA) and fly ash and their strength tests were conducted in the laboratory. The potential of rice husk ash (RHA) and fly ash when blended as a swell reduction layer between footing of a foundation and the sub-grade was then studied. In order to examine importance of study, a cost comparison was then made for preparation of sub-base of a highway project when blended with admixture and when unblended. Stress-strain behavior of the unconfined compressive strength shows that the failure stresses and strains get increased by 106% and 50% when fly ash content is increased from 0 to 25%. When the rice husk ash (RHA) content is increased from 0 to 12%, the unconfined compressive stress get increased by 97% while that of CBR was improved by 47%. Therefore, a rice husk ash (RHA) content of 12% and fly ash content of 25% was recommended for the strengthening of the expansive sub-grade soil. The fly ash content of 15% is recommended for blending into rice husk ash (RHA) for forming swell reduction layer because of the satisfactory performance in laboratory tests.

Yadu et al. (2011)[44] conducted laboratory study of the black cotton soil stabilizing it with fly- ash (FA) and the rice husk ash (RHA). The samples of the soils were been collected from rural road which was located in district Raipur of Chhattisgarh state. The soil was then stabilized with the different percentages of fly-ash with percentage content of 5, 8, 10, 12, and 15 and rice husk ash (RHA) with the different percentage content of 3, 6, 9 11, 13, and 15. Then the experiments were conducted in the laboratory such as Atterberg limits, specific gravity, CBR, and UCS tests for raw and stabilized soils. Results shows that the addition of fly-ash (FA) and rice husk ash (RHA) reduced plasticity index (PI) and the specific gravity of soil. The density and water content curves indicates that the addition of rice husk ash (RHA) results in an increase in the optimum moisture content (OMC) and the decrease in the maximum dry density (MDD), while these both values decreases with the addition of fly-ash (FA). The additions of the stabilizers i.e. fly-ah (FA) and the rice husk ash (RHA) increases the value of UCS and CBR, indicates the improvement in strength properties of soil. Based on the UCS and CBR tests, an optimum amount of fly-ash (FA) and rice husk ash (RHA) were found to be 12% and 9%, respectively.

Koteswara et al. (2011)[45] used rice husk ash (RHA), lime and gypsum as an additive to expansive soil which results in a considerable improvement in strength characteristics of expansive soil. It was also found that the rice husk ash (RHA) can have the potential to stabilize expansive soil solely or when mixed with the lime and gypsum. The utilization of these industrial wastes such as rice husk ash (RHA), lime and gypsum which can be an alternative so to reduce construction cost of the roads particularly in rural areas. It was also observed that liquid limit of expansive soil have been decreased by 22% with addition of the 20% rice husk ash (RHA) + 5% lime. It was also noticed that the free swell index of expansive soil had been reduced by 88% when there is an addition of 20% rice husk ash (RHA) + 5% lime. The UCS of expansive soil had been increased by 548% when there is an addition of 20% rice husk ash (RHA) + 5% lime + 3% gypsum after 28 days of curing.

Mtallib and Bankole (2011)[46] carried out an experimental study on the lime stabilized lateritic soils by using rice husk ash (RHA) as an admixture. The index property of the soil is classified as A-7-6 under AASHTO soil classification scheme. The Index and the geotechnical properties tests were conducted on soil containing the lime and rice husk ash (RHA) combinations shows a significant improvement in the properties. The Atterberg limits was significantly changed with the lime and rice husk ash (RHA) combination and the plasticity of soils was significantly get reduced from 18.10 to 6.70 for the sample A and 26.6 to 5.92 for the sample B for 6 % lime and 12.5% for rice husk ash (RHA) combination. The characteristic of the compaction, in addition of the lime and rice husk ash (RHA) decreases the MDD and increased OMC. The CBR values get peaked at 50% for un-soaked values for 8 % lime and 10 % rice husk ash (RHA) combinations for the sample A while that for sample B was 30%with a content of 6% lime and 12.5% rice husk ash (RHA) combinations.

Muntohar and Hantoro (2000)[47] found that liquid limit reduces with an increase in the content of lime and rice husk ash (RHA) combinations. They also found out that the plastic limit increases with the increasing in the content of lime and rice husk ash (RHA).

Osinubi and Katte (1997)[48] found out that there is a decrease in the maximum dry unit weight which can be attributed to replacement of the soil by the rice husk ash (RHA) in mixture which has relatively the lower specific gravity of 2.04 as compared to that of soil which has specific gravity between 2.69 to 2.71. There is also an increase in optimum moisture content with an increase in rice husk ash (RHA) contents.

Osula, (1991)[49] concluded that there may be an attribute to the coating of the soil by the rice husk ash (RHA) which results to form large particles having larger voids and hence less density.

RajuSarkar et al. (2011)[50] have studied detailed analysis of Pond Ash samples collected around Delhi from NTPC'S three Power Plants – Badarpur, Dadri, Rajghat and carried series of laboratory tests with various Admixtures like Bentonite, Polypropylene Fibre (1,2&3%),Lime, Marble dust & Gelatine Starch(1:2) and Characterize them by determining their Physical properties &Geotechnical characteristics with the conclusion that all the pond ash particles collected from Badarpur, Dadri and Rajghat are predominantly sand. The ratio of light (stand Proctor) compaction characteristics and heavy (Modified Proctor) compaction characteristics of Badarpur, Dadri and Rajghat pond ashes were different. The Badarpur pond ash could be compacted to a somewhat greater dry unit weight than the other two pond ashes. The ratio of MDD and OMC of the Proctor tests were fund for the three pond ashes as 75% and 133% ; 77% and 126% and 86% and 125% respectively. In the consolidation untrained triaxial shear of the Badarpur, Dadri and the Rajghat pond ash specimen (MDD-OMC stste),the deviator stress attained peak value at axial strains in the range of 1.5%-3.0% for the samples and the thereafter almost constant. The drained cohesion and angle of shearing resistance were 0 KPa and 30.4^0, 0 KPa and 32.0^0 and 0 KPa and 28.9^0 respectively.

Gupta et al. (2013) [51] studied the geotechnical behavior of fine sand with pond ash and lime. They have showed the formation of cementitious compound formation when pond ash reacts with lime in fine sand due to which the strength of the fine sand have increased. The SEM result shows a strong interlocking between the sand particles, pond ash and lime, thus giving enhanced strength to sand. They have showed that the maximum dry density of the sand increases with an increase in content of lime and it also increases with an increase in content of pond ash but only till 8%, after that it starts decreasing. There results also show an increase in the value of CBR with an increase in the percentage of lime and pond ash. There triaxial result shows that there is an increase in the angle of internal friction and cohesion of sand up to an optimum value of 9% lime and 16% pond ash.

Chapter 3
Experimental work and Methodology

3.1 Introduction

Large scale utilization of microsilica and rice husk ash in geotechnical constructions will reduce the problems faced by the silicon production and rice husk industry for its disposal as the property of these additives are closely related with natural earth material mostly because of its property closely related with the natural earth material. So assessment of the behavior of these additives (microsilica and rice husk ash) at different condition is required before its use as a construction material in Civil engineering structure. Even through adequate substitute for full scale field tests are not available; tests at laboratory scale provide a measure to control many of the variable encountered in practice. Details of material used, sample preparation and testing procedure adopted have been outlined in this chapter.

3.2 Material used

3.2.1 Expansive soil

The soil in this project was collected from the district Shivpuri in Northern Madhya Pradesh, India. The soil is excavated from 1 meter depth below the ground level. The physical appearance of soil is:

1. Black or darkish grey to brown color
2. Shrinkage and swelling behavior when it contact with moisture condition
3. Crack and heave formed and the cracks width are 15-20 cm.

3.2.2 Microsilica

The microsilica was brought from DBS building products Pvt. Ltd. , New Delhi. The product is DELKEM DENSIFIED SILICA FUME/MICROSILICA of grade 920-D. It compiles the following standards:

- ASTM C 1240 : 2005
- IS 15388 : 2003

It was supplied in 25Kgs bag. Following below is the properties of DELKEM microsilica.

Table 1: Microsilica properties and specifications provided by DELKEM

Properties		
1	State	Amorphous ultrafine powder
2	Color	Grey to medium grey
3	Bulk density -densified	500-700Kg/m^3
4	Bulk density – undensified	180-200 Kg/m^3
5	Specific gravity	2.25
Specification		
1	Silicon dioxide(SiO_2)	Minimum 90%
2	Moisture Content	Maximum 2%
3	Loss on ignition(LOI)%	Maximum 3%
4	Oversize percent retained on 45-µm	Maximum 8%
5	Specific Surface	15-27m^2/g
6	Pozzolanic activity index (%)	105% @ 7 D

3.2.3 Rice husk ash

The rice husk ash used in this was brought from NK Enterprises, Singhania house, Jharsuguda, Orrisa, India. Silpozz (Rice husk ash) has an amorphous silica/ microsilica with silica content of ver 89% having particle size of 25 µm mostly. The RHA was supplied in a 25 kg bag.

Table 2: Rice husk ash properties and specifications provided by NK ENTERPRISES

Physical Properties	Values
Specific gravity	2.05
Fineness – median particle size, µm	8.3
Nitrogen absorption, m^2/g	20.6
Water requirement, %	104
Pozzolanic activity index, %	99
Chemical Properties	
Silicon dioxide (SiO_2)	90.7
Aluminium oxide (Al_2O_3)	0.4

Ferric oxide (Fe_2O_3)	0.4
Calcium oxide (CaO)	0.4
Magnesium oxide (MgO)	0.5
Sodium oxide (Na_2O)	0.1
Potassium oxide (K_2O)	2.2
Equivalent alkali (Na_2O+0.658K_2O)	1.5
Phosphorous oxide (P_2O_5)	0.4
Titanium oxide (TiO_2)	0.03
Sulphur trioxide (SO_3)	0.1
Loss of ignition	4.8

3.3 Sample preparation and experimental program

Six different microsilica and RHA content, i.e.5%, 7%, 9%, 11%, 13% and 15% by weight of the expansive soil were used for preparing the samples. As from literature review, it was certain to use additive i.e. microsilica and RHA initiating from 5%.

The overall testing program is conducted in three phases. In first phase the geotechnical properties of the expansive soil samples were studied by conducting Sieve analysis, Hydrometer analysis, Standard proctor test, UCS test, CBR test and Direct Shear test. Details of percentage of RHA and microsilica are given in table 3.

Figure 6: DELKM microsilica bag

Figure 7: photo of RHA Sample

Table 3: Test program for stabilization of expansive soil using Microsilica and RHA

S.No	Test method	Complying standards	Sample variables taken individually		Parameter
			Microsilica	RHA	
1	Standard Proctor Test	IS:2720(Part-VII)-1987 and (Part VIII)-1987	5%,7%,9% 11%,13% 15%	5%,7%,9% 11%,13% 15%	OMC, MDD
2	UCS Test	IS:2720(Part-X)-1991	5%,7%,9% 11%,13% 15%	5%,7%,9% 11%,13% 15%	UCS
3	CBR Test	IS:2720(Part-XVI)-1987	5%,7%,9% 11%,13% 15%	5%,7%,9% 11%,13% 15%	CBR
4	Direct Shear Test	IS:2720(Part1)-(1983)	5%,7%,9% 11%,13% 15%	5%,7%,9% 11%,13% 15%	Shear strength, Cohesion, Angle of internal friction

3.4 Determination of particle size distribution

3.4.1 Sieve analysis (IS: 2720 (part iv), 1985)

The results from sieve analysis of the soil when plotted on a semi-log graph with particle diameter or the sieve size as the abscissa with logarithmic axis and the percentage passing as the ordinate gives a clear idea about the particle size distribution. From the help of this curve, D-10 and D-60 are determined. This D-10 is the diameter of the soil below which 10% of the soil properties lie. The ratio of D-10 and D-60 gives the uniformity coefficient (Cu) which in turn is a measure of the particle size range.

3.4.2 Hydrometer analysis (IS: 2720 (part iv), 1985)

The hydrometer analysis is used to determine the particle size distribution of the soil having size 0.075mm to 0.01mm .It means that hydrometer is used to find out the particle range of fine grained soil. The principal of hydrometer analysis is to obtain the clay fraction. Like sieve analysis similar data sheet is presented on semi-log graph and find out the particle size of the soil. Hydrometer analysis have identify the particle size<

0.02mm. This test is done when more than 20% of soil passes through 0.075mm sieve and 90% or more passes through 4.75mm sieve. The hydrometer analysis based on stokes's law. The formula for calculating the particle size is

Particle size $D = M*\sqrt{He/t}$

Where $M = 1.33*10^{-3}$

H_e = height of hydrometer

T = time elapsed in sec

3.5 Determination of index property of soil

3.5.1 Specific gravity IS: 2720 (Part 3/Sec 1), 1987

The specific gravity of the soil is the ratio between the weight of the soil solids and weight of equal volume of water. It is measured by the help of a volumetric flask in a very simple experimental setup where volume of the soil is found out and its weight is divided by the equal volume of water. The specific gravity is used to find out the degree of saturation and unit weight of moist soil. Ultimately the unit weight of soil is used to determine pressure, settlement and stability problem.

The specific gravity of the soil = $(m_2 - m_1) / [(m_4 - m_1) - (m_3 - m_2)]$

Where, M_1 = mass of density bottle in gm.

M_2= mass of bottle and dry soil in gm.

M_3 = mass of bottle, soil and water in gm.

M_4 = mass of bottle full of water only in gm.

Specific gravity is always measured at room temperature and reported to the nearest 0.1 decimal.

3.5.2 Liquid limit IS: 2720(Part V)(1985)

The Casargrande tool cuts a groove of size 2mm wide at the bottom and 11 mm at the top and 8mm high. The number of blows used for the two soil samples to come in contact is noted down. Graph is plotted taking number of blows on a logarithmic scale on the abscissa and water content on the ordinate. Liquid limit corresponds to 25 blows from the graph.

3.5.3 Plastic limit IS: 2720(Part V)(1985)

This is determined by rolling out soil till diameter reaches approximately 3 mm and measuring water content for the soil which crumbles on reaching this diameter.

3.5.4 Plasticity index: 2720(Part V)(1985)

The plasticity index is calculated from liquid limit and plastic limit of the soil and it is defined as the difference between the liquid limit and plastic limit of the soil.

$I_p = W_L - W_P$

W_L = Liquid limit , W_P = plastic limit

Table 4: Representation of group symbol in Plasticity chart: (IS:1498-1970)

S.no	Group symbol	Typical name
1	CL	Clay with low plasticity
2	CI	Clay with intermediate plasticity
3	CH	Clay with high plasticity
4	ML	Silt with low plasticity
5	OL	Organic silt
6	MI	Intermediate silt
7	OI	Organic silt
8	MH	Silt of high plasticity
9	OH	Organic clay

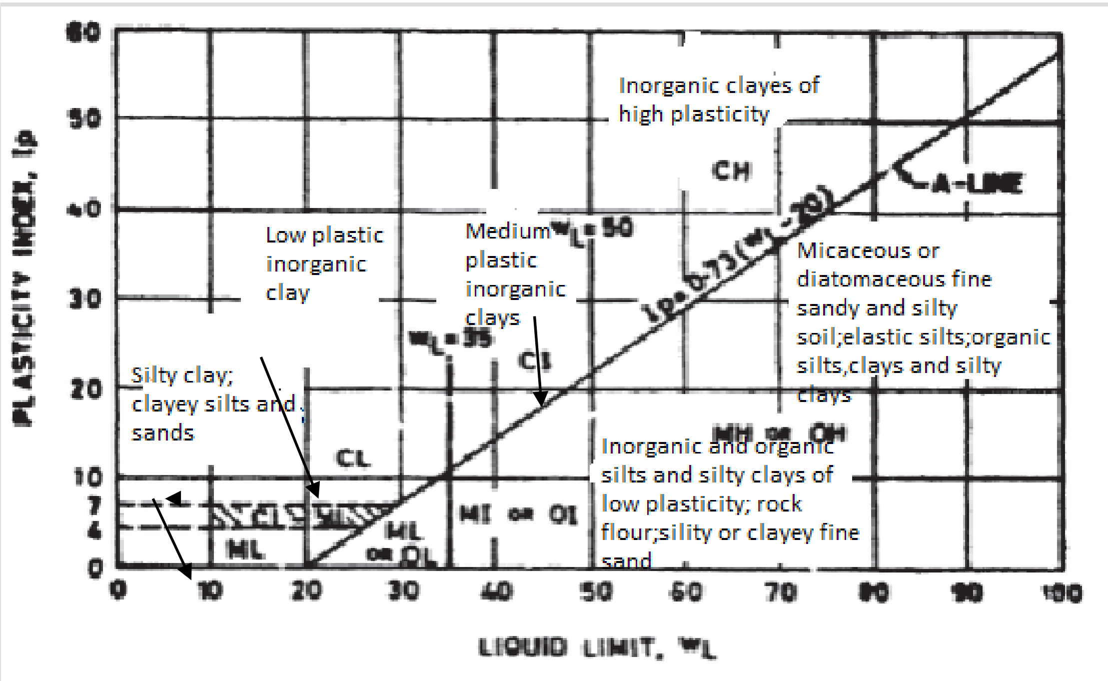

Figure 8: Plasticity chart (IS: 1498-1970)

3.5.5 Free swell index IS:2720 (part XL) (1977)

Free Swell Index is the increase in volume of a soil, without any external constraints, on submergence in water. Free swell index test is used to study the expansive behaviour of the soil. The determination of free swell index can be done by using following procedure. Take two specimens of oven-dry soil each weigh 10gm, passes through 425µm IS sieve. Pour each soil specimen into graduated cylinder of 1000ml capacity and one cylinder is filled with water and other is filled with kerosene up to 100ml. Remove the entrapped air by gently shaking with glass rod. Now allow it to state of equilibrium for 24hours. After 24 hours the final volume of soil in each sample read out. The free swell index can be found out by using following formula.

$$\text{Free swell index (\%)} = (V_d - V_k)/V_k \times 100$$

Where,

V_d= volume of soil specimen read from graduated cylinder containing distilled water.
V_k = volume of soil specimen read from the graduated cylinder containing kerosene.

Table 5: Free swell index V/s Degree of Expansiveness (IS: 2720 (part XL) (1977)

S.no	Free swell index	Degree of expansiveness
1	<20	Low
2	20-35	Moderate
3	35-50	High
4	>50	Very high

3.5.6 Energy Dispersive X-ray Spectrometry (EDS)

EDS uses the X-ray spectrum emitted by a sample is bombarded with a fixed focused beam of electrons localized chemical analysis. All elements of atomic number 4 (Ser) to 92 (U) can be detected in principle, but they are not equipped for all instruments elements "Light" (z <10). Qualitative analysis involves the identification of the lines in the spectrum and is very easy due to the simplicity of the spectra of X-ray quantitative analysis (determination of the concentrations of the elements) involves measuring line intensities for each element in the sample and for the same elements in the calibration standards known composition. By scanning the beam in a manner similar to a television display screen and the intensity of selected Line X-ray images of element distribution or "maps" can be produced. Also, that by Electrons collected sample reveal the surface topography or the average atomic number varies depending on the selected mode. The scanning electron microscope (SEM), which closely related to the probe of electrons, it is primarily designed for producing images of electrons but Also for mapping elements even point analysis as X-ray spectrometer, is added. There is considerable overlap in the functions of these instruments.

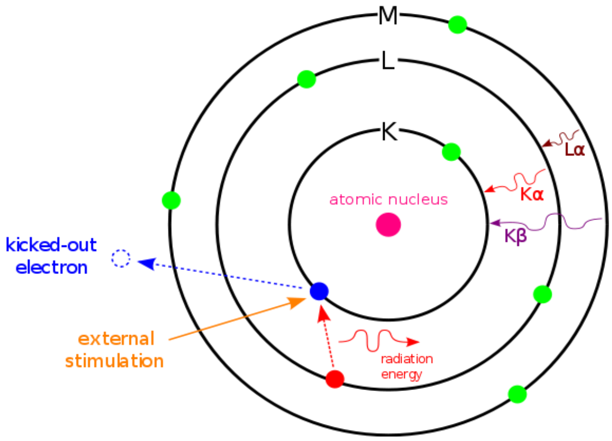

Figure 9: Principle of EDS (Wikipedia)

3.5.7 Scanning electron microscope (SEM)

The scanning electron microscope (SEM) is one of the most versatile instruments available for the examination and analysis of the microstructure, morphology, chemical composition characterizations and electrical conductivity of the soil. It is a type of electron microscope that images the surface of soil specimen by scanning it with high energy beam of electron, this high energy beam electron generate a variety of signals at the surface of soil specimen. The signals that drive electron-sample interactions reveal information about the sample including external texture, chemical composition, and crystalline structure and orientation of materials. Areas ranging from 1 cm to 5 microns in width can be imaged in a scanning mode, using conventional SEM techniques and the range of magnification of this technique from 20X to 30,000X and the range of spatial resolution from 50 to 100 nm. The SEM is also used to analyses the selected point, locations on the sample and this approach is especially useful in studying qualitatively or semi-quantitatively determining chemical compositions (using EDS), crystalline structure, and crystal orientations (using EBSD).

Secondary electron and back scattered electron is the types of signals that is produced by Scanning electron microscope. Basically we have use the primary signals for scanning the topography of the structure for the visualization of surface texture and roughness, but in the inclusion of secondary signals it contrast the images of the topography, According to which we can visualize the texture , roughness in large scale. The topographical image is dependent on how many of the secondary electrons actually reach the detector. A secondary electron signal can resolve surface structures down to the order of 10 nm or better. Although an equivalent number of secondary electrons might be produced as a result of the specimen primary beam interaction, only those that can reach the detector will contribute to the ultimate image of soil specimen. Back scanning electron is another method to produce image. A BSE is defined as one which has undergo a single or multiple scattering events with energy greater than 50eV. The elastic collision between the electron and specimen bounce back the electron with great energy. About 10-50% of beam electron scattered back and 60-80% retained. So these backscattered beams contrast the image of specimen. But According to the study of the various researchers they find out that secondary electron is give better result as compared to the back scattering electron signals in SEM. SEM method is very useful when the soil specimen in bulk.

3.6 Determination of engineering property of the soil

3.6.1 Proctor compaction test: IS: 10074(1982)

The standard compaction test gives the relationship between dry density and the moisture content. Compaction is one kind of densification that is preceded by rearrangement of expansive soil particles with content at different percentage without outflow of water. The Experimental set up is consist of .1) Cylindrical mould having internal diameter 101.6mm and internal height 116.43mm. 2) Detachable base plate 3) Collar having diameter of 114.3mm. 4) The weight of the rammer is 2.5kg, numbers of blow at each three layer is 25 and the height is 304.8mm. As the water content increases the bulk density of the soil increases because compaction effort removes all the air from the compacted soil. But at particular water content it will fall, at that water content we will stop the experiment. Graph is plotted between water content (abscissa) and dry density (ordinate). According to graph we will find out the maximum dry density at optimum moisture content. This equation is used in the following experiment.

$$\text{Bulk density} = \frac{\text{mass of compacted soil(gm)}}{\text{volume of mould(cc)}}$$

$$\text{Moisture content} = \frac{\text{mass of water(gm)}}{\text{mass of soil(gm)}} \text{X } 100$$

$$\text{Dry density} = \frac{\text{Bulk density}}{1 + \frac{\text{moisture content}}{100}}$$

3.6.2 Unconfined compressive test IS: 2720 (Part 10)(1987)

This experiment is used to determine the unconfined compressive strength of soil/composites samples of soil blended with microsilica and rice husk ash which in turn is used to calculate the unconsolidated, undrained shear strength of unconfined soil/soil samples. The unconfined compressive strength (qu) is the compressive stress at which the unconfined cylindrical composite sample fails under simple compressive test. The experimental setup constitutes of the compression device and dial gauges for load and deformation. The load was taken for different reading of strain dial gauge taken. The corrected cross-sectional area was calculated by dividing the area by (1- ε) and then the compressive stress for each step was calculated by dividing the load with the corrected area.

$$\text{Unconfined compressive strength(qu)} = \frac{\text{Unconfined load(KN)}}{\text{Corrected area(m2)}}$$

Where,

Corrected area=cross-sectional area / (1-ε).

$$\varepsilon = \frac{\text{change in length}(\nabla \text{l})}{\text{orignal length(l)}}$$

3.6.3 California bearing ratio test (Unsoaked) IS: 2720 (Part 16), 1987

Using the moisture content and corresponding dry density the amount of soil/samples of soil blended with microsilica and rice husk ash used for CBR was calculated. The sample was tested using the CBR instruments and the samples are Unsoaked for which CBR values was found out.

The California bearing ratio test is a measure of a material to penetration of standard plunger under controlled density and moisture conditions. The test procedure should be strictly adhered if high degree of productivity is required. The CBR test may be conducted in remoulded or undisturbed specimen in the laboratory. US corps of engineers have also recommended a test procedure for in situ test. B penetration test meant for the evaluation of sub-grade strength of roads and pavements. The results obtained by these tests are used with the empirical curves to determine the thickness of pavement and its component layers. This is the most widely used method for the designing of flexible pavement.

The experimental set up consist of .1) Cylindrical mould with inside diameter 150 mm and height 175 mm, provided with a detachable extension collar 50 mm height and a detachable perforated base plate 10 mm thick. 2) Spacer disc 148 mm in diameter and 47.7 mm in height along with handle. 3) Weight of hammer is 2.6 kg with a drop of 310 mm.4) The annular surcharge weight of metal is 2.5 kg each, 147 mm in diameter, with a central hole 53 mm in diameter. The test consists of cylindrical plunger 50mm in diameter and 100 mm in length is penetrating in mould at the rate of 1.25mm/min. The load is recorded corresponding to 2.5mm and 5.0mm. Now by drawing the load penetration curve find the value of load at 2.5mm and 5.0mm and recorded higher value of both penetration. Generally 2.5mm give higher value.

Calibration factor of the proving ring 1 Div. = 4.518 kg

Surcharge weight used (kg) = 2.750 kg

Least count of penetration dial 1 Div. = 0.01 mm

$$\mathrm{CBR}(\%) = \frac{\text{Test load(N)}}{\text{Standard load(N)}} \mathrm{X}\ 100$$

D= diameter of sample (meter)

L= length of sample (meter)

Table 6: Standard load for CBR test

S.no	Penetration of plunger(mm)	Standard load (Kg)
1	2.5	1370
2	5.0	2055
3	7.5	2630
4	10	3180
5	12.5	3600

3.6.4 Direct shear test IS:2720 (Part 11)(1983):

In many engineering problems such as design of foundation, retaining walls, slab bridges, pipes, sheet piling, the value of the angle of internal friction and cohesion of the soil play an important role. Direct shear test is used to predict these parameters quickly. The test can be carried out at different moisture contents, before running the test, the sample must be saturated with water. To find the reliable results, the test is often carried out on three or four samples of undisturbed soil. The soil sample is placed in a cubic shear box having dimensions 6cm X 6cmX 3cm composed of a upper and lower box. The limit between the two parts of the box is approximately at the mid height of the sample. The sample is subjected to a controlled normal stress and the upper part of the sample is pulled laterally at a controlled strain rate or until the sample fails. The applied lateral load and the induced strain are recorded at given internals. These induced strains are used to find out the corrected area. Now divide the corrected area to the applied load will give shearing stress. Plot the graph between shearing stress and horizontal shear displacement. Record the maximum value of shearing stress at each value of normal stress. Plot the graph between shearing stress (ordinate) and normal stress (abscissa). A linear curve fitting is often made on the test result points. The intercept of this line with the vertical axis gives the cohesion and its slope gives the peak friction angle. The formula used to find out the value of shearing stress is:

$$\text{Shearing stress(KN/m2)} = \frac{\text{Shearing load(KN)}}{\text{Corrected area(m2)}}$$

Chapter 4
Result and Discussion

4.1 Physical property of the expansive soil

4.1.1 Grain size distribution 2720(part IV), 1985

An Extensive investigation carried on Expansive soil. Based on sieve analysis, it was found that more than 50% of soil passes through 0.075mm sieve and it can be classified as fine grained soil. I also found that the liquid limit of the soil is greater than 50%. Thus the Soil categorized above can be either MH or CH. Plasticity chart also helps to classify the soil. According to this, it was found that the plasticity index of the soil is 28.55% and it can be safely classified as CH i.e. Clay of high plasticity.

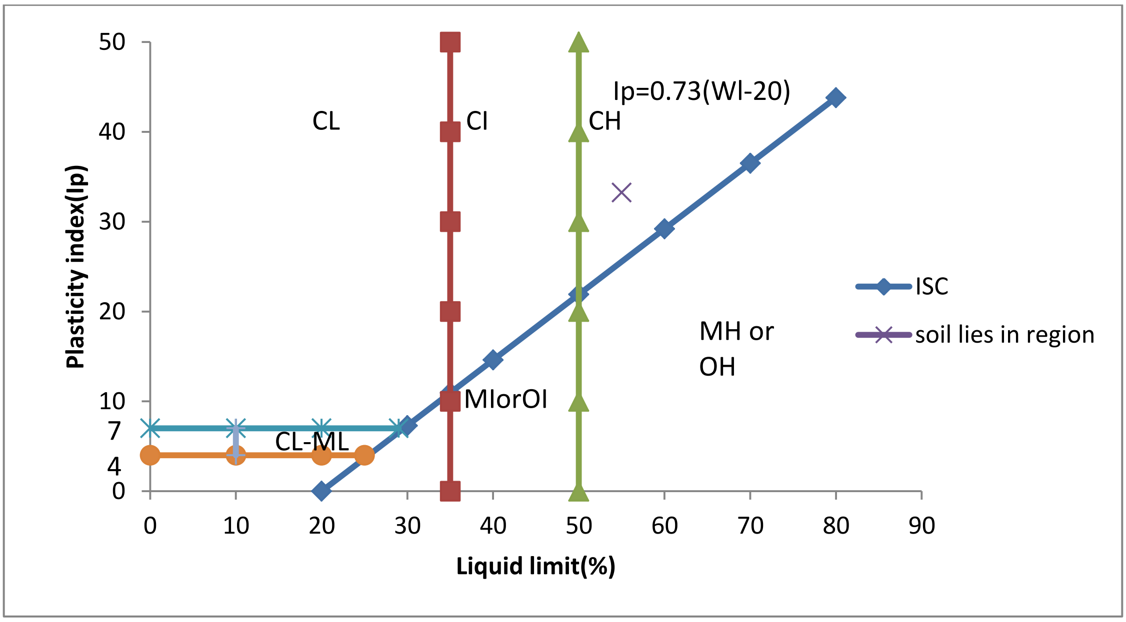

Figure 10: Plasticity chart

Based on the particle size distribution, the soil is graded in to different category that is , Percent Gravel(>4.75mm)=100-100=0%,Percent Coarse sand(4.75-2mm)=100-100=0% , Percent medium sand(2-0.425mm) =100-97= 3% , Percent of fine sand (0.425-0.075mm) =97- 91=6%, Percent of silt/clay(<0.075mm)=91%

As we know that if the percentage of the silt and clay fraction is greater than 12 percentage, then it is required for us to determine the fine grain fraction size to assess the percentage of the clay particles and silt particles which can be determined by hydrometer analysis.

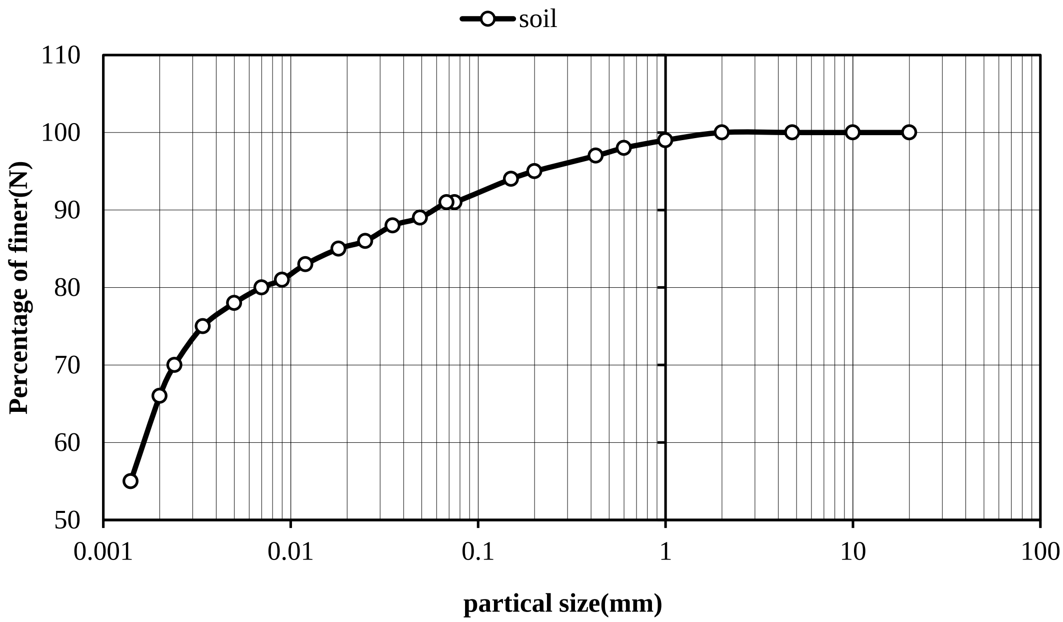

Figure 11: Hydrometer analysis of soil

Based on the hydrometer analysis, I graded the fine grained soil in different category that is, Percent coarse silt (0.063-0.02mm) =3%, Percent medium silt(0.02-0.0063mm) =8%,Fine silt(0.063-0.002)=13%, Clay(<0.002)=55%. According to hydrometer analysis it was found that expansive soil is clay with some percentage of silt fractions.

4.1.2 Specific gravity IS: 2720(Part1V), 1987

The Specific gravity of Expansive soil is determined by IS: 2720 (Part 1V section 1 (1985) and it is found to be 2.68. For clayey and silty soil the range of specific gravity is 2.67-2.9.The application of specific gravity in geotechnical engineering is used to study the types of soil and to calculate the void ratio and degree of saturation. The specific gravity of the expansive soil is very high, due to this it has more pores and have more void ratio. Due to the high void ratio it cannot control the water through it and failure of structure occurs. If we use additives in soil it fills the void and decreases the void ratio and specific gravity which increases the strength of soil. Due to such stabilization of soil there comes effect in specific gravity, so we can use this soil in embankment, slope and retaining wall.

4.1.3 Liquid limit IS: 2720 (Part V), 1985

According to the experimental result it is found that the liquid limit of the soil is 52% which is greater than 50%, so soil is classified as Expansive soil. The range of liquid limit for Expansive soil is 50-100%.

Liquid limit is an important physical property. The application of liquid limit in geotechnical engineering is used to classify soil, correlate various soil properties with strength, estimate swelling potential of soil, and hundreds of similar uses in geotechnical engineering.

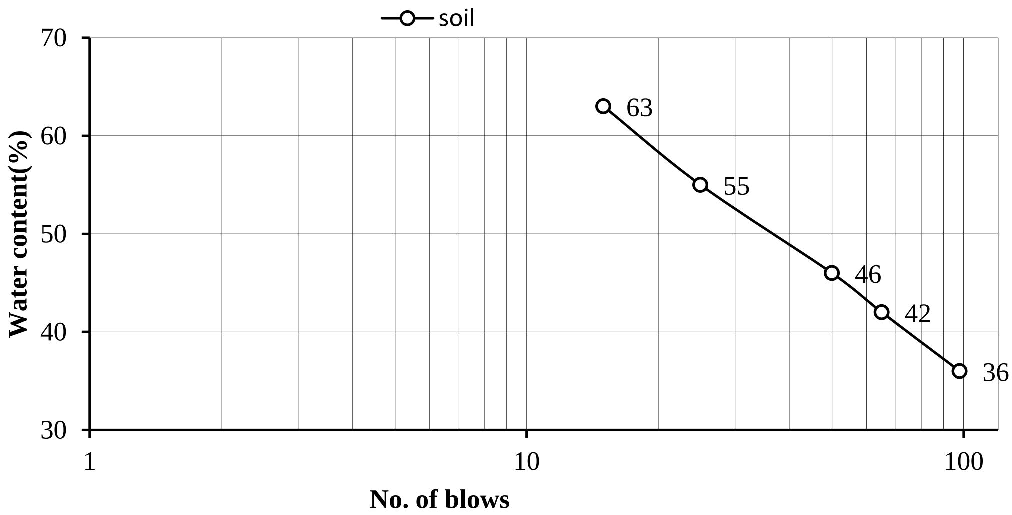

Figure 12: Liquid limit of soil

4.1.4 Plastic limit IS: 2720(Part V), 1985

Plastic limit of the soil is determined by IS specification. According to the experimental result it was found that the plastic limit of the soil is 23.45%. The range of plastic limit for expansive soil is 20-65%.

The application of plastic limit in the geotechnical engineering is to find out the plasticity index of soil.

4.1.5 Plasticity index: 2720(Part V), 1985

The plasticity index is defined as the difference between the liquid limit and plastic limit of the soil and it was found that the plasticity index of Expansive soil is 28.55% which is very high.

The application of plasticity index in geotechnical engineering is used to classify the soil which lies either above A-line or below A-line by using plasticity chart. A plasticity index lower than 20 to 24 was generally a safe area but if it is higher than this value respond to swelling behavior of clay.

4.1.6 Free swell index IS: 2720 (part XL) , 1977

The free swell index is determined by IS specification. According to the experimental result it was found that free swell index of the soil is 51% which is greater than 50%. If we study the expansive range of soil then I found that this soil is very expansive in nature.

The application of free swell index in geotechnical engineering is to determine the swelling behavior of soil because most of the structure like road, dam, embankment, canals fails due to the swelling and shrinkage behavior of soil. So the study of swelling behavior of the soil before constructing such type of structure is very important.

4.1.7 Energy Dispersive X-ray Spectroscopy (EDX)

4.1.7.1 EDX of soil sample

The EDX shows the peaks which are caused by X-rays given off as electrons which return to the K- electron shell and one peak resulting from the L-shell of the mineral. Here I have done EDX of soil, microsilica and RHA which is shown in figure 13, figure 14 and figure 15. The peak in every figure shows the highest number of compounds which are present in a mineral.

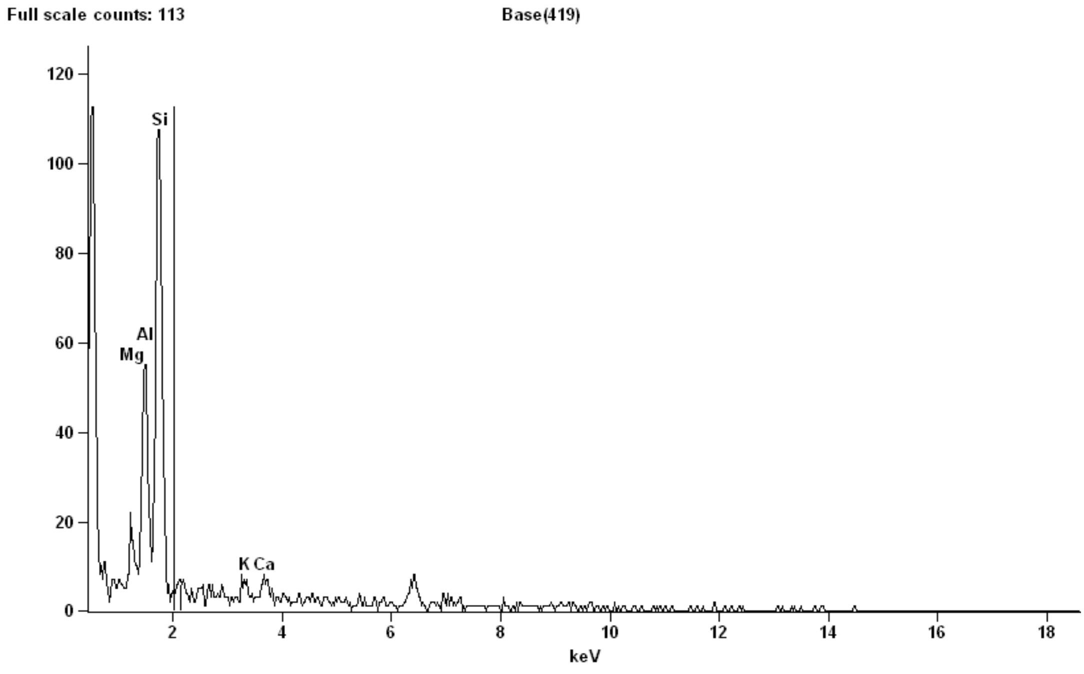

Figure13: Electronic Dispersive Spectrum of Soil

Table.7: Mineralogical Characteristics of Soil

Element Line	Net Counts	Int. Cps/nA	Weight %	Weight % Error	Atom %	Atom % Error	Formula
Mg K	88	---	3.91	+/- 0.62	4.61	+/- 0.73	Mg
Al K	450	---	20.31	+/- 1.76	21.53	+/- 1.87	Al
Si K	1171	---	64.66	+/- 2.65	65.85	+/- 2.70	Si
Si L	0	---	---	---	---	---	
K K	49	---	4.74	+/- 0.97	3.47	+/- 0.71	K
K L	0	---	---	---	---	---	
Ca K	59	---	6.38	+/- 1.19	14.55	+/- 0.85	Ca
Ca L	0	---	---	---	---	---	
Total			100.00		100.00		

4.1.7.2 EDX of microsilica

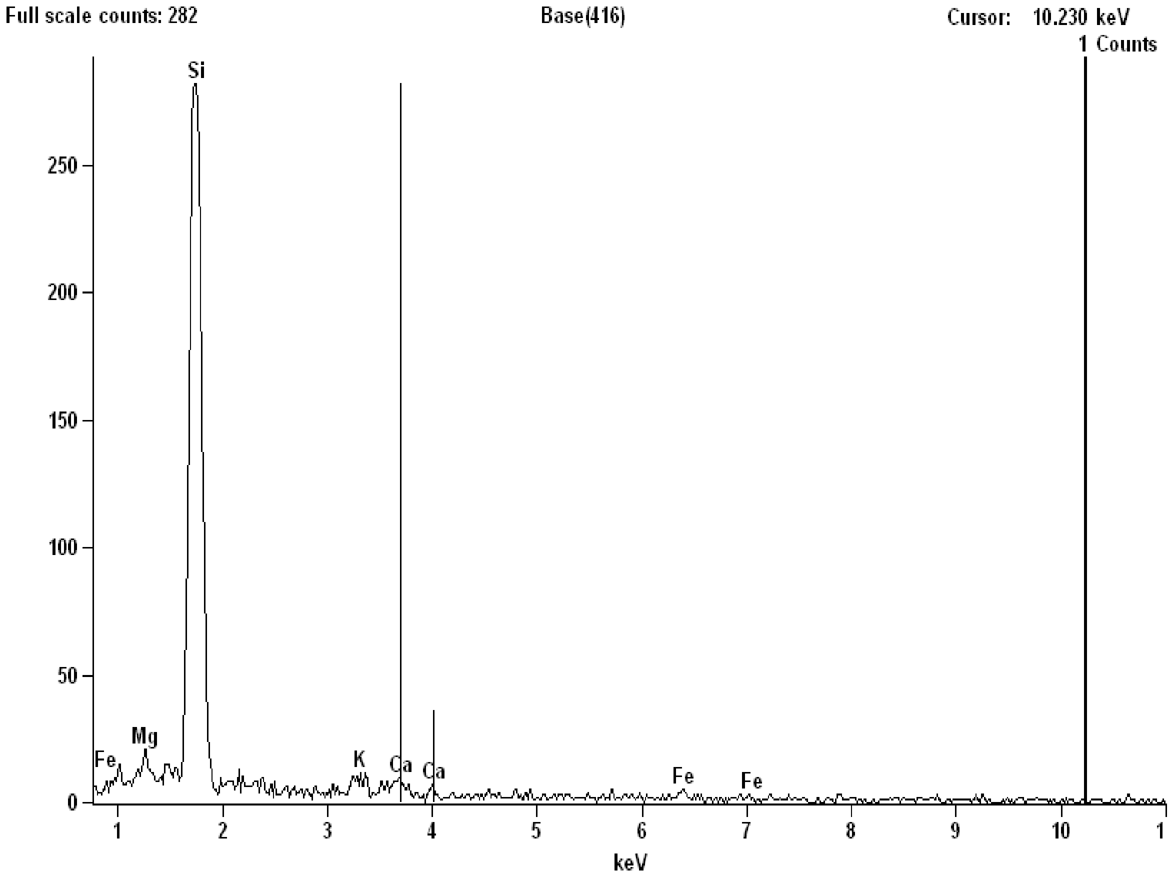

Figure 14: Electronic Dispersive Spectrum of Microsilica

Table.8: Mineralogical Characteristics of Microsilica

Element Line	Net Counts	Int. Cps/nA	Weight %	Weight % Error	Atom %	Atom % Error	Formula
Mg K	91	---	2.08	+/- 0.32	1.59	+/- 0.39	Mg
Si K	3518	---	85.91	+/- 1.66	95.23	+/- 1.74	Si
Si L	0	---	---	---	---	---	
K K	86	---	4.12	+/- 0.67	1.10	+/- 0.50	K
K L	0	---	---	---	---	---	
Ca K	27	---	1.46	+/- 0.65	0.70	+/- 0.48	Ca
Ca L	0	---	---	---	---	---	
Fe K	40	---	6.43	+/- 2.09	1.38	+/- 1.10	Fe
Fe L	0	---	---	---	---	---	
Total			100.00		100.00		

4.1.7.3 EDX of RHA

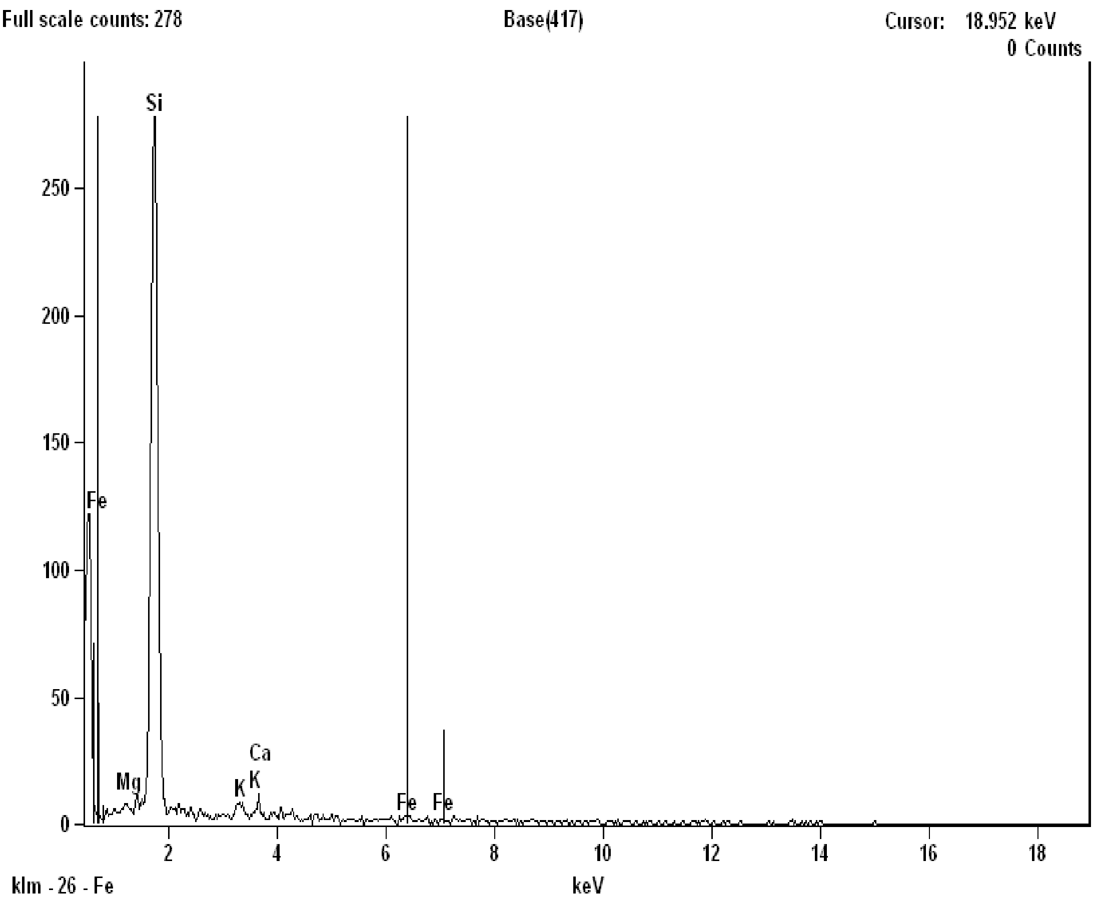

Figure 15: Electronic Dispersive Spectrum of RHA

Table.9: Mineralogical Characteristics of RHA

Element Line	*Net Counts*	*Int. Cps/nA*	*Weight %*	*Weight % Error*	*Atom %*	*Atom % Error*	*Formula*
Mg K	13	---	0.34	+/- 0.26	1.41	+/- 0.31	Mg
Si K	3312	---	91.08	+/- 1.84	94.00	+/- 1.90	Si
Si L	0	---	---	---	---	---	
K K	65	---	3.57	+/- 0.66	2.65	+/- 0.49	K
K L	0	---	---	---	---	---	
Ca K	28	---	1.70	+/- 0.67	0.23	+/- 0.48	Ca
Ca L	0	---	---	---	---	---	
Fe K	18	---	3.31	+/- 1.65	1.72	+/- 0.86	Fe
Fe L	0	---	---	---	---	---	
Total			100.00		100.00		

4.1.8 Scanning electron microscope (SEM)

Scanning electron microscope gives the information about the morphology of the sample. In this method a high electron beam is concentrated on the sample to provide the precision of the sample. In this report SEM for three samples i.e. soil, microsilica and RHA and their results have been shown in figure 16, figure 17 and figure 18.

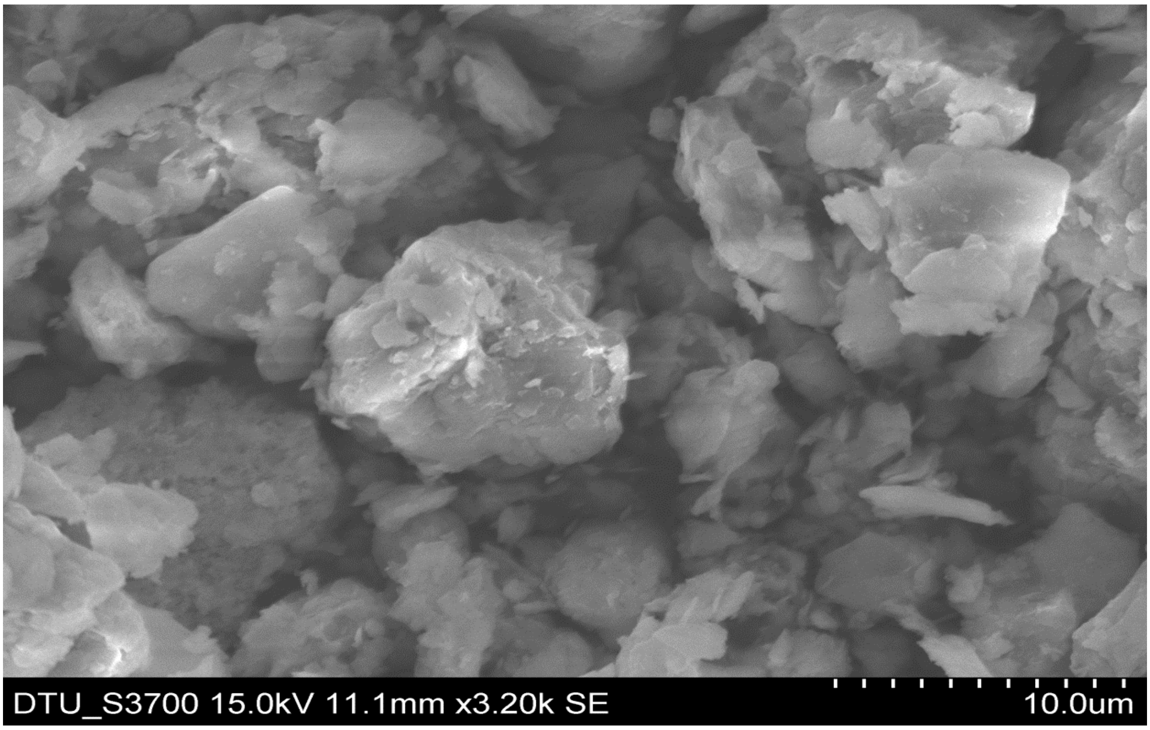

Figure 16: SEM for Soil at 10.0um Scale

These SEM results are important considering the filling of voids by microsilica and RHA in the soil particles thus giving good interlocking.

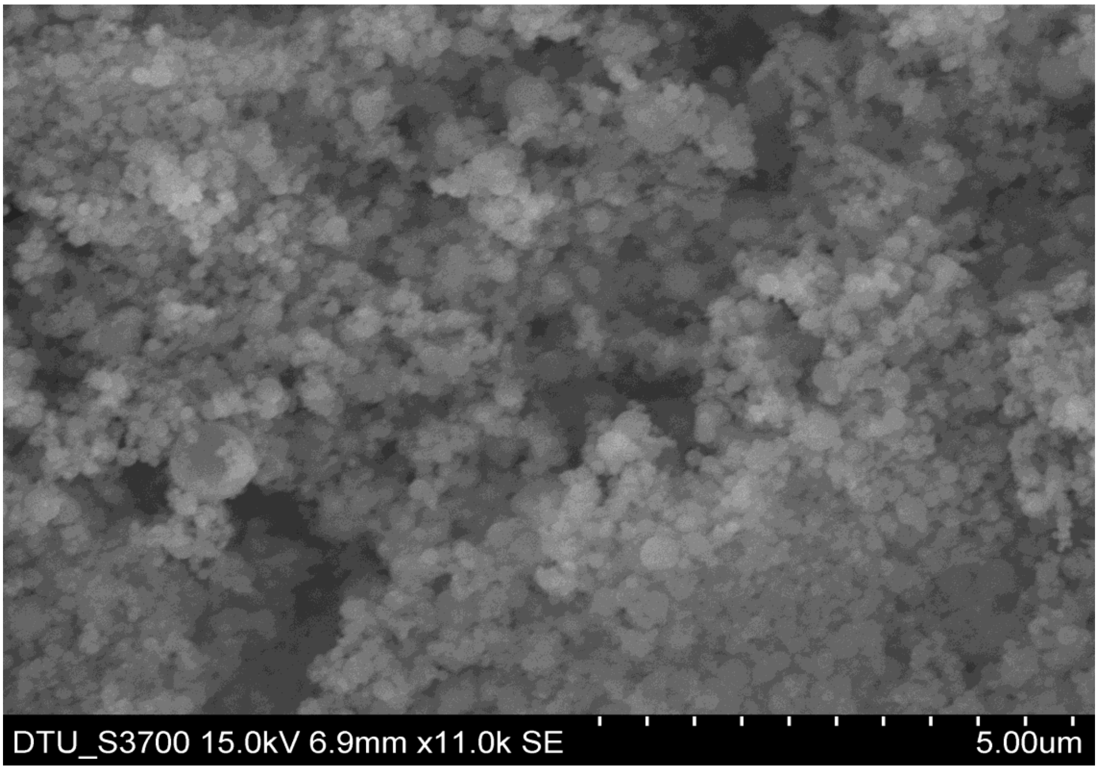

Figure 17: SEM for Microsilica at 5.0um Scale

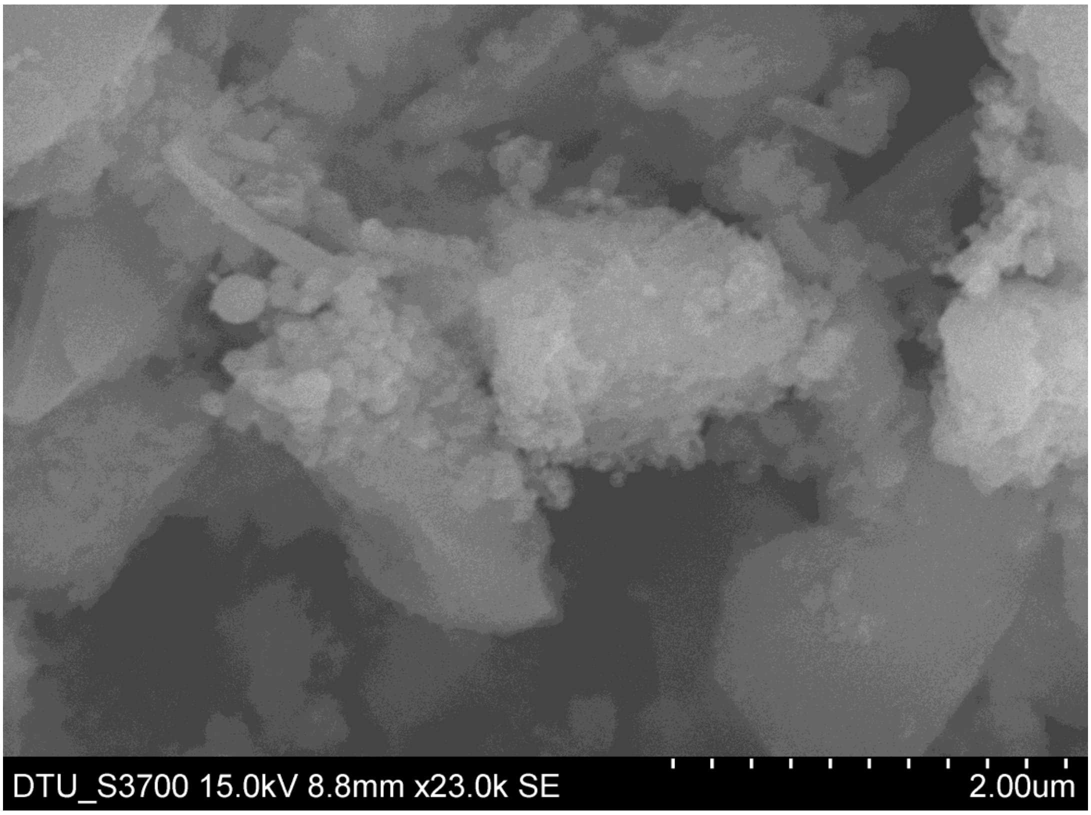

Figure 18: SEM for RHA at 2.0um Scale

4.1.9 Proctor compaction test: IS: 10074, 1982

Sample weight = 2.5Kg

Mass of mold + base plate (W) = 4286gm

Volume of mold (V) =1000cc

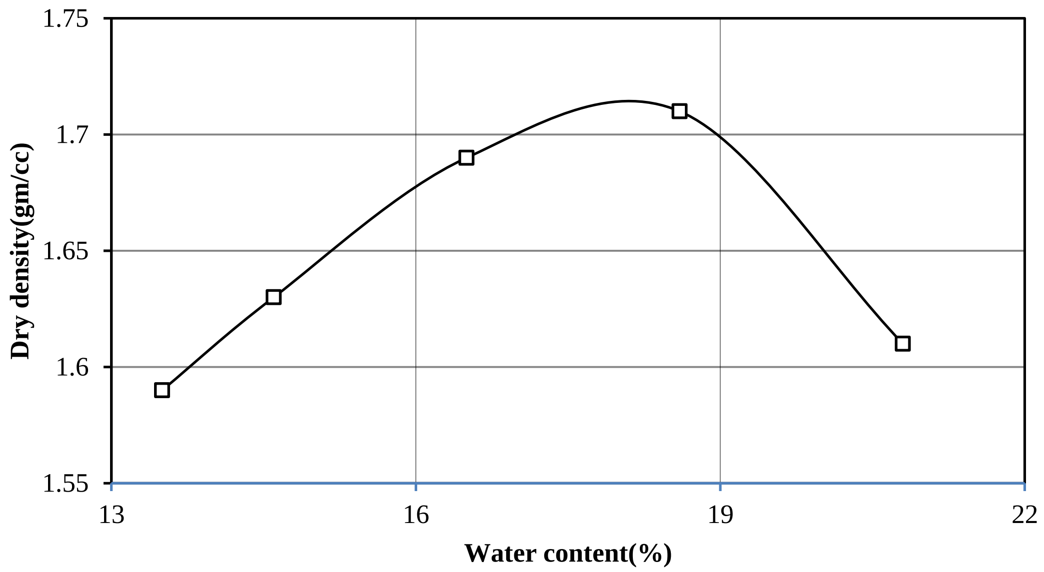

Figure 19: Variation of Dry Density with Water Content of soil

According to figure 19, I found that the Maximum dry density of soil 1.71gm/cc at optimum moisture content of 18%.

The application of proctor compaction test in soil engineering used to determine the optimal moisture content at which a given soil type will become most dense and achieve its maximum dry density. Too much water will make the soil squishy and temporarily reduce bearing capacity and too dry soil won't make the bond thoroughly. So we have to take the optimum value at which the soil becomes compact at MDD.

The application of Optimum moisture content and maximum dry density is used to study the various Experiments like unconfined compressive strength, indirect tensile test, CBR, Direct shear test.

4.1.10 Unconfined compressive test IS: 2720(Part 10)(1987):

Optimum water content =19% Mass of soil = 500gm

Dry density = 1.8gm/cc Length of specimen = 115mm

Diameter of specimen = 50mm

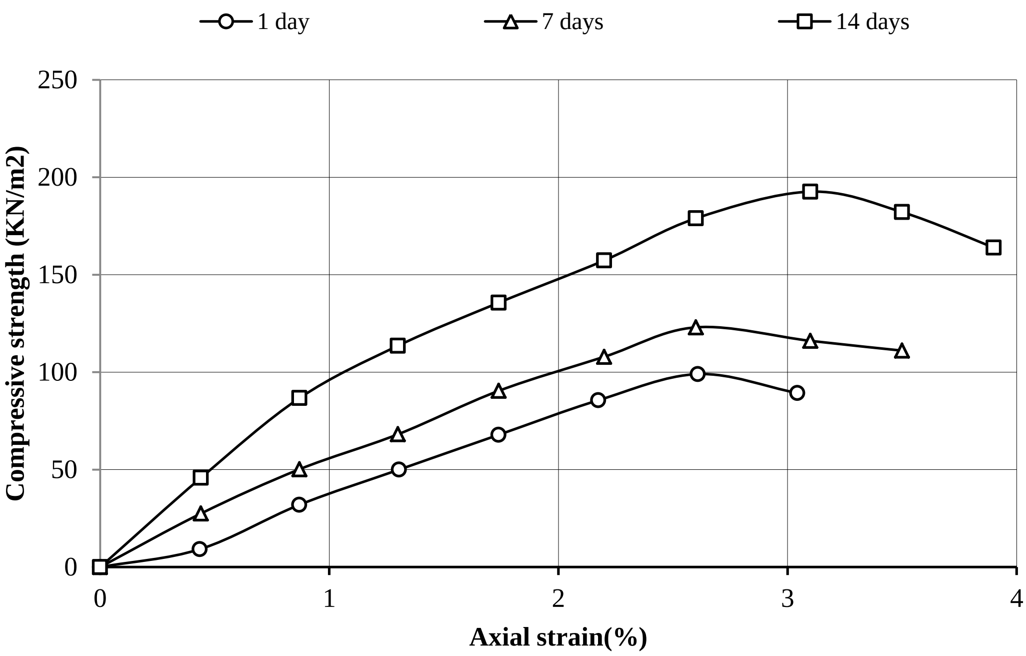

Figure 20: Variation of compressive strength with axial strain of soil at different days of curing

According to figure 20, it is observed that the Unconfined Compressive Strength of the soil is different for different curing period. For one day, seven days and fourteen days curing period the Unconfined Compressive Strength of the soil is 99KN/m^2, 124KN/m^2 and 192.62KN/m^2, also the undrained shear strength of the soil is 49.5KN/m^2,62KN/m^2 and 96.31KN/m^2.

From above result it is concluded that as the curing period of the unconfined compressive strength sample increases the Unconfined Compressive Strength and undrained shear strength of the soil increases. The significance of this test is that it quickly finds out the shear strength of the cohesive soil. The practical application of Unconfined Compressive Strength test is used to obtain the rough estimates of soil strength for the design and stability analysis of foundation, retaining wall, slope sand embankments.

4.1.11 California bearing ratio test (Unsoaked condition) IS: 2720 (Part 1), 1987

Optimum water content=18%

Mass of mould=7950gm

Mass of soil=500gm

Dry density=1.71gm/cc

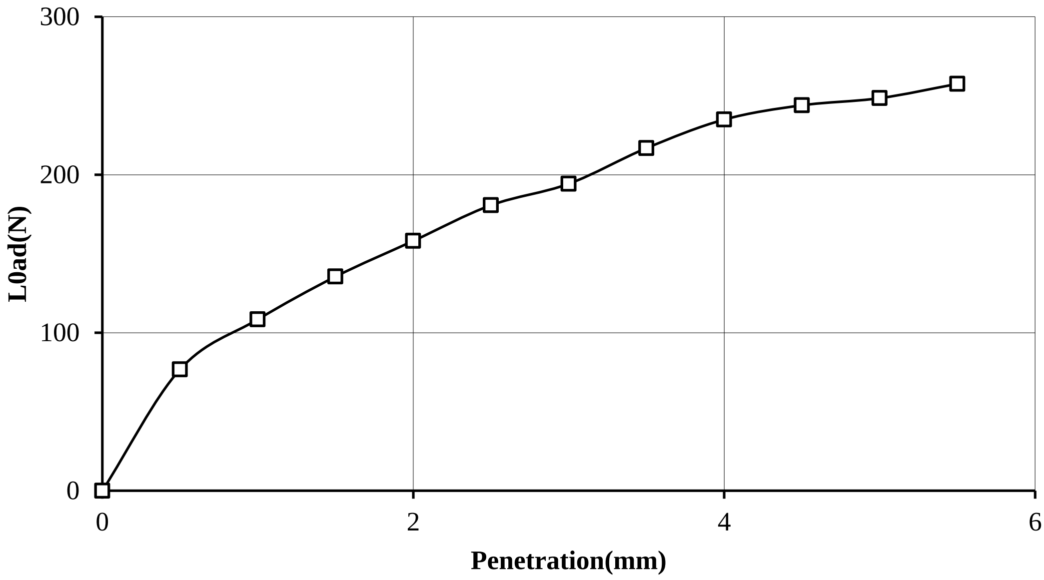

Figure 21: Variation of Load with Penetration of soil

From the Load-Penetration curve in figure 21, it is observed that the CBR value of soil at 2.5mm of penetration is 1.31% and for 5mm of penetration the value came 1.20%. So the higher value of CBR is adopted. Generally the CBR value of 2.5mm of penetration is higher than 5mm of penetration.

4.1.12 Direct shear test IS: 2720 (Part 11), 1983

Size of box=6cm*6cm*3cm

Area of box=36cm2

Mass of box + base plate + porous stones +grid plate=3250gm

Mass of box +base plate + porous stones +grid plate +soil specimen=3500gm

Normal stress=50KN/m2, 100KN/m2, 150KN/m2

Table 10: Shearing stress Vs Normal stress of soil

S.no	Normal stress(KN/m)	Shearing stress(KN/m)
1	50	56.04
2	100	67.36
3	150	78.36

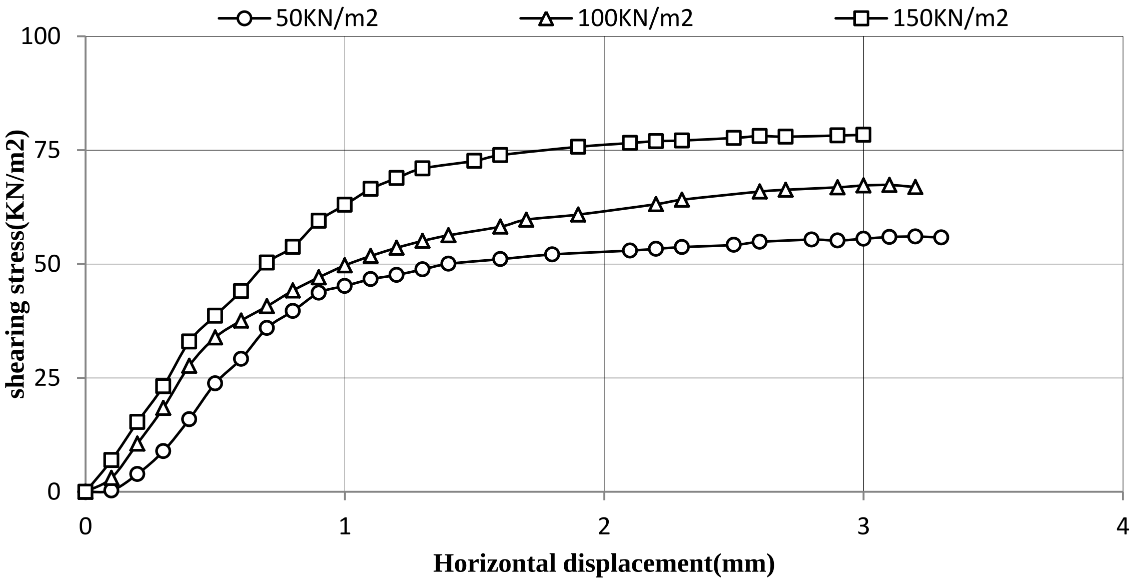

Figure 22: Variation of Shearing stress with horizontal displacement of soil at different normal stresses

From the figure 22, it is observed that as the normal stress increases, the shearing stress of the soil is also increased. This test is conducted at three different normal stresses that are 50KN/m^2, 100KN/m^2 and 150KN/m^2 and it was found that shearing stress corresponding to normal stress is 56.04KN/m^2, 67.36KN/m^2,78.36KN/m^2. Then the graph is plotted between normal stress and shear stress to calculate the cohesion and angle of internal friction.

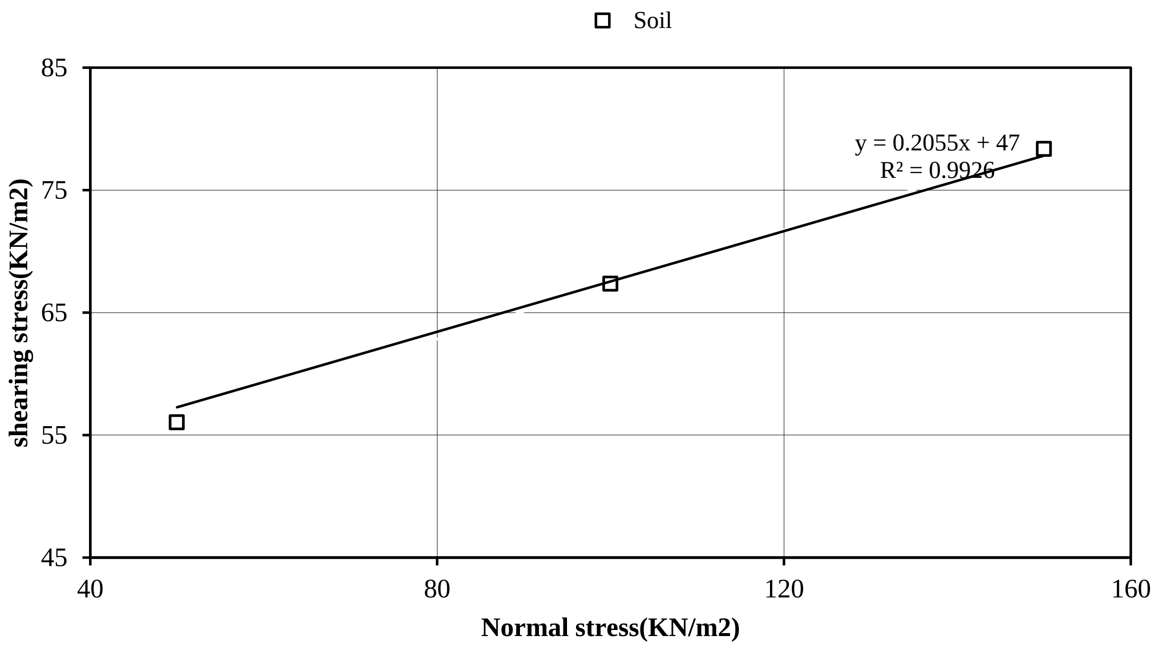

Figure 23: Variation of Shearing stress with Normal stress of soil

From the figure 23, it is found that, y=0.205x+47 is the equation of line. Where 0.205 is the slope and 47 is the intercept of line. The intercept shows the cohesion of the soil that is C=47KN/m. and the angle of internal friction is 11.58 degree. From the values investigated it can be seen that shear strength of the soil is less and should be enhanced.

The practical application of the direct shear test is used to study the shear strength of soil on which structure like embankment, dams, canal is constructed. Shear strength is an important parameter. If the shear strength of the soil is less, then the structure will fail due to slippage condition. So for designing the embankment, retaining wall it is important to carry out the direct shear test so that we can find the shear strength of the soil.

4.2 Soil blended with microsilica

4.2.1. Proctor compaction test: IS: 10074(1982)

Sample weight=2.5Kg

Mass of mold +base plate (W) =4286gm

Volume of mold (V) =1000cc

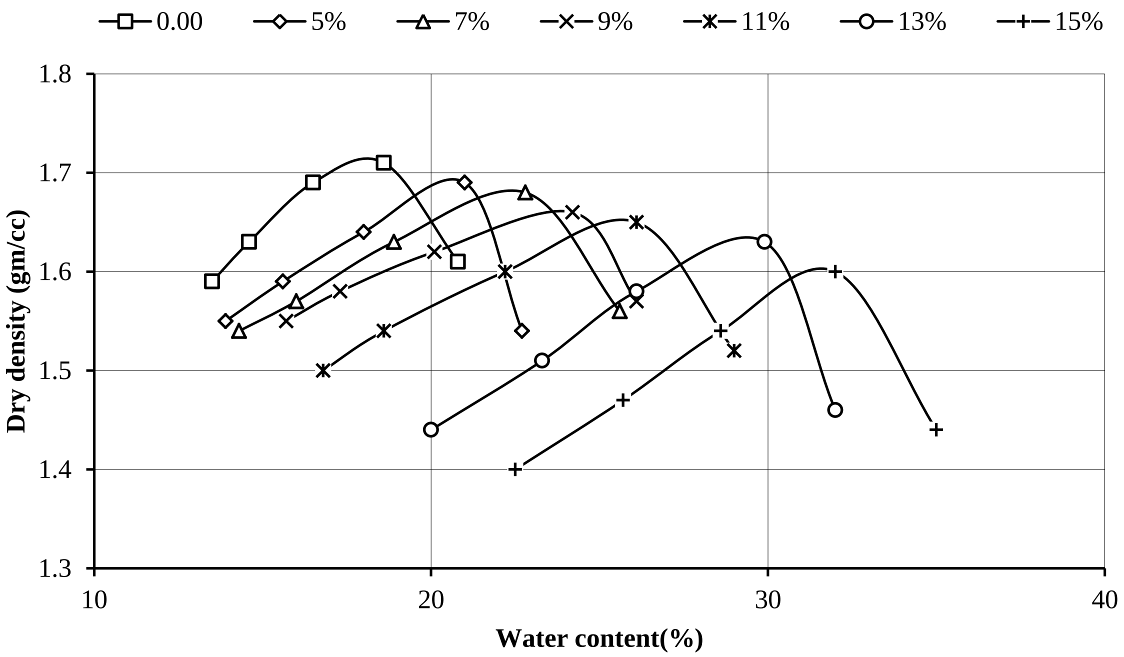

Figure 24: Variation of dry density with water content of soil blended with different percentage of microsilica

From the figure 24, it is observed that as the microsilica content increases from 5% to 15% with an increment of 2% initially starting from 5% it is seen that the optimum moisture content increases while on the other hand the dry density decreases.

The reason for this increase in the optimum moisture content can be attributed to the change in the surface area of the composite samples. The silica fume is seen to change the particle size distribution and the surface area of the soil stabilized samples made. In the same way, the reason for decrease in the maximum dry density is attributed to the addition of higher amount of silica fume with low density, which tends to sill the voids of composite samples. The increment in optimum moisture content and decrement in maximum dry density along with microsilica is given below in table:

Table 11: Variation of MDD and OMC with different percentage of microsilica content

S.no	% of Microsilica content	Maximum dry density(gm/cc)	Optimum moisture content (%)
1	0	1.71	18.1
2	5	1.69	21
3	7	1.68	22.8
4	9	1.66	24.2
5	11	1.65	26.1
6	13	1.63	29.9
7	15	1.6	32

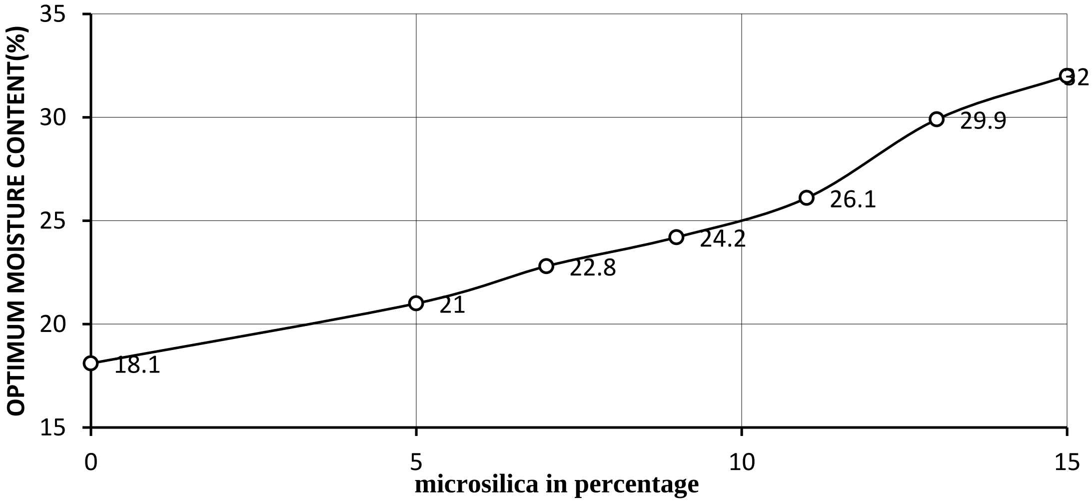

Figure 25: Variation of optimum moisture content with microsilica in different percentage

Table 11 shows the results of OMC and MDD, when expansive soil reinforced is blended with microsilica. Comparing the result with untreated soil it is observed that OMC increases by 18.1%, 21%, 22.8%, 24.2%, 26.1%, 29.9% and 32% and MDD decreases by 1.71gm/cc, 1.69gm/cc, 1.68gm/cc, 1.66gm/cc, 1.65gm/cc, 1.63gm/cc and 1.6gm/cc at microsilica content of 5%, 7%, 9%, 11%, 13% and 15%.

Figure 26: Sample prepared in standard proctor test

It is seen from the figure 25 that optimum moisture content is increasing linearly with the increase in microsilica content, maximum at 15%.

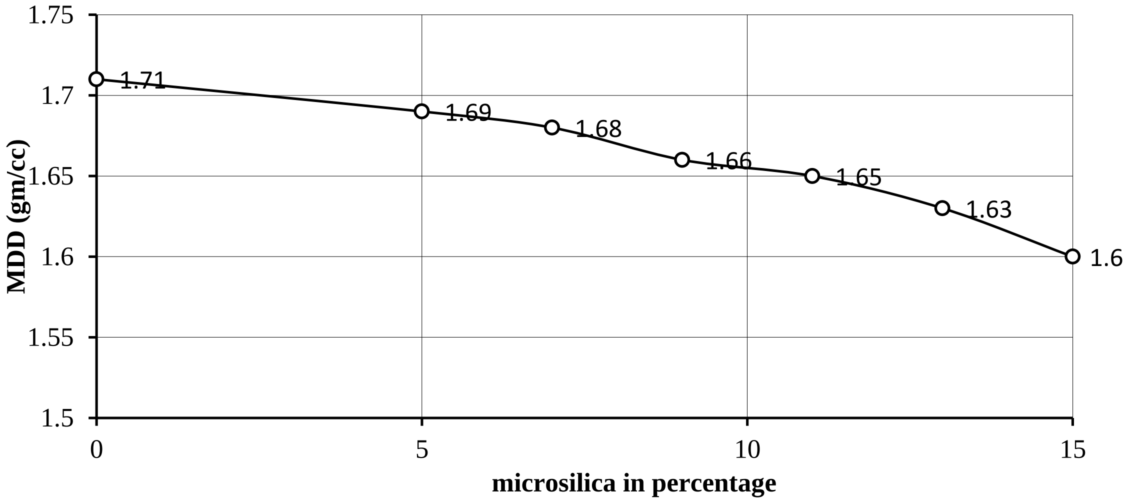

Figure 27: Variation of maximum dry density with microsilica in different percentage

The figure 27, shows that maximum dry density with increase in microsilica content up to 15% is decreasing.

4.2.2. Unconfined compressive test IS: 2720 (Part 10), 1987

Figure 28: UCS samples Figure 29: Installation of UCS sample Figure 30: Failure of UCS sample

Optimum water content at different Percentage of microsilica content

Dry density at different percentage of microsilica content

Mass of soil+ microsilica=500gm

Length of specimen=115mm

Diameter of specimen =50mm

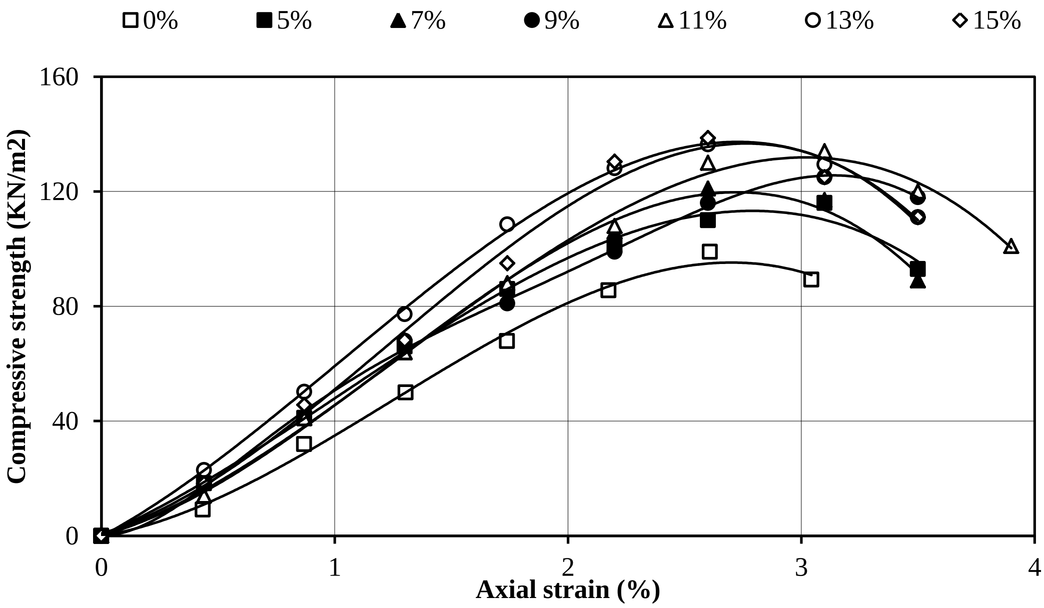

Figure 31: Variation of compressive strength with axial strain of soil blended with different percentage of microsilica at one day curing

From the figure 31, it is seen that with the increase in microsilica content the Unconfined Compressive Strength of the composite sample increased linearly up to 11% after that the Unconfined Compressive Strength slightly changes with the addition of microsilica.

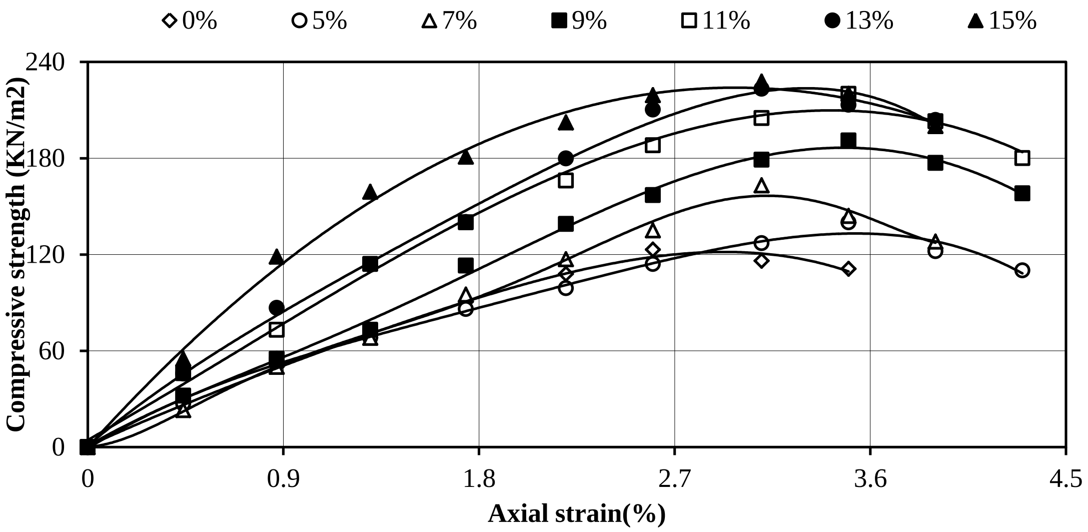

Figure 32: Variation of compressive strength with axial strain of soil blended with different percentage of microsilica at seven days curing

From figure 32, it is seen that the Unconfined Compressive Strength of the composite sample at 7 days curing period increases up to 11%, further then the Unconfined Compressive Strength become constant with further addition of microsilica content up to 15%. This shows that after 11% of microsilica content, the unconfined compressive is slightly affected.

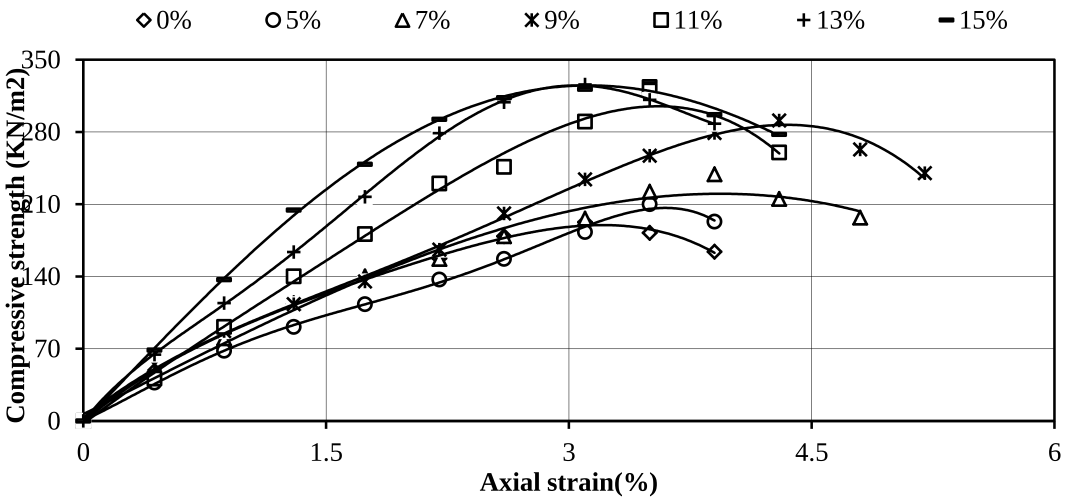

Figure 33: Variation of compressive strength with axial strain of soil blended with different percentage of microsilica at Fourteen days curing

From figure 33, it is seen that the Unconfined Compressive Strength of the composite samples at 14 days curing period increases linearly up to 11%, further then the Unconfined Compressive Strength become constant with further addition of microsilica content up to 15%. This shows that after 11% of microsilica content, the unconfined compressive is slightly affected.

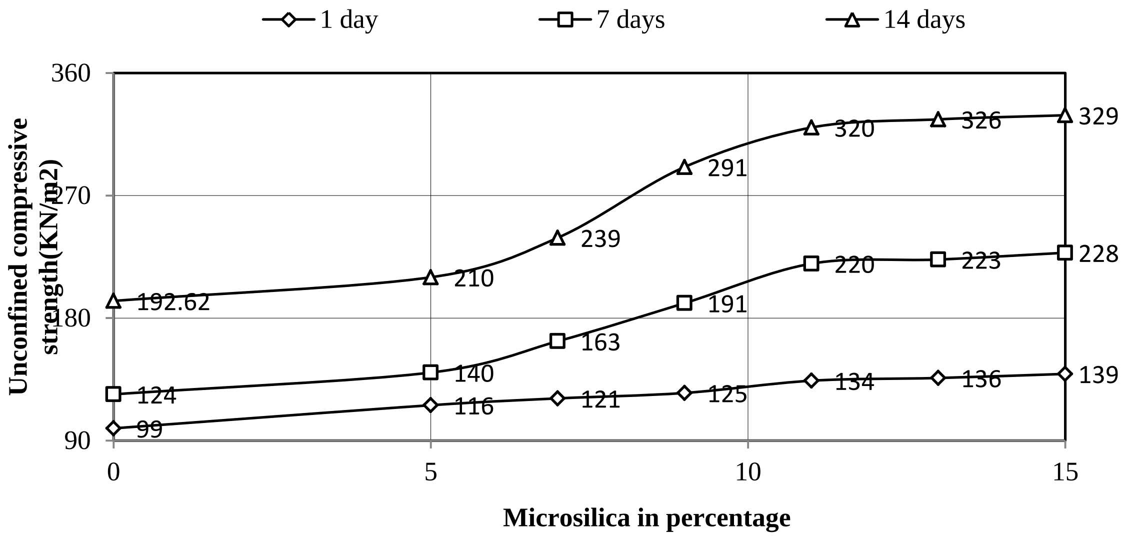

Figure 34: Unconfined Compressive Strength with microsilica in different percentage at different days in curing

As the figure 34, gives the variation of different samples of soil prepared at different percentage of microsilica content from 5% to 15% at 1day, 7 days and 14 days curing period, it can be seen that the Unconfined Compressive Strength increases with the increase in the curing period of the composite samples. The above graph also shows the linear proportionality of the Unconfined Compressive Strength with percentage of microsilica content. But at 7 days and 14 days it can be seen that unconfined increases rapidly with the percentage of microsilica content up to 11% then after that the Unconfined Compressive Strength slightly changes. The maximum strength of Unconfined Compressive Strength is between 5% - 11%. The increase in the Unconfined Compressive Strength is attributed to the internal friction of silica fume particles and chemical reaction between silica fume and soil. An increase in silica fume content in soil had made the stabilized soil samples more brittle than the natural soil samples which is ductile as compared to all stabilized samples. The Unconfined Compressive Strength test is widely used as a quick, economical method of obtaining the approximate compressive strength of all cohesive soils.

Table12: Unconfined Compressive Strength at different percentage of microsilica at different curing period

Sample	Curing days	Percent of content	Unconfined compressive strength(KN/m^2)
Soil with different percentage of microsilica content	1days	0	99
		5	116
		7	121
		9	125
		11	134
		13	136
		15	139
	7 days	0	124
		5	140
		7	163
		9	191
		11	220
		13	223
		15	228
	14 days	0	192.62
		5	210
		7	239
		9	291
		11	320
		13	326
		15	329

According to the table 12, following result can be seen:

One day Unconfined Compressive Strength of Expansive soil at 0%,5%, 7% ,9% ,11% ,13% and 15% is $99KN/m^2$, $116KN/m^2$, $121KN/m^2$, $125KN/m^2$, $134KN/m^2$, $136KN/m^2$ and $139KN/m^2$ and the percentage of variation of Unconfined Compressive Strength with respect to soil is 17.17% ,22.22% ,26.26% ,35.35% ,37.37% and 40.4%.The maximum value of Unconfined Compressive Strength is $139KN/m^2$ at content of 15%

Seven days Unconfined Compressive Strength of Expansive soil at 0%, 5%, 7%, 9%,11%,13% and 15% is 124KN/m^2, 140KN/m^2, 163KN/m^2, 191KN/m^2, 220KN/m^2, 223KN/m^2 and 228KN/m^2 and the percentage of variation of Unconfined Compressive Strength with respect to soil is 13.13% ,31.45% , 54.03% , 77.41% ,80% and 83.87%.The maximum value of Unconfined Compressive Strength is 228KN/m^2 at content of 15% .

Fourteen days Unconfined Compressive Strength of Expansive soil at 0%, 5%, 7%, 9%,11%,13% and 15% is 192.62KN/m^2 ,210KN/m^2, 239KN/m^2, 291KN/m^2, 320KN/m^2, 326KN/m^2 and 3429KN/m^2 and the percentage of variation of Unconfined Compressive Strength with respect to soil is 9%, 24.07%, 51.56%, 66.66%, 69.79% and 71.35%. The maximum value of Unconfined Compressive Strength is 329KN/m^2 at content of 15%.

From the above discussion it is found that with the increase in the microsilica content and with the increase in curing period the Unconfined Compressive Strength get increased. It is clear more curing period samples showing good compressive strength with the addition of microsilica content.

4.2.3. California bearing ratio test (Un soaked condition) IS: 2720 (Part 1), 1987

Optimum water content at different % microsilica

Dry density at different percentage of microsilica

Mass of mould=7950gm

Mass of soil=500gm

n

Figure 35: Sample prepared in CBR mold

Figure 36: testing of CBR specimen

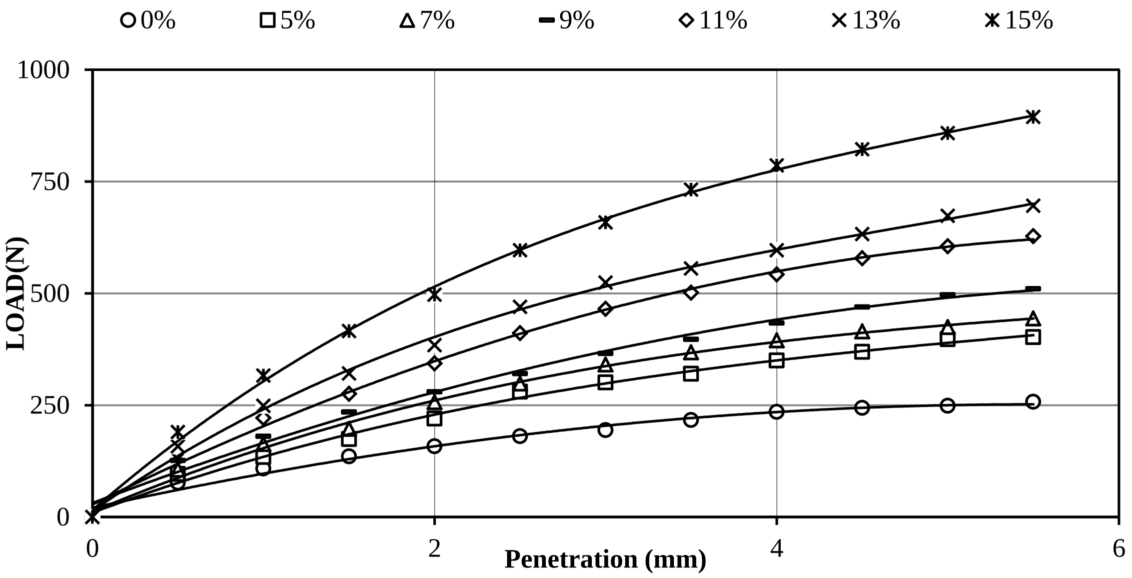

Figure 37: Variation of load with penetration of soil blended with different percentage of microsilica

From the figure 37, it can be seen that with the addition of the microsilica from 5% to 15%, the California bearing ratio value get increased. There comes no optimum value in the CBR value up to 15%, the value of the composite samples is increasing.

From the above discussion it can be found out that as there is an increase in the CBR value, the load carrying capacity of the soil also increases. The result shows that with the improvement in the value of CBR value the soil can now be safely used in the sub grade in road construction and in the embankments in dams.

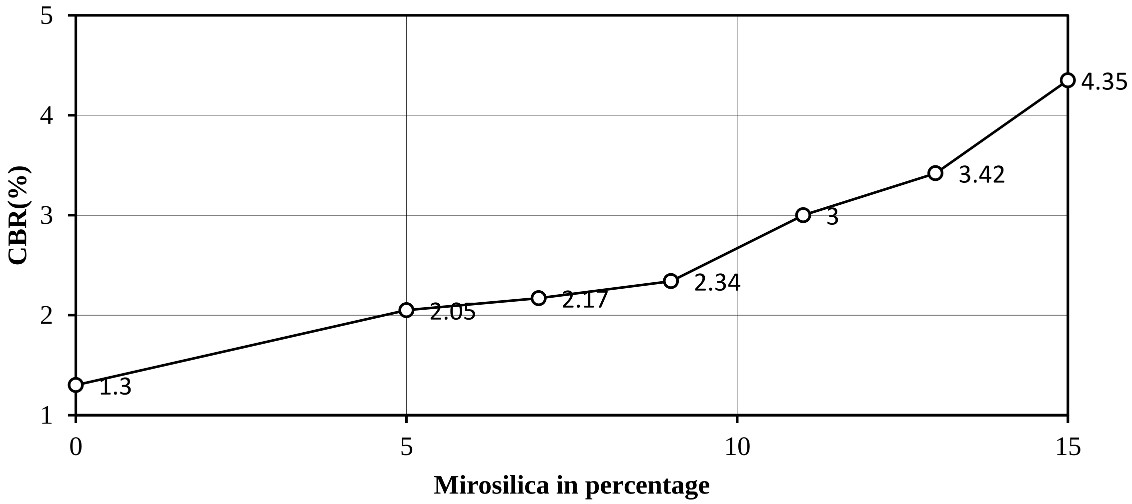

Figure 38: Variation of percentage of CBR with microsilica

From the figure 38, it can be seen that there is an initial hike in CBR value up to 5%, then becomes constant till 9%, and then increases rapidly to 15%. The remarks would be that with the increase in microsilica content the particles of microsilica takes the voids of soil, holding soil more strong, making it more elastic.. The variation of CBR with microsilica content can be seen in table

Table 13: CBR value at different percentage of microsilica

S.no	percentage of microsilica	CBR (%) (Un soaked condition)
1	0	1.3
2	5	2.05
3	7	2.17
4	9	2.34
5	11	3
6	13	3.42
7	15	4.35

Table 13, represents the value of CBR value with different percentage of microsilica content. When it is compared with the unblended soil the percentage in CBR value was improved by 57.69%, 66.92%, 80%, 130.7%, 163% and 234.61% at microsilica content of 5%, 7%, 9%, 11%, 13% and 15%.

When the value of CBR is compared with that of unblended soil, it is found that with the percentage of microsilica content, the CBR value are observed to be improved by 2.05%, 2.17%, 2.34%, 3%, 3.42% and 4.35%.

4.2.4. Free swell index IS:2720 (part XL) , 1977

Free swell index is determined by IS specification. According to the experimental investigation it is found that as the microsilica content increases from 5% to 15%, the free swell index of the expansive get decreased.

From the figure 39, it can be seen that the free swell index get decreased with increase in the microsilica content from 5% to 15%.

The reason for reduction in the free swell index of the expansive soil when blended with microsilica might be the pozzolanic reaction between the free calcium compound and silica compound to form a hydration compound which generate heat, with this the water that should come in contact with the expansive soil, make hydroxyl ions to produce heat of hydration, so less water goes in the shells of expansive soil. Basically free swell index test is used in embankments, dams and canal like structure built on expansive soils where there is high amount of seepage of water, due to which the probability of failure of structure increases. The decrease of free swell index shown in table below:

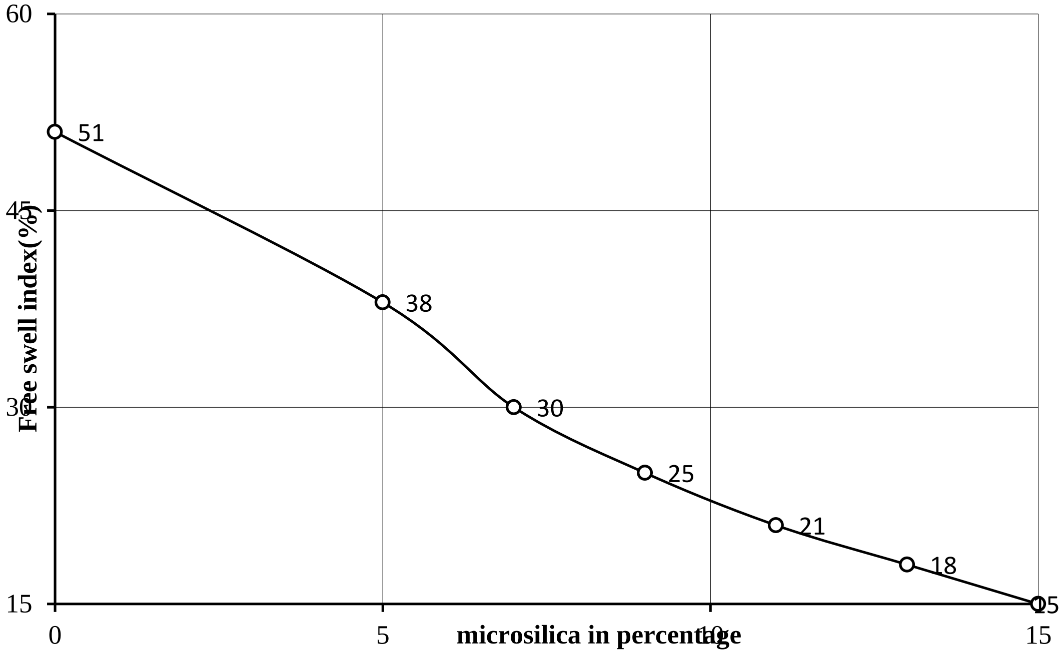

Figure 39: Variation of Free swell index with microsilica in different percentage

Table 14: Free swell index at different value of microsilica

S.No	percentage of microsilica	Free swell index (%)
1	0	51
2	5	38
3	7	30
4	9	25
5	11	21
6	13	18
7	15	15

Table 14, presents the value of free swell index of the samples with the varying percentage of microsilica content. The decrease in the percentage of the free swell index value with respect to unblended soil is 25.5%, 41.11%, 51%, 59%, 75% and 71% at 5%, 7%, 9% , 11%, 13% and 15% microsilica content respectively.

4.2.5. Direct shear test IS:2720 (Part 11)(1983):

Figure 40: Sample of direct shear test

Figure 41: Failed sample after testing

Size of box = 6cm*6cm*3cm

Area of box = 36cm2

Mass of box + base plate + porous stones + grid plate=3250gm

Mass of box +base plate + porous stones +grid plate +soil specimen+ microsilica=3500gm

Normal stress=50KN/m2, 100KN/m2, 150KN/m2

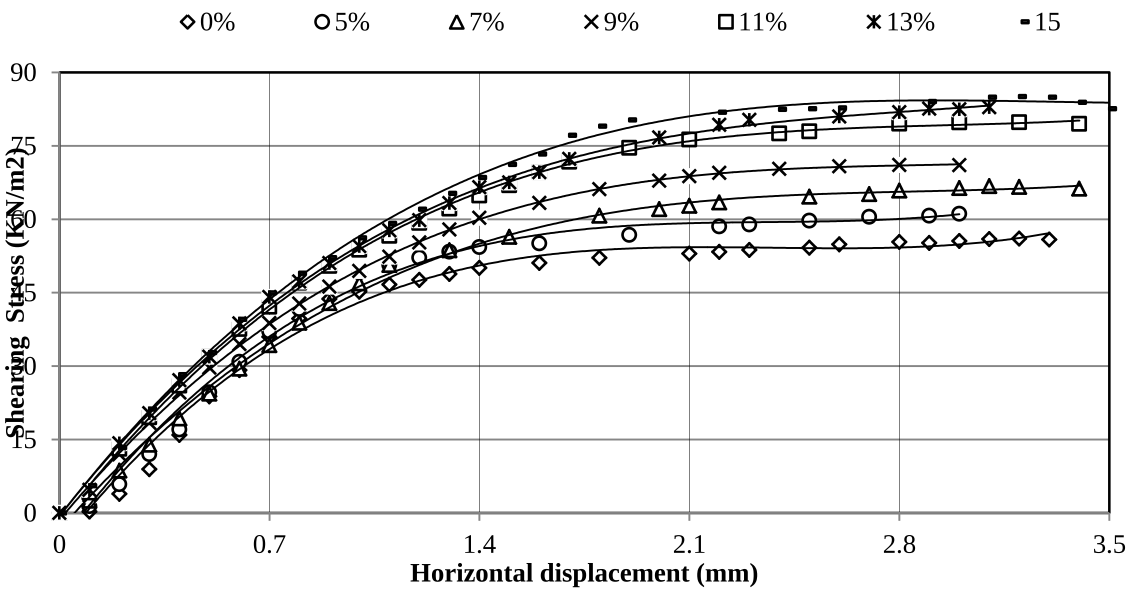

Figure 42: Variation of Shearing Stress with horizontal displacement at different percentage of microsilica at 50kN/m^2

According to the figure 42, it can be seen that the shearing stress of the composite samples increases with increase in the microsilica content. Above graph shows the shearing stress corresponding to the normal stress at 50KN/m^2. It can be seen that for microsilica content from 5% to 11%, the variation in shearing stress moves sharply with respect to the horizontal displacement. At microsilica content above 13%, there is not much increment in the shearing stress with respect to the horizontal displacement.

According to the below figure 43, it can be seen that with the addition of microsilica content the shearing stress tends to increase at a linear state at a normal stress of 100KN/m^2, with the maximum variation at 9%.As it can be seen that the shearing stress at 100KN/m^2 is more than the shearing stress at 50KN/m^2 for all values of horizontal displacement. The shearing stress at 15% microsilica tends to show the maximum peak which clearly shows that addition of microsilica increases the shearing stress. Thus making stabilized soil more compatible in working in various structures.

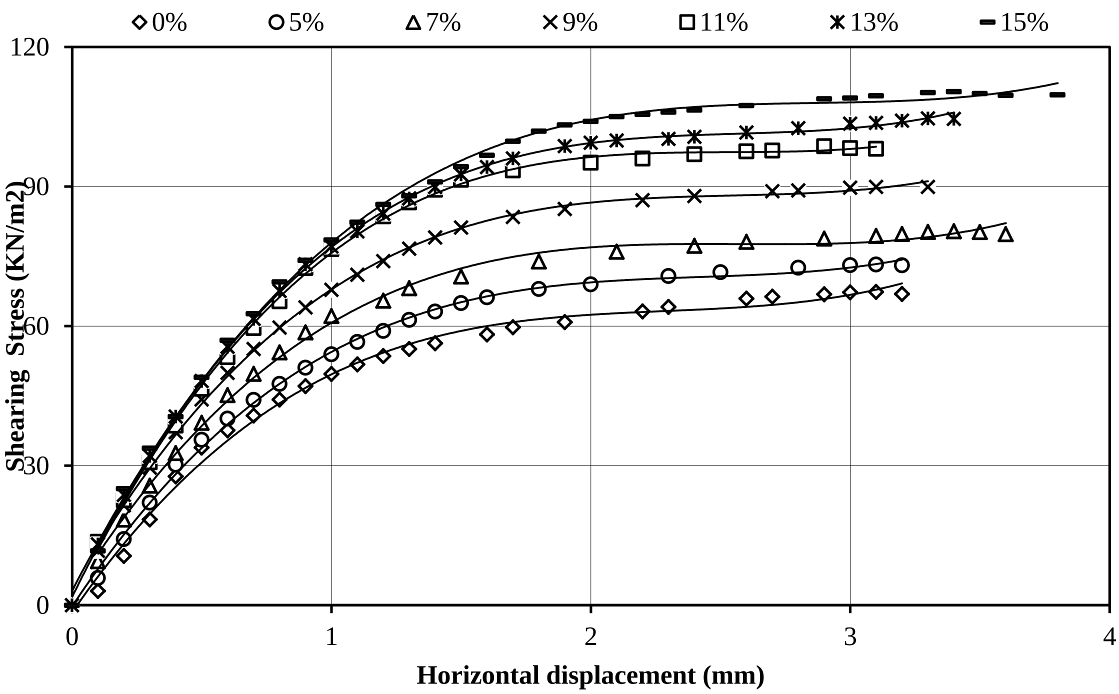

Figure 43: Variation of Shearing Stress with horizontal displacement at different percentage of microsilica at 100kN/m^2

According to the figure 44, it can be seen that the shearing stress of the samples is increasing with respect to the addition of microsilica content. It can be seen that at 5% and 7% microsilica content, the shearing stress is not having such large variation, and the graph tends to overlap each other, the same case can be seen at 13% and 15%. There tends to a heavy increase in shearing stress with respect to horizontal displacement at 9% and 11% with a normal stress of 150KN/m^2. The shearing stress for all values at 150KN/m^2 is more than that of shearing stress at 100KN/m^2 normal stress.

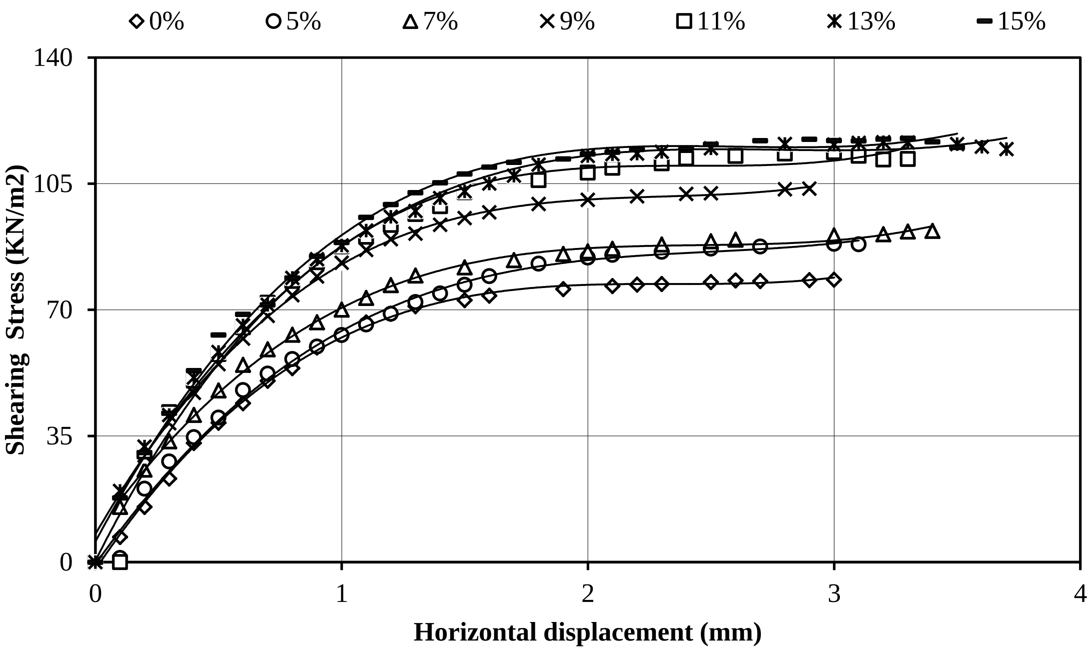

Figure 44: Variation of Shearing Stress with horizontal displacement at different percentage of microsilica at 150kN/m^2

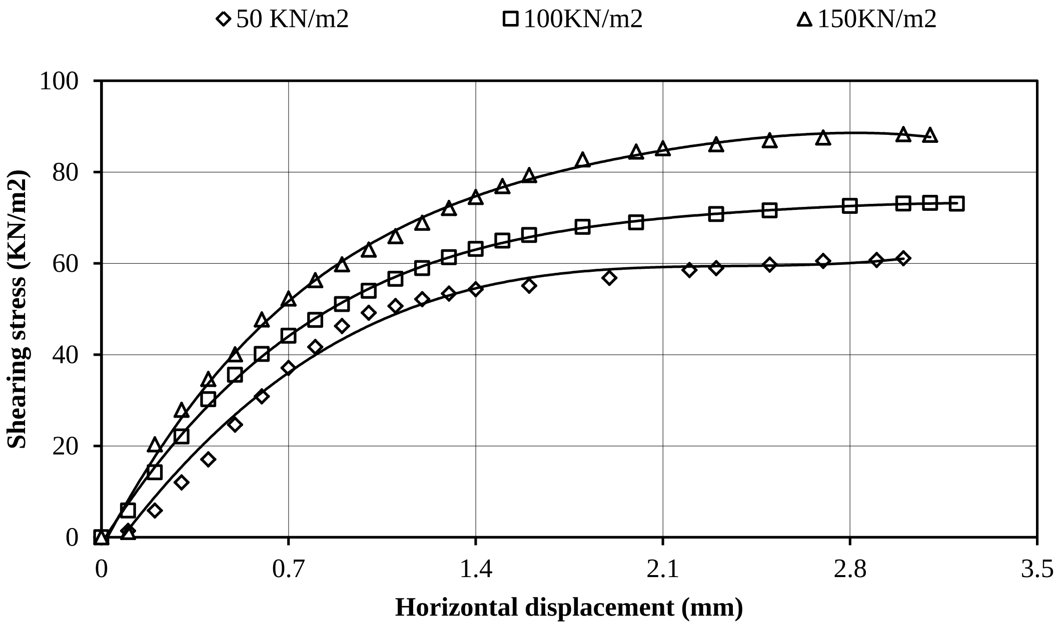

Figure 45: Variation of Shearing stress with horizontal displacement of soil blended with microsilica at five percent at different normal stress

Figure 45, shows that the shearing stress graph with respect to the horizontal displacement at different normal loads, it can be as the normal stress is increased the shearing stress increases. The normal stress of 50KN/m^2, 100 KN/m^2 and 150 KN/m^2 is been loaded. The shearing stress at different normal stress comes to 61.11 KN/m^2, 73.22 KN/m^2 and 86.12 KN/m^2 at a microsilica content of 5%.

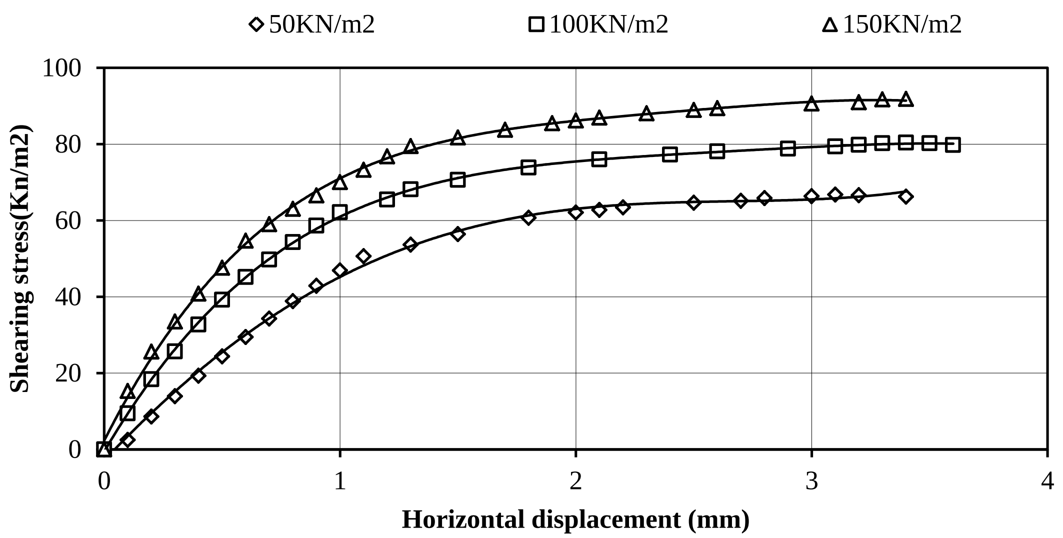

Figure 46: Variation of Shearing stress with horizontal displacement of soil blended with microsilica at seven percent at different normal stress

Figure 46, shows that the shearing stress graph with respect to the horizontal displacement at different normal loads, it can be as the normal stress is increased the shearing stress increases. The normal stress of 50KN/m^2, 100 KN/m^2 and 150 KN/m^2 is been loaded. The shearing stress at different normal stress comes to 66.78 KN/m^2, 80.38 KN/m^2 and 93.56 KN/m^2 at a microsilica content of 7%.

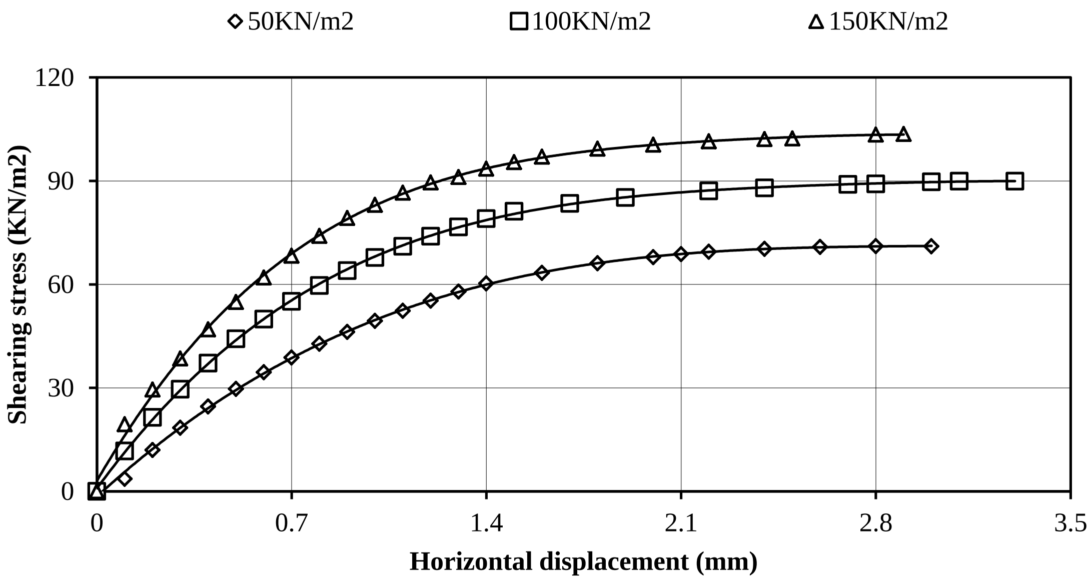

Figure 47: Variation of Shearing stress with horizontal displacement of soil blended with microsilica at nine percent at different normal stress

Figure 47, shows that the shearing stress graph with respect to the horizontal displacement at different normal loads, it can be as the normal stress is increased the shearing stress increases. The normal stress of 50KN/m^2, 100 KN/m^2 and 150 KN/m^2 is been loaded. The shearing stress at different normal stress comes to 71.09 KN/m^2, 89.94 KN/m^2 and 103.61 KN/m^2 at a microsilica content of 9%.

According to the figure 48, it can be seen that with the addition of microsilica content the shearing stress tends to increase at a linear state at a normal stress of 100KN/m^2, with the maximum variation at 9%.As it can be seen that the shearing stress at 100KN/m^2 is more than the shearing stress at 50KN/m^2 for all values of horizontal displacement. The shearing stress at 15% microsilica tends to show the maximum peak which clearly shows that addition of microsilica increases the shearing stress. Thus making stabilized soil more compatible in working in various structures.

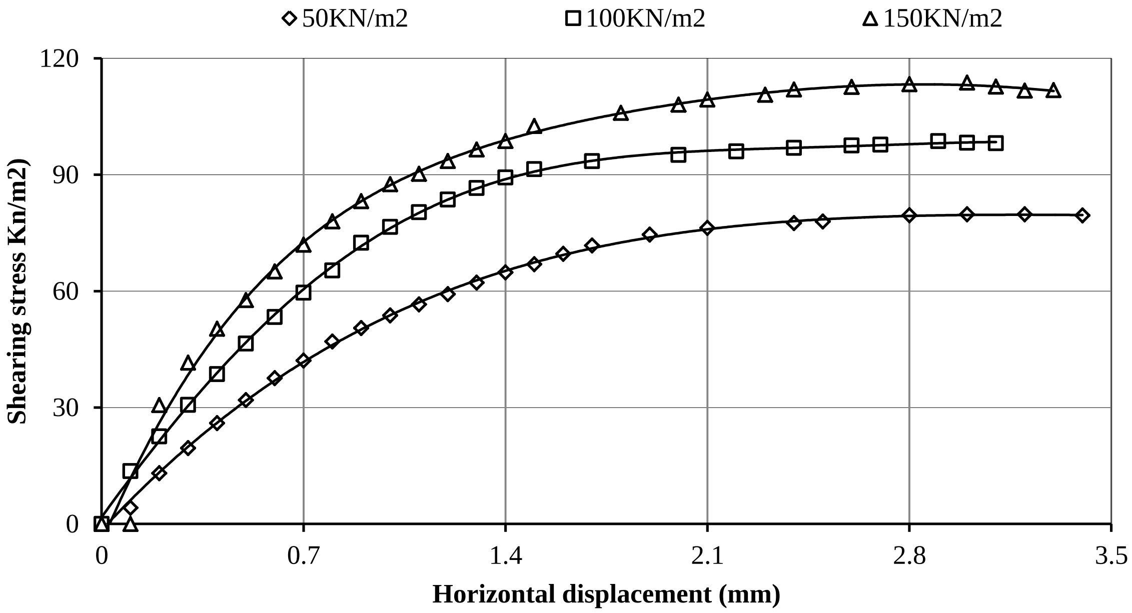

Figure 48: Variation of Shearing stress with horizontal displacement of soil blended with microsilica at eleven percent at different normal stress

Figure 49, shows that the shearing stress graph with respect to the horizontal displacement at different normal loads, it can be as the normal stress is increased the shearing stress increases. The normal stress of 50KN/m^2, 100 KN/m^2 and 150 KN/m^2 is been loaded. The shearing stress at different normal stress comes to 82.894 KN/m^2, 101.122 KN/m^2 and 16.491 KN/m^2 at a microsilica content of 13%.

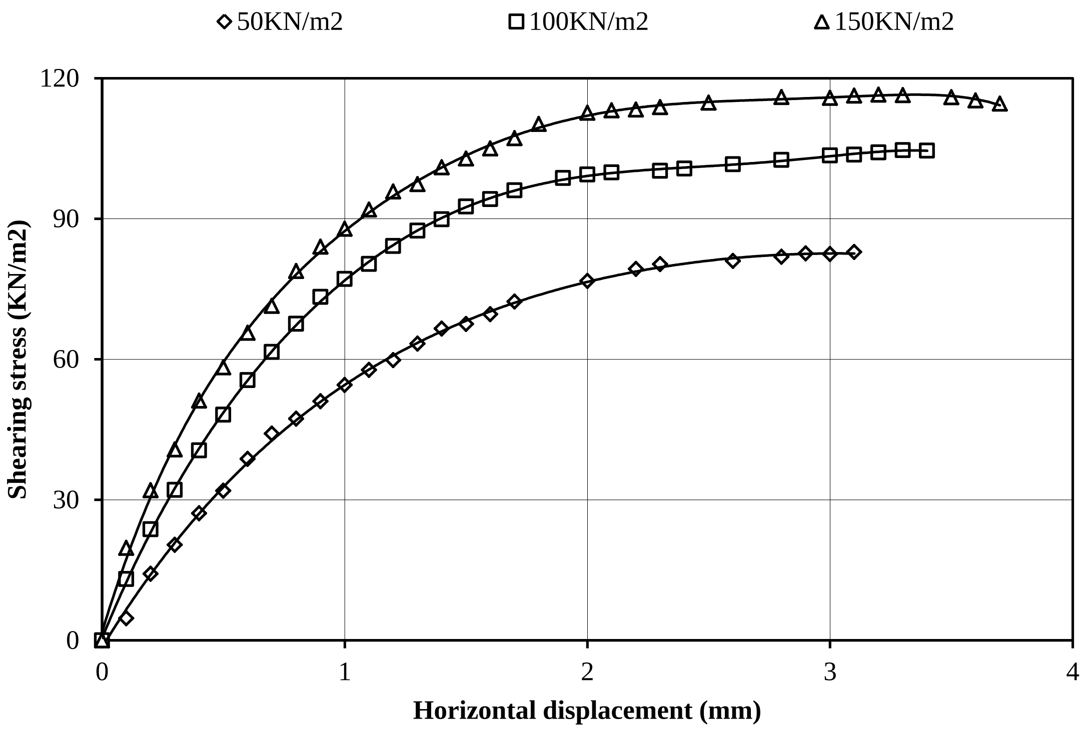

Figure 49: Variation of Shearing stress with horizontal displacement of soil blended with microsilica at thirteen percent at different normal stress

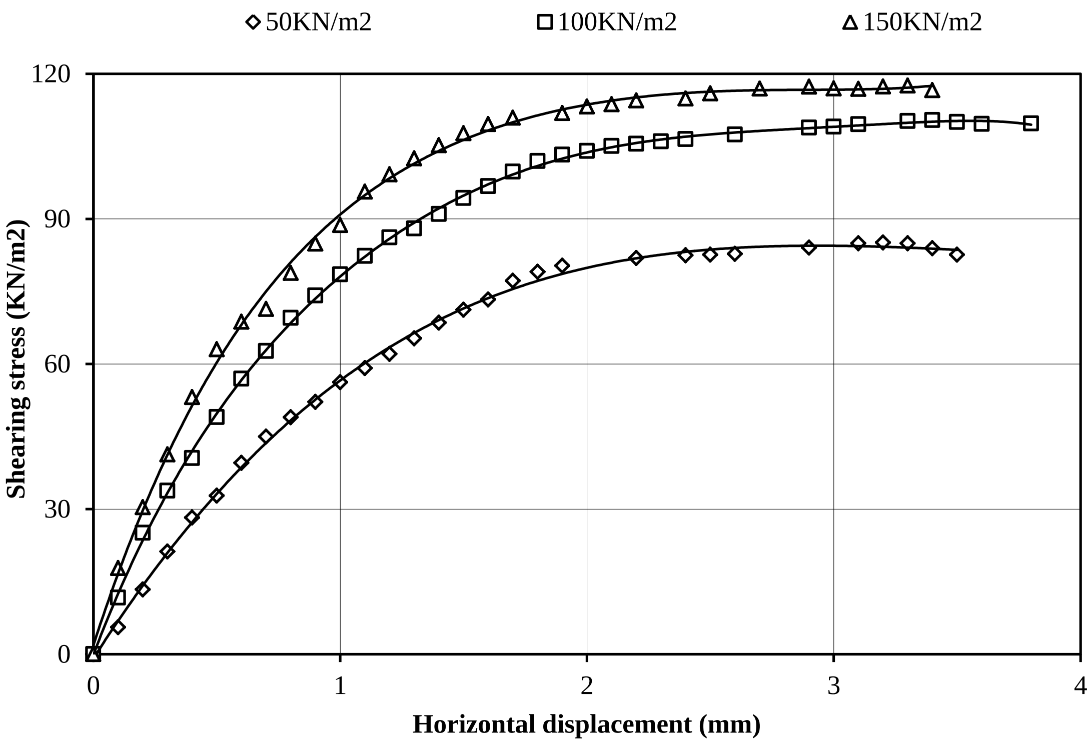

Figure 50: Variation of Shearing stress with horizontal displacement of soil blended with microsilica at fifteen percent at different normal stress

Figure 50, shows that the shearing stress graph with respect to the horizontal displacement at different normal loads, it can be as the normal stress is increased the shearing stress increases. The normal stress of 50KN/m^2, 100 KN/m^2 and 150 KN/m^2 is been loaded. The shearing stress at different normal stress comes to 85.0939 KN/m^2, 104 KN/m^2 and 119.64 KN/m^2 at a microsilica content of 15%.. The Normal stress and shearing stress at different percentage of microsilica are given below in table 15

Table 15: Shearing stress & Normal stress at different percentage of microsilica

Sample	Percent of microsilica	Normal stress(KN/m2)	Shearing stress(KN/m2)
Soil blended with different percentage of microsilica	0	50	56.04
		100	67.36
		150	78.36
	5	50	61.11
		100	73.22
		150	86.12
	7	50	66.78
		100	80.38
		150	93.56
	9	50	71.09
		100	89.94
		150	103.61
	11	50	79.81
		100	98.65
		150	113.74
	13	50	82.894
		100	101.122
		150	116.491
	15	50	85.0939
		100	104
		150	119.64

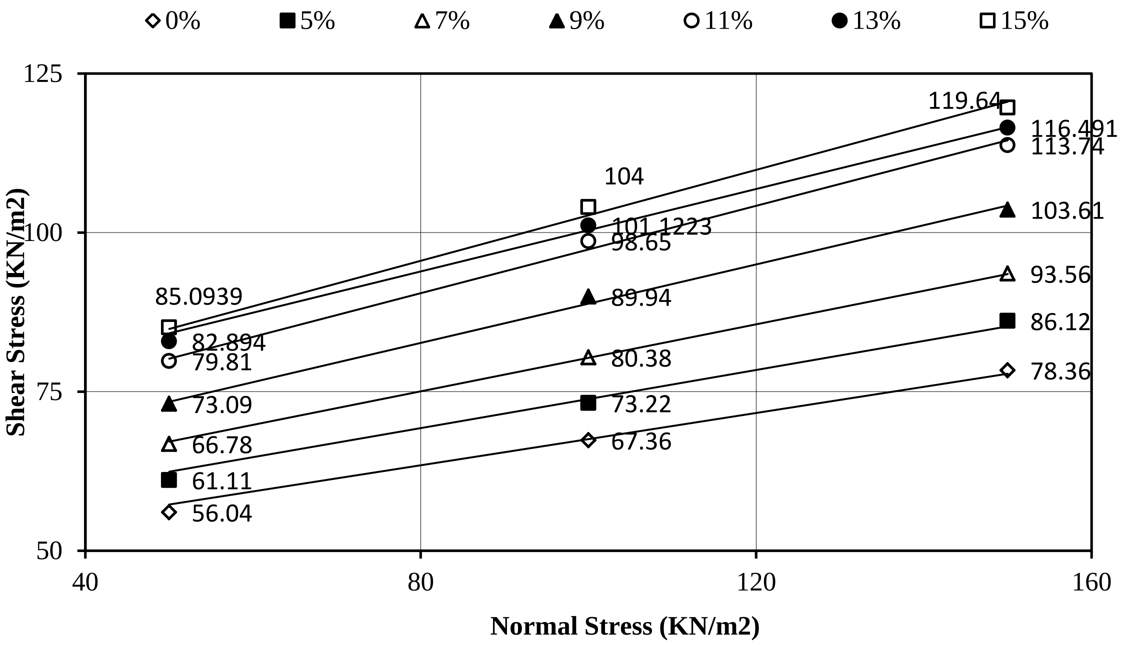

Figure 51: Variation of shear stress with normal stress to calculate the shear parameters i.e. cohesion and angle of internal friction

From the figure 51, it can be seen that with the increase in the percentage of microsilica content the cohesion of soil increases to 13%, then goes down at 15%. Relative to this the angle of friction which is also a property used in the construction of embankment, dams is seen here to increase with increase in percentage of microsilica content to the value of 11%, then decrease at 13% and then again increase at 15% microsilica content. The increase in the cohesion of the composite samples prepared at different percentage of microsilica content can be result from a good binding between the soil particles and microsilica particles which provide interlocking and makes it strong. The Equation of line, slope and intercept are given below in table:

Table 16: Equation of slope & intercept of line at different percentage of microsilica

S.no	Percent of microsilica	Equation of line	Slope= Tanθ	Intercept (cohesion)KN/m2
1	0	Y=0.205x+47	0.205	47
2	5	Y=0.228x+51	0.228	51
3	7	Y=0.263x+54	0.263	54
4	9	Y=0.308x+58	0.308	58
5	11	Y=0.343x+63	0.343	63
6	13	Y=0.323x+68	0.323	68
7	15	Y=0.357x+67	0.357	67

The table 16, shows the value of cohesion and angle of internal friction in degree with varying percentage of microsilica. The angle of internal friction and cohesion is finding out by the graph between shear stress and normal stress with varying percentage of microsilica. Simply we have to only draw an intercept line and slope which will calculate both cohesion and angle of internal friction. The value of cohesion comes to be 47KN/m,51KN/m, 54KN/m, 58KN/m, 63KN/m, 68KN/m and 67KN/m, while the value for angle of internal friction comes in degree 11.58, 12.84, 14.73, 17.11, 18.93, 17.9 and 19.64 corresponding to the 5%, 7%, 9%, 11%,13% and 15% microsilica content.

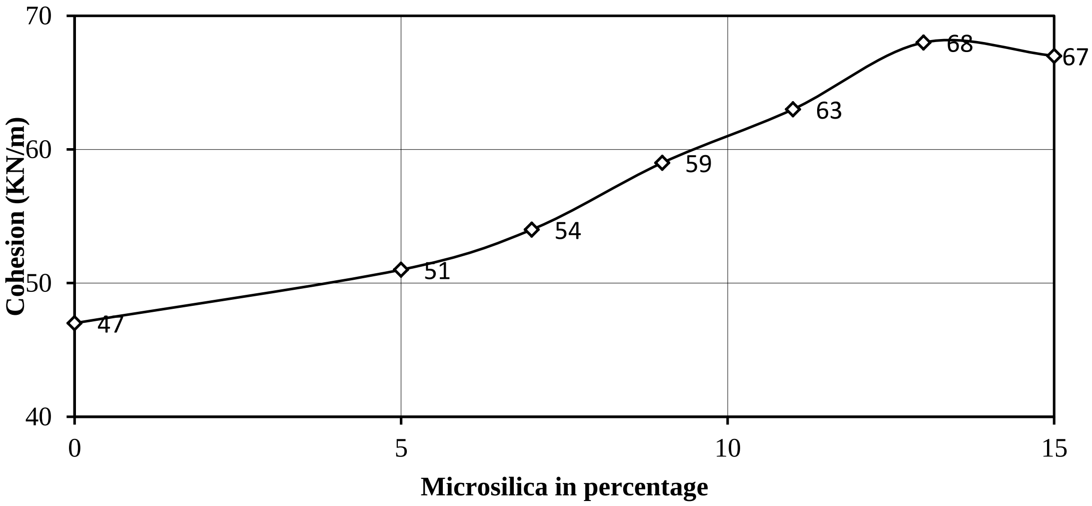

Figure 52: Variation of cohesion with microsilica at different percentage

From the figure 52, it can be seen that there is a liner increase in the cohesion of the stabilized soil with different percentage of microsilica to the value of 13%, after that it decreases.

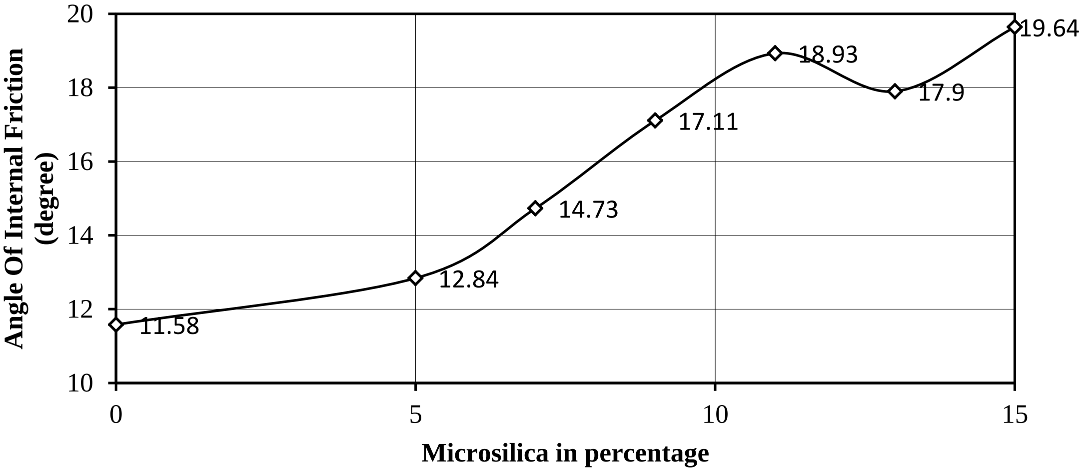

Figure 53: Variation of angle of internal friction with microsilica at different percentage

From the figure 53, it can be seen that the angle of internal friction increases with the increase in the value of microsilica content to the value of 11%, then decreases to 13% and then again increase to 15%.

4.3 Soil blended with rice husk ash (RHA)

4.3.1. Proctor compaction test: IS: 10074, 1982

Sample weight = 2.5Kg

Mass of mold +base plate (W) = 4286gm

Volume of mold (V) = 1000cc

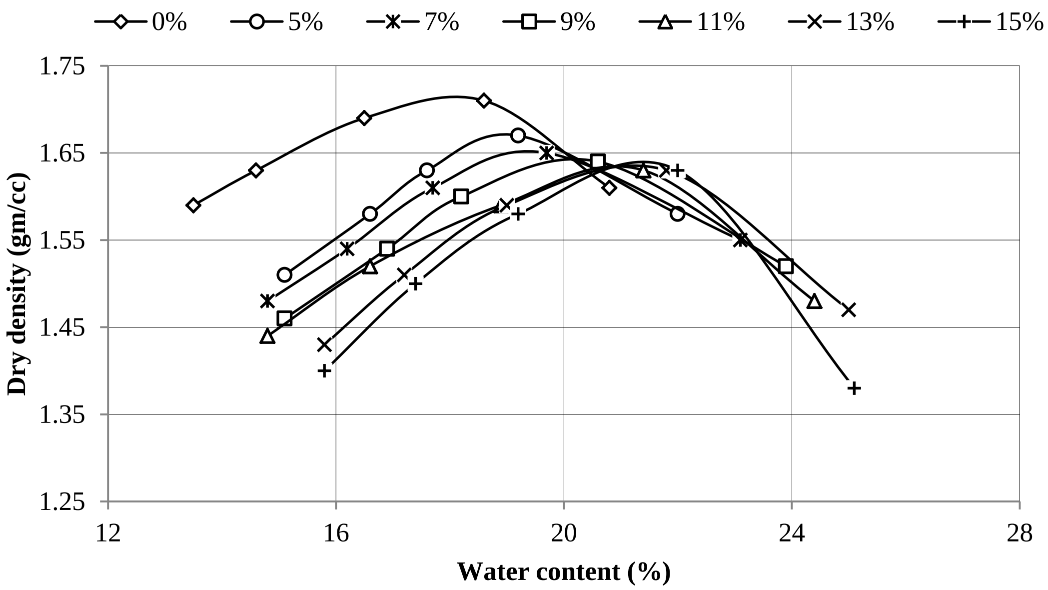

Figure 54: Variation of dry density with water content of soil blended with different percentage of RHA

From the figure 54, it can be seen that with the increase in the percentage of RHA content in the soil the optimum moisture content increases while the maximum dry density decreases. It can be seen that the optimum moisture content increases at a very slow rate with increase in percentage of RHA content, the reason must be that the RHA are finer than soil. The more fine the particle, the more surface area it have, so more water is required to have a full lubrication. The RHA content also decreases the amount of free silt and clay fraction, forming coarser materials, which occupy large spaces for retaining water. Another point in increase of optimum moisture content is attributed by pozzalanic reaction of RHA with soil. On the other hand we can see that the maximum dry density at a constant rate up to 15%, the reason behind it is comparatively is low specific gravity value (2.25) of RHA than that of the replaced soil(2.68) and the initial simultaneous flocculation and agglomeration of clayey particles caused by the cat ion exchange may be the another cause behind it. Another cause can be due to the coating of soil by RHA which results in large particles with larger voids and thus less density.

Table 17: Variation of MDD and OMC with different percentage of RHA content

S.no	% of RHA	Maximum dry density(gm./cc)	Optimum moisture content (%)
1	0	1.71	18.1
2	5	1.67	19.2
3	7	1.65	19.7
4	9	1.64	20.6
5	11	1.63	21.4
6	13	1.63	21.8
7	15	1.63	22

Table 17, represents the result of optimum moisture content and maximum dry density when blended with different percentage of RHA content. We can see with the addition of RHA in the soil, the optimum moisture content increases with a value of 19.2%, 19.7%, 20.6%, 21.4%, 21.8% and 22% and the value of maximum dry density decreases with a value of 1.67gm/cc, 1.65gm/cc, 1.64gm/cc, 1.64gm/cc, 1.63gm/cc, 1.63gm/cc and 1.63gm/cc.

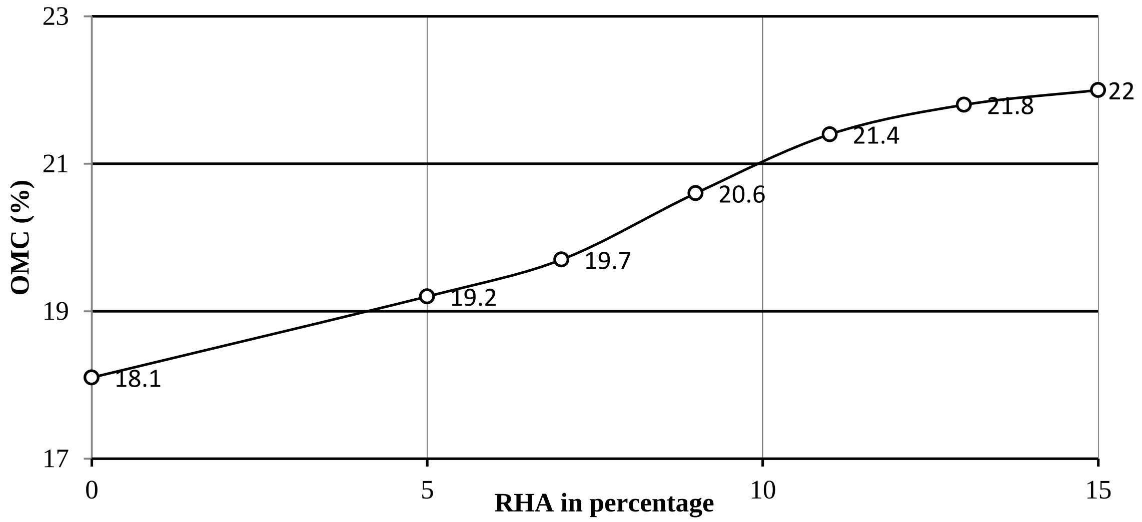

Figure 55: Variation of OMC with RHA in different percentage

From the figure 55, it can be seen that the optimum moisture content increases, with the highest value of 22% at 15% RHA content.

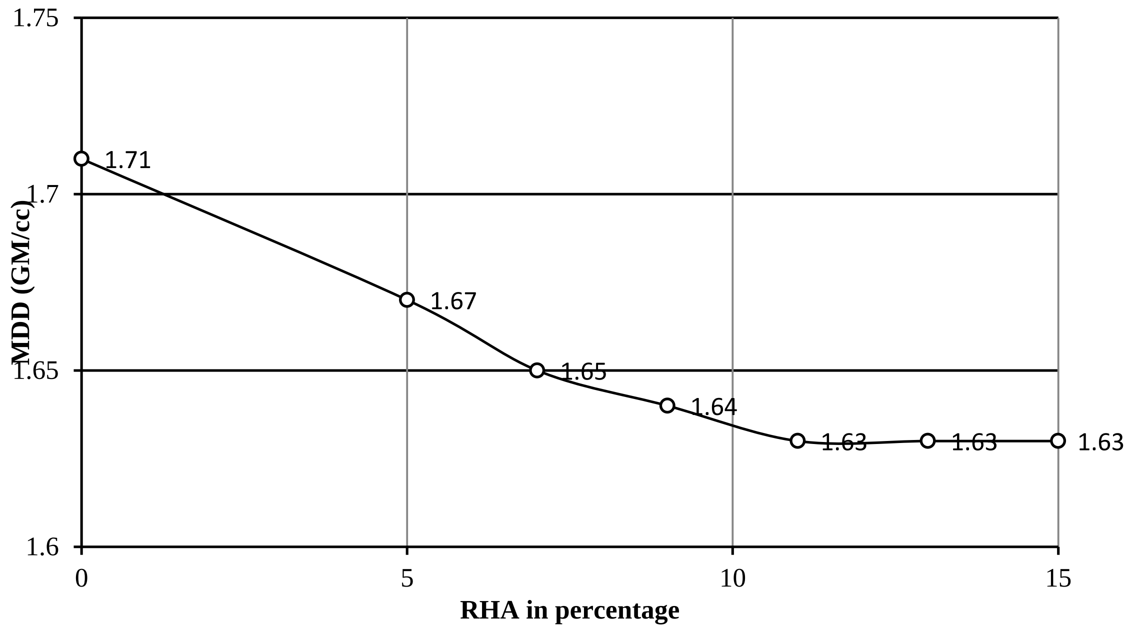

Figure 56: Variation of MDD with RHA in different percentage

From the figure 56, it can be seen that the maximum dry density decrease up to 11%, then it becomes constant, that is above 11% RHA content there is no increase or decrease in maximum dry density.

4.3.2. Unconfined compressive test IS: 2720 (Part 10), 1987

Optimum water content at different Percentage of RHA content

Dry density at different percentage of RHA content

Mass of soil+ RHA=500gm

Length of specimen=115mm

Diameter of specimen =50mm

From the figure 57, it can be seen that the Unconfined Compressive Strength of the blended soil with different percentage of RHA achieve higher strength till 11%, after that it start decreasing. It shows the optimum value of RHA to be used in Unconfined Compressive Strength to be till 11% as mixing after that will decrease the Unconfined Compressive Strength of the sample.

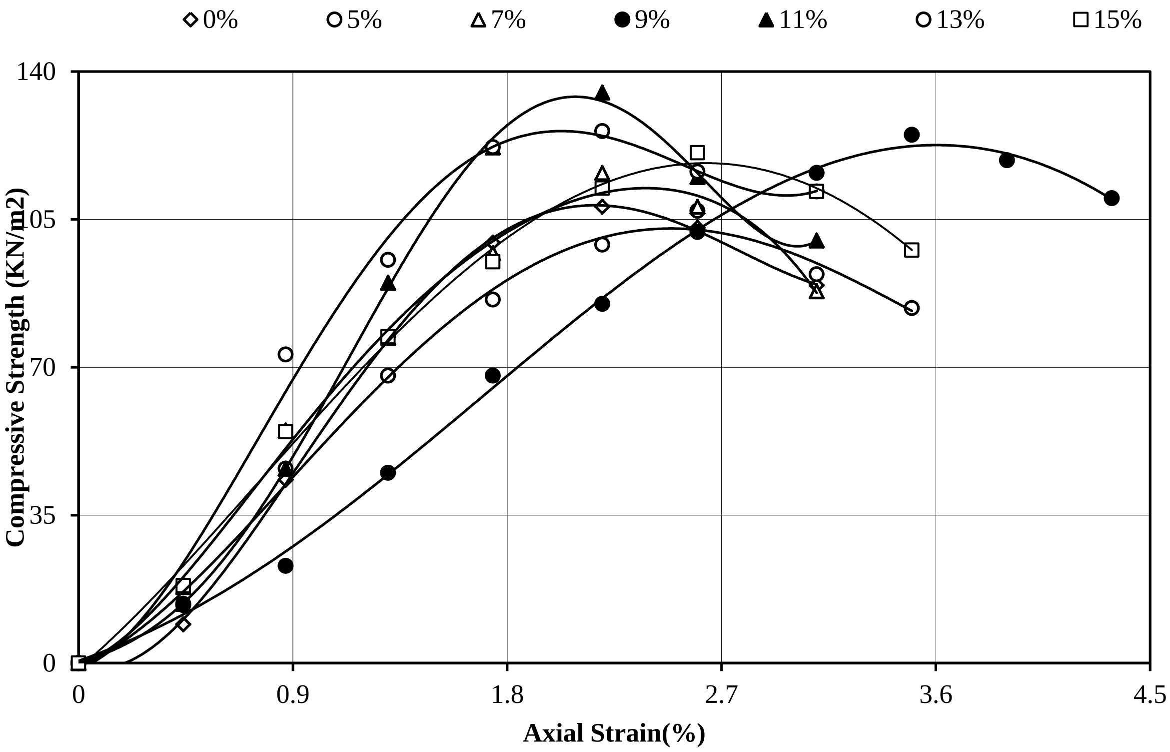

Figure 57: Variation of compressive strength with axial strain of soil blended with different percentage of RHA at one day curing

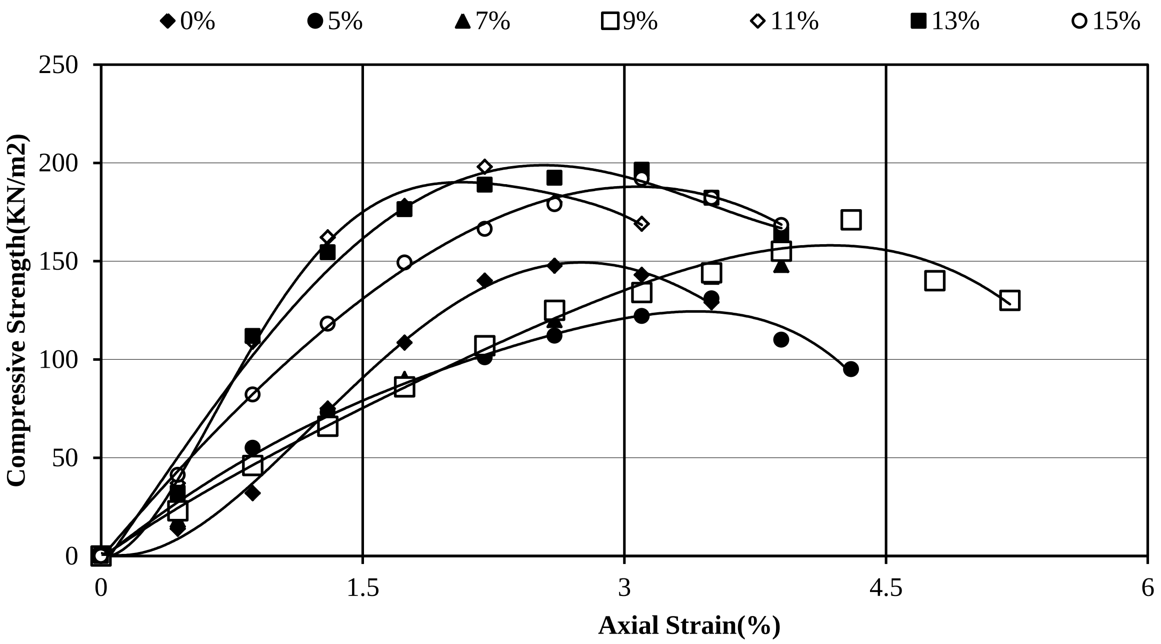

Figure 58: Variation of compressive strength with axial strain of soil blended with different percentage of RHA at seven days curing

From the figure 58, it can be seen that the Unconfined Compressive Strength of the blended soil with different percentage of RHA achieve higher strength till 11%, after that it start decreasing. It shows the optimum value of RHA to be used in Unconfined Compressive Strength to be till 11% as mixing after that will decrease the Unconfined Compressive Strength of the sample. The above graph also shows the higher Unconfined Compressive Strength of the sample at 7 days curing period than the Unconfined Compressive Strength of the samples at 1 day curing period. This shows that curing somehow affects the strength of the sample when RHA is mixed.

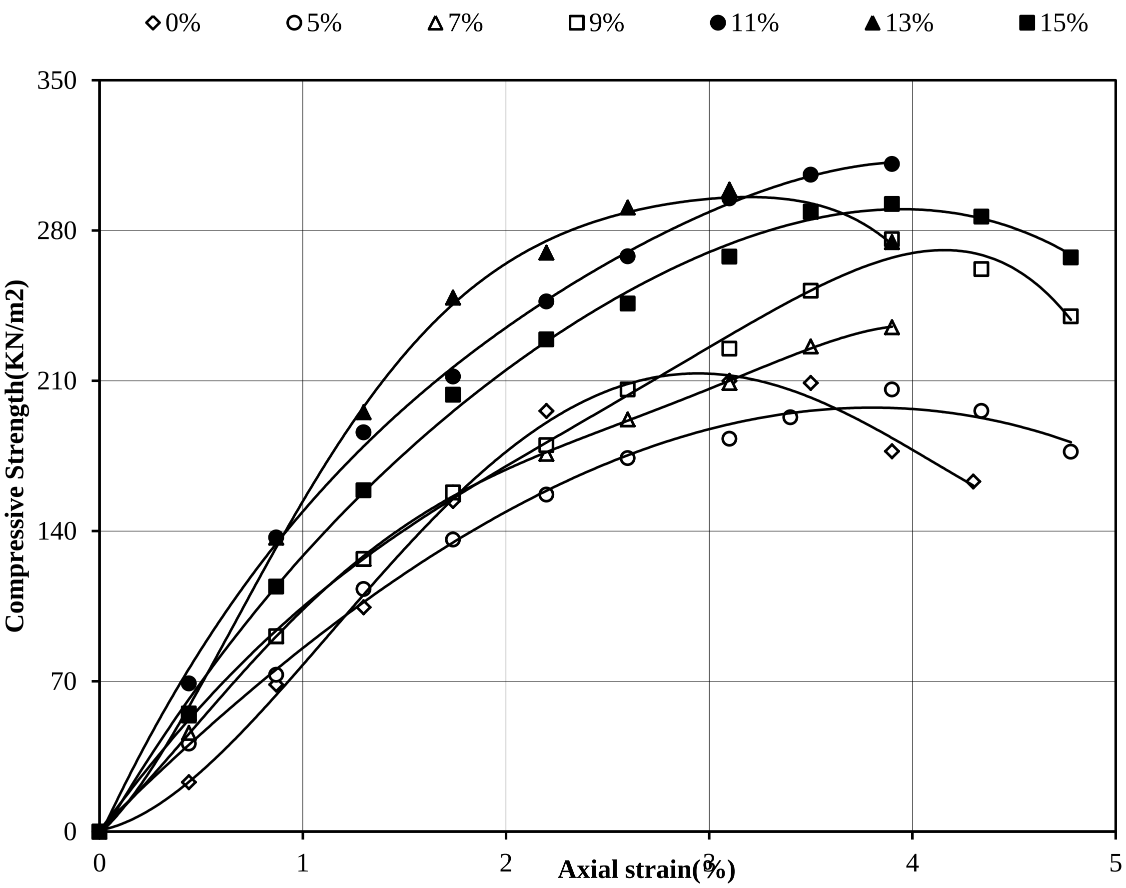

Figure 59: Variation of compressive strength with axial strain of soil blended with different percentage of RHA at fourteen days curing

From the figure 59, it can be seen that the Unconfined Compressive Strength of the blended soil with different percentage of RHA achieve higher strength till 11%, after that it start decreasing. It shows the optimum value of RHA to be used in Unconfined Compressive Strength to be till 11% as mixing after that will decrease the Unconfined Compressive Strength of the sample. The above graph also shows the higher Unconfined

Compressive Strength of the sample at 14 days curing period than the Unconfined Compressive Strength of the samples at 7 days curing period. This shows that curing somehow affects the strength of the sample when RHA is mixed.

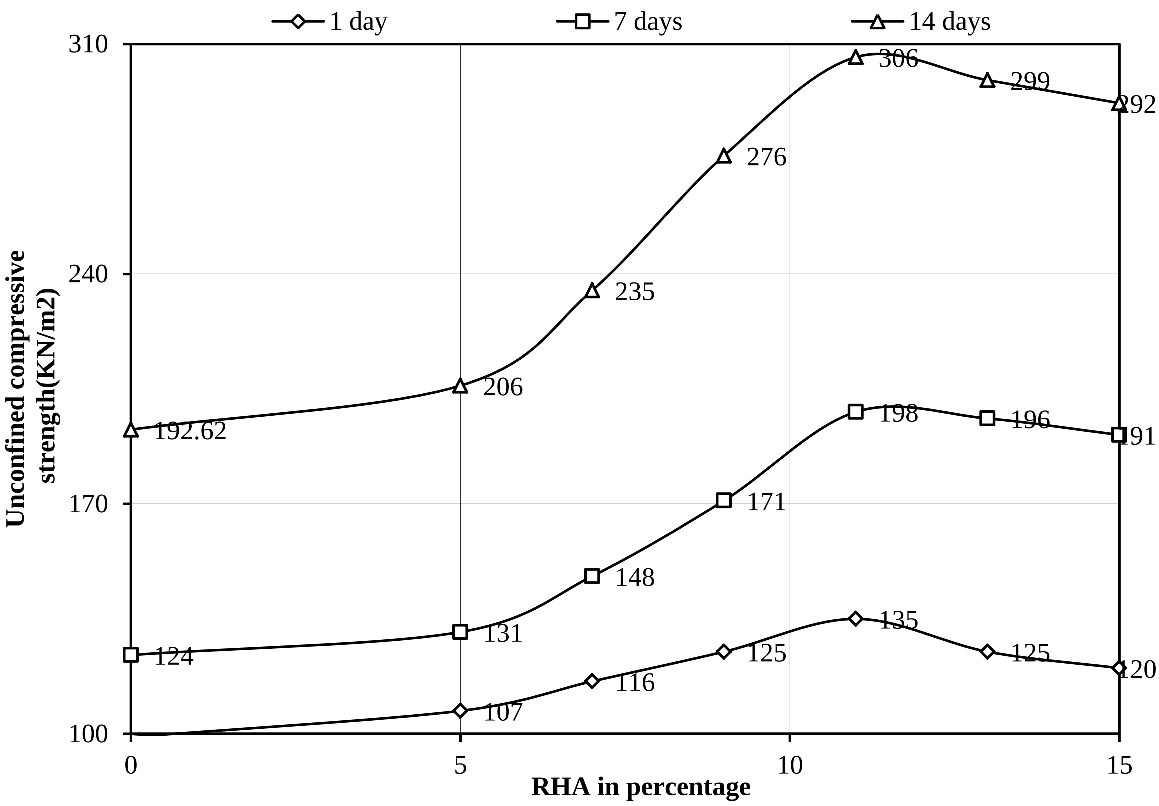

Figure 60: Unconfined Compressive Strength with RHA in different percentage at different days of curing period

It can be seen from the figure 60, plotted between Unconfined Compressive Strength and percentage of RHA at 1, 7 and 14 days curing period, the outcome results shows that the Unconfined Compressive Strength value increases with subsequent addition of RHA to a maximum value of 11%, after which it starts decreasing. The subsequent increase in the value of Unconfined Compressive Strength is attributed to the formation of a cementitious compound between CaOH present in the soil and the pozzolans present in the RHA. The decrease in the value after 11% RHA may be due to excessive RHA introduced in the soil and therefore forming weaker bonds between the soil and the cementitious compound formed. There is a positive impact of RHA on Unconfined Compressive Strength due to pozzolanic reaction of RHA with soils. This results in agglomeration of large size particles and causes an increase in the compressive strength.

The variation of Unconfined Compressive Strength with RHA is given below in table 18 and how much percentage it will vary.

Table 18: Unconfined Compressive Strength at different percentages of RHA content at different curing period

Sample	Curing days	Percent of RHA	Unconfined compressive strength(KN/m^2)
Soil blended with different percentage of RHA content	1days	0	99
		5	107
		7	116
		9	125
		11	135
		13	125
		15	120
	7 days	0	124
		5	131
		7	148
		9	171
		11	198
		13	196
		15	191
	14 days	0	192.62
		5	206
		7	235
		9	276
		11	306
		13	229
		15	292

The following results can be discussed from the following table 18:

One day Unconfined Compressive Strength of Expansive soil at 0%, 5%, 7% , 9% ,11% ,13% and 15% of RHA is 99KN/m^2, 107KN/m^2, 116KN/m^2, 125KN/m^2, 135KN/m^2, 125KN/m^2 and 120KN/m^2 and the percentage of variation of unconfined confined strength with respect to soil is 8.08% ,17.17% ,26.26% ,36.36% ,26.26% and

21.21%.The maximum value of Unconfined Compressive Strength comes to be 135KN/m^2 at 11% RHA.

Seven days Unconfined Compressive Strength of Expansive soil at 0%, 5%, 7% , 9% ,11% ,13% and 15% of RHA is 124KN/m^2, 131KN/m^2, 148KN/m^2, 171KN/m^2, 198KN/m^2, 196KN/m^2 and 191KN/m^2 and the percentage of variation of unconfined confined strength with respect to soil is 5.64% ,19.35% ,37.90% ,59.67% ,58.06% and 54.03%.The maximum value of Unconfined Compressive Strength comes to be 198KN/m^2 at 11% RHA.

Fourteen days Unconfined Compressive Strength of Expansive soil at 0%, 5%, 7% , 9% ,11% ,13% and 15% of RHA is 192.62KN/m^2, 206KN/m^2, 235KN/m^2, 276KN/m^2, 306KN/m^2, 299KN/m^2 and 292KN/m^2 and the percentage of variation of unconfined confined strength with respect to soil is 7.29% ,22.39% ,43.75% ,59.37% ,55.72% and 52.02%.The maximum value of Unconfined Compressive Strength comes to be 306KN/m^2 at 11% RHA.

From the above discussion it can be seen that with increase in the percentage of RHA content and curing period the Unconfined Compressive Strength of samples get increased.

4.3.3. California bearing ratio test (Unsoaked condition) IS: 2720 (Part 1), 1987

Optimum water content at different % of RHA

Dry density at different % of RHA

Mass of mould=7950gm

Mass of soil=500gm

From the figure 61, it can be seen that as the percentage of RHA content increases, the value of CBR also increases till it reaches to the maximum at 11% RHA content. After 11% RHA content the CBR value starts a downfall. So it is clear that the optimum value of 11% RHA content is needed for the maximum value of CBR.

From the above discussion it can be found out that as there is an increase in the CBR value, the load carrying capacity of the soil also increases. The result shows that with the improvement in the value of CBR value the soil can now be safely used in the sub grade in road construction and in the embankments in dams.

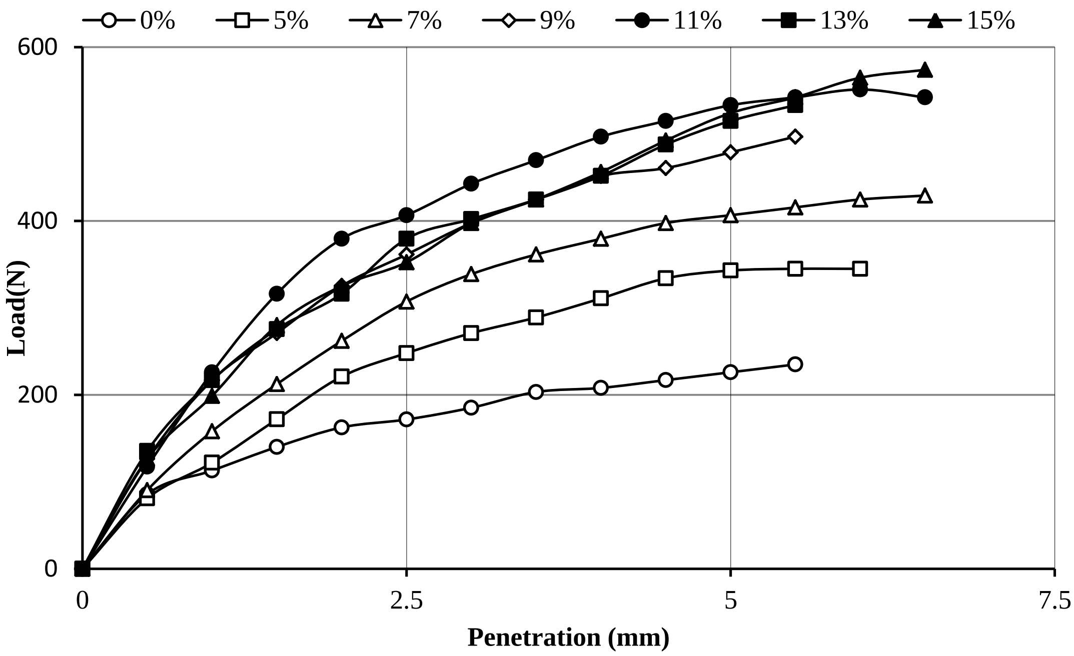

Figure 61: Variation of Load with penetration of soil with different percentage of RHA content

The reason behind the increase in CBR value is the effect of the pozzolanic reaction between the amorphous silica and ammonia present in RHA to react with free lime present in the expansive soil to form a cementitious compound, but this will occur up to certain limit of RHA content of 11%, after that there will be insufficient lime to react with the amorphous silica and alumina to form cementitious compound, so the strength of the soil samples starts decreasing with increase in percentage of RHA content.

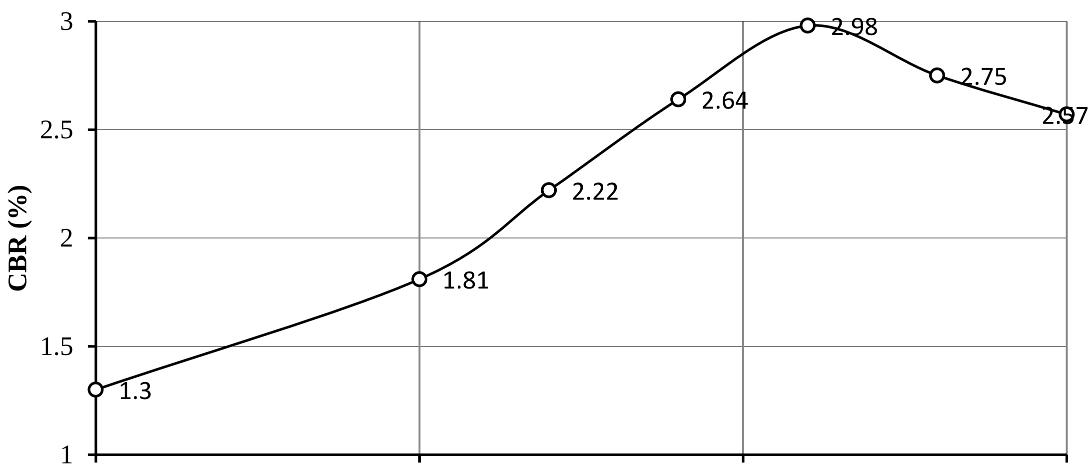

Figure 62: Variation of CBR with RHA in different percentage

From the figure 62, it can be seen that there is a maximum hike in the CBR value between 5% - 11% of RHA content. The strength provided in this percentage is maximum, with a maximum value of 2.98% at 11% RHA content. The variation of CBR with RHA can be seen in table below

Table 19: CBR value at different percentages of RHA

S.no	percentage of RHA	CBR(%)(Un soaked condition)
1	0	1.30
2	5	1.81
3	7	2.22
4	9	2.64
5	11	2.98
6	13	2.75
7	15	2.57

Table 19, represents the value of CBR value with different percentage of RHA content. When it is compared with the unblended soil the percentage in CBR value is improved by 39.23%, 70.76%, 103.07%, 129.23%, 111.53% and 97.69% at RHA content of 5%, 7%, 9%, 11%, 13% and 15%.

When the value of CBR is compared with that of unblended soil, it is found that with the percentage of RHA content, the CBR value are observed to be improved by 1.81%, 2.22%, 2.64%, 2.98%, 2.75% and 2.57%.

4.3.4. Free swell index IS:2720 (part XL) (1977):

Free swell index is determined by IS specification. According to the experimental investigation it is found that as the content increases from 5% to 15% the free swell index of the soil decreases.

From the figure 63, it can be seen with the increase in the percentage of RHA content, the value of free swell index is decreasing at a constant rate. Here it can be seen there is constant decrease in free swell index, which is due to the pozzolanic reaction between free amorphous silica in RHA and free lime in expansive soil, due to which heat of hydration generates, making water which otherwise coming in contact with soil is now processed in the reaction.

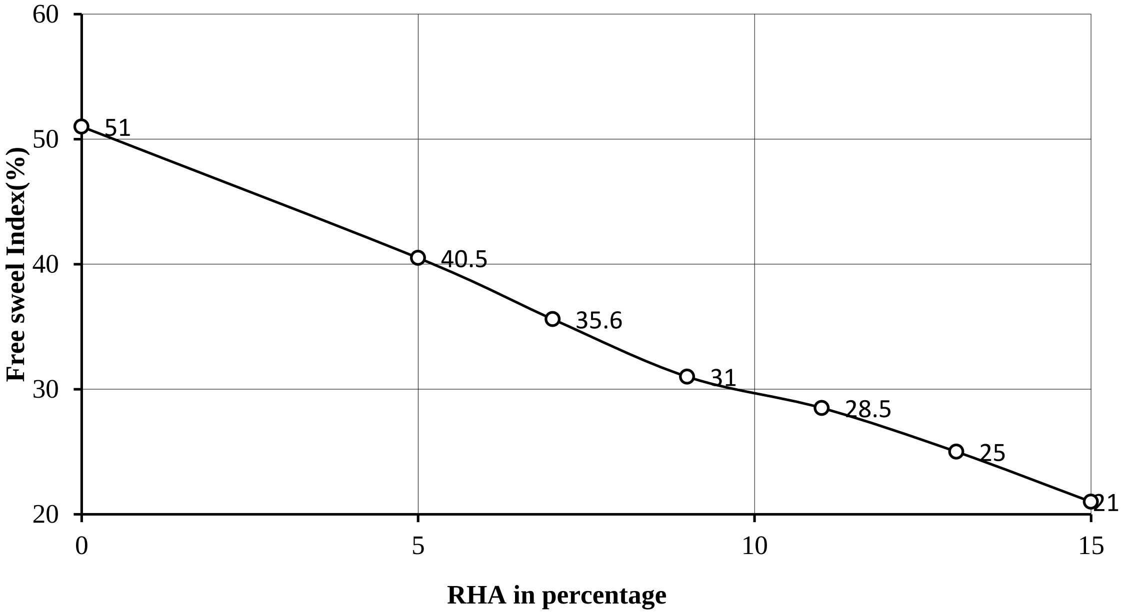

Figure 63: Variation of Free swell index with RHA in different percentage

Basically free swell index test is used in embankments, dams and canal like structure built on expansive soils where there is high amount of seepage of water, due to which the probability of failure of structure increases. The decrease of free swell index shown in table below:

Table 20: Free swell index at different percentages of RHA

S.No	Percentage of RHA	Free swell index (%)
1	0	51
2	5	40.5
3	7	35.6
4	9	31
5	11	28.5
6	13	25
7	15	21

Table 20, presents the value of free swell index of the samples with the varying percentage of RHA content. The decrease in the percentage of the free swell index value with respect to unblended soil is 20.58%, 30.19%, 39.21%, 44.11%, 50.98% and 58.82% at 5%, 7%, 9% , 11%, 13% and 15% RHA content respectively.

4.3.5. Direct shear test IS: 2720 (Part 11), 1983

Size of box=6cm*6cm*3cm   Area of box=36cm2

Mass of box + base plate + porous stones +grid plate=3250gm

Mass of box +base plate + porous stones +grid plate +soil specimen+ RHA=3500gm

Normal stress=50KN/m^2, 100KN/m^2, 150KN/m^2

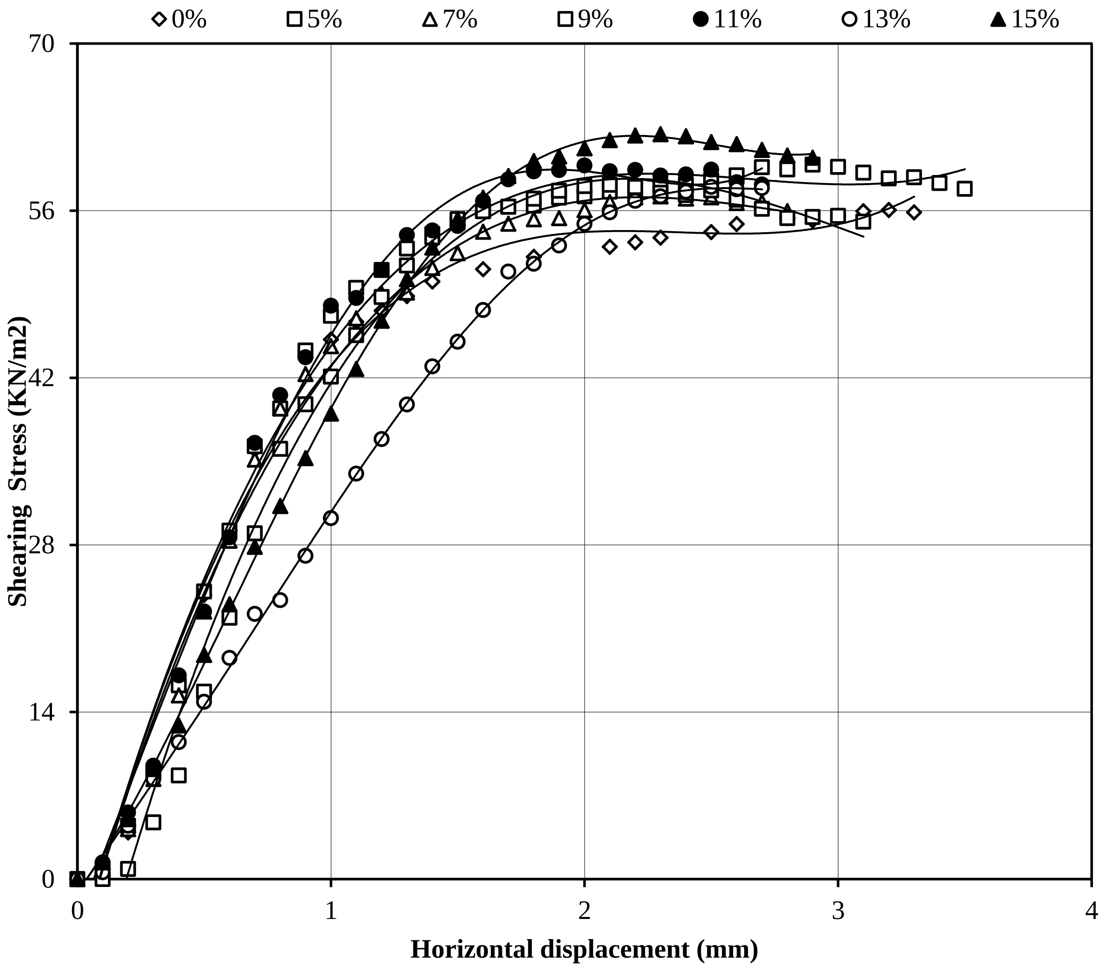

Figure 64: Variation of Shearing Stress with horizontal displacement at different percentages of RHA at normal stress 50KN/m^2

According to the figure 64, it can be seen that with the increase in the percentage of RHA content, there is not much increase in the shear stress of the composite samples. It can be seen that the maximum value of shear stress came at the percentage of 15% of RHA content. As the graph shows the shear stress corresponds to the normal stress of 50KN/m^2, there is increment in shear stress as compared to the untreated soil.

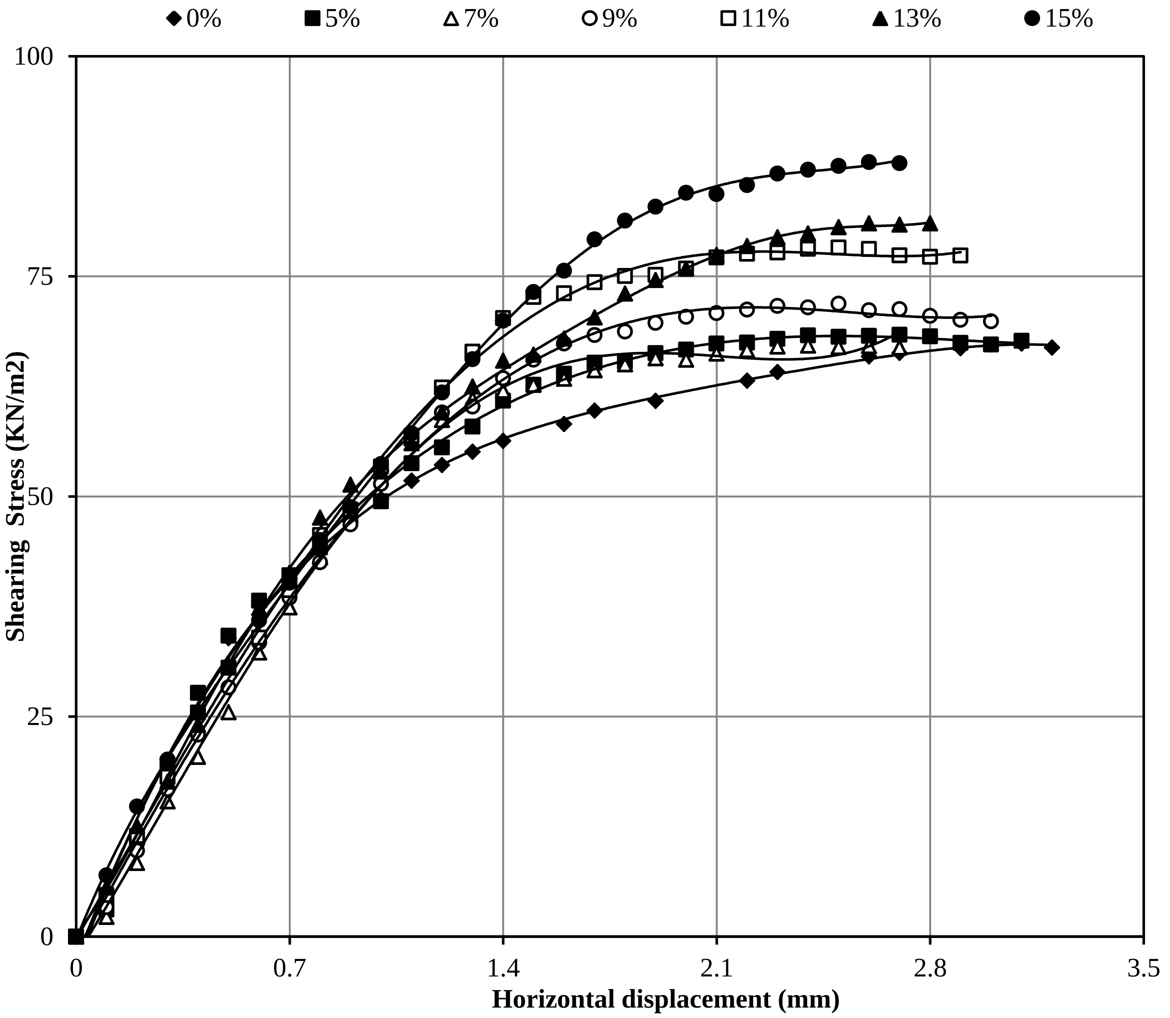

Figure 65: Variation of Shearing Stress with horizontal displacement at different percentages of RHA at normal stress 100KN/m^2

According to figure 65, it can be seen that there is increment in the shear stress of the composite samples blended with different percentage of RHA content. It can be seen at 5% RHA content, the shear stress increases, but at 7% RHA content, there can be seen a constant value and after that there is a increase in the shear stress with increase in RHA content up to 15%. All the samples are loaded at a normal stress of 100KN/m^2. It shows that the value at each point is more than shear stress generated at a normal stress of 50KN/m^2. So it shows that with increase in the vertical stress there is increase in the shear stress at each stage of horizontal displacement.

According to the figure 66, it can be seen that with the increase in the percentage of RHA content, the shear stress corresponding to horizontal displacement increases at a steady rate up to 15% RHA content. The maximum value corresponds to 15% RHA content. Here we can see that the value of shear stress corresponding to horizontal displacement at a normal stress of 150KN/m2 is more than the normal stress of 100 KN/m2 and 50 KN/m2.

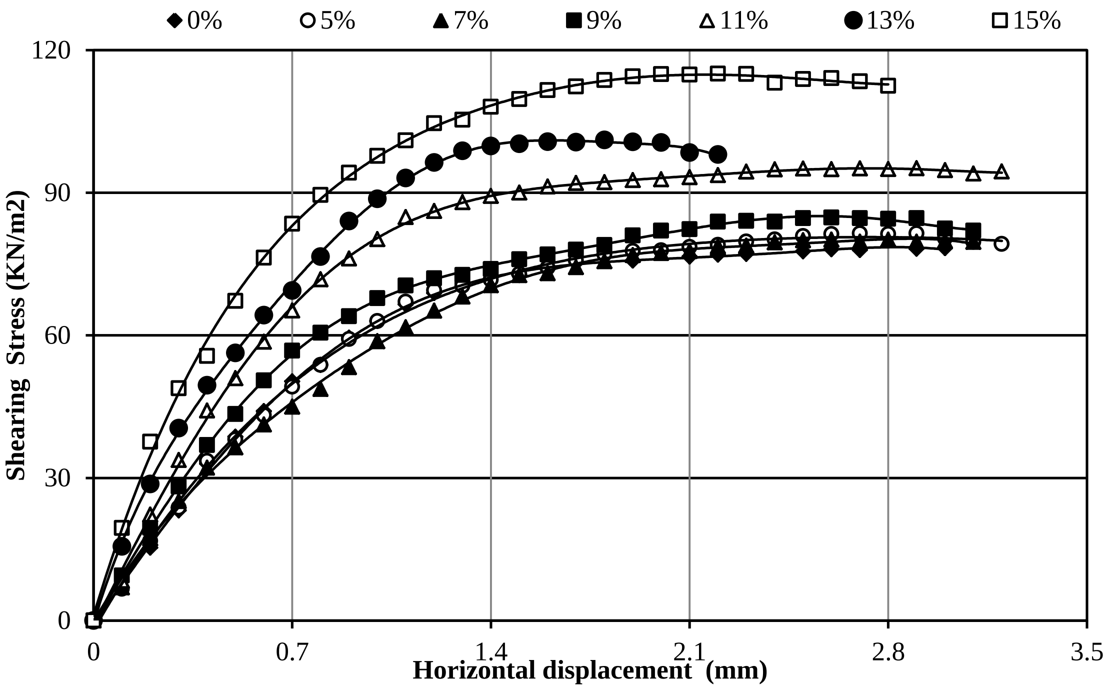

Figure 66: Variation of Shearing Stress with horizontal displacement at different percentages of RHA at normal stress 150KN/m^2

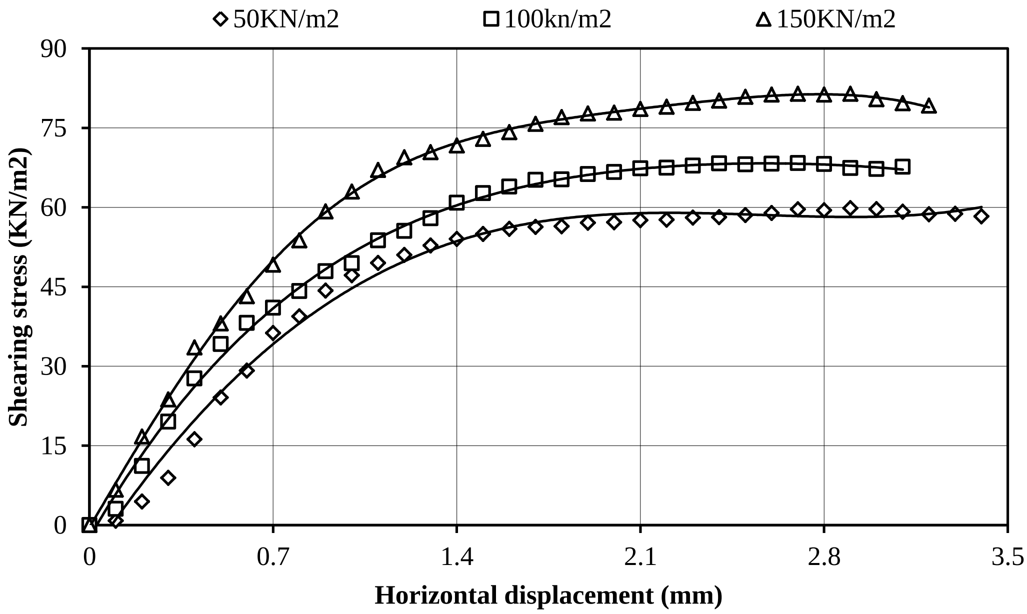

Figure 67: Variation of Shearing stress with horizontal displacement of soil blended with RHA at five percent at different normal stress

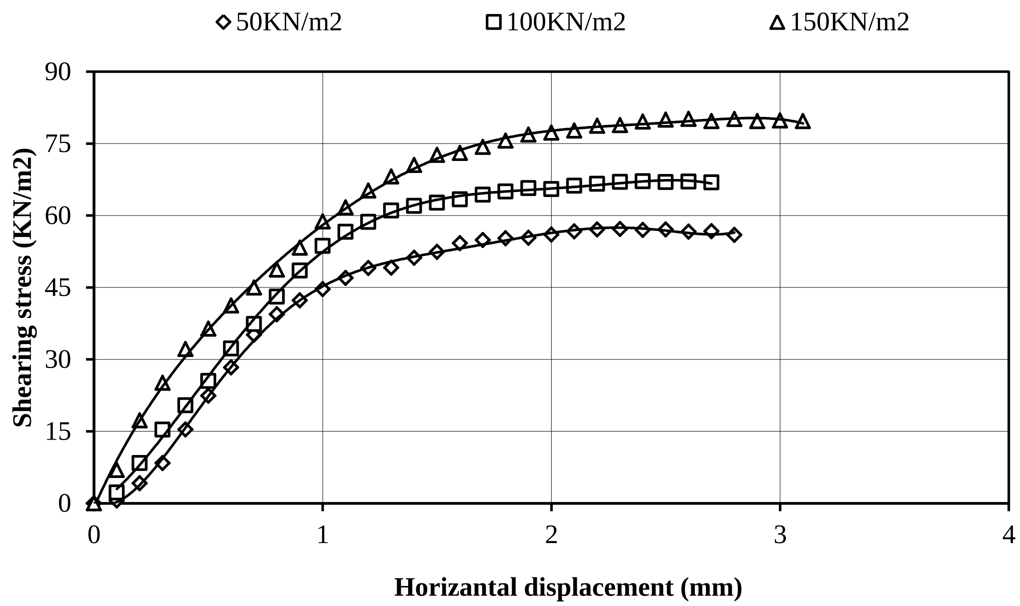

Figure 68: Variation of Shearing stress with horizontal displacement of soil blended with RHA at seven percent at different normal stress

Figure 68, shows that the shearing stress graph with respect to the horizontal displacement at different normal loads, it can be as the normal stress is increased the shearing stress increases. The normal stress of 50KN/m^2, 100 KN/m^2 and 150 KN/m^2 is been loaded. The shearing stress at different normal stress comes to 60.12 KN/m^2, 67.12 KN/m^2 and 81.5 KN/m^2 at a RHA of 7%.

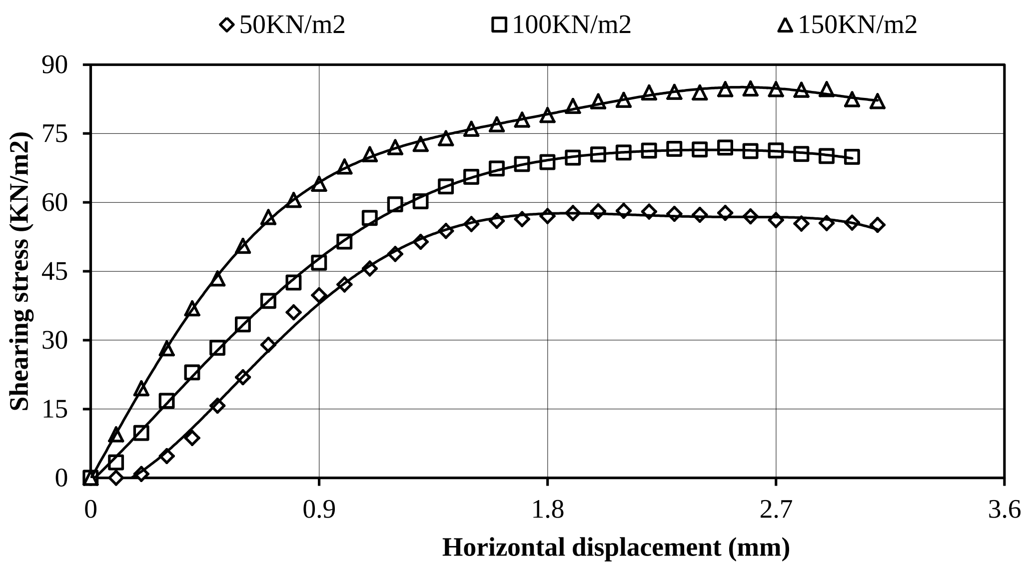

Figure 69: Variation of Shearing stress with horizontal displacement of soil blended with RHA at nine percent at different normal stress

Figure 69, shows that the shearing stress graph with respect to the horizontal displacement at different normal loads, it can be as the normal stress is increased the shearing stress increases. The normal stress of 50KN/m^2, 100 KN/m^2 and 150 KN/m^2 is been loaded. The shearing stress at different normal stress comes to 58.14 KN/m^2, 71.88 KN/m^2 84.64 KN/m^2 at a RHA of 9%.

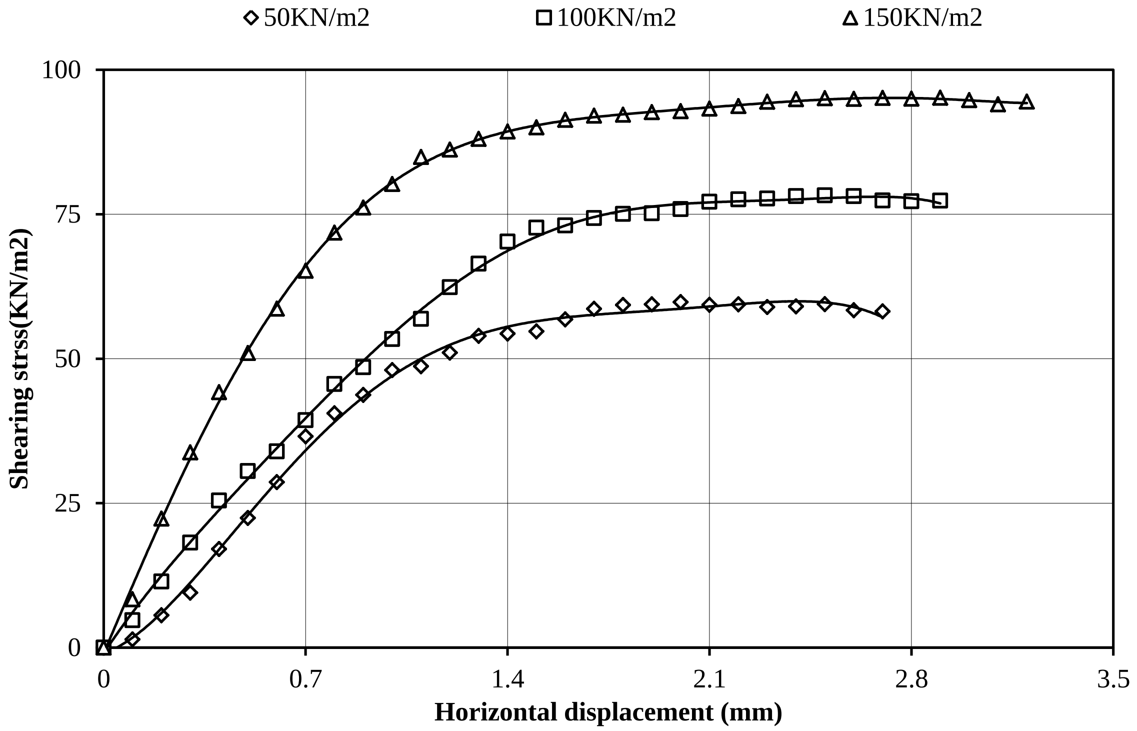

Figure 70: Variation of Shearing stress with horizontal displacement of soil blended with RHA at eleven percent at different normal stress

Figure 70, shows that the shearing stress graph with respect to the horizontal displacement at different normal loads, it can be as the normal stress is increased the shearing stress increases. The normal stress of 50KN/m^2, 100 KN/m^2 and 150 KN/m^2 is been loaded. The shearing stress at different normal stress comes to 59.07 KN/m^2, 78.26 KN/m^2 and 95.11 KN/m^2 at a RHA of 11%.

Figure 71, shows that the shearing stress graph with respect to the horizontal displacement at different normal loads, it can be as the normal stress is increased the shearing stress increases. The normal stress of 50KN/m^2, 100 KN/m^2 and 150 KN/m^2 is been loaded. The shearing stress at different normal stress comes to 57.97 KN/m^2, 81.01 KN/m^2 and 101.08 KN/m^2 at a RHA of 13%.

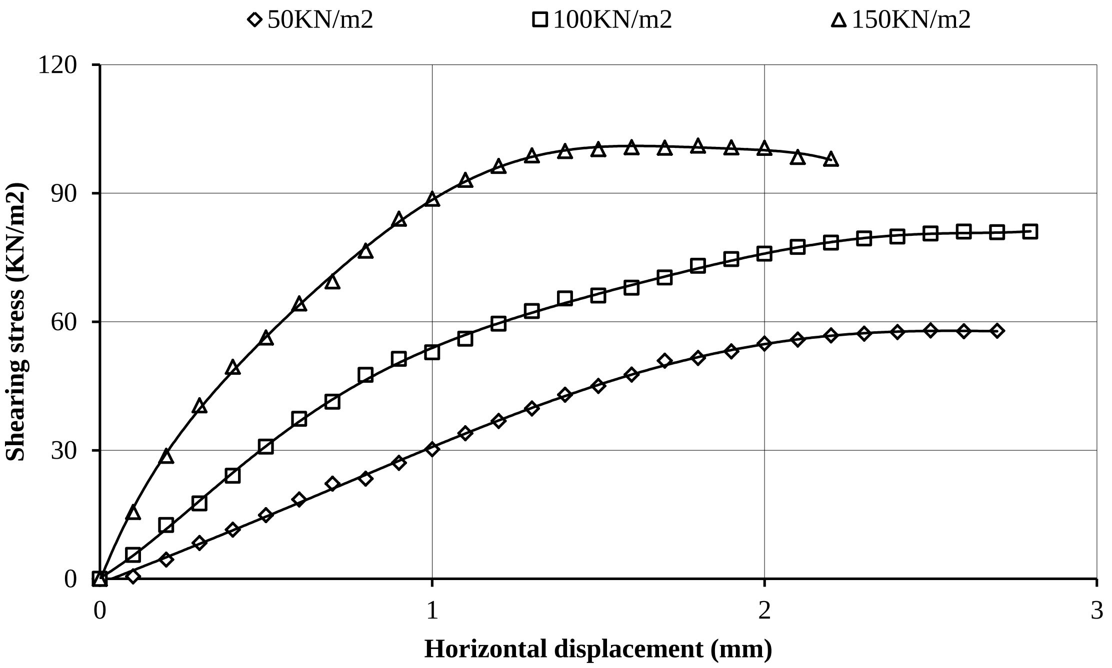

Figure 71: Variation of Shearing stress with horizontal displacement of soil blended with RHA at thirteen percent at different normal stress

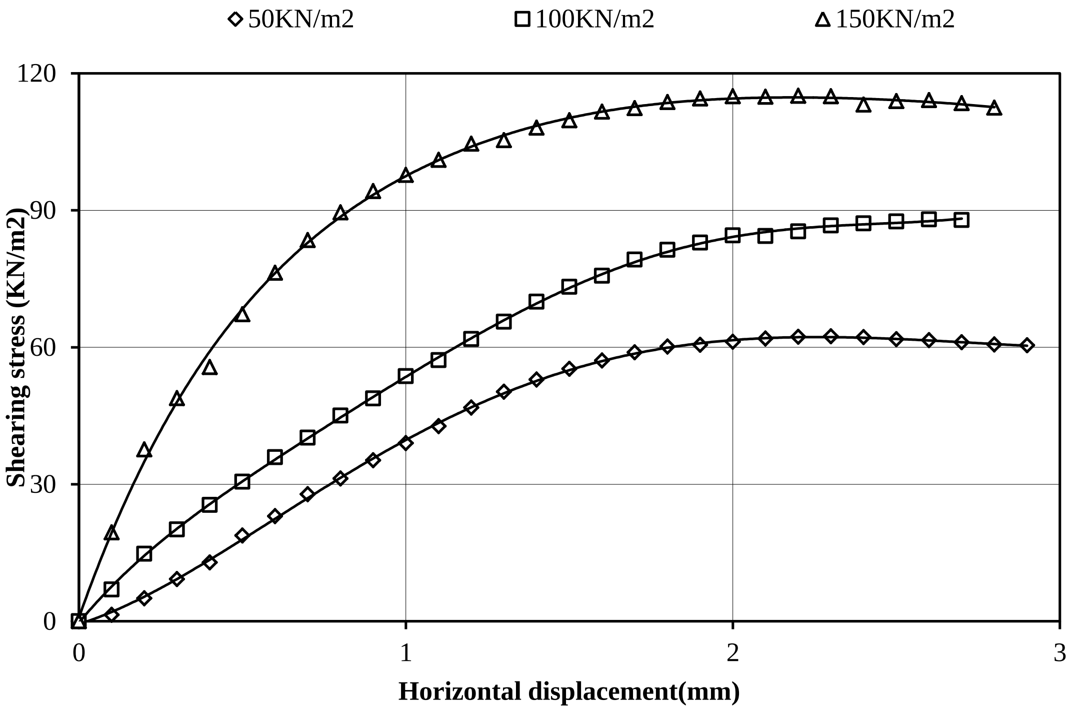

Figure 72: Variation of Shearing stress with horizontal displacement of soil blended with RHA at fifteen percent at different normal stress

Figure 72, shows that the shearing stress graph with respect to the horizontal displacement at different normal loads, it can be as the normal stress is increased the shearing stress increases. The normal stress of 50KN/m^2, 100 KN/m^2 and 150 KN/m^2 is been loaded. The shearing stress at different normal stress comes to 61.19 KN/m^2, 87.97 KN/m^2 and 115.05 KN/m^2 at a RHA of 15%. The Normal stress and shearing stress at different percentage of RHA are given below in table 21

Table 21: Shearing stress & Normal stress at different percentage of RHA content

Sample	Percent of RHA	Normal stress(KN/m2)	Shearing stress(KN/m2)
Soil blended with different percentage of RHA content	0	50	56.04
		100	67.36
		150	78.36
	5	50	59.64
		100	68.35
		150	81.43
	7	50	60.12
		100	67.12
		150	81.5
	9	50	58.14
		100	71.88
		150	84.64
	11	50	59.07
		100	78.26
		150	95.11
	13	50	57.97
		100	81.01
		150	101.08
	15	50	61.19
		100	87.97
		150	115.05

This table 21, shows the complete behavior of shearing stress with respect to the different percentage of RHA content.

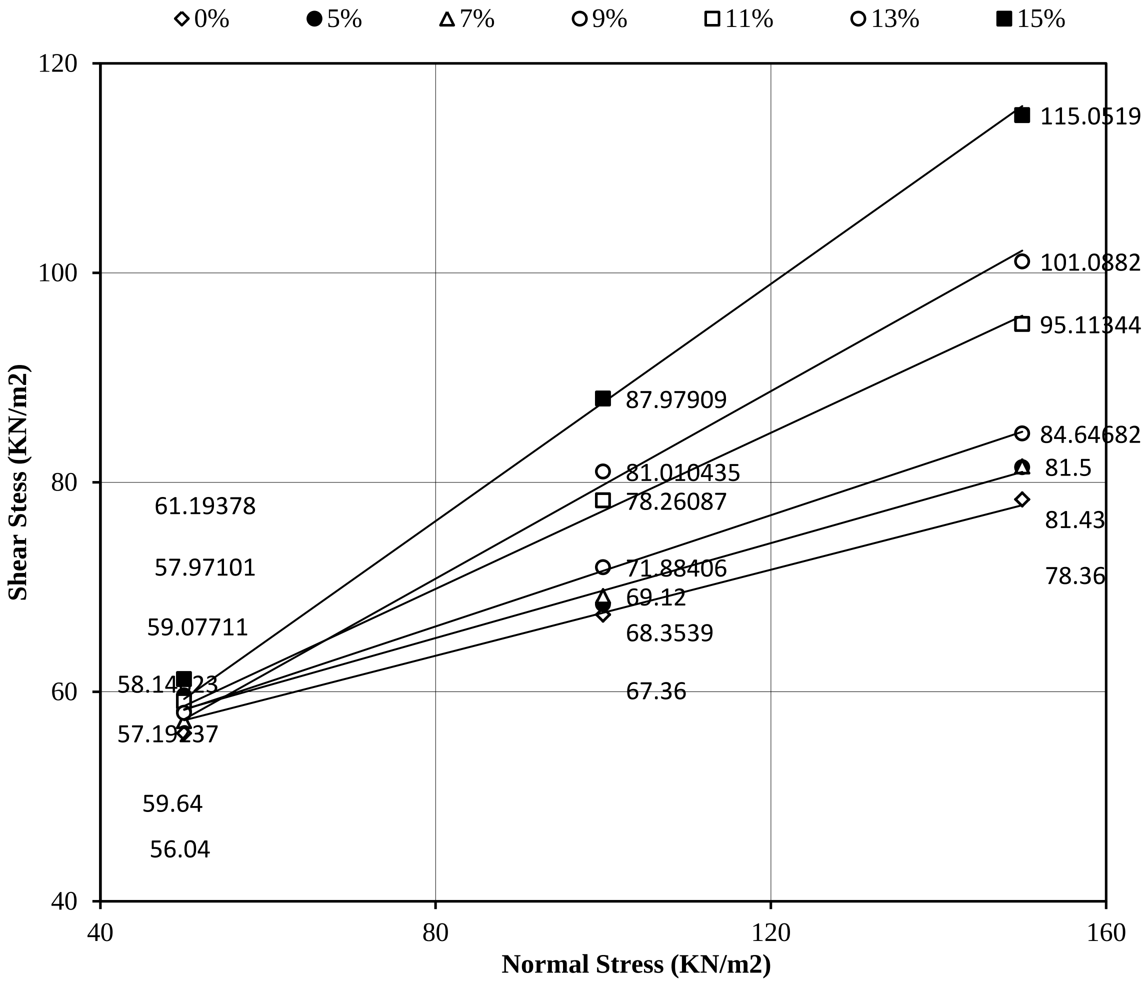

Figure 73: Variation of shear stress with normal stress of soil blended with RHA at different percentage

From the figure 73, it can be seen that with the increase in the percentage of RHA content the cohesion of soil starts decreasing. At 5% RHA content, it achieves the same value of cohesion as untreated soil, but after increasing the percentage of RHA, the cohesion starts to decrease with a downfall till 15%. Low cohesion makes RHA a poor cushioning and construction material. On the other hand, it can be seen that with the increase in the percentage of RHA content, the angle of internal friction increases rapidly. The reason behind must be silica content in RHA which act as a binder which agglomerate the particles into lager one and the soil changes from clay o silt.

The Equation of line, slope and intercept are given below in table:

Table 22: Equation of line, slope & intercept of line at different percentage of RHA

S.no	Percent of RHA	Equation of line	Slope=TanQ	Intercept (cohesion)KN/m2
1	0	Y=0.205x+47	0.205	47
2	0.25	Y=0.226x+47	0.226	47
3	0.50	Y=0.242x+45	0.242	45
4	0.75	Y=0.291x+42	0.291	42
5	1.00	Y=0.372x+40	0.372	40
6	1.25	Y=0.447x+35	0.447	35
7	1.50	Y=0.566x+31	0.566	31

The table 22 shows the value of cohesion and angle of internal friction in degree with varying percentage of RHA. The angle of internal friction and cohesion is found out by the graph between shear stress and normal stress with varying percentage of RHA. Simply we have to only draw an intercept line and slope which will calculate both cohesion and angle of internal friction. The value of cohesion comes to be 47KN/m, 47KN/m, 45KN/m, 42KN/m, 40KN/m, 35KN/m and 31KN/m, while the value for angle of internal friction comes in degree 11.58, 12.73, 13.60, 16.20, 20.40, 24.08 and 29.07 corresponding to the 5%, 7%, 9%, 11%,13% and 15% RHA content.

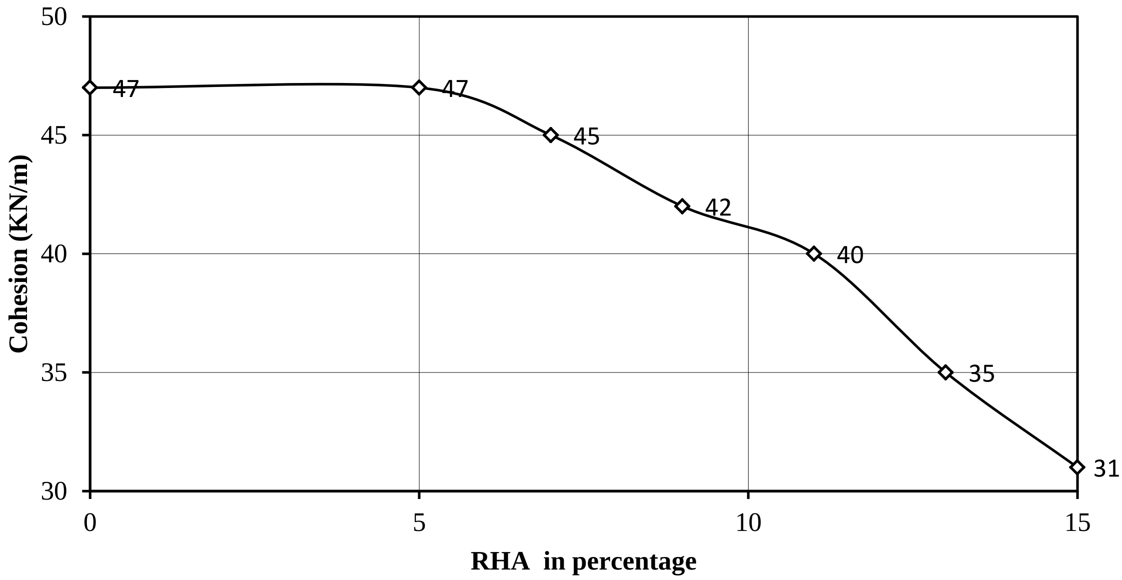

Figure 74: Variation of cohesion with RHA in different percentage

From the figure 74, it can see that the value of cohesion is constant up to 5% RHA, after then the cohesion starts decreasing at a constant rate with a minimum value at 15% RHA.

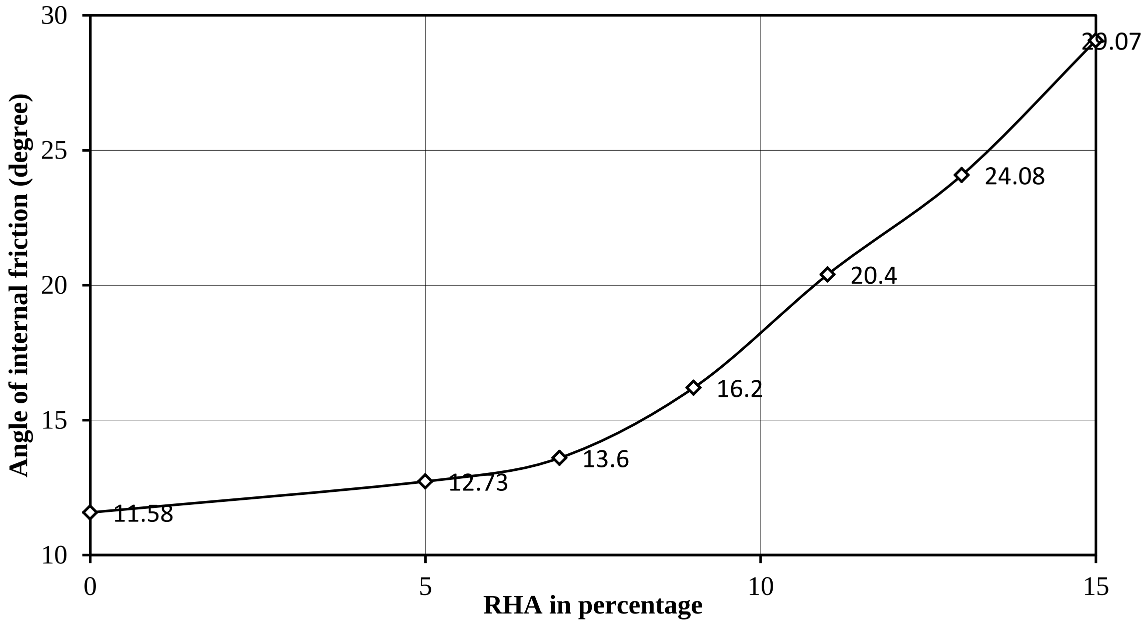

Figure 75: Variation of angle of internal friction with RHA in different percentage

From the figure 75, it can be seen that the angle of internal friction is likely to be constant till 5% RHA content, after that there is a upward slope or increase in the angle of internal friction with the addition of percentage of RHA.

Chapter 5
Comparison of results

5.1. Compaction test

According to the experimental investigation of both additives, it is seen in figure 76 that as the percentage of additives is increased the OMC of composite samples increases. It also shows that the OMC of soil with different percentage of microsilica is likely to have increased values as compare to the OMC values of RHA.

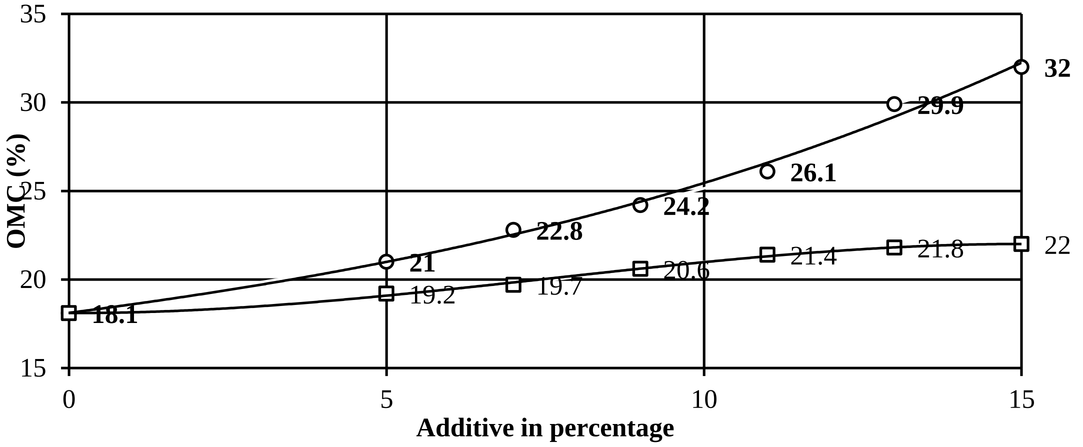

Figure 76: Variation of OMC with additives in different percentage

According to the experimental investigation of both the additives, it is seen in figure 77 that as the additive content is increased the maximum dry density of the soil decreases in both the cases of microsilica and RHA.

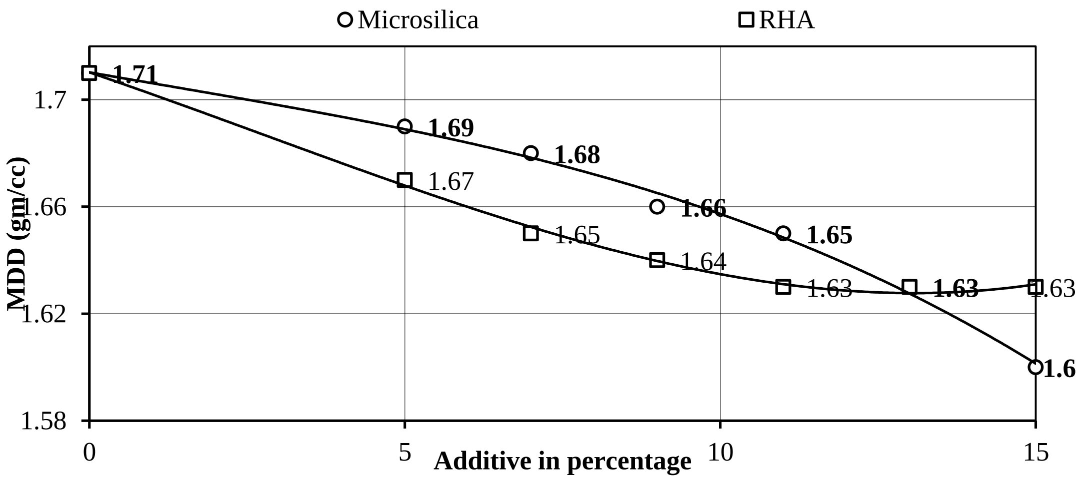

Figure 77: Variation of MDD with additives in different percentage

5.2. Unconfined compressive strength

According to the result, it can be seen in figure 78 that the Unconfined Compressive Strength of the soil increases with increase in microsilica and RHA content. The Unconfined Compressive Strengthat1 day curing period with different percentage of microsilica and RHA content can be seen in the graph and their variation, but of microsilica is more as compared with the additive RHA. Although, at one point the unconfined compressive strength of RHA comes out to be more than that of microsilica.

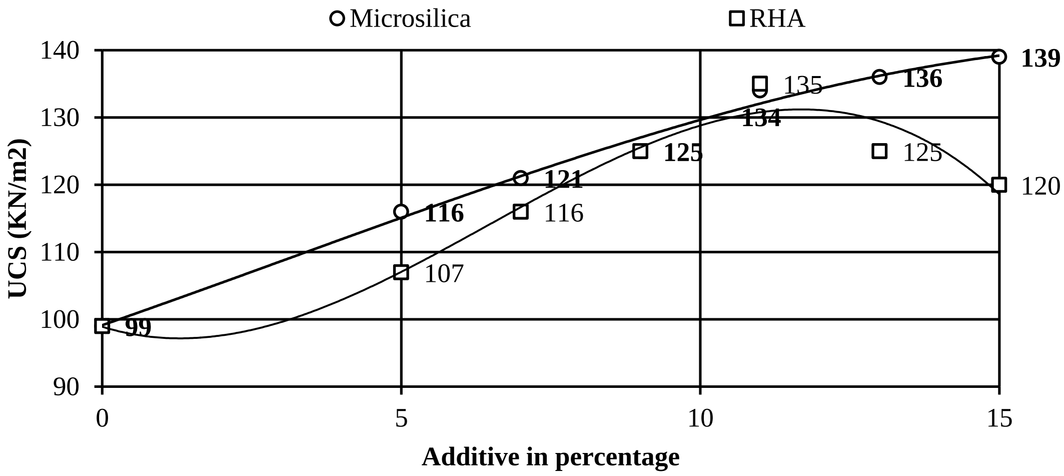

Figure 78: Variation of UCS with additives in different percentage at one day in curing period

According to the experimental results, it is found in figure 79 that the Unconfined Compressive Strength of soil at 7 days curing comes more than Unconfined Compressive Strength at one day curing period with blending the soil with additives microsilica and RHA.

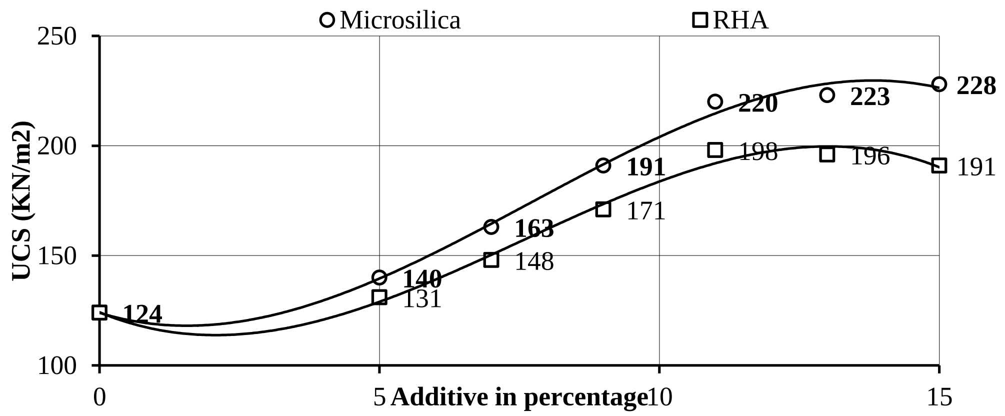

Figure 79: Variation of UCS with additives in different percentage at seven days in curing period

According to the experimental results, it is found in figure 80 that the Unconfined Compressive Strength of soil at fourteen days curing comes more than Unconfined Compressive Strength at seven days curing period with blending the soil with additives microsilica and RHA.

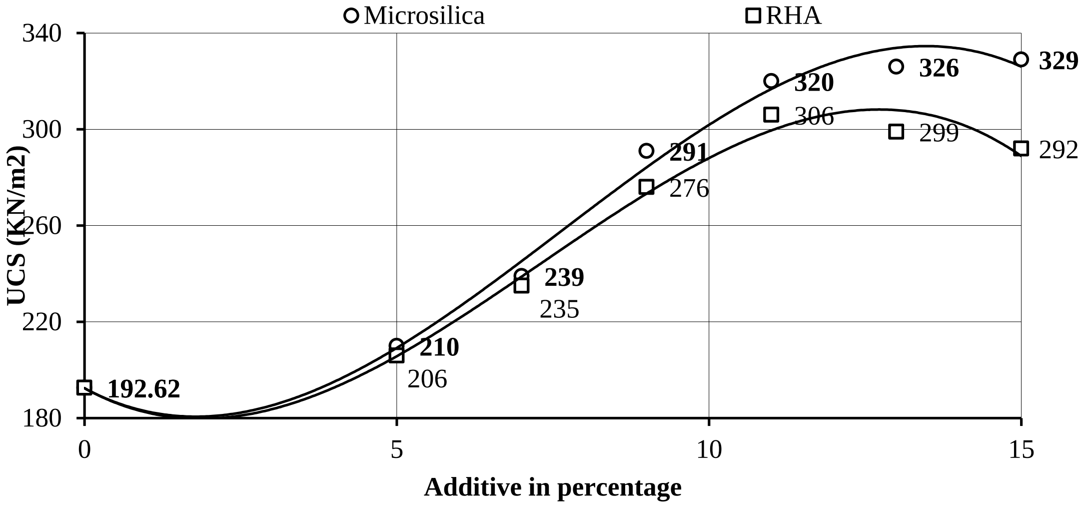

Figure 80: Variation of UCS with additives in different percentage at fourteen days in curing period

5.3. CBR test

CBR test is very important test to evaluate the strength of a sub-grade soil, sub-base and base course material for design of the thickness for road construction. According to the in the design of pavement, sub-grade. According to the experimental investigation, it can be seen in figure 81 that the CBR value of soil increases with increase in microsilica content, on the other hand the CBR value of soil with RHA content increases to maximum at 11%, then starts decreasing.

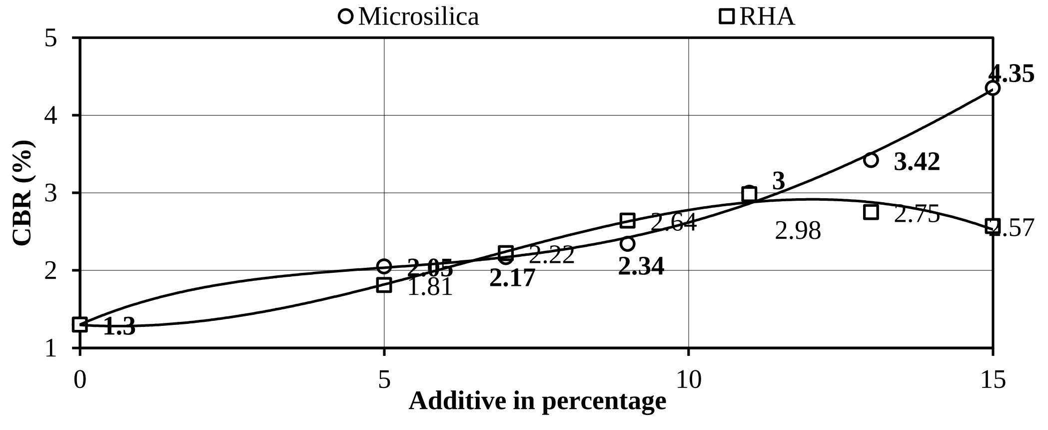

Figure 81: Variation of CBR with additives in different percentage

5.4. Free swell Index

According to experimental investigation it is seen in figure 82 that as the percentage of additives(microsilica and RHA) is increased the Free swell index decreases.

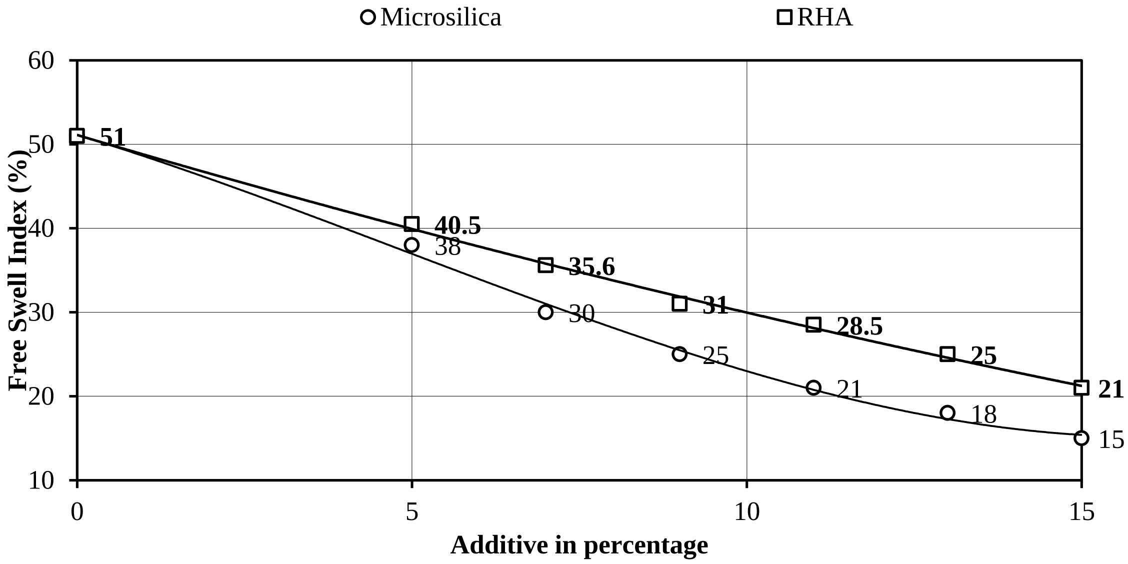

Figure 82: Variation of free swell index with additives in different percentage

5.5. Direct Shear Test

According to experimental investigation, it is seen in figure 83 that the shear stress corresponding to normal stress at 50kN/m^2 with additive microsilica is high as compared to that of RHA.

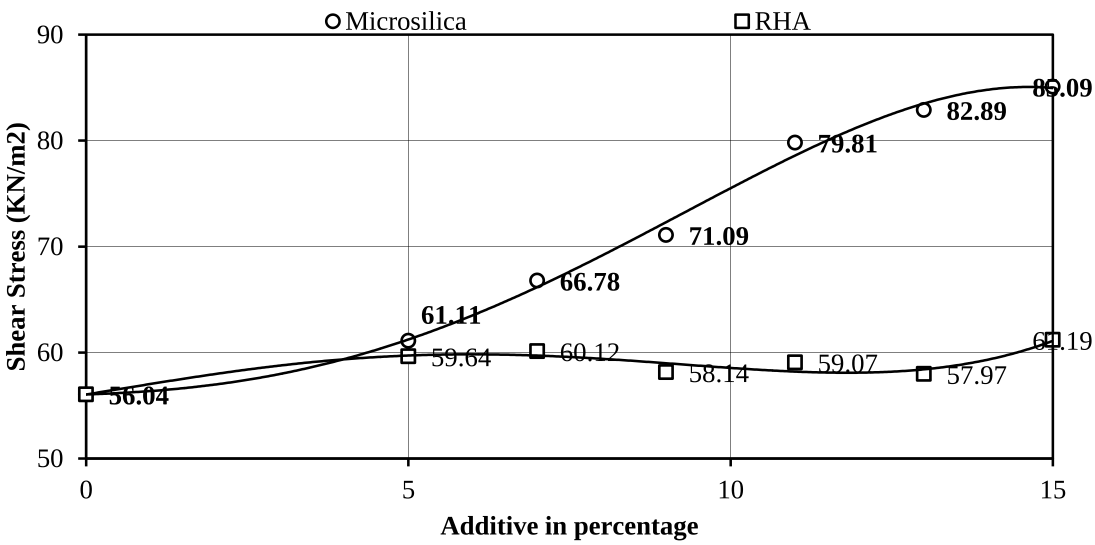

Figure 83: Variation of shear stress with additive in percentage for a normal stress of 50kN/m^2

According to experimental investigation, it is seen in figure 84 that the shear stress corresponding to normal stress at 100kN/m^2 with additive microsilica is high as compared to that of RHA.

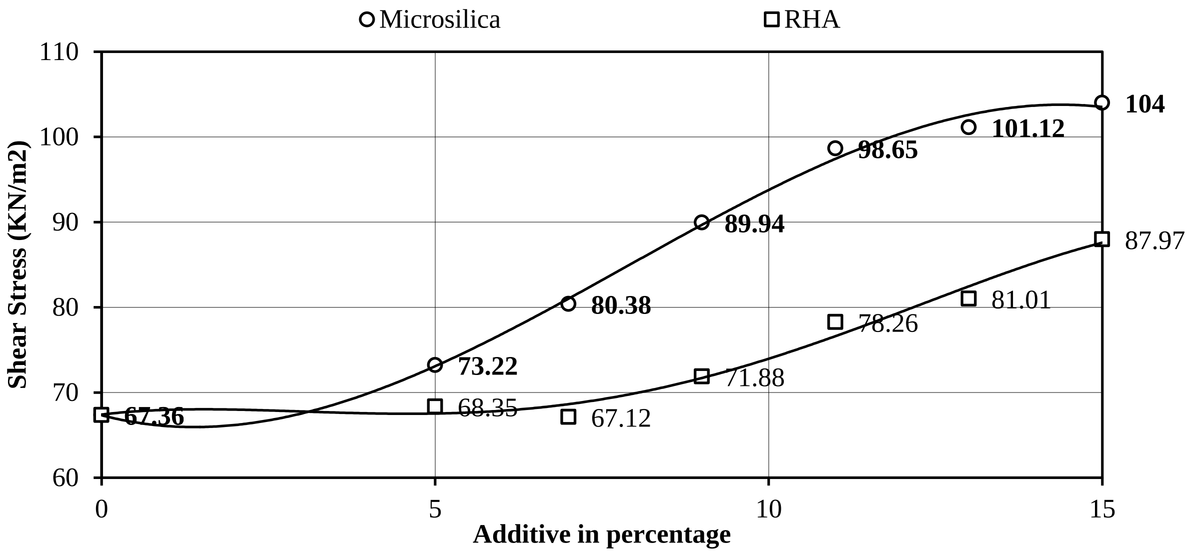

Figure 84: Variation of shear stress with additive in percentage for a normal stress of 100 kN/m^2

According to experimental investigation, it is seen in figure 85 that the shear stress corresponding to normal stress at 150kN/m^2 with additive microsilica is high as compared to that of RHA.

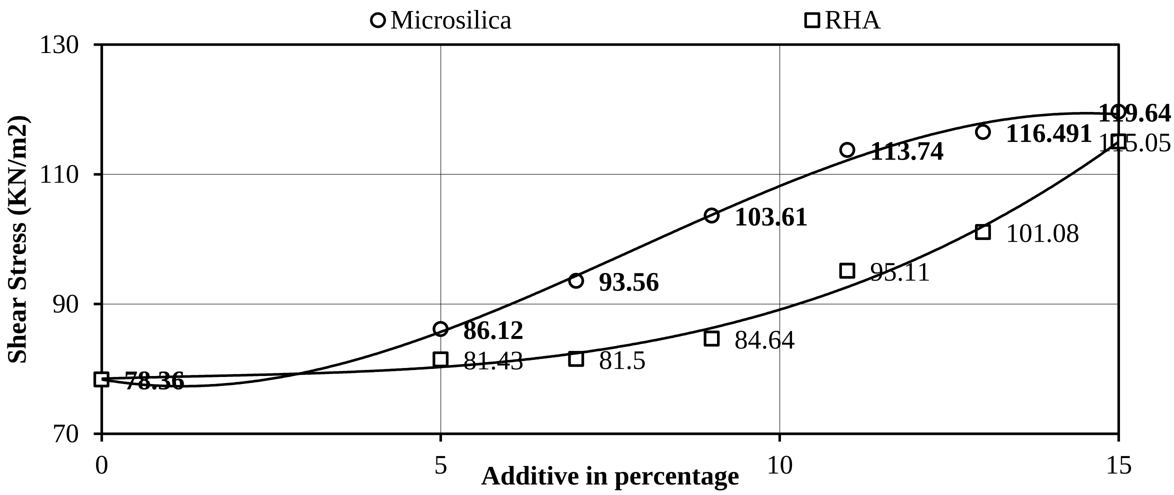

Figure 85: Variation of shear stress with additives in different percentage for a normal stress of 150 kN/m^2

According to experimental investigation, it is seen in figure 86 that the cohesion with additive microsilica is high as compared to that of RHA.

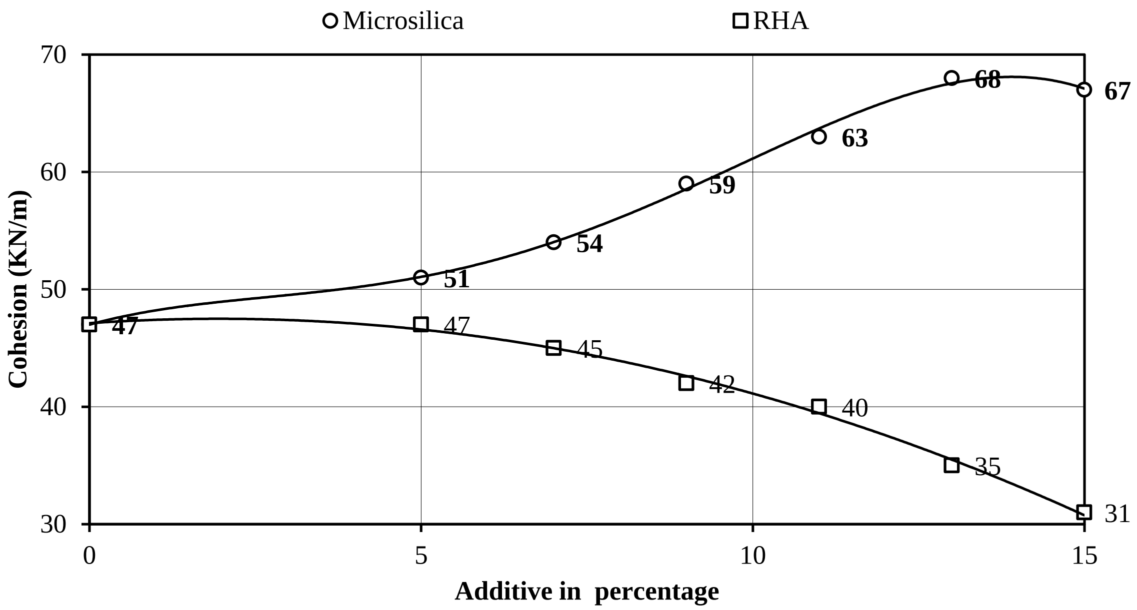

Figure 86: Variation of cohesion with additive in different percentages in soil

According to experimental investigation, it is seen in figure 87 that the angle of internal friction with additive microsilica is low as compared to that of RHA.

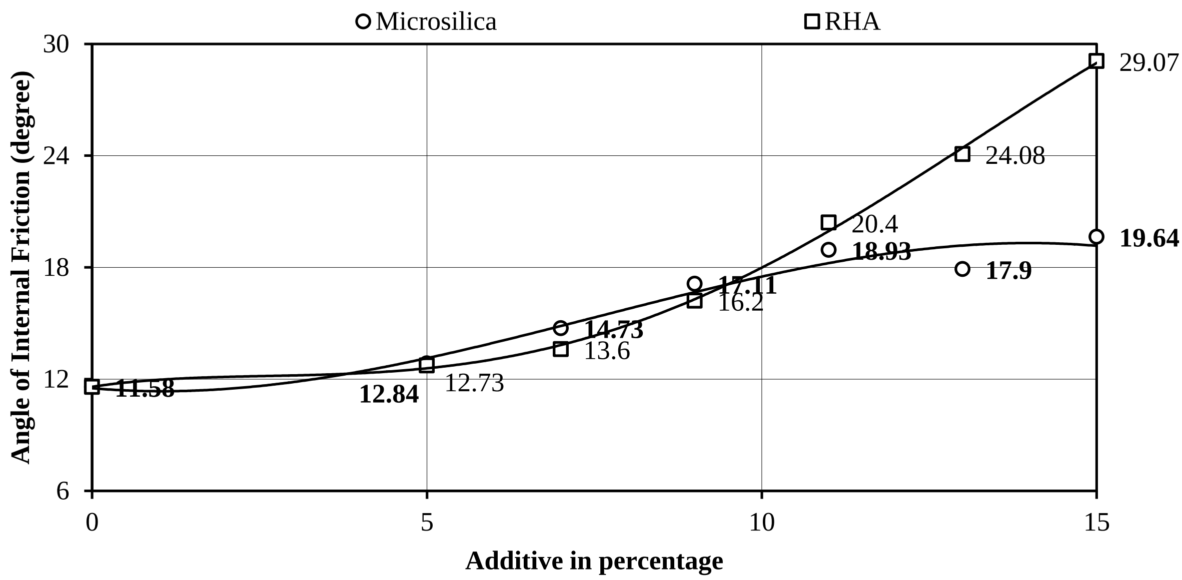

Figure 87: Variation of angle of internal friction with additive in different percentages in soil

Chapter 6
Conclusion

6.1 Conclusion

1. The standard proctor test indicates that OMC of soil with additive microsilica comes more than that of OMC of soil with additive RHA. This shows that the additives especially microsilica imbibes more water to achieve MDD. Further compaction test results shows that the value of MDD of soil for microsilica comes to be less than that for RHA, it can be due to the good interlocking of the RHA particles in the soil to achieve the MDD soon.
2. When the microsilica content was increased from 0 to 15%, the UCS of soil increased by 70.80%, while in RHA it increased by 58.86% from 0 to 11% after then it starts decreasing. So it shows that with the addition of microsilica content there comes no optimum value up to 15%, while that in RHA the optimum value comes to be 11% in soil.
3. When the microsilica content was increased from 0 to 15%, the CBR of soil increased by 234.61%, while in RHA it increased by 129.23% from 0 to 11%, after then it starts decreasing, resulting in an optimum value of 11%.
4. The optimum value in case of RHA in UCS and CBR comes to be at 11%, while in case of microsilica there comes no decreasing value from 0 to 15%, so it means that microsilica can give much more strength if its content is increased.
5. The degree of expansiveness of soil tends to decrease by 70.58% when blended with microsilica from 0 to 15%, while in case of RHA this value comes to be 58.82%, it shows that the microsilica decreases the swelling characteristics of soil more than that of RHA.
6. The shear strength of soil when blended with microsilica comes to be more than that of RHA. The value of cohesion for soil when blended with microsilica increases by 42.55% from 0 to 15%, while that in RHA the value of cohesion decreases to a value of 34.04% from 0 to 15% it is due to the low cohesion of RHA. While the angle of friction increases for RHA and microsilica both.

7. With this study and the above points it is concluded that microsilica is a better additive in the soil to impart more strength and stability as compared to additive RHA in soil.

6.2 Scope of future work

1. Other engineering properties such as consolidation and triaxial test should be considered so that much more deep study can be done.
2. The CBR test in soaked condition should be carried out to study the strength of expansive soil in soaked condition.
3. Liquefaction properties can be studied out.
4. Combination of different additives mixed with microsilica and RHA can be carried out.

References

1. Kariuki, P. C. (2004), Spectroscopy and swelling soils – An integrated approach, Department of Applied Science, Delf University of Technology in the Netherlands.
2. Jones, D. E., and Holtz, W. G. (1973), Expansive soils – the hidden disaster. Civil Engineering- ASCE, 43(8), 49-51.
3. Chen, F. H. (1988), Foundations on expansive Soils, Elsevier Science Publishers B. V.
4. Meehan, R. L. and Karp, L. B. (1994), California housing damage related to expansive soils, Journal of Performance of Constructed Facilities. 8, no. 2, 139-157.
5. Day, R.W(1999), Geotechnical and foundation engineering design and construction, McGraw-Hill Companies, New York.
6. Patrick, D. M and Snethen, D. R. (1976), An occurrence and distribution survey of expansive materials in the United States by physiographic areas, Interim Report, Federal Highway Administration Office of Research and development, Washington, D.C. 20590
7. Ingles, O.G., Metcalf, J.B., 1972. Soil Stabilization, Principles and Practice, John Wiley &Son's, Sydney, Australia.
8. Al-Zoubi,M.S./ Swell Characteristics of Natural and Treated Compacted Clays /,The Electronic Journal of Geotechnical Engineering,Vol.13,Bund.D.(2008)
9. Fattah, M.Y., Salman, F.A. and Bestun, J. N., (2010). "A Treatment of Expansive Soil using Different Additives", ActaMontanisticaSlovaca, Ročnik 15, čislo 4, pp. 290-297.
10. Ali,M.S. and Koranne,S.S., (2011)."Performance Analysis of Expansive Soil Treated with Stone Dust and Fly Ash", EJGE, Vol.16, Bund,I, pp. 973-982.
11. Ramadas, T.L., Kumar,N.D. and Yesuratnam, G., (2011)." Geotechnical Characteristics of Three Expansive Soils Treated with Lime and Fly Ash", International Journal of Earth Sciences and Engineering, Vol. 4, pp. 46-49. ISSN 0974-5904.
12. Abdullah,W.S. and Alsharqi, A.S., (2011)." Rehabilitation of Medium Expansive Soil Using Cement Treatment", Jordan Journal of Civil Engineering, Vol.5, No.3, pp. 343-356.
13. Puppalaa, A.J. and Musenda, C., (2002). "Effects of Fiber Reinforcement on Strength and Volume Change in Expansive Soils", Transportation Research, pp. 134-140. Record No.00-0716.
14. Moayed, R.Z., Heiderari, S. and Sayadi, M.: "Stabilization of saline soil with

lime and silica fume", 2nd International Conference on New Developments in Soil Mechanics and Geotechnical Engineering, Near East University, Nicosia, North Cyprus, 2009.

15. Makarchian, M. and Roshanomid, H.: "The effect of gypsum on the clay stabilized with lime and silica fume". Building Research Technical Paper, Vol. 55, pp.15-20, 2007.
16. Abd El-Aziz, M.A., Abo-Hashema, M.A. and El-Shourbagy, M.:"The effect of lime-silica fume stabilizer on engineering properties of clayey sub grade", Engineering Conference, Faculty of Engineering, Mansoura University, Paper No. 96, Sharm, Holiday Inn Hotel, 2004.
17. Halstead, W. J. (1986). "Use of Fly Ash in Concrete." Rep. No. NCHRP Report 127, Transportation Research Board, Washington, D.C
18. Saranjeet Rajesh Soni *et al. “Disposal of solid waste for black cotton soil stabilization”,International Journal of Advanced Engineering Sciences and Technologies Vol No. 8,Issue No. 1,113 -120.
19. Dr. Adel A. Al-Azzawi, Khalida A. Daud ,Muhammed A. Abdul Sattar “Effect of Silica Fume Addition on the Behavior of Silty-Clayey Soils” Journal of Engineering and Development, Vol. 16, No.1, March 2012.
20. VenuGopal.N,Project report on “Study of Soil Properties with Silica Fume as Stabilizer and Comparing the same with RBI-81 and Cost Estimation”,PG diploma in Highway Engineering,Visvesvaraya Technological University,Belgaum.
21. Biswas, etal. (2012) .“Utilization of Rice husk with Lime in Subgrade Soil for a Rural Road.”International Conference on Emerging Frontier in Technology for Rural Area.
22. Sabat, A.K., Nanda,R.P. (2011). “Effect of marble dust on strength and durability of Rice husk ash stabilized expansive soil.”International Journal of Civil and Structural Engineering,Volume 1,No 4, pp.-939-948.
23. Kalkan,E. and Akbulut,S., “The Positive Effects of Silica Fume on the Permeability,Swelling Pressure and Compressive Strength of Natural Clay Liners”,Journal of Engineering Geology,Vol. 73,2004,pp. 145-156.
24. M Karimi& Ali G.“Stabilization of silty sand soils with lime and micro silica admixture in pressure of sulphates”.Pan –Am CGS ,Geotechnical Conference 2011.
25. IshraqKhudhairAbass “Studying some of the Geotechnical Properties of Stabilized Iraqi Clayey Soils” , Eng. & Tech. Journal, Vol.31,Part (A), No.6, 2013 Tech. ,1117-1130 Journal.

26. Taylor, M.R., Lydon, F.D, Barr, B.I.G. 1996. Mix Properties for High Strength Concrete Construction and Building Materials. Volume 10, Issue6,September 1996, Pages 445-450
27. Das, B.M. 1999. Principle of Foundation Engineering. 4th Edition, Cole Company, California, USA.
28. National lime association. 2006. Technical Brief. Lime, the Versatile Chemical.
29. Beagle, E. C. (1978). "Rice husk conversion to energy." Rep. No. 37, FAO Agricultural Services Bulletin, Rome, Italy.
30. Velupillai, L., Mahin, D. B., Warshaw, J. W., and Wailes, E. J. (1997). "A Study of the Market for Rice Husk-to-Energy Systems and Equipment." Rep. No. Louisiana State University Agricultural Center, USA.
31. Mehta, P. K., 1986, “Concrete structure, properties and materials”, Prentice Hall, Englewood Cliffs, N.J.
32. Ramakrishna, A.N. and Pradeep Kumar, A.V., 2008, “Effect of moisture content on strength behaviour of BC soilricehusklime mixes.” IndianHighways, 36(1), pp.4958.
33. Payá, J., Monzó, J., Borrachero, M.V., Mellado, A., Ordoñez L.M., Determination ofamorphous silica in rice husk ash by rapidanalytical method, Journal of Cement and Concrete Research, Vol. 31, 2001, pp. 212– 231..
34. Della, V. P., Knhn, I., and Hotza, D. (2002). "Rice husk ash as an alternate source for active silica production." Materials Letters, 57(4), 818-821.
35. Ganesan, K., Rajagopal, K., and Thangavel, K. (2008). "Rice husk ash blended cement: Assessment of optimal level of replacement for strength and permeability properties of concrete." Construction and Building Materials, 22(8), 1675-1683.
36. Rahman, M. A. (1987). "Properties of clay-sand-rice husk ash mixed bricks." Internationa Journal of Cement Composites and Lightweight Concrete, 9(2), 105-108.
37. Gromko, G.J., Review of expansive soils,Journal of Geotecnical Engineering Division, ASCE, Vol. 100, No. 6., 1974, pp. 667–687.
38. Qijun Yu., Sawayama, K., Sugita, S., Shoya,M., Isojima, Y., The reaction between ricehusk ash and Ca(OH)2 solution and thenature of its product, Journal of Cement and Concrete Research. Vol. 29 (1), 1999, pp. 37–43.
39. Balasubramaniam, A.S., Lin, D.G., Acharya, S.S.S., Kamruzzaman, A.H.M., Behaviour ofsoft Bangkok clay treated with additives,Proceeding of 11th Asian Regional

Conferenceon Soil Mechanic and Geotechnical Engineering. Seoul. Vol. 1: 11–14, 1999.

40. Chapman, H.D., "Cation Exchange Capacity in Methods of soil analysis", American society of Soil Agronomy, 1965, C.A. Black et al., Eds., Madison, WI, pp.891-901.
41. Kate, J.M. and Katti, R.K., "Effect of CNS layer on the behavior of underlying expansive soils media: an experimental study", 1980, Indian Geotechnical Journal, 281-305.
42. Ito, K. K, Senge, M., Adomako, J. T., and Afandi, (2008), "Amendment of Soil Physical and Biological Properties Using Rice Husk and Tapioca Wastes", Journal of Japnanese Society of Soil Physics, No. 108, pp. 81-90.
43. Brooks, R. M., (2009), "Soil Stabilization with Fly ash and Rice Husk Ash", International Journal of Research and Reviews in Applied Sciences, Volume 1, Issue 3, pp. 209-217.
44. Yadu, L., Tripathi, R. K., Singh, D., (2011), "Comparison of Fly Ash and Rice Husk Ash Stabilized Black Cotton Soil", International Journal of Earth Sciences and Engineering, Vol. 04, No 06 SPL, pp. 42-45.
45. Koteswara, R. D., Pranav, P. R. T., and Anusha, M., (2011), "Stabilization of Expansive Soil with Rice Husk Ash, Lime and Gypsum –An Experimental Study", International Journal of Engineering Science and Technology, Vol. 3 No. 11, pp. 8076 – 8085.
46. Mtallib, M. O. A., and Bankole, G. M., (2011), "The Improvement of the Index Properties and Compaction Characteristics of Lime Stabilized Tropical Lateritic Clays with Rice Husk Ash (RHA) Admixtures", Electronic Journal of Geotechnical Engineering, Vol. 16, Bund. I, pp. 984-996.
47. Muntohar, S., and Hantoro, G., (2000), "Influence of Rice Husk Ash and Lime on Engineering Properties of a Clayey Sub-grade", Electronic Journal of Geotechnical Engineering, Vol. 5,pp 1-9, 2000
48. Osinubi, K.J. and Katte, V.Y., (1997), "Effect of Elapsed Time after Mixing on Grain Size and Plasticity Characteristic, I: Soil-Lime Mixes", NSE Technical Transactions Vol. 32, No. 4, pp 88-96, 1997
49. Osula D. O. A., (1991), "Lime Modification of Problem Laterite", Engineering Geology, Vol. 30, pp. 141-149.
50. Sarkar, Raju., Abbas, S.M., and Sahu, J.D. (2011) "Geotechnical Characterization Of Pond Ash Avalaible in National Capital Region –Delhi" International Journal of Earth Sciences and Engineering Volume 04, No 06 SPL, pp. 138-142.

51. Gupta, Ashok., Gupta, Shilpi., and Negi, Prateek.(2013) "Geotechnical Behaviour of Fine Sand Miced with Pond Ash and Lime" International Journal of Scientific & Engineering Research, volume 4, Issue 4, April-2013 281