SOIL STABILIZATION BY USING MUNICIPAL SOLID WASTE ASH

Submitted in partial fulfilment of the requirements
for the degree of
Bachelor of Engineering

Mr. Khan Asif Hasib	14CES14
Mr. BelimNavaidYakub	15DCES61
Mr. Khan Mashrakain Ibnain	15DCES72
Mr. Siddiqui Ashraf Mohammad Rafique	15DCES94

Under Guidance of
Prof. Umesh Jadhav

Department of Civil Engineering,
School of Engineering and Technology

Anjuman-I-Islam's Kalsekar Technical Campus
Plot No. 2 3, Sector – 16, Near Thana Naka, KhandaGaon,
New-Panvel, Navi-Mumbai. 41026.

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CERTIFICATE

This is to certify that the project entitled **"Soil Stabilization By Using Municipal Solid Waste Ash"**is a bonafide work of **Mr. Khan Asif Hasib (14CES14), Mr. Belim Navaid Yakub (15DCES61), Mr. Khan Mashrakain Ibnain (15DCES72) , Mr. Siddiqui Ashraf Mohammad Rafique, (15DCES94),** submitted to the University of Mumbai in partial fulfilment of the requirement for the award of the degree of "Undergraduate" in "Civil Engineering"

Prof. Umesh Jadhav
Name of Supervisor/Guide
(Supervisor)

Dr. R. B. Magar
(Head of Department)

Dr. Abdul RazakHonnutagi
(Director, AIKTC)

PROJECT REPORT APPROVAL FOR B.E

This project report entitled "**Soil Stabilization Using Municipal Solid Waste Ash**" is a bonafide work of "**Mr. Khan Asif Hasib (14CES14), Mr. Belim Navaid Yakub (15DCES61), Mr. Khan Mashrakain Ibnain (15DCES72), Mr. Siddiqui Ashraf Mohammed Rafique (15DCES94).**"is approved for the degree of "**Bachelor of Engineering**" in "**Department of Civil Engineering**''

Examiners

1. …………………………

2. …………………………

Supervisors:

1. …………………………

2. …………………………

Date: __________

Place:- Panvel

DECLARATION

We declare that this written submission represents my ideas in our own words and where others ideas or words have been included, we have adequately cited and referenced the original sources. We also declare that, we have adhered to all principles of academic honesty and integrity and have not misrepresented or fabricated or falsified any idea/data/fact/source in our submission. I understand that any violation of the above will be cause for disciplinary action by the Institute and can also evoke penal action from the sources which have thus not been properly cited or from whom proper permission has not been taken when needed.

Mr. Khan Asif Hasib (14CES14)

Mr. Belim Navaid Yakub (15DCES61)

Mr. Khan Mashrakain Ibnain (15DCES72)

Mr. Siddiqui Ashraf Mohammed Rafique (15DCES94)

Date:

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ABSTRACT

This report presents the analysis of soil using municipal waste ash in soil stabilization for different engineering properties. As solid waste disposal being a major environmental problem, because of unavaiblity of lands for disposal. Due to fast growth in construction industry, land is not available as per construction requirements. For dealing with these problems, solid waste can be incinerating to reduce its volume by 80%, and further can be used as soil stabilization material. It enhances the soil properties and also solves the problem of solid waste disposal in city. This work deals with estimation of engineering properties of soil i.e. shear test, maximum dry density vs. optimum moisture content and permeability. Experimental work shows that use of 10% MSW ash by weight of soil can improve the properties at larger extends. The foundation of a building or road is an essential part for effective transmission of load to the subsoil present beneath it.

The experimental work is carried out to find the effect of varying proportions of waste materials on shear strength, compaction and permeability of soil. Pursuant to this, following objectives are proposed in the present investigation.

1) To use municipal waste ash as a stabilizing material and to solve the problem of waste disposal.

2) To evaluate the strength characteristics of soil for different proportions of ash in replacement of 5%, 10%, 15% and 20%

3) To study the results of replacement and concentration on future use.

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ABBREVIATION NOTATION AND NOMENCLATURE

RHA	Rice Husk Ash
MSW	Municipal Solid Waste
SWM	Solid Waste Management
BMC	Brihan Mumbai Mahanagar Palika
MoUD	Ministry of Urban Development
OMC	Optimum Moisture Content
MDD	Maximum Dry Density
CBR	California Bearing Ratio
SP	Standard Proctor
WAS	West African Standard
MP	Modified Proctor
UCS	Unconfined Compressive Strength
MPa	Mega Pascal
AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
CKD	Cement Kiln Dust
LKD	Lime Kiln Dust
LOI	Loss On Ignition
PI	Plasticity Index
wP	Plastic Limit
wL	Liquid Limit
wS	Shrinkage Limit

Chapter 1

Introduction

1.1 General

The foundation of a building or road is an essential part for effective transmission of load to the subsoil present beneath it. The quality of soil has large impact on type of structure and its design. The expansive soils are examples of weak soils, which encountered in foundation engineering for bridges, highways, buildings, embankments etc. Expansive soil undergoes volume changes when they come in contact with water. They show alternate swelling and shrinkage properties. It expands during rainy season and shrinks during summer season. Solid waste term includes all those solid and semi-solid materials that are discarded by the community. Improper management of solid wastes causes adverse effects on the ecology which may lead to cause possible outbreak of diseases and epidemics. Solid wastes are broadly classified in to three group's namely Industrial waste, Agricultural waste, and Municipal waste apart from other categories of wastes.

Today, world faces a serious problem in disposing the large quantity of Municipal waste. The disposal of Municipal waste without proper attention creates impact on environmental health. It disturbs ecosystem, causes air pollution, water pollution etc. The engineers have to take challenge for safe disposal of municipal waste. This research undertakes use of Municipal waste in stabilizing soil, various attempts have been made to improve the strength of soil using different chemical additives in combination with lime and cement, but research work has to focus more on use of cheaper and locally available material.

Soil stabilization is a procedure in which existing properties of soil are improved by means of addition of cementing materials or chemicals. One of the more common methods of stabilization includes the mixing of natural coarse grained soil and fine grained soil to obtain a mixture that develops adequate internal friction and cohesion and thereby provides a material that is workable during placement. Stabilization of soil can be carried out by using mechanical stabilization, cementing stabilization and chemical stabilization. Rearrangement of soilparticles by some of mechanical compaction is referred as “Mechanical Stabilization”, use of cementing material such as cement, lime, bitumen/asphalt etc. is added to soil is “Cementing Stabilization” and use of chemicals in soil such as calcium chloride; sodium chloride etc. is “Chemical Stabilization”. Today, world faces a serious problem in disposing the large quantity of agricultural waste. The disposal of agricultural waste without proper attention creates impact on environmental health. It disturbs ecosystem, causes air pollution, water pollution etc. The engineers have to take challenge for safe disposal of agricultural waste. various attempts have been made to improve the strength of soil using different chemical additives ,but research work has to focus more cheaper and locally available material.

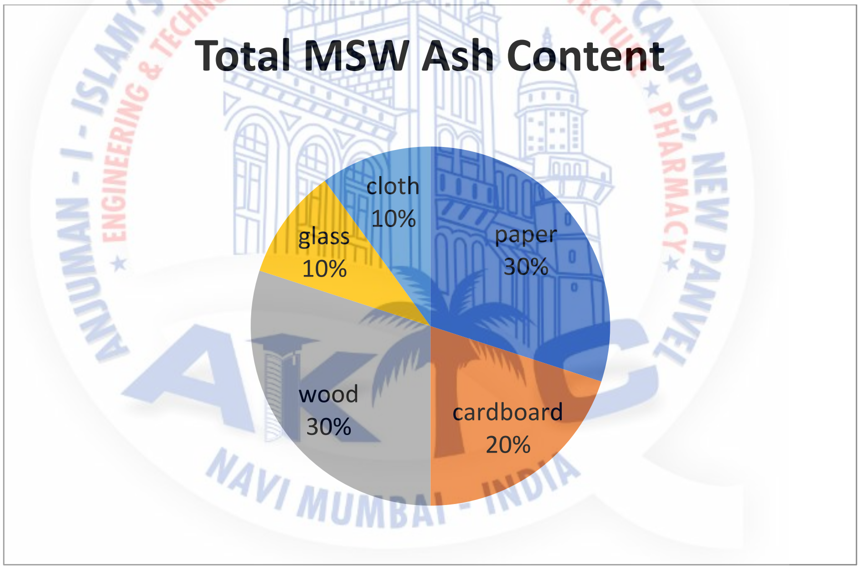
Different ways are available for enhancing the engineering performances of soils are soil stabilization, soil reinforcement etc. Admixtures like lime, cement were used traditionally for stabilization purposes.In recent times with the increase in the demand for infrastructure and feasible foundation design in not applicable due to poor bearing capacity of ground soil stabilization has started to take a new shape. Stabilization is process of fundamentally changing the chemical properties of soft soils by adding binders or stabilizers, either in wet or dry conditions to increase the strength and stiffness of the originally weak soils. With the availability of better research, materials andequipment soil stabilization is emerging as a popular and cost-effective method for soil improvement. With the availability of better research, materials and equipment soil stabilization is emerging as a popular and cost-effective method for soil improvement. In the present investigation attempt is made to stabilize black cotton soil.

During the last few decades, rapid growth of population, industrialization and urbanization have resulted in increase in environmental pollution in the form of wastes. Wastes are re unwanted residual materials, which cannot be discharged directly or before suitable treatment, to the atmosphere or to any receiving site. The solids, semi-solid and some liquid wastes are matter of great concern and have to be utilized suitably in different construction applications. Research is being carried out to utilized different solid wastes like fly ash from thermal power stations, steel slags from steel industries, hospital wastes, red mud from aluminium industries, quarry wastes etc. Research is also therefore required to be carried out regarding utility of municipal solid wastes, which are available in huge quantities. Quite a large amount of solid rubbish is contributed by households in the form of domestic wastes. Some of the constituents of these wastes are groceries food scraps, vegetable remains, packing materials, paper, remains of used coal, ash, wood, metals, plastic, ceramic, glass etc. if these wastes are not properly disposed off, this can prove perilous and environmental hazard such places often become a home for rat, flies, bacteria, mosquitoes all having the potential of causing many human diseases. The damage of the environment by the uncontrolled disposal of solid waste can be clearly visualized.

The generation of huge quantities of MSW poses serious disposal and environmental problems. Thus, it is imperative for a large-scale utilization of MSW and is bulk utilized in the construction of roads. Extensive research work has also been done in India as well as abroad and the studies have shown that MSW has a great potential for its use in road works. As a result, millions of tons of MSW are getting accumulated and causing serious environmental problems. At the present time, most operating facilities in the U.S. recover the ferrous metal fraction present in municipal waste combustor ash, which can comprise approximately 10 to 20 percent of the total ash fraction. Only a very small fraction of the non-ferrous fraction of the ash generated in the U.S. is recovered and utilized. Most of this fraction is used in landfill cover applications. A small unknown fraction is used as an aggregate substitute in road base applications. MSWA has been used as a granular base in road construction, as a fill material, and as an embankment material in Europe for almost two decades. Municipal waste combustor ash has also been tested for use as an aggregate substitute in asphalt paving mixes, where it has performed in a satisfactory manner, particularly in base or binder course applications. In this application, the ash is used to replace the sand-size or fine aggregate portion of the mix. There are presently no known commercial uses of municipal waste combustor ash in this application.

1.2 Municipal Solid Waste

2 Municipal solid waste (MSW) more commonly known as trash or garbage consist of everyday items we use and then throw away such as product packaging, grass, clipping, furniture ,clothing ,bottles, food, scraps, newspaper, appliances, paint and batteries. This come from our homes , schools, hospitals and business .



1.3 Soil Stabilization.

Soil Stabilization is the alteration of soils to enhance their physical properties. Stabilization can increase the shear strength of a soil and/or control the shrink-swell properties of a soil, thus improving the load bearing capacity of a sub-grade to support pavements and foundations.

Soil Stabilization can be utilized on roadways, parking areas, site development projects, airports and many other situations where sub-soils are not suitable for construction. Stabilization can be used to treat a wide range of sub-grade materials, varying from expansive clays to granular materials. This process is accomplished using a wide variety of additives, including lime, fly-ash, and Portland cement.

1.4 Needs and Advantages.

Soil properties vary a great deal and construction of structures depends a lot on the bearing capacity of the soil, hence, we need to stabilize the soil which makes it easier to predict the load bearing capacity of the soil and even improve the load bearing capacity. The gradation of the soil is also a very important property to keep in mind while working with soils. The soils may be well-graded which is desirable as it has less number of voids or uniformly graded which though sounds stable but has more voids. Thus, it is better to mix different types of soils together to improve the soil strength properties. It is very expensive to replace the inferior soil entirely and hence, soil stabilization is the thing to look for in these cases.

• It improves the strength of the soil, thus, increasing the soil bearing capacity.

• It is more economical both in terms of cost and energy to increase the bearing capacity of the soil rather than going for deep foundation or raft foundation.

• It is also used to provide more stability to the soil in slopes or other such places.

• It helps in reducing the soil volume change due to change in temperature or moisture content.

•Stabilization improves the workability and the durability of the soil.

1.5 **Objectives of Project.**

The dissertation study aims to achieve the following objectives:

1) To use municipal waste ash as a stabilizing material and to solve the problem of waste disposal.

2) To evaluate the different characteristics of soil for different proportions of ash in replacement of 0%,5%, 10%, 15% and 20%

3) To study the results of replacement and concentration on future use.

1.6 Scope of Work.

This study will contribute in improvement of soil properties by using waste ash. As soil and waste is variable material in characteristics from place to place, this experimental research work will be applicable for particular region soil with particular type of Municipal solid waste. For different area's soil and different waste, we can have different Optimum percentage of waste ash which will enhance soil properties. Use of solid waste ash as stabilizing material, is cheap as well as eco-friendly method of soil stabilization, which will solve the waste disposal problems as well as enhances soil properties.

Chapter 2

Review of Literature

2.1 General

Many researchers attempt to stabilize the different type of soil with use of cementious materials or waste or as a combination; here we discuss some of works based on use of waste Ash which used in combination with cementious material or used separately. For soil stabilization, various materials are being used like stone dust, rubber tyre, bagasse ash, industrial waste etc.

2.2 Overview of Literature

Sivapullaiah et al. (1996) presented the effect of fly ash and lime, on the index properties of expansive soils such as liquid limits, plastic limits and free swell. The studied soil was black cotton soil. The results showed that the index properties of this soil were significantly varied by addition fly ash. It is observed that the domain of alteration depends on the particle size distribution, free lime content and pozzolanic reactivity of the fly ash. The effect of the coarseness of fly ash particles is to decrease the activity. Thus, fly ash can decrease the plasticityindex of the soil. The effect of the addition of fly ash is to significantly improve the physical properties and workability of the black cotton soil.

Mirsa (1998) examined clay stabilization with Class C fly ash. Physical and chemical properties of fly ash and compaction and strength behaviour of soils stabilized with Class C fly ash were discussed. Examples were prepared by blending a small proportion of bentonite with kaolinite. Furthermore, fly ash had a rapid hydration characteristic. So, higher densities and strengths were achieved when the compaction is performed with little or no delay. However, delayed compaction produces low densities and strength. It was observed that the stabilization characteristics are related to the soil mineral type and plasticity. The laboratory studies indicated that use of Class C fly ash in soil stabilization was dependent on the ash contents, water content, compaction delay, strength development with time and curing methodology and the type of clay mineral. Thus, these Class C fly ashes are particularly suited for use as soil improvement agents.

Temimi et al. (1998) studied the addition of fly ash in the clay soils. Different clay-flyash samples were tested in order to find the effect of fly ash on the mechanical properties of clay materials. It indicated that the inclusion of fly ash in the clay material improved the mechanical properties of the clay, like the compressibility and the consolidation. So, compressibility and the settlement, decreased and the consolidation of clay increased.

Cokca (2001) used from high-calcium and low-calcium class C fly ashes for stabilization of an expansive soil and evaluation of the expansive soil-lime, expansive soil-cement, and expansive soil-fly ash systems. Lime, cement and fly ash were added to the expansive soil at different percentages. The specimens were subjected to chemical composition, grain size distribution, consistency limits, and free swell tests. Also, the Specimens with fly ash were cured and after that they were subjected to free swell tests. It can be concluded that the expansive soil can be successfully stabilized by fly ashes. Furthermore, plasticity index, activity, and swelling potential of the samples decreased with increasing percentage of stabilizer and curing time.

Puppala et al. (2001) used from fly ash and fiber reinforcement methods, to treat and increase the strength of two expansive soils. In this regard, Physical tests such as Atterberg limits, standard Proctor compaction and other tests like unconfined compressive strength, shrinkage, and free swell were conducted on both raw and treated clay samples. Both methods showed an increase in unconfined compression strength of the soils. Improvement with flyash decreased free swell, plasticity and linear shrinkage strains of raw soils. Fiber reinforcement decreased the vertical shrinkage strains. Whereas, it increased the free swell values. In general, the fly ash treatment method can be used to stabilize expansive soils, and fibers can be used to increase the strength and decrease the shrinkage potentials of expansive soils. In addition, the important point is that fibers alone will not provide comprehensive stabilization. Another advantage of the two methods was that both stabilizers were recycled waste products and therefore their use in soil stabilization will reduce landfilling costs.

Kumar and Sharma (2004) presented a study of the efficacy of fly ash in improving the engineering characteristics of expansive soils. An experimental program evaluated the effect of the fly ash on the free swell index, swell potential, swelling pressure, plasticity, compaction, strength, and hydraulic conductivity characteristics of expansive soil. The results showed that the plasticity, hydraulic conductivity and swelling properties of the blends decreased and the dry unit weight and strength increased with an increase in fly ash content. The resistance to penetration of the mixtures increased significantly with an increase in fly ash content for a certain water content. Excellent correlation was obtained between the measured and predicted undrained shear strengths and the undrained cohesion of the expansive soil blended with fly ash increased with the fly ash content.

Misra et al. (2005) studied the laboratory evaluation of the stabilization characteristics of clay soils blended with self-cementing class C fly ash and residual self-cementation of ponded class C fly ash. The stabilization characteristics were evaluated with reaching to the uniaxial compressive strength stiffness, and swelling potential. So, twelve set of mixtures of clay soils with the percentages of kaolinite and montmorillonite, self-cementing fly ash and appropriate amount of water were compacted and cured. Furthermore, unconfined compression and CBR tests were used. The results showed that the optimum moisture content changes due to the addition of fly ash. The samples rapidly gained compressive strength and stiffness within seven days curing period, and the greatest increase occurred in one day due to the rapid hydration reaction of fly ash. With increasing in montmorillonite content, strength of the samples increased significantly. By increasing in fly ash content, swelling potential of stabilized clay may be reduced. CBR values showed that the ponded class C fly ash can be a good substitute as a base course material.

Prabhakar et al. (2006) investigated the behaviour of soils mixed with fly ash to improve the load bearing capacity of the soil. Three different types of soil and different percentage of fly ash were used. The objectives of this investigation were reached to the usefulness of fly ash-soil mixtures, and focused to improve the engineering properties of soil with better load bearing capacity. This study also mentioned the cost effective of fly ash for soil improvement and covered the compaction behaviour, settlement, California bearing ratio, shear strength parameters and swelling characteristics. The results showed that addition of fly ash reduced the dry density of the soil and unit weight of soil. The void ratios and porosity changed with increasing content of fly ash in soils. The shear strength of the mixture was improved due to the addition of fly ash and increasing of it was nonlinearly. The value of cohesion increased by the addition of fly ash and this alteration was linear. CBR value of soil improved by the addition of fly ash. The results indicated that the shear strength and the angle of internal friction of soil admixed with fly ash caused a better strength. Using fly ash in soil also reduced swelling in the soil. Even, the fly ash improved the shear strength, cohesion and bearing capacity. So, this mixture can be used as the base materials for the roads, back filling and etc.

Kumar et al. (2007) studied the effects of polyester fiber inclusions and lime stabilization on the geotechnical characteristics of fly ash-soil mixtures. The geotechnical characteristics of fly ash soil specimens, lime-soil specimens and lime-fly ash-soil specimens mixed with different proportions of randomly oriented fibers were investigated. Test specimens were subjected to compaction tests, unconfined compression tests and split tensile strength tests. Specimens were cured after which they were tested for unconfined compression tests and split tensile tests. The results showed that with the increase in lime content, the maximum dry density of soil-lime mixes decreases and optimum moisture content increases but fly ash decreases further maximum dry density and optimum moisture contentincreases. However polyester fibers had no significant effect on maximum dry density and optimum moisture content. With the increase in the percentage of fly ash while the lime is constant, strength tends to increase and reaches a certain maximum value and after that it starts decreasing. The ratio of split tensile strength and unconfined compressive strength increases with increase in fiber content. So, polyester fibers are more efficient when soil was subjected to tension rather than to compression. Furthermore, a good stabilization can be formed on expansive soils by the combination of fibers, lime, and fly ash.

Kumar and Sharma (2007) studied the effect of fly ash on the volume change of a highly plastic expansive clay and a non-expansive clay with low plasticity. The effect of fly ash on free swell index, swell potential, and swelling pressure of expansive clays were evaluated. Moreover, Compression index and secondary consolidation characteristics of both clays were also determined. The results showed that Swell potential and swelling pressure, when determined at constant dry unit weight of the mixture, decreased and when determined at constant weight of clay, increased. Compression index and coefficient of secondary consolidation of both the clays decreased by addition fly ash. So, the settlement of structures built on this stabilized clays decreased and consolidation happened in shorter time. Furthermore, maximum dry unit weight increased and optimum moisture content decreased with increasing fly ash content.

Saifuddin and Salam (2010) studied the potential use of various solid wastes for producing construction materials. The study is based on the comprehensive review of available literature on the construction materials including different kinds of solid wastes. The traditional methods for producing construction materials are using the valuable natural resources. Besides, the industrial and urban management systems are generating solid wastes, and most often dumping them in open fields. These activities pose serious detrimental effects on the environment. To safeguard the environment, many efforts are being made for the recycling of different types of solid wastes with a view to utilizing them in the production of various construction materials. This study discusses the environmental implications caused by the generation of various solid wastes, and highlights their recycling potentials and possible use for producing construction materials. In addition, study shows the applications of solid waste based construction materials in real construction, and identifies the research needs.

Chittaranjan et.al (2011) studied the 'Agricultural wastes as soil stabilizers'. In this study, Agricultural wastes such as sugar cane bagasse ash, rice husk ash and groundnut shell ash are used to stabilize the weak sub grade soil. The weak sub grade soil is treated with the above three wastes separately at 0%, 3%, 6%, 9%,12%and 15% and CBR test is carried out for each percent. The results of these tests showed improvement in CBR value with the increase in percentage of waste.

Amit and Vishal (2012) studied on the properties of black cotton soil, many researchs has been conducted for increase in properties of B.K.C by cementing, mechanical and chemical means. By taking environmental into consideration there is huge amount of waste gernerated from sugarcane and the waste are incinerated and baggase ash is prepared and properties of B.K.C are found by replacement of Bagasse ash by (3%,6%,9% and 12%) on replacement on this amount this is variation in index properties of soil and the optimum quantity found was 6%.

UdayashankarHakari (2012) studied the stabilization of black cotton soil in the region of twin-city of Huballi-Dharwad due urbanization and growth in the economy lead to increase of industries which tends in production of fly ash in large quantity and building construction also increase on large scale. He investigate the engineering properties of soil and how to improve it. A large amount of fly ash generated from industries are mixed with soil in terms of volume (%) at 10%,20%,40%,60% and the optimum content between 20% to 40% was found to be effective on engineering properties of soil like there was large and effective variation of plastic limit, liquid limit and shrinkage limit and also increase the bearing capacity of soil upto large extent.

Bose (2012) used fly ash to stabilize a highly plastic clay. The geo-engineering properties such as, Atterberg limits, grain size distribution, linear shrinkage, free swell index, welling pressure, compaction characteristics, unconfined compressive strength and CBR value of virgin clay and stabilize with fly ash were evaluated. Expansive soil was stabilized with various proportion of fly ash. The results showed that plasticity index of clay-fly ash mixes decreased with increase in fly ash content. Thus, addition of fly ash increases its workability by colloidal reaction and changing its grain size. The free swell index value and swelling pressure of expansive clay mixed with fly ash decreased with increase in fly ash content. Furthermore, addition of fly ash reduced the optimum moisture content but the dry density increased and unconfined compressive strength of clay-fly ash mixes is found to be maximum. So, it is concluded that the fly ash has a good potential for improving the engineering properties of expansive soil.

Kiran (2013) studied the black cotton soil which is taken from Harihara, Davanagere district, Karnataka. Under this study laboratory experiments are carried out for different percentages (4%, 8% and 12%) of bagasse ash and additive mix proportions. The strength parameters like CBR, UCS are determined. It is observed that, the blend results of bagasse ash with different percentage of cement for black cotton soil gave change in density, CBR and UCS values. The density values got increased from 15.16 KN/m3 to 16.5 KN/m3 for addition of 8% bagasse ash with 8% cement, Then CBR values got increased from 2.12 to 5.43 for addition of 4% bagasse ash with 8% cement and UCS values got increased to 174.91 KN/m2 from 84.92 KN/m2 for addition of 8% bagasse ash with 8% cement.

Abdulfatah et al (2013) studied the compaction characteristics of Lateritic Soil-Stabilized Municipal Solid Waste (MSW) Bottom Sediment. In particular, work has been directed towards determining to what extent the results of the British Standard Compaction Test for Lateritic Soils are affected by MSW Bottom Sediments. The bottom sediments of MSW from some selected dumping sites in Kano metropolis Nigeria were mixed with lateritic soils in different proportions and a compaction test was conducted on the mixtures. Maximum dry densities (MDD) of the mixtures were found to range between 1.600 and 1.700 Mg.m-3 and optimum moisture contents (OMC) were between 12% and 17%. The results are similar to those of Silty Clay Soils of MDD between 1.600 and 1.845 Mg.m-3 and OMC between 15% and 25%. It is recommended that the bottom sediments be used as landfill or road construction materials after sorting out the re-cycled materials.

Borthakur and Singh (2014) studied Peat soil has geotechnical properties such as high water content, high organic matter content, low shear strength, low bearing capacity and high compressibility which makes it as a difficult soil for construction of structures in its natural state. In Manipur, India around 4,24,000 hectares of land area is covered by peat soil, so, improvement mechanism is needed for construction of structures upon it. Industries are very less in Manipur, so commonly used stabilising industry by products like fly ash, lime etc. are not economically available. Therefore, locally available admixtures such as stone dust (SD) and kiln dust (KD) have been selected for stabilization of peat soil. Admixtures are mixed with peat soil in different seven percentages: 0%, 2%, 5%, 8%, 10%, 12%, and 15%, respectively. Again, the peat soil is also mixed with both stone dust (SD) and kiln dust (KD) in the ratio of (soil: stone dust: kiln dust); 94:3:3, 92:4:4, 90:5:5, 88:6:6and 86:7:7. Proctor compaction test, unconfined compressive strength test, triaxial shear strength test and California bearing ratio tests are conducted on stabilised soil to determine admixture impact on peat soil properties. Laboratory test results shows that Maximum dry density (MDD) and unconfined compressive strength of stabilised soil are maximum at 10% of admixtures. Maximum value of shear strength is observed at 8%, for soils with KD and SD+KD, & at 5% for soils with SD.CBR values increases in all cases & bearing capacity is maximum at 8% Results of this study show that, if properly optimized, the use of these locally available admixtures may be available alternative for the stabilization of peat soil.

Vizcarra et al. (2014) presented the characteristics of municipal solid waste (MSW) incineration ash and evaluates this ash in road pavement layers through the mixture of ash with a clay soil. Chemical, physical, and mechanical tests and the mechanistic-empirical design for a pavement structure were carried out on the pure soil and also in the soil mixture with the addition of different ash content. The results showed that fly ash reduced the expansion of the material, showing an increase in the California bearing ratio (CBR) and resilient modulus value. Furthermore, content and type of ash was important in final results and it showed the efficacy of MSW fly ash for its use in base road pavement layers.

Prasad and Sharma (2014) evaluated the effectiveness of clayey soil blended with sand and fly ash for soil stabilization by studying the subgrade characteristics. The purpose of this work is to find a solution for proper disposal of fly ash and also provides good subgrade material for pavement construction. The results showed hat substantial improvement in compaction and California bearing ratio of composite containing clay, sand and fly ash. The swelling of the clay also reduced after stabilization. The maximum dry density of clay-sand-fly ash mix decreased with the addition of fly ash and optimum moisture content increased. Thus, the stabilized soil can be used for construction of flexible pavements in low traffic area fly ash mix decreased with the addition of fly ash and optimum moisture content increased. Thus, the stabilized soil can be used for construction of flexible pavements in low traffic areas.

Kumar and Munilakshmi (2015) studied that clayey soils usually have the potential to demonstrate undesirable engineering behaviour, such as low bearing capacity, high shrinkage and swell characteristics and high moisture susceptibility. Stabilization of these soils is a usual practice for improving the strength. Study reports the improvement in the strength of a locally available cohesive soil by addition of Municipal Solid Waste (MSW) incinerator ash as a soil stabilizing agent and to evaluate its influence on soil properties and shear strength of the soil when use in different proportion.

Shaikh Wasim (2016) studied about the management of municipal solid waste in soil stabilization, he present the analysis of municipal waste ash in soil stabilization for different engineering properties.As solid waste disposal being a major environmental problem, because of unviability of lands for disposal.solid waste can be incinerated to reduce its volume by 80%,and further can be used as a soil stabilizer. Experiment were carried out to evaluate the characteristics strength of soil for different proportions of ash in replacement of 5%,10%,15% and 20% but at 10% of addition of MSWA, soil showed improved index properties of soil like shear, compaction and permeability.

2.3 Summary

Research on soil stabilization by using any waste material separately or in combination shows that soil properties can be enhanced but till certain limit, above that limit these materials doesn't affect soil properties. Different soil and different waste material has different physical and chemical properties, so while selecting any waste material for stabilization we should consider chemical and physical properties of soil as well as waste material.It will result in effective and cheap stabilization method.

Chapter 3

Methods and Methodology

3.1 General

Long term performance of pavement structures often depends on the stability of the underlying soils. Engineering design of these constructed facilities relies on the assumption that each layer in the pavement has the minimum specified structural quality to support and distribute the super imposed loads. These layers must resist excessive permanent deformation, resist shear and avoid excessive deflection that may result in fatigue cracking in overlying layers. Available earth materials do not always meet these requirements and may require improvements to their engineering properties in order to transform these inexpensive earth materials into effective construction materials.

3.2 Mechanisms of stabilization

The stabilization mechanism may vary widely from the formation of new compounds binding the finer soil particles to coating particle surfaces by the additive to limit the moisture sensitivity. Therefore, a basic understanding of the stabilization mechanisms involved with each additive is required before selecting an effective stabilizer suited for a specific application.

Chemical stabilization involves mixing or injecting the soil with chemically active compounds such as Portland cement, lime, fly ash, calcium or sodium chloride or with viscoelastic materials such as bitumen. Chemical stabilizers can be broadly divided in to three groups: Traditional stabilizers such as hydrated lime, Portland cement and Fly ash; Non-traditional stabilizers comprised of sulfonated oils, ammonium chloride, enzymes, polymers, and potassium compounds; and By-product stabilizers which include cement kiln dust, lime kiln dust etc. Among these, the most widely used chemical additives are lime, Portland cement and fly ash. Although stabilization with fly ash may be more economical when compared to the other two, the composition of fly ash can be highly variable. The mechanisms of stabilization of the traditional stabilizers are detailed below.

3.2.1 Traditional Stabilizers

Traditional stabilizers generally rely on pozzolanic reactions and cation exchange to modify and/or stabilize. Among all traditional stabilizers, lime probably is the most routinely used. Lime is prepared by decomposing limestone at elevated temperatures. Lime-soil reactions are complex and primarily involve a two-step process.The primary reaction involves cation exchange and flocculation/agglomeration that bring about rapid textural and plasticity changes. The altered clay structure, as a result of flocculation of clay particles due to cation exchange and short-term pozzolanic reactions, results in larger particle agglomerates and more friable and workable soils. Although pozzolanic reaction processes are slow, some amount of pozzolanic strength gain may occur during the primary reactions, cation exchange and flocculation/agglomeration. Extent of this strength gain may vary with soils depending on differences in their mineralogical composition. Therefore, mellowing periods, normally about one-day in length but ranging up to about 4-days, can be prescribed to maximize the effect of short term reactions in reducing plasticity, increasing workability, and providing some initial strength improvement prior to compaction. The second step, a longer-term pozzolanic based cementing process among flocculates and agglomerates of particles, results in strength increase which can be considerable depending on the amount of pozzolanic product that develops, and this, in turn depends on the reactivity of the soil minerals with the lime or other additives used in stabilization.

The pozzolanic reaction process, which can either be modest or quite substantial depending on the mineralogy of the soil, is a long-term process. This is because the process can continue as long as a sufficiently high pH is maintained to solubilize silicates and aluminates from the clay matrix, and in some cases from the fine silt soil. These solubilized silicates and aluminates then react with calcium from the free lime and water to form calcium-silicate-hydrates and calcium aluminates hydrates, which are the same type of compounds that produce strength development in the hydration of Portland cement. However, the pozzolanic reaction process is not limited to long term effects.

The pozzolanic reaction progresses relatively quickly in some soils depending on the rate of dissolution from the soil matrix. In fact, physio-chemical changes at the surface of soil particles due to pozzolanic reactions result in changes in plasticity, which are reflected in textural changes that may be observed relatively rapidly just as cation exchange reactions are.Portland cement is comprised of calcium-silicates and calcium-aluminates that hydrate to form cementitious products. Cement hydration is relatively fast and causes immediate strength gain in stabilized layers. Therefore, a mellowing period is not typically allowed between mixing of the components (soil, cement, and water) and compaction. In fact it is general practice to compact soil cement before or shortly after initial set, usually within about2 hours. Unless compaction is achieved within this period traditional compaction energy may not be capable of developing target density. However, Portland cement has been successfully used in certain situations with extended mellowing periods, well beyond 2 to 4 hours. Generally, the soil is remixed after the mellowing periods to achieve a homogeneous mixture before compaction. Although the ultimate strength of a soil cement product with an extended mellowing period may be lower than one in which compaction is achieved before initial set, the strength achieved overtime in the soil with the extended mellowing period may be acceptable and the extended mellowing may enhance the ultimate product by producing improved uniformity. Nevertheless, the conventional practice is to compact soil cement within 2 hours of initial mixing. During the hydration process, free lime, $Ca(OH)_2$ is produced. In fact, up to about 25 percent of the cement paste (cement and water mix) on a weight basis is lime. This free lime in the high pH environment has the ability to react pozzolanically with soil, just as lime does and this reaction continues as long as the pH is high enough, generally above about 10.5.

Fly ash is also generally considered as a traditional stabilizer. While lime and Portland cement are manufactured materials, fly ash is a by-product from burning coal during power generation.As with other by-products, the properties of fly ash can vary significantly depending on the source of the coal and the steps followed in the coal burning process. These by-products can broadly be classified into class C (self-cementing) and class F (non-self-cementing) fly ash based on AASHTO M 295 (ASTM C 618). Class C fly ash contains a substantial amount of lime, CaO, but almost all of it is combined with glassy silicates and aluminates. Therefore, upon mixing with water, a hydration reaction similar to that which occurs in the hydration of Portland cement occurs. As with Portland cement, this hydration reaction produces free lime. This free lime can react with other unreacted pozzolana, silicates and aluminates, available within the fly ash to produce a pozzolanic reaction, or the free lime may react pozzolanically with soil silica and/or alumina. Class F ash, on the other hand, contains very little lime and the glassy silica and/or alumina exists almost exclusively as pozzolana. Therefore, activation of this pozzolana requires additives such as Portland cement or lime, which provide a ready source of free lime. The hydration or "cementitious" reactions and the pozzolanic reactions that occur when fly ash is blended with water from the products that bond soil grains or agglomerates together to develop strength within the soil matrix. As discussed previously, maintenance of a high system pH is required for long term strength gain in fly ash-soil mixtures.

The kinetics of the cementitious reactions and pozzolanic reactions that occur in fly ash stabilized soils vary widely depending on the type of ash and its composition. Normally, class C ashes react rapidly upon hydration. However, class F ashes activated with lime or even Portland cement produce substantially slower reactions than Portland cement - soil blends. Generally, compaction practice of fly ash - soil blends varies depending on the type of ash used or whether or not an activator is used, but the standard practice is to compact within6 hours of initial mixing.

3.2.2 By Product Stabilizer

Like traditional stabilizers, pozzolanic reactions and cation exchange are the primary stabilization mechanisms for many of the by-product stabilizers. Lime kiln dust (LKD) and cement kiln dust (CKD) are by-products of the production of lime and Portland cement, respectively.

Lime kiln dust (LKD) normally contains between about 30 to 40 percent lime. The lime may be free lime or combined with pozzolana in the kiln. The source of these pozzolana is most likely the fuel used to provide the energy source. LKDs may be somewhat pozzolanically reactive because of the presence of pozzolana or they may be altogether non- reactive due to the absence of pozzolana or the low quality of the pozzolana contained in the LKD. Cement kiln dust (CKD) is the by-product of the production of Portland cement.

The fines captured in the exhaust gases of the production of Portland cement are more likely (than LKD) to contain reactive pozzolana and therefore, to support some level of pozzolanic reactivity. CKD generally contains between about 30 and 40 percent CaO and about 20 to 25 percent pozzolanic material.

Specific procedure of composition of by-product LKD or by-product CKD as the oxide composition of each can vary widely depending on the composition of the feed stock, the nature of the fuel, the burning efficiency, and the mechanism and efficiency of flue dust capture. For example, if coal is used, then ash produced as a by-product of burning coal could be captured in the bag house or other mechanism used to capture exhaust fines with the by-product lime. If the source of the LKD is from the production of dolomite lime, then magnesium oxide may form a significant part of the LKD. Magnesium oxide, MgO, takeslonger and is more difficult to fully hydrate than CaO, and upon hydration it expands. If the LKD contains more than about 5 percent MgO then care should be taken to insure full hydration of the MgO if this LKD is used for modification or stabilization. Again, it is incumbent upon the agency involved to determine acceptable levels of oxides and trace elements that comprise the by-product. As a general guide on the level of risk associated with the presence of oxides and trace elements in these by product stabilizers, the development of expansive mineral products may become intolerable when the SO3 content exceeds about 3 percent or when the MgO content exceeds about 3 to 5 percent. The impact of organics can also be a problem as their presence can interfere with the availability of calcium to the soil or aggregate being treated. Several tests can be used to screen for the presence of organics. One quick test if loss on ignition (LOI). Although it does not identify the type of organic, which is definitely important, an LOI of greater than about 8 to 10 percent flags a potentially problematic quantity of organics.

3.2.3 Non-Traditional Stabilizers

This standard practice is limited to traditional, chemical stabilizers like: Portland cement, lime and fly ash. However, it is important when considering treatment with these traditional products to broach the subject of non-traditional or alternative stabilizers.

The mechanism of stabilization for non-traditional stabilizers varies greatly among the stabilizers. Asphalt may or may not be grouped as a traditional stabilizer depending on perspective. Asphalt is not a "chemical" stabilizer in the sense that it does not react chemically with the soil to produce a product that alters surface chemistry of the soil particles or that binds particles together. Instead asphalt waterproofs aggregate and soil particles by coating them and developing an adhesive bond among the particles and the asphalt binder. The process is dependent on the surface energies of the aggregate or soil and the asphalt binder. Consequently, since this mechanism is more physical than chemical, soilswith very high surface areas are not amenable to asphalt stabilization and such stabilization is normally limited to granular materials such as gravels or sands, and perhaps some silty sands. As a visco-elastic, visco-plastic material, temperature and/or dilution methods are required to make asphalt stabilization effective in soils. Either lower viscosity liquid asphalts (normally developed by mixing bitumen with diluents) or emulsified asphalts are used in soil stabilization. Because the nature of asphalt stabilization is so mechanistically different from chemical stabilization, asphalt stabilization is not considered as a candidate in this standard practice.

3.3 Classification of Soil

Soil texture is defined, at least initially, by its appearance and is dependent on the size, shape and distribution of particles in the soil matrix. Soil particle sizes may vary from boulders or cobbles, roughly a meter in diameter, to very fine clay particles, roughly a few microns in diameter. Engineering properties of coarse fractions are dependent on physical interlocking of grains and vary with the size and shape of individual particles. Finer fractions in soil have a significantly higher specific surface area and their behaviour is influenced more by electro-chemical and physio-chemical aspects than particle interaction. Among finer particles, clays exhibit varying levels of consistency and engineering behaviour and demonstrate various levels of plasticity and cohesiveness in the presence of water. Silt fractions are also classified as fine-grained soils because more than 50 percent of the soil mass is smaller than 75 µm, which fits in the designation of fine-grained material according to the Unified Classification System (AASHTO M 145). However, the specific surface area of silt fines is several orders of magnitude larger than that of clay soil particles. This difference is part of the reason that clay particles are more reactive than silt particles. In addition, clay minerals have a unique sheet particle structure and a crystalline layer structure that is amenable to significant isomorphous substitution. As a result of the isomorphous substitution of lower valence cations for higher valence cations within the layer structure, clay mineral surfaces carry a significant negative surface charge that can attract positively charged ions and dipolar water molecules. The cumulative effect of high surface area and surface charge makes clay particles particularly reactive, especially with water, and is the root cause of the propensity of clay particles to shrink and swell depending on the availability of water.

Soil classification system differentiates soils, first based on particle size and secondly based on Atterberg limits. If 35 percent or more of the mass of the soil is smaller than 75 µm in diameter, then the soil is considered either a silt or clay and if less than 35 percent of particles are smaller than 75-micron sieve, then the soil is considered to be coarse-grained, either a sand or gravel. For stabilization purposes, soils can be classified into sub grade and base materials based on fractions passing No. 200 sieve. If 25 percent or more passes through the no. 200 sieve the soil can be considered as a sub grade, and if not, they may be classified as a base material. However, more than simple gradation impacts the definition of a sub grade or base. In order to be termed a base material, the material in question must also be targeted for use as a base layer from a structural perspective. On the other hand, an in situ coarse-grained soil with less than 25 percent fines, may be, by definition a native sub grade even though it may achieve the required classification of a base. For stabilization purposes, the soils may be differentiated into sub grade (soil) stabilization and base stabilization (coarse-grained) on the basis on the fine content indexSoil is a broad term used in engineering applications which includes all deposits of loose material on the earth's crust that are created by weathering and erosion of underlying rocks.

3.3.1 Guidelines for soil stabilization

Stabilization projects are site specific and require integration of standard test methods, analysis procedures and design steps to develop acceptable solutions. Many variables should be considered in soil treatment, especially if the treatment is performed with the intent of providing a long-term effect on soil properties. Soil-stabilizer interactions vary with soil type and so does the extent of improvement in soil properties. Hence developing a common procedure applicable for all types of stabilizers is not practical.

Soil exploration and sampling should be performed as described in the preceding sections. The soil can be classified as either a sub grade category or base category material on the basis of AASHTO M145. A key decision factor in selecting the appropriate sub grade additive is the concentration of water soluble sulphates in the soil. Sulphate testing should be done in accordance with the modified version of AASHTO T 290 or equivalent. Soils with sulphate levels above 3,000 ppm may be considered problematic and should beaddressed separately from the standpoint of additive selection all the way through mix design and construction. Sampling, testing, stabilizer selection, and mix design for these soils should follow the draft recommended practice for stabilizing sulphate-bearing soils. A second key factor to be considered when deciding on the type of stabilizer to be used is the concentration of organic matter in the soil.Organic contents can interfere with strength gain mechanisms and should be determined prior to proceeding with mix design with any calcium-based stabilizer AASHTO M 147 also provides guidance in distinguishing among classes of base materials.

3.3.2 Guidelines for Stabilizer Selection

Soil characteristics including mineralogy, gradation and physio-chemical properties of fine-grained soils influence the soil-additive interaction. Hence stabilizer selection should be based on the effectiveness of a given stabilizer to improve the physio-chemical properties of the selected soil. The preliminary selection of the appropriate additive(s) for soil stabilization should consider:

- Soil consistency and gradation

- Soil mineralogy and composition
- Desired engineering properties
- Purpose of treatment
- Mechanisms of stabilization
- Environmental conditions and engineering economics

Soil index properties (i.e., sieve analysis, Atterberg limit testing, and moisture density testing) should be determined based on laboratory testing of field samples. Soil samples should be prepared following AASHTO T 87. The initial processing of most soils involves thorough air drying or assisted drying at a temperature not to exceed 60oC. Aggregations of soil particles should be broken down into individual grains to the extent possible. A representative soil fraction should be selected for testing following AASTHO T 248. The required quantity of soil smaller than 0.425 mm (No. 40 sieve) should be used to determine the soil index properties. Liquid limit testing should be performed following AASHTO T 89 and plastic limit and plasticity index testing should be measured following AASHTO T 90.

3.4 Soil Stabilization Methods with Different Materials

Soil stabilization with cement, bitumen, lime, chemical stabilization, geotextile, grouting etc. are discussed. It is a method of improving soil properties by blending and mixing other materials.Following are the various soil stabilization methods and materials:

3.4.1 Soil Stabilization using Cement

The soil stabilized with cement is known as soil cement. The cementing action is believed to be the result of chemical reactions of cement with siliceous soil during hydration reaction. The important factors affecting the soil-cement are nature of soil content, conditions of mixing, compaction, curing and admixtures used.The appropriate amounts of cement needed for different types of soils may be as follows:

- Gravels – 5 to 10%
- Sands – 7 to 12%
- Silts – 12 to 15%, and
- Clays – 12 – 20%

The quantity of cement for a compressive strength of 25 to 30 kg/cm^2 should normally be sufficient for tropical climate for soil stabilization.

Lime, calcium chloride, sodium carbonate, sodium sulphate and fly ash are some of the additives commonly used with cement for cement stabilization of soil.

Lime, calcium chloride, sodium carbonate, sodium sulphate and fly ash are some of the additives commonly used with cement for cement stabilization of soil.

Fig 3.4.1: Cement stabilization

(Source: www.google.com)

3.4.2 Soil Stabilization using Lime

Slaked lime is very effective in treating heavy plastic clayey soils. Lime may be used alone or in combination with cement, bitumen or fly ash. Sandy soils can also be stabilized with these combinations. Lime has been mainly used for stabilizing the road bases and the subgrade.Lime changes the nature of the adsorbed layer and provides pozzolanic action. Plasticity index of highly plastic soils are reduced by the addition of lime with soil. There is an increase in the optimum water content and a decrease in the maximum compacted density and he strength and durability of soil increases.Normally 2 to 8% of lime may be required for coarse grained soils and 5 to 8% of lime may be required for plastic soils. The amount of fly ash as admixture may vary from 8 to 20% of the weight of the soil.

Fig 3.4.2: Lime stabilization

(Source: www.google.com)

3.4.3 Soil Stabilization with Bitumen

Asphalts and tars are bituminous materials which are used for stabilization of soil, generally for pavement construction. Bituminous materials when added to a soil, it imparts both cohesion and reduced water absorption. Depending upon the above actions and the nature of soils, bitumen stabilization is classified in following four types:

Figure 3.4.3:- Bitumen Stabilization.

(Source: www.google.com)

3.4.4 Chemical Stabilization of Soil

Calcium chloride being hygroscopic and deliquescent is used as a water retentive additive in mechanically stabilized soil bases and surfacing. The vapor pressure gets lowered, surface tension increases and rate of evaporation decreases. The freezing point of pure water gets lowered and it results in prevention or reduction of frost heave.The depressing the electric double layer, the salt reduces the water pick up and thus the loss of strength of fine grained soils. Calcium chloride acts as a soil flocculent and facilitates compaction.

Figure 3.4.4:-Chemical Stabilization

Source :-(www.google.com)

Frequent application of calcium chloride may be necessary to make up for the loss of chemical by leaching action. For the salt to be effective, the relative humidity of the atmosphere should be above 30%. Sodium chloride is the other chemical that can be used for this purpose with a stabilizing action similar to that of calcium chloride.

Sodium silicate is yet another chemical used for this purpose in combination with other chemicals such as calcium chloride, polymers, chrome lignin, alkyl chlorosilanes, siliconites, amines and quarternary ammonium salts, sodium hexametaphosphate, phosphoric acid combined with a wetting agent.

3.4.5 Soil Stabilization by Grouting

In this method, stabilizers are introduced by injection into the soil. This method is not useful for clayey soils because of their low permeability. This is a costly method for soil stabilization.

Figure 3.4.5 Stabilization by grouting

Source (www.google.com)

This method is suitable for stabilizing buried zones of relatively limited extent. The grouting techniques can be classified as following:

- Clay grouting
- Chemical grouting
- Chrome lignin grouting
- Polymer grouting, and
- Bituminous grouting

3.4.6 Soil Stabilization by Geotextiles and Fabrics

Geotextiles are porous fabrics made of synthetic materials such as polyethylene, polyester, nylons and polyvinyl chloride. Woven, non-woven and grid form varieties of geotextiles are available. Geotextiles have a high strength.

When properly embedded in soil, it contributes to its stability. It is used in the construction of unpaved roads over soft soils.

Reinforcing the soil for stabilization by metallic strips into it and providing an anchor or tie back to restrain a facing skin element.

Figure 3.4.6 Stabilization by Geo-Textiles

Source (www.google.com)

3.4.7 Fly Ash Stabilization

Fly ash can be used effectively to stabilize coarse grained particles with little or no fines. In coarser aggregates, fly ash generally acts as a pozzolana and/or filler to reduce the void spaces among larger size aggregate particles to float the coarse aggregate particles. After the appropriate amount of fly ash is added to coarse grained soils to fill the voids, optimize density, an activator is often used to maximize the pozzolanic reaction in the mixture. The activator content is generally in the range of 20 to 30 percent of the fly ash used to fill the voids. The activator is normally either lime or Portland cement, but lime kiln dust or cement kiln dust can also be used. Similarly, consider a clay soil that is stabilized with lime but the clay is not pozzolanically reactive. The addition of fly ash and lime can substantiallyincrease strength in the blend due to the reactive pozzolana provided by the ash. In these fine-grained soils, fly ash is typically used in conjunction with lime or cement to enhance the reactivity of the fine-grained soil with lime or cement. Class C fly ash has been used alone to stabilize moderately plastic soils. The basis for stabilization is free lime that becomes available upon hydration of the ash. The largemajority of this lime is combined with the silica and alumina, but upon hydration, just as in the hydration of Portland cement, cementitious products are formed which stabilize the soil. However, during this hydration process, just as in the hydration of cement, free lime is released, which can react pozzolanically with the clay. This reaction reduces clay particle plasticity and improves strength. Fig 3.5.3 shows flyash stabilization at site.

3.4.8 In-Situ Stabilization

The method involves on site soil improvement by applying stabilizing agent without removing the bulk soil. This technology offer benefit of improving soils for deep foundations, shallow foundations and contaminated sites. Planning of the design mix involves the selection and assessment of engineering properties of stabilized soil and improved ground. The purpose is to determine the dimensions of improved ground on the basis of appropriate stability and settlement analyses to satisfy the functional requirements of the supported structure. The technology can be accomplished by injection into soils a cementitious material such cement and lime in dry or wet forms. The choice to either use dry or wet deep mixingmethods depend among other things; the in-situ soil conditions, in situ moisture contents, effectiveness of binders to be used, and the nature of construction to be founded. Depending on the depth of treatment, the in-situ stabilization may be regarded as either deep mixing method or mass stabilization.

3.4.9 Quality control

Quality Control QC and QA is obtained from the installation records of the columns and from the results of appropriate laboratory and field verification tests. Each column is provided with a chart-log, which comprises: date and time of execution, length of column shaft, penetration/withdrawal rates of the mixing tool, mixing speed, pressure and flow rate of pumped slurry, total slurry consumption per column. Specimen of stabilised soils for testing are usually obtained from fresh columns with the wet grab method. Advanced core drilling and other field testing methods can be also used to obtain specimens and to inspect continuity, uniformity and stiffness of DSM columns. The selection of suitable verification methods depends on their relevance, accuracy and applicability in relation to the purpose and pattern of soil treatment and strength of stabilised soil.

3.4.10 Mass stabilization

Mass Stabilization for shallow mixing Shallow dry mixing offers a cost-effective solution for ground improvement works or site remediation when dealing with substantial volumes of very weak or contaminated superficial soils with high water content, such as deposits of dredged sediments, wet organic soils or waste sludges. In this method special mixing tools are used, which are in most cases fixed to an excavator's rig arm. Mixing is executed vertically or horizontally, with mixing tools that resemble screw propellers having a centrally provided nozzle for binder. The binder is fed from a separate unit which houses the pressurised binder container, compressor, air dryer and supply control unit. Stabilization is executed in phases, according to the operational range of the drilling rig, which generally comprises an area of 8 to 10 m² and depth up to approx 4 m. Once the required binder volume has been applied, mixing is continued to assure the optimum mixing properties.

3.5 Soil Properties

3.5.1 Atterberg Limits

The Atterberg limits are a basic measure of the critical water contents of a fine-grained soil its shrinkage limit, plastic limit, and liquid limit. As a dry, clayey soil takes on increasing amounts of water, it undergoes distinct changes in behaviour and consistency. Fig 3.3.1 shows state of soil at different limits graphically.

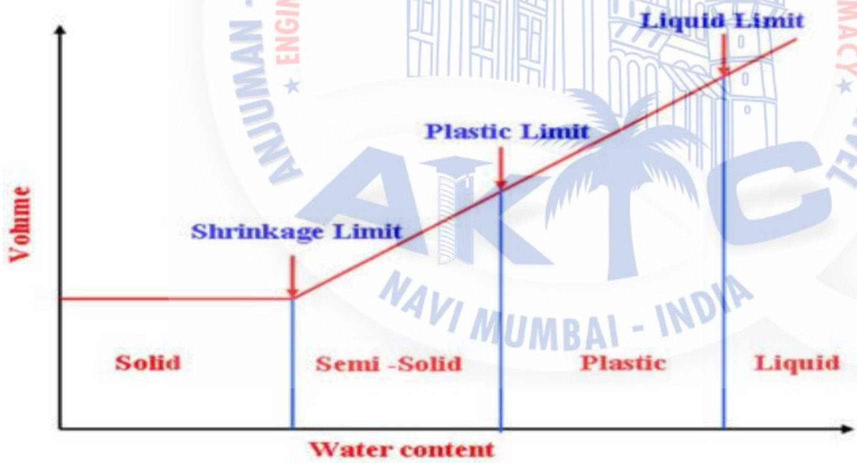


Fig 3.5.1: Atterberg limits

Soil is in form of solids until shrinkage limit approaches. Soil is in semi solid formfrom shrinkage to plastic limit. Soil is in plastic form from plastic to liquid limit. Soil is inliquid form after liquid limit.

3.5.2 Liquid Limit:

It is the water content of the soil between the liquid state and plastic state of the soil. Itcan be defined as the minimum water content at which the soil, though in liquid state, showssmall shearing strength against flowing. It is measured by the Casagrande's apparatus and isdenoted by wl.

3.5.3 Plastic Limit:

This limit lies between the plastic and semi-solid state of the soil. It is determined by rolling out a thread of the soil on a flat surface which is non-porous. It is the minimum water content at which the soil just begins to crumble while rolling into a thread of approximately 3mm diameter. Plastic limit is denoted by wp.

3.5.4 Shrinkage Limit:

This limit is achieved when further loss of water from the soil does not reduce the volume of the soil. It can be more accurately defined as the lowest water content at which the soil can still be completely saturated. It is denoted by ws.

3.6 Characteristics of Municipal Solid Waste:

The concentration of mercury was 23.76 mg/kg, slightly exceeding the maximum allowable environment amount of 23mg/kg as prescribed by the environment policy. This high mercury concentration could have been introduced by burning of industrial products in the MSW. The ash had high concentration of calcium (220240 mg/kg), with its trace elements like copper (89.12 mg/kg), zinc (325.36 mg/kg), and lead (352.62 mg/kg) all being within range of environment maximum allowable concentrations in soil. The loss on ignition of MSWA was 83.49%, which was obtained on burning 6 kilograms of municipal solid waste. This value is more than 12% as the maximum requirement for pozzolana as set in. It means that MSWA contains un-burnt carbon and this can reduce the pozzolanic activity of the ash. However, suggested that the un-burnt carbon in pozzolanic materials can serve as filler in the material being stabilized

3.7 Collection and Preparation of Municipal Solid Waste Ash:

Municipal solid waste will be collected from our houses. The municipal solid waste will be incinerated in a muffle furnace that allowed controlled incineration of MSW. MSW experienced loss on ignition of about 80%. Remaining ashes will be collected at the bottom of the muffle furnace. The ash was then screened through a 0.3 mm sieve to remove unburnt MSW.

Figure 3.7.1 Cardboard Ash

Figure 3.7.2 Paper Ash

Figure 3.7.3 Cloth Ash

Figure 3.7.4 Wooden Ash

3.8 Testing on soil:

Liquid Limit & Plastic Limit Test

In accordance with IS 2720 (Part 5) – 1985.

3.8.1 Liquid Limit Test

A) Apparatus

Casagrande's liquid limit device, Grooving tools of both standard and ASTM types, Oven Evaporating dish, Spatula, IS Sieve of size 425 μm, Weighing balance, with 0.01g accuracy

Fig.3.8.1Casagrande's liquid limit device
Source (www.google.com)

B) Test Procedure

1. Place a portion of the paste in the cup of the liquid limit device.

2. Level the mix so as to have a maximum depth of 1cm.

3. Draw the grooving tool through the sample along the symmetrical axis of the cup, holding the tool perpendicular to the cup.

4. For normal fine grained soil: The Casagrande's tool is used to cut a groove 2mm wide at the bottom, 11mm wide at the top and 8mm deep.

5. For sandy soil: The ASTM tool is used to cut a groove 2mm wide at the bottom, 13.6mm wide at the top and 10mm deep.

6. After the soil pat has been cut by a proper grooving tool, the handle is rotated at the rate of about 2 revolutions per second and the no. of blows counted, till the two parts of the soil sample come into contact for about 10mm length.

7. Take about 10g of soil near the closed groove and determine its water content

8. The soil of the cup is transferred to the dish containing the soil paste and mixed thoroughly after adding a little more water. Repeat the test.

9. By altering the water content of the soil and repeating the foregoing operations, obtain at least 5 readings in the range of 15 to 35 blows. Don't mix dry soil to change its consistency.

10. Liquid limit is determined by plotting a 'flow curve' on a semi-log graph, with no. of blows as abscissa (log scale) and the water content as ordinate and drawing the best straight line through the plotted points.

3.8.2 Plastic Limit Test

A)Apparatus

Porcelain evaporating dish about 120mm diameter, Spatula, Container to determine moisture content, Container to determine moisture content, Oven, Ground glass plate – 20cm x 15cm,Rod – 3mm dia. and about 10cm long..

Fig.3.8.2 Plastic limit apparatus…from google

B) Test Procedure

1. Take about 8g of the soil and roll it with fingers on a glass plate. The rate of rolling should be between 80 to 90 strokes per minute to form a 3mm dia.

2. If the dia. of the threads can be reduced to less than 3mm, without any cracks appearing, it means that the water content is more than its plastic limit. Knead the soil to reduce the water content and roll it into a thread again.

3. Repeat the process of alternate rolling and kneading until the thread crumbles.

4. Collect and keep the pieces of crumbled soil thread in the container used to determine the moisture content.

5. Repeat the process at least twice more with fresh samples of plastic soil each time.

3.8.3 Shrinkage Limit Test

In accordance with IS 2720-1972 .

A) Apparatus

Oven, Sieve 425 micron, Mercury, Desiccator, Weighing balance, with 0.01g accuracy

Fig 3.6.3 Shrinkage Apparatus
Source www, google.com

B) Test Procedure

1. 100 gm. of soil sample from a thoroughly mixed portion of the material passingthrough 425 micron IS sieve is taken.

2. About 30 gm. of above soil sample is placed in the evaporating dish and thoroughly mixed with distilled water to make a paste.

3. The weight of the clean empty shrinkage dish is determined and recorded.

4. The dish is filled in three layers by placing approximately 1/3rd of the amount of wet soil with the help of spatula.

5. Then the dish with wet soil is weighed and recorded immediately.

6. The wet soil cake is air dried until the color of the pat turns from dark to light. Then it is oven dried at a temperature of 1050 C to 1100 C for 12 to 16 hours. The weight of the dish with dry sample is determined and recorded. Then the weight of oven dry soil pat is calculated (W0).

7. The shrinkage dish is placed in the evaporating dish and the dish is filled with mercury, till it overflows slightly. Then it is be pressed with plain glass plate firmly on its top to remove excess mercury. The mercury from the shrinkage dish is poured into a measuring jar and the volume of the shrinkage dish is calculated. This volume is recorded as the volume of the wet soil pat (V).

8. A glass cup is placed in a suitable large container and the glass cup removed bycovering the cup with glass plate with prongs and pressing it. The outside of the glasscup is wiped to remove the adhering mercury. Then it is placed in the evaporating dishwhich is clean and empty.

9. Then the oven dried soil pat is placed on the surface of the mercury in the cup andpressed by means of the glass plate with prongs, the displaced mercury being collected in the evaporating dish.

10. The mercury so displaced by the dry soil pat is weighed and its volume (Vo) is calculated by dividing this weight by unit weight of mercury.

3.9 Testing on MSW Ash

3.9.1 Proctor Compaction Test:

The Proctor compaction test is a laboratory method of experimentally determining theoptimal moisture content at which a given soil type will become most dense and achieve its maximum dry density.

Equipments for Proctor's Test for Compaction of Soil:-

1. Compaction mould, capacity 1000ml.
2. Rammer, mass 2.6 kg
3. Detachable base plate
4. Collar, 60mm high
5. IS sieve, 4.75 mm
6. Oven
7. Desiccator
8. Weighing balance, accuracy 1g
9. Large mixing pan
10. Straight edge
11. Spatula
12. Graduated jar
13. Mixing tools, spoons, trowels, etc.

Procedure:

1. Take 16 kg of air-dried soil sample passing through 20 mm IS sieve.

2. Add water to soil as per the table given in 7.0. Mix soil and water thoroughly.

3. Allow soil to mature for 20hours. For this keep moist soil in water tight container.

4. Take out soil sample in tray. Spread it uniformly using spoon/spatula. Divide it in equal 6parts.

5. Clean empty compaction mould with base plate, dry and weigh to a nearest gm.

6. Apply grease to inside surface of mould, top of base, internal surface of collar and bottom surface of rammer.

7. Fix collar to top of the mould. Place mould on floor.

8. Divide soil (1st part) ion three equal sub parts. Put 1 sub part of the soil specimen to themould using spoon. Spread it uniformly in mould. Apply 25 blows distributed uniformly over

entire soil surface by rammer. Insure that drop of rammer is full example 310mm.

9. Using spatula make scratches of compacted layer. Add 2 subpart of soil. And apply 25blows using rammer as stated in step 8.

10. Repeat step9 for 3 sub parts so that Mould is fully filled in 3comapcted layers of soil ,Compacted soil should not protrude/project in collar more than 6mm.

11. Remove collar and using straight edge trim top of compacted layer to level flush with ringof mould.

12. Weigh mould filled with compacted soil.

13. Extrude soil from mould. Take representative soil sample from middle layer from watercontain determination by oven drying method.

14. Now take 2 part of moist soil sample in a separate tray. Add water to it to increase itswater content as desired. Repeat step 8 to 13.

15. Conduct test repeatedly for remaining 4parts of moist soil by increasing its water containmore than previous specimen.

16. Tabulate observation. Calculate bulk density, actual water contain and dry density of eachpart of moist soil specimen.

17. Plot a smooth curve between water contains % as abscissa and dry density as ordinate onnatural scale

18. Read water contain % corresponding to max. Dry density and report it as optimummoisture content.

Fig. 3.9.1: Proctor Compaction Apparatus

Source…from google

Fig 3.9.1 shows Proctor compactor test apparatus, which consists of mould with baseplate and collar, rammer with height projector. Depend upon weight of rammer, height ofcompaction and blows given test is called simple or modified proctor test.

3.9.2 Unconfined compressive test

This is the simplest and quickest test for determining the cohesion and the shear strength of the cohesive soils. These values are used for checking the short term stability of foundations and slopes.

Apparatus:

1. Compression device
2. Sample ejector
3. Dials gauge
4. Stopwatch oven
5. Balance

Procedure:

1. Soil which is to be tested is mixed with water. This sample is than filled in the mould which is oiled in advance. The mould is having the same internal diameter as that of specimen which is to be tested

2. The mould is opened carefully and sample is taken out

3. Prepare two or three such samples for testing.

4. Measure the initial length and diameter of the specimen.

5. Put the specimen on bottom of the loading device. Adjust upper plate to make contact with the specimen. Set the dial gauge (compression) at zero. The dial gauge reading provides the deformation in the sample and in turn strain.

6. Compress the specimen until crakes are developed or the strain curve is well past its peak or until a vertical deformation of 20% is reached. Take the dial reading approximately at every 1 mm deformation of the specimen.

7. The proving ring reading provides the corresponding load in- turn axial stress on the sample.

8. Repeat of the specimen.

9. Determine water content of each sample.

10. Repeat the above procedure by varying content of ash by 5%,10%,15%&20%.

3.9.3 California bearing ratio (CBR) test

The California Bearing Ratio test is penetration test meant for the evaluation of subgrade 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 design of flexible pavement

Apparatus: Following Equipments and tool are for this test required.

1. Cylindrical mould with inside dia. 150 mm and height 175 mm, provided with a detachable extension collar 50 mm height and a detachable perforated base plate 1 mm thick.

2. Spacer disc 148 mm in dia and 47.7 mm in height along with handle.

3.Metal rammers weight 2.6 kg with a drop of 310 mm (or) weight 4.89 kg a drop 450 mm.

4 Weights, one annular metal weight and several slotted weights weighing 2.5 kg each, 147 mm in dia. with a central hole 53 mm in diameter.

5. Loading machine, with a capacity of at least 5000 kg and equipped with a movable head or base that travels at a uniform rate of 1.25 mm/min. Complete with load indicating device.

6. Metal penetration piston 50 mm dia. and minimum of 100 mm in length.

7. Two dial gauges reading to 0.01 mm.

8. Sieves. 4.75 mm and 20 mm I.S. Sieves.

9. Miscellaneous apparatus, such as a mixing bowl, straight edge, scales soaking tankor pan, drying oven, filter paper and containers.

Preparation of test specimen:

Undisturbed specimen: Attach the cutting edge to the mould and push it gently into the ground. Remove the soil from the outside of the mould which is pushed in. When the mould is full of soil, remove it from weighing the soil with the mould or by any field method near the spot. Determine the density.

Remoulded specimen: Prepare the remoulded specimen at Proctors maximum dry density or any other density at which C.B.R> is required. Maintain the specimen at optimum moisture content or the field moisture as required. The material used should pass 20 mm I.S. sieve but it should be retained on 4.75 mm I.S. sieve. Prepare the specimen either by dynamic compaction or by static compaction.

Dynamic Compaction:

1. Take about 4.5 to 5.5 kg of soil and mix thoroughly with the required water.
2.Fix the extension collar and the base plate to the mould. Insert the spacer disc over the base. Place the filter paper on the top of the spacer disc.

3. Compact the mix soil in the mould using either light compaction or heavy compaction. For light compaction, compact the soil in 3 equal layers, each layer being given 55 blows by the 2.6 kg rammer. For heavy compaction compact the soil in 5 layers, 56 blows to each layer by the 4.89 kg rammer.

4.Remove the collar and trim off soil.

5. Turn the mould upside down and remove the base plate and the displacer disc.

6. Weigh the mould with compacted soil and determine the bulk density and dry density.

7. Put filter paper on the top of the compacted soil (collar side) and clamp the perforated base plate on to it.

For soaked test, the filter paper is now placed on the base plate and the mould is turned upside down, so that the top of the sample now placed in water tank for soaking. A filter paper is placed over the sample top along with the perforated plate with the adjustable stem. Over this surcharge weight of 2.5 kg is placed. Soaking is done for 4 days (or for a shorter period if by then soil is thoroughly saturated, showing no further expansion). The initial and final readings of the dial gauge are taken to measure the expansion.

The sample is allowed to drain off water in a vertical position for 15 min. The sample along with the mould is again weighed to calculate the % of water absorbed.

Procedure for Penetration Test:

1. Place the mould assembly with the surcharge weights on the penetration test machine.

2. Seat the penetration piston at the center of the specimen with the smallest possible load, but in no case in excess of 4 kg so that full contact of the piston on the sample is established.

3. Set the stress and strain dial gauge to read zero. Apply the load on the piston so that the penetration rate is about 1.25 mm/min.

4. Record the load readings at penetrations of 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 7.5, 10 and 12.5 mm. Note the maximum load and corresponding penetration if it occurs for a penetration less than 12.5 mm.

5. Detach the mould from the loading equipment. Take about 20 to 50 g of soil from the top 3 cm layer and determine the moisture content.

6. Repeat the above procedure by varying content of ash by 5%,10%,15%&20%.

3.9.4. Permeability test

It can be defined as the ability of a porous mass to allow passage of water through themedium. For finding permeability Constant head and differential head test will be conduct.

Equipments:

1) Permeameter

2) Tamper

3) Balance

4) Scoop

5) 1000ml graduated cylinder

6) Watch (or stopwatch)

7) Thermometer

8) Filter paper

Procedure:

1.Take 2.5kg of air dried soil (sand size) sample passing through 4.75mm sieve and retain 2mm sieve. Add and mix water to bring the moisture contain to desired level. Leave the soilair–tight container for same time.

2. Saturate porous plate and boiling water.

3. Apply grease to inside surface of the mould, base plate and collar.

4. Clamp the mould to the compaction base plate, place to the top of the mould.

5. Prepare soil specimen filling it in 3 layers, each layer should give 25 blows by standardrammer.

6. Remove the collar and trim off excess soil

7. Cover the soil specimen at both end with filter paper and saturated porous stones. Place themould assembly in drainage base fix the top cap on to it using rubber sealing gasket.

8. Open air vent on top of permeameter mould. Immerse the mould with soil specimen in a water tank (to be used as bottom water tank during test) for saturation for 24 hours.

9. Connect the inlet nozzle of permeameter to the stand pipe filled with water. Close air valve of permeameter.

10. Open outlet of permeameter and allow water to flow out. Wait for some time to establish steady flow.

11. Measure head h1 and note time t1.

12. Let the water level in stand pipe to fall to a lower head. Not h2and corresponding time t2.

13. Repeat step 11 and 12 twice to take additional reading h1, h2, t1& t2.

14. Tabulate observation and calculate average value of coefficient of permeability.

15. Repeat the above procedure by varying content of ash by 5%,10%,15%&20%.

Fig.3.9.4: Permeability Test Apparatus

Source …From google

Chapter 4

Results and Discussion

4.1 General

Soil with different amount percentage of Municipal solid waste ash is used for soil stabilization. Here we are discussing test results of shear test, permeability and compaction test on soil with 0%, 5%, 10%, 15% and 20% municipal solid waste ash. As Municipal solid waste is varying in percentage, properties of soil are also varying. Here we are discussing results of shear strength, permeability and compaction of soil with different percentage of Municipal solid waste ash through test reading and graphs plotted for comparison of results.

4.2 Permeability Test:

Permeability test is carried out on soil with varying percentage of soil to check permeability effect of soil with different amount of MSWA. Table 4.2 shows percentage of ash used and Permeability achieved to corresponding percentage.

Percentage of Ash	coefficent of permeability cm/s
0%	4.516x10^-4
5%	3.262x10^-4
10%	2.801x10^-4
15%	2.402x10^-4
20%	1.921x10^-4

Table 4.2

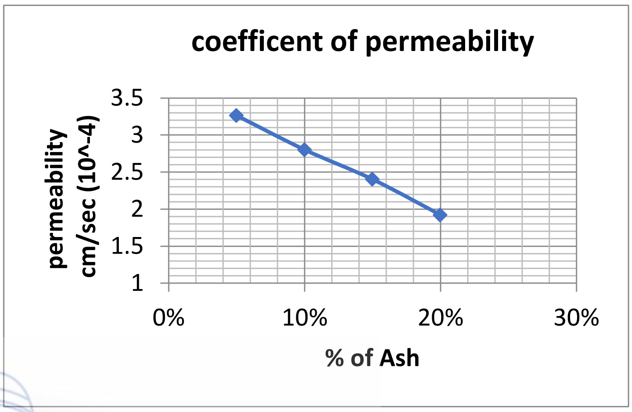


Figure 4.2:-Graph of Permeability Results

At 0% of ash addition i.e. soil without ash gives Permeability as 4.516x10^-4 cm/sec. As ash content increases in soil, permeability of soil decreases as 2.801x10^-4 cm/sec at 10%. After 10% addition of ash, permeability of soil continues to decrease. Accumulation of fine ash particles in soil voids results in blockage of pore path hence permeability of soil decreases.

4.3 California Bearing Ratio Test:-

C.B.R test is carried out on soil with varying percentage of soil to check bearing capacity of soil with different amount of MSWA. Table 4.3 shows percentage of ash used and bearing capacity achieved to corresponding percentage.

Percentage of Ash	Load =p.r.r*c.f	Penetration in mm
0%	0.248	2.5
	0.3008	5
5%	0.726	2.5
	1.198	5
10%	2.53	2.5
	2.303	5
15%	2.9	2.5
	2.54	5
20%	2.6	2.5
	2.43	5

Table 4.3

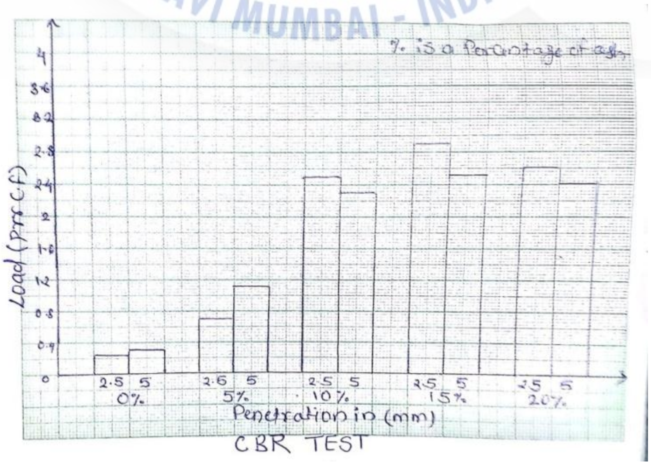


Figure 4.3 Graph of C.B.R results

4.4 Unconfined Compression Test:-

Unconfined Compression test is carried out on soil with varying percentage of soil to shear strength of soil with different amount of MSWA. Table 4.3 shows percentage of ash used and shear strength achieved to corresponding percentage

4.4.1 :- Without Ash

DGR	PRR	Deformation	Load	Strain	Corrected Area	Compressive stress =P/A
		Δ=DGR*G/10	P=PRR*CF*100	□=(ΔL/L0)	A=A0/(1-□)	(Kg/cm2)
		(cm)	(Kg)			
30	0.8	0.03	18.4	0.0042857	11.3899540	1.61545866
60	1.6	0.06	36.8	0.0085714	11.4391902	3.21701093
90	4	0.09	92	0.0128571	11.4888538	8.00776137
120	6.2	0.12	142.6	0.0171428	11.5389505	12.3581428
150	8	0.15	184	0.0214285	11.5894861	15.8764588
180	9.2	0.18	211.6	0.0257142	11.6404662	18.1779659
210	10.2	0.21	234.6	0.03	11.6918969	20.0651786
240	10.6	0.24	243.8	0.0342857	11.7437840	20.7599185
270	10.8	0.27	248.4	0.0385714	11.7961337	21.0577470
300	10.8	0.3	248.4	0.0428571	11.8489522	20.9638789

Table 4.4.1 U.C.S Test without Ash

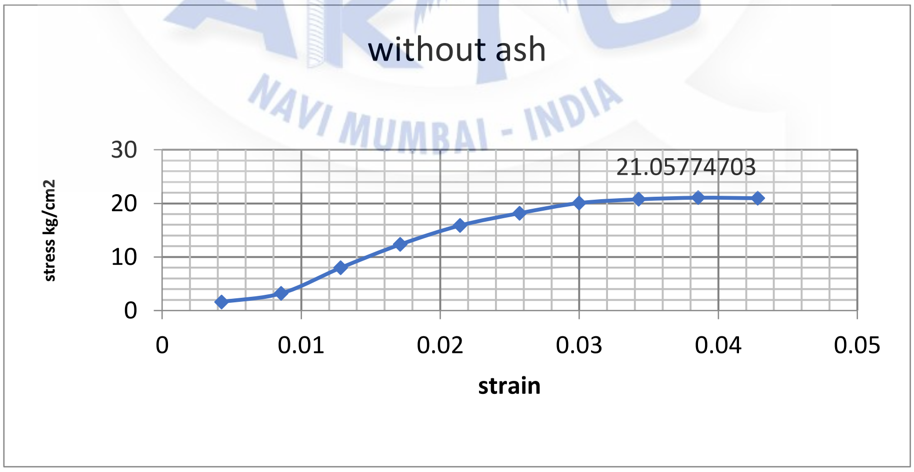


Figure 4.4.1:-Graph of U.C.S Test

4.4.2 :-Soil and 5% MSW ash.

DGR	PRR	Deformation	Load	Strain	Corrected Area	Compressive stress =P/A
		Δ=DGR*G/10	P=PRR*CF*100	▯ =(ΔL/L0)	A=A0/(1-▯)	(Kg/cm2)
60	0.6	0.06	13.8	0.008571429	11.4391902	1.206379102
90	1.2	0.09	27.6	0.012857143	11.4888538	2.402328413
120	1.6	0.12	36.8	0.017142857	11.5389505	3.189198163
150	3.4	0.15	78.2	0.021428571	11.5894861	6.747495024
180	5.4	0.18	124.2	0.025714286	11.6404662	10.66967569
210	6.2	0.21	142.6	0.03	11.6918969	12.19648113
240	7.2	0.24	165.6	0.034285714	11.7437840	14.10107676
270	7.9	0.27	181.7	0.038571429	11.7961337	15.40335199
300	8.6	0.3	197.8	0.042857143	11.8489522	16.69345914
330	9.2	0.33	211.6	0.047142857	11.9022458	17.77815735
360	9.8	0.36	225.4	0.051428571	11.9560210	18.85242577
390	10.2	0.39	234.6	0.055714286	12.0102844	19.53325932
410	10.6	0.41	243.8	0.058571429	12.0467344	20.23784961
440	10.8	0.44	248.4	0.062857143	12.1018262	20.52582771
470	11.2	0.47	257.6	0.067142857	12.1574242	21.18869884
500	11	0.5	253	0.071428571	12.2135353	20.71472281

Table 4.4.2 U.C.S Test with 5% Ash

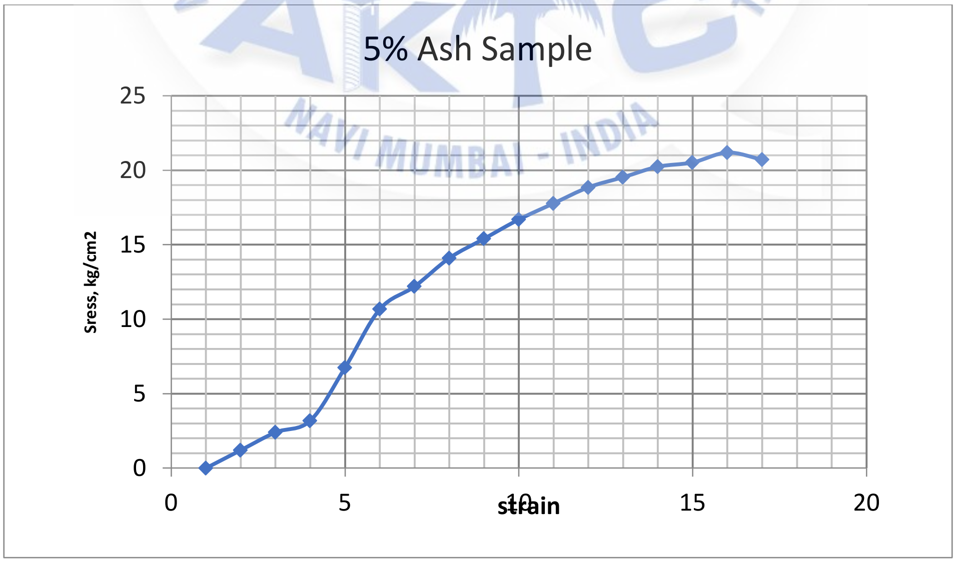


Figure 4.4.2:-Graph of U.C.S Test

4.4.3 :- Soil and 10% ash

DGR	PRR	Deformation	Load	Strain	Corrected Area	Compressive stress =P/A
		Δ=DGR*G/10	P=PRR*CF*100	⍰=(ΔL/L0)	A=A0/(1-⍰)	(Kg/cm2)
60	1.6	0.06	36.8	0.008571429	11.4391902	3.21701093
90	3	0.09	69	0.012857143	11.48885384	6.00582103
120	3.8	0.12	87.4	0.017142857	11.53895058	7.5743456
150	5.2	0.15	119.6	0.021428571	11.58948613	10.3196982
180	6.8	0.18	156.4	0.025714286	11.64046628	13.4358879
210	7.2	0.21	165.6	0.03	11.69189691	14.1636555
240	9.4	0.24	216.2	0.034285714	11.74378402	18.4097391
270	9.8	0.27	225.4	0.038571429	11.79613373	19.1079556
300	11.2	0.3	257.6	0.042857143	11.84895224	21.7403188
330	12.2	0.33	280.6	0.047142857	11.90224588	23.5753825
360	12.8	0.36	294.4	0.051428571	11.95602108	24.6235765
390	13.2	0.39	303.6	0.055714286	12.01028442	25.2783355
410	13.2	0.41	303.6	0.058571429	12.04673445	25.2018504
440	13	0.44	299	0.062857143	12.10182622	24.7070148

Table 4.4.3 U.C.S Test with 10%Ash

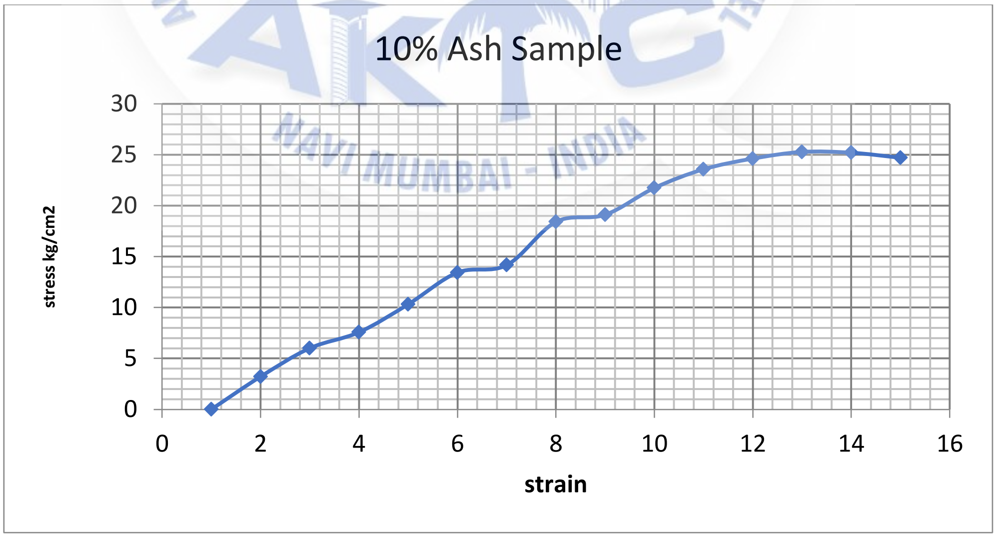


Figure 4.4.3:-Graph of U.C.S Test

4.4.4:- Soil and 15% ash.

DG R	PRR	Deformation	Load	Strain	Corrected Area	Compressive stress =P/A
		Δ=DGR*G/10	P=PRR*CF*100	□=(ΔL/L0)	A=A0/(1-□)	(Kg/cm2)
60	0.6	0.06	13.8	0.00857142	11.4391902	1.20637910
90	1.4	0.09	32.2	0.01285714	11.48885384	2.80271648
120	2.2	0.12	50.6	0.0171428	11.53895058	4.38514747
150	3.2	0.15	73.6	0.02142857	11.58948613	6.35058355
180	6.6	0.18	151.8	0.02571428	11.64046628	13.0407147
210	9.2	0.21	211.6	0.03	11.69189691	18.0980042
240	10.4	0.24	239.2	0.03428571	11.74378402	20.3682219
270	11.6	0.27	266.8	0.03857142	11.79613373	22.6175801
300	12.4	0.3	285.2	0.04285714	11.84895224	24.0696387
330	13.2	0.33	303.6	0.04714285	11.90224588	25.5077909
360	14.2	0.36	326.6	0.05142857	11.95602108	27.3167802
390	14.2	0.39	326.6	0.05571428	12.01028442	27.1933610
410	14	0.41	322	0.05857142	12.04673445	26.7292353

Table 4.4.4 U.C.S Test With 15%Ash

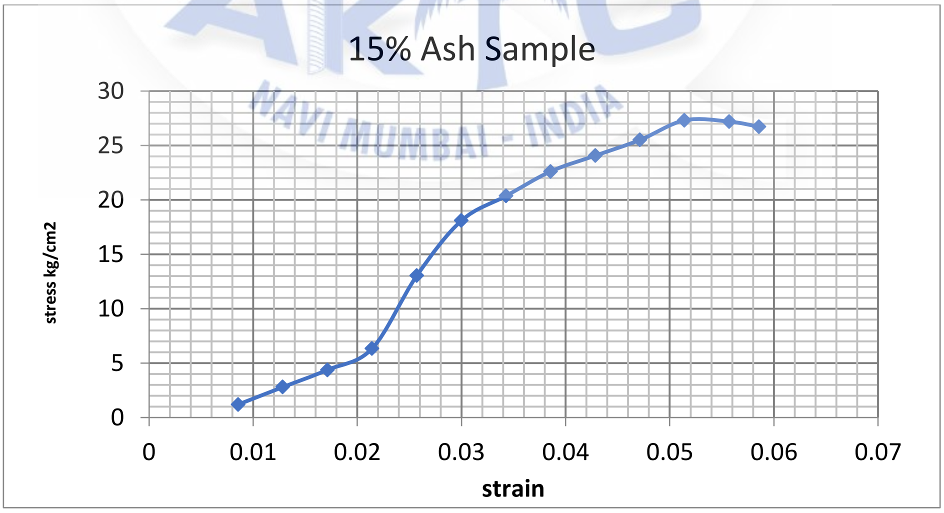


Figure 4.4.4:-Graph of U.C.S Test.

4.4.5:- Soil and 20 % ash.

DGR	PRR	Deformation	Load	Strain	Corrected Area	Compressive stress =P/A
		Δ=DGR*G/10	P=PRR*CF*100	⍰ =(ΔL/L0)	A=A0/(1-⍰)	(Kg/cm2)
60	1.2	0.06	27.6	0.00857142	11.4391902	2.412758203
90	1.8	0.09	41.4	0.01285714	11.48885384	3.603492619
120	4.4	0.12	101.2	0.01714285	11.53895058	8.770294949
150	6.2	0.15	142.6	0.02142857	11.58948613	12.30425563
180	8.6	0.18	197.8	0.02571428	11.64046628	16.99244646
210	10.4	0.21	239.2	0.03	11.69189691	20.45861351
240	11.8	0.24	271.4	0.03428571	11.74378402	23.11009803
270	12.6	0.27	289.8	0.03857142	11.79613373	24.56737153
300	13.4	0.3	308.2	0.04285714	11.84895224	26.01073865
330	13.6	0.33	312.8	0.04714285	11.90224588	26.28075434
360	14.2	0.36	326.6	0.05142857	11.95602108	27.3167802
390	14.4	0.39	331.2	0.05571428	12.01028442	27.57636609
410	14.4	0.41	331.2	0.05857142	12.04673445	27.49292777
440	14	0.44	322	0.06285714	12.10182622	26.60755444

Table 4.4.5 U.C.S Test With 20 % Ash

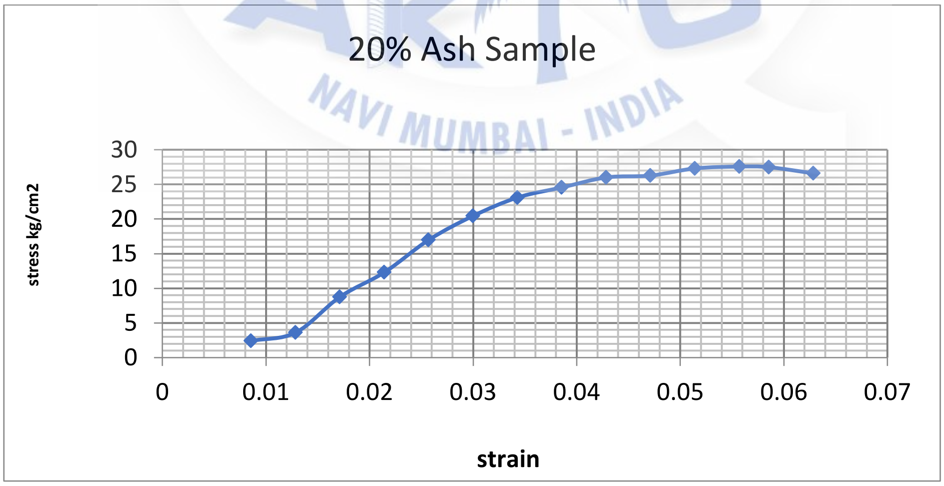


Figure 4.4.5:-Graph of U.C.S Test

4.5 Standard Proctor test:-

OMC is carried out on soil with varying percentage of soil to check density effect of soil with different amount of Water content . Table 4.5.1 shows percentage of water used and Optimum moisture content achieved to corresponding percentage.

sr.no	% of water added	optimum moisture content	dry dencity in kN/m^3	bulk density in kN/m^3
1	8	20.68	14.37	17.338
2	10	21.9	14.987	18.27
3	12	25	15.44	19.305
4	14	28	15.85	20.288
5	16	30.5	15.65	40.43

Table 4.5

Figure 4.5 Graph of O.M.C

Chapter 5

Conclusion

5.1 General

Study of soil stabilization by using municipal solid waste ash gives us wider use of Solid waste in soil stabilization which definitely reduces the environmental pollution level as well as enhances the soil properties.

5.2 Conclusion

1 Experimental study of soil stabilization with Municipal Solid waste ash shows use of Municipal solid waste in 10% with soil enhances soil properties viz. Shear strength, permeability and compaction.

2. This study shows instead of having simply disposal of Municipal Solid Waste, which is also not possible due to lack of land availability, we can improve soil properties by using waste ash.

3.As soil and waste is variable material in characteristics from place to place, thisexperimental work is only applicable to our Panvel region soil with particular type of Municipal solid waste. For different area's soil and different waste, we can have different Optimum percentage of waste ash which will enhance soil properties.

4. Using waste ash as stabilising material is cheap as well as eco-friendly method of soil stabilization, which solves the waste disposal problems as well as enhances soil properties.

5.3 Future scope of work

1. We can analyse this technique for larger extent. In this study only shear strength, permeability and compaction is assessed where in future we can go for soil bearing capacity, consolidation etc.

2.Due to incineration of Municipal solid waste for this technique, air pollution may credit in atmosphere, for that purpose air assessment is a major scope of work.

3. Municipal solid waste contains some organic fraction also which may responsible for leachate formation in soil, if this leachate infiltrate through soil and joins ground water it will contaminate ground water. For analysing this problem we can concentrate on leachate formation and remedial measures in future.

Chapter 6

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