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Strength Characterstics of Stabilized Embankment Using Fly Ash

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Abstract: *Infrastructure projects such as highways, railways, water reservoirs, reclamation etc. requires earth material in very large quantity. In urban areas, borrow earth is not easily available which has to be hauled from a long distance. Quite often, large areas are covered with highly plastic and expansive soil, which is not suitable for such purpose. Extensive laboratory / field trials have been carried out by various researchers and have shown promising results for application of such expansive soil after stabilization with additives such as sand, silt, lime, fly ash, etc. As fly ash is freely available, for projects in the vicinity of a Thermal Power Plants, it can be used for stabilization of expansive soils for various uses. The present paper describes a study carried out to check the improvements in the properties of expansive soil with fly ash in varying percentages. Laboratory tests like 1) grain size analysis 2) atterberg limits 3) standard compaction 4) permeability test have been carried out on soil and CBR (California bearing ratio) test is conducted on soil sample mixed with fly ash in various proportions and results are reported in this paper. One of the major difficulties in field application is thorough mixing of the two materials (expansive soil and fly ash) in required proportion to form a homogeneous mass. The fly ash is mixed with soil in required proportion until the CBR value for the embankment is achieved.*
Keywords: *compaction, CBR test, flies ash, laboratory tests, plastic clay, stabilization, expansive soils, and embankment.*

I. Introduction

A land-based structure of any type is only as strong as its Foundation. For that reason, soil is a critical element influencing the success of a construction project. Soil is either part of the foundation or one of the raw materials used in the construction process. Therefore, understanding the engineering properties of soil is crucial to obtain strength and economic permanence. Soil stabilization is the process of maximizing the suitability of soil for a given construction purpose

1.1 What Is Soil Stabilization?

Soil is one of nature's most abundant construction materials. Almost all construction is built with or upon soil. When unsuitable construction conditions are encountered, a contractor has four options:
(1) Find a new construction site
(2) Redesign the structure so it can be constructed on the poor soil
(3) Remove the poor soil and replace it with good soil
(4) Improve the engineering properties of the site soils

In general, Options 1 and 2 tend to be impractical today, while in the past, Option 3 has been the most commonly used method. However, due to improvement in technology coupled with increased transportation costs, Option 4 is being used more often today and is expected to dramatically increase in the future. Improving an on-site (in situ) soil's engineering properties is referred to as either "soil modification" or "soil stabilization." The term "modification" implies a minor change in the properties of a soil, while stabilization means that the engineering properties of the soil have been changed enough to allow field construction to take place.

Nearly every road construction project will utilize one or both of these stabilization techniques. The most common form of "mechanical" soil stabilization is compaction of the soil, while the addition of cement, lime, bituminous, or other agents is referred to as a "chemical" or "additive" method of soil stabilization. There are two basic types of additives used during chemical soil stabilization: mechanical additives and chemical additives. Mechanical additives, such as soil cement, mechanically alter the soil by adding a quantity of a material that has the engineering characteristics to upgrade the load-bearing capacity of the existing soil. Chemical additives, such as lime, chemically alter the soil itself, thereby improving the load-bearing capacity of the soil.

1.2 Why And When Is It Used?

Traditionally, stable sub-grades, sub-bases and/or bases have beenconstructed by using selected, well-graded aggregates, making it fairly easy to predict the load-bearing capacity of the constructed layers. By using select material, the engineer knows that the foundation will be able to support the design loading.

Gradation is an important soil characteristic to understand. A soil is considered either "well-graded" or "uniformly-graded" (also referred to as "poorly-graded"). This is a reference to the sizes of the particles in the materials. Uniformly-graded materials are made up of individual particles of roughly the same size. Well-graded materials are made up of an optimal range of different sized particles.

It is desirable from an engineering standpoint to build upon a foundation of ideal and consistent density. Thus, the goal of soil stabilization is toprovide a solid, stable foundation. “Density” is the measure of weight by volume of a material, and is one of the relied-upon measures of the suitability of a material for construction purposes. The more density a material possesses, the fewer voids are present. Voids are the enemy of road construction; voids provide a place for moisture to go, and make the material less stable by allowing it to shift under changing pressure, temperature and moisture conditions.

Uniformly-graded materials, because of their uniform size, are much less dense than well-graded materials. The high proportion of voids per volume of uniformly-graded material makes it unsuitable for construction purposes. In well-graded materials, smaller particles pack in to the voids between the larger particles, enabling the material to achieve high degrees of density. Therefore, well-graded materials offer higher stability, and are in high demand for construction.

With the increased global demand for energy and increasing local demand for aggregates, it has become expensive from a material cost and energy use standpoint to remove inferior soils and replace them with choice, well-graded aggregates. One way to reduce the amount of select material needed for base construction is to improve the existing soil enough to provide strength and conform to engineering standards. This is where soil stabilization has become a cost-effective alternative.

Fig 1: laying of pavement by soil stabilization process

Essentially, soil stabilization allows engineers to distribute a larger load with less material over a longer life cycle.

There are many advantages to soil stabilization:

- Stabilized soil functions as a working platform for the project
- Stabilization waterproofs the soil
- Stabilization improves soil strength
- Stabilization helps reduce soil volume change due to temperature or moisture
- Stabilization improves soil workability
- Stabilization reduces dust in work environment
- Stabilization upgrades marginal materials
- Stabilization improves durability
- Stabilization dries wet soils
- Stabilization conserves aggregate materials
- Stabilization reduces cost
- Stabilization conserves energy

1.3 Applications:-

Soil stabilization is used in many sectors of the construction industry. Roads, parking lots, airport runways, building sites, landfills, and soil remediation all use some form of soil stabilization. Other applications include waterway management, mining, and agriculture

There are two primary methods of soil stabilization used today:

• Mechanical stabilization

• Chemical or additive stabilization

1.4 Chemical Soil Stabilization:-

One method of improving the engineering properties of soil is by adding chemicals or other materials to improve the existing soil. This technique is generally cost effective: for example, the cost, transportation, and processing of a stabilizing agent or additive such as soil cement or lime to treat an in-place soil material will probably be more economical than importing aggregate for the same thickness of base course. Additives can be mechanical, meaning that upon addition to the parent soil their own load-bearing properties bolster the engineering characteristics of the parent soil. Additives can also be chemical, meaning that the additive reacts with or changes the chemical properties of the soil, thereby upgrading its engineering properties. Placing the wrong kind or wrong amount of additive – or, improperly incorporating the additive into the soil – can have devastating results on the success of the project. So, in order to properly implement this technique, an engineer must have:

1) A clear idea of the desired result
2) An understanding of the type(s) of soil and their characteristics on site
3) An understanding of the use of the additive(s), how they react with the soil type and other additives, and how they interact with the surrounding environment
4) An understanding of and means of incorporating (mixing) the additive
5) An understanding of how the resulting engineered soil will perform

Combining the additives with the soil is typically done with various machines. The method used is usually based on three factors: what machines are available, the location (urban or rural), and the additives that are being used. The mixing should be as uniform as possible.

The most economic and time-efficient method is to use a rotary mixer, a large machine that incorporates additives with the soil by tumbling them in a large mixing chamber equipped with a rotor designed to break up and mix the materials. It is capable of uniformly introducing additives and water while breaking up the soil into an optimal homogenous grade.

The rotary mixer does all mixing in place, and is unrivalled in production by other methods. For some applications that require more precision, a pug mill is used. A pugmill is essentially a large mixing chamber that is similar to a cement mixer. Measured pre-graded aggregates, additives, and usually water are mixed in the pugmill and then applied to uniform thickness. Pug mills produce high quality stabilization, but at higher costs and slower production.

Blade mixing is done with the use of a motor grader. Blade mixing is not nearly as efficient as the previously described systems, but it is far less complex. Essentially, the additive is placed in flat windrows and the blade of the grader mixes the additive with the soil in a series of turning and tumbling actions. Other machines are similarly used for mixing aswell, including scarifies, plows, and disks. It is very difficult to uniformly control mixing percentages and mixing depth using this technique

Fig 2: Addition of chemical to the soil

1.4.1additives:-

There are many kinds of additives available. Not all additives work for all soil types, and a single additive will perform quite differently with different soil types. Generally, an additive may be used to act as a binder, alter the effect of moisture, increase the soil density or neutralize the harmful effects of a substance in the soil. Following are some of the most widely used additives and their applications:

1) Portland Cement:

Portland cement is a mechanical additive that can be used for soil modification (to improve soil quality) or soil stabilization (to convert the soil to a solid cement mass). The amount of cement used will dictatewhether modification or stabilization has occurred. Nearly all types ofsoil can benefit from the strength gained by cement stabilization.

However, the best results have occurred when used with well-graded fines that possess enough fines to produce a floating aggregate matrix

2) Quicklime/Hydrated Lime:

Lime is a chemical additive that has been utilized as a stabilizing agent in soils for centuries. Experience has shown that lime will react well with medium, moderately fine, and fine-grained clay soils. In clay soils, the main benefit from lime stabilization is the reduction of the soil's plasticity: by reducing the soil's water content, it becomes more rigid. It also increases the strength and workability of the soil, and reduces the soil's ability to swell. It is very important to achieve proper gradation when applying lime to clay soils. By breaking up the clay into small- sized particles, you allow the lime to introduce homogeneously and properly react with the clay.

Lime can be applied dry to the soil, but if blowing dust is of concern or the work is being done in a populated area, the lime can be mixed with water to form slurry. A curing time of 3 to 7 days is normal to allow the lime to react with the soil, during which the surface of the stabilized soil should be wetted periodically.

3) Fly Ash:

Fly ash, a chemical additive consisting mainly of silicon and aluminium compounds, is a by-product of the combustion of coal. Fly ash can be mixed with lime and water to stabilize granular materials with few fines, producing a hard, cement-like mass. Its role in the stabilization process is to act as a pozzolan and/or as a filler product to reduce air voids. A common application is as part of a lime/cement/fly ash mixture (LCF) to stabilize coarse-grained soils that possess little or no fine grains. Because voids, increasing the density of soil. It is essentially a waste product; it can be obtained rather inexpensively.

Fig 3: fly ash

4) Calcium Chloride:

Calcium chloride is a chemical additive that has the ability to absorb moisture from the air until it liquefies into a solution. The presence of calcium chloride in the moisture of a soil lowers the freezing temperature of that moisture. For this reason, calcium chloride is a proven stabilizing additive for cold-climate applications. If the water in the soil can't freeze, there is less soil movement (i.e., frost heaves), making it much more stable. Calcium chloride also works well as a binder, making the soil easier to compact and reducing dust.

5) Bitumen:

Bitumen is a mechanical additive that occurs naturally or as a by-product of petroleum distillation. It is the black pitch used to make asphalt. Asphalt cement, cutback asphalt, tar, and asphalt emulsions are all used to achieve bituminous soil stabilization. Soil type, construction method and weather are all factors in choosing which bitumen to use. Bitumen makes soil stronger and resistant to water and frost. The use of bitumen can lead to fewer weather-related delays during construction, and makes to reclaimed roadbed. Compactionis easier and more consistent.

Fig 4: bitumen mixed with aggregate

6) Chemical or Bio Remediation:

Our industrial society produces many benefits, but occasionally there are unintentional, accidental, or criminal problems that occur. Petroleum hydrocarbons, lead, PCBs, solvents, pesticides, and other hazardous natural and man-made substances often contaminate soil. Because even contaminated real estate can be valuable – and because pollution is undesirable to begin with – efforts are made to return contaminated soil to an acceptable condition for human habitation.

The goal of chemical or bio remediation is to convert hazardous substances into inert ones and to prevent hazardous substances from spreading or leaching. The type of additive depends on the contaminant(s) and the environment. Chemical additives are often proprietary chemical cocktails, but the science is well understood and they are quite effective at neutralizing hazardous substances. Bio remediation is typically done by the introduction of natural means: bacteria or insects that eat contaminants and convert them to inert waste, or plants that filter out contaminants and convert them to natural substances.

1.5 Mechanical Soil Stabilization:-

Mechanical soil stabilization refers to either compaction or the introduction of fibrous and other non-biodegradable reinforcement to the soil. This practice does not require chemical change of the soil, although it is common to use both mechanical and chemical means to achieve specified stabilization. There are several methods used to achieve mechanical stabilization:

1) Compaction:

Compaction typically employs a heavy weight to increase soil density by applying pressure from above. Machines are often used for this purpose; large soil compactors with vibrating steel drums efficiently apply pressure to the soil, increasing its density to meet engineering requirements. Operators of the machines must be careful not to over- compact the soil, for too much pressure can result in crushed aggregates that lose their engineering properties.

Fig 5: compaction by rolling

2) Soil Reinforcement:

Soil problems are sometimes remedied by utilizing engineered or non- engineered mechanical solutions. Geo-textiles and engineered plastic mesh are designed to trap soils and help control erosion, moisture conditions and soil permeability. Larger aggregates, such as gravel, stones, and boulders, are often employed where additional mass and rigidity can prevent unwanted soil migration or improve load-bearing properties.

3) Addition of Graded Aggregate Materials:

A common method of improving the engineering characteristics of a soil is to add certain aggregates that lend desirable attributes to the soil, such as increased strength or decreased plasticity. This method provides material economy, improves support capabilities of the subgrade, and furnishes a working platform for the remaining structure.

Fig 6: addition of aggregate to the soil

1.5.1 Mechanical Remediation:-

Traditionally, mechanical remediation has been the accepted practice for dealing with soil contamination. This is a technique where contaminated soil is physically removed and relocated to a designated hazardous waste facility far from centres of human population. In recent times, however, chemical and bio remediation have proven to be a better solution, both economically and environmentally. It is often cheaper to solve the problem where it exists rather than relocate the problem somewhere else and possibly need to deal with it again in the future.

1.6 The Basic Soil Stabilization:-

Both new construction and rehabilitation projects are candidates for soil stabilization. While the precise stabilization procedures will vary depending on many factors – including location, environment, time requirements, budget, available machinery, and weather – the following process is generally practiced:

Assessment and Testing:

The soils of the site are thoroughly tested to determine the existing conditions. Based on analysis of existing conditions, additives are selected and specified. Generally, a target chemical percentage by weight and a design mix depth are defined for the sub-base contractor. The selected additives are subsequently mixed with soil samples and allowed to cure. The cured sample is then tested to ensure that the additives will produce the desired results.

Site Preparation:

The existing materials on site, including existing pavement if it is being reclaimed, is pulverized utilizing a rotary mixer. Any additional aggregates or base materials are introduced at this time. The material is brought to the optimal moisture content by drying overly wet soil or adding water to overly dry soil. The grade is shaped if necessary to obtain the specified material depth.

Fig 7: excavation of surface soil

Introduce Additives:

Cement, lime or fly ash can be applied dry or wet. When applied dry, it is typically spread at a required amount per square yard (meter) or station utilizing a cyclone spreader or other device. When lime is applied as slurry, it is either spread with a tanker truck or through the rotary mixer's on-board water spray system. Calcium chloride is usually applied by a tanker truck equipped with a spray bar. Bituminous additives are typically added utilizing an on-board emulsion spray system on a rotary mixer. It can also be sprayed on the surface, but this method requires several applications and additional mixing.

Fig 8: introduction of chemical additives

Mixing:

To fully incorporate the additives with the soil, a rotary mixer makes several mixing passes until the materials are homogenous and well- graded. It is crucial that the rotary mixer maintains optimal mixing depth, as mixing too shallow or too deep will create undesirable proportions of soil and additive. Inappropriate proportions of soil and additive will decrease the load-bearing properties of the cured layer.

Some projects require multiple layers of treated and compacted soil. When applying cement and fly ash, it is important to finish mixing as soon as possible due to the quick-setting characteristics of the additives.

Fig 9: mixing of fly ash with soil

Compaction and Shaping/Trimming:

Compaction usually follows immediately after mixing, especially when the additive is cement or fly ash. Some bituminous additives require a delay between mixing and compaction to allow for certain chemical changes to occur. Compaction is accomplished through several passes using different machines. Initial compaction is begun utilizing a vibratory pad foot compactor. The surface is then shaped and trimmed to remove pad marks and provide a more suitable profile. Intermediate compaction follows utilizing a pneumatic compactor, which provides a certain kneading action that further increases soil density. A tandem drum roller is used on the finishing pass to provide a smooth surface. A final shaping gives the material a smooth finish and a proper crown and grade.

Fig 10: compaction

Curing:

Sufficient curing will allow the additive to fully achieve its engineering potential. For cement, lime, and fly ash stabilization, weather and moisture are critical factors, as the curing can have a direct bearing on the strength of the stabilized base. Bituminous-stabilized bases often require a final membrane of medium-curing cutback asphalt or slow- curing emulsified asphalt as a moisture seal. Generally, a minimum of seven days are required to ensure proper curing. During the curing period, samples taken from the stabilized base will reveal when the moisture content is appropriate for surfacing.

Fig 11: curing process

Fig 12: preparation of layer for curing

II. Literature Review

Fly ash by itself has little cementatious value but in the presence of moisture it reacts chemically and forms cementatious compounds and attributes to the improvement of strength and compressibility characteristics of soils. It has a long history of use as an engineering material and has been successfully employed in geotechnical applications.

Erdalcokca (2001):

Effect of Fly ash on expansive soil was studied by Erdal,Cokca,Fly ash consists of often hollow spheres of silicon, aluminium and iron oxides and unoxidized carbon. There are two major classes of fly ash, class C and class F. The former is produced from burning anthracite or bituminous coal and the latter is produced from burning lignite and sub bituminous coal. Both the classes of fly ash are puzzolans, which are defined as siliceous and aluminous materials. Thus Fly ash can provide an array of divalent and trivalent cations ($Ca2+$,$Al3+$,$Fe3+$etc) under ionized conditions that can promote flocculation of dispersed clay particles. Thus expansive soils can be potentially stabilized effectively by cation exchange using fly ash. He carried out investigations using Soma Flyash and Tuncbilekfly ash and added it to expansive soil at 0-25%. Specimens with fly ash were cured for 7days and 28 days after which they were subjected to Oedometer free swell tests. And his experimental findings confirmed that the plasticity index, activity and swelling potential of the samples decreased with increasing per cent stabilizer and curing time and the optimum content of fly ash in decreasing the swell potential was found to be 20%. The changes in the physical properties and swelling potential is a result of additional silt size particles to some extent and due to chemical reactions that cause immediate flocculation of clay particles and the time dependent puzzolanic and self-hardening properties of fly ash and he concluded that both high –calcium and low calcium class C fly ashes can be recommended as effective stabilizing agents for improvement for improvement of expansive soils.

Pandian et.al. (2002):

Studied the effect of two types of fly ashes Raichur fly ash (Class F) and Neyveli fly ash (Class C) on the CBR characteristics of the black cotton soil. The fly ash content was increased from 0 to 100%. Generally the CBR/strength is contributed by its cohesion and friction. The CBR of BC soil, which consists of predominantly of finer particles, is contributed by cohesion. The CBR of fly ash, which consists predominantly of coarser particles, is contributed by its frictional component. The low CBR of BC soil is attributed to the inherent low strength, which is due to the dominance of clay fraction. The addition of fly ash to BC soil increases the CBR of the mix up to the first optimum level due to the frictional resistance from fly ash in addition to the cohesion from BC soil. Further addition of fly ash beyond the optimum level causes a decrease up to 60% and then up to the second optimum level there is an increase. Thus the variation of CBR of fly ash-BC soil mixes can be attributed to the relative contribution of frictional or cohesive resistance from fly ash or BC soil, respectively. In Neyveli fly ash also there is an increase of strength with the increase in the fly ash content, here there will be additional puzzolonic reaction forming cementitious compounds resulting in good binding between BC soil and fly ash particles

Phanikumar and Sharma (2004):

A similar study was carried out by Phanikumar and Sharma and the effect of fly ash on engineering properties of expansive soil through an experimental programme. The effect on parameters like free swell index (FSI), swell potential, swelling pressure, plasticity, compaction, strength and hydraulic conductivity of expansive soil was studied. The ash blended expansive soil with fly ash contents of 0, 5, 10,15 and 20% on a dry weight basis and they inferred that increase in fly ash content reduces plasticity characteristics and the FSI was reduced by about 50% by the addition of 20% fly ash. The hydraulic conductivity of expansive soils mixed with fly ash decreases with an increase in fly ash content, due to the increase in maximum dry unit weight with an increase in fly ash content. When the fly ash content increases there is a decrease in the optimum moisture content and the maximum dry unit weight increases. The effect of fly ash is akin to the increased compactive effort. Hence the expansive soil is rendered more stable. The undrained shear strength of the expansive soil blended with fly ash increases with the increase in the ash content.

The comprehensive review of literature shows that a considerable amount of work related to the determination of deformation characteristics and strength characteristics of expansive soil is done worldwide.From various contributions, the investigations on strength characteristics of expansive soil conducted by S.Narasimharaoet.al (1987, 1996); Sridharan et.al (1989); Mathew et.al (1997); G.RajaSekaranet.al (2002); Ali.M.A. Abd-Allah (2009) are worthy of note.

Improving the strength of soil by stabilization technique was performed by SupakjiNontananandh et.al (2004) and Can BurakSisman and ErhanGezer (2011).

The effect of electrolytes on soft soils were explained by Sivanna, G.S (1976);Anandakrishnan et.al (1966); Saha et.al (1991); Rao, M.S et.al(1992);Sivapullaiah,P.V. et al(1994); Bansal et.al(1996); S. Narasimha Rao et.al(1996); Appamma, P.,(1998); Chandrashekar et.al (1999);G. Rajasekaran et.al (2000); J. Chu et.al (2002);MatchalaSuneelet.al (2008).

The effect of steel industrial wastes on soft soils were presented by Ashwani Kumar et.al (1998); Bhadra, T. K et.al (2002); Dr. D. D. Higgins (2005**);** Koteswara Rao (2006).

III. Materials

The materials that are used in this are mainly-

1) Black cotton soil
2) Fly ash

3.1 Black cotton soil:-

The soil used in our project is black cotton soil which is collected the from a field in sankarpalli village nearer to patancheruvu, greater Hyderabad (dist.) The soil is fully exposed to the atmosphere and several crops are grown in it throughout the year. Due to exposure to several atmospheric conditions there is frequent expansion and contraction in the soil mass. By this uneven expansion and contraction alternative cracks are formed in the soil mass which seriously affect the performance of pavement when used as subgrade.

Rich proportion of montmorillonite is found in Black cotton soil from mineralogical analysis. High percentage of montomonillonite renders high degree of expansiveness. These property results cracks in soil without any warning. These cracks may sometimes extent to severe limit like ½" wide and 12" deep. So building to be founded on this soil may suffer severe damage with the change of atmospheric conditions.

As plasticity index and linear shrinkage decreased with the increase of lime content, a mixture of both lime and cement is necessary for adequate stabilization of road bases for heavy wheel loads on the black cotton soils.

The black cotton soil is tested in our college laboratory to access various soil properties like plastic limit of the clay, liquid limit, max dry density and optimum moisture content, grain size distribution, strength etc. which will give the basic idea of the type of soil.

1) They are very fertile
2) They are black in colour
3) They are high in organic matter.
4) They often form in grasslands and wetlands.
5) Organic matter contains plant nutrients and it also improves the physical properties of the soil, enhancing it for plant growth.
6) It is also known as regur soil.

Fig 13: black cotton soil

3.2 Fly ash:-

Fly ash is a fine, glass powder recovered from the gases of burning coal during the production of electricity. These micron-sized earth elements consist primarily of silica, alumina and iron. These power plants grind coal to powder fineness before it is burned. Fly ash - the mineral residue produced by burning coal - is captured from the power plant's exhaust gases and collected for use.

Fly ash, also known as flue-ash, is one of the residues generated in combustion, and comprises the fine particles that rise with the flue gases. Ash which does not rise is termed bottom ash. In an industrial context, fly ash usually refers to ash produced during combustion of coal. Fly ash is generally captured by electrostatic precipitators or other particle filtration equipment before the flue gases reach the chimneys of coal-fired power plants and together with bottom ash removed from the bottom of the furnace is in this case jointly known as coal ash. Depending upon the source and makeup of the coal being burned, the components of fly ash vary considerably, but all fly ash includes substantial amounts of silicon dioxide (SiO2) (both amorphous and crystalline) and calcium oxide (CaO), both being endemic ingredients in many coal-bearing rock strata.

Fig 14: fine powdered fly ash

3.2.1 Properties Of Fly Ash:

Fineness: The fineness of fly ash is important because it affects the rate of pozzolonic activity and the workability of the concrete. Specifications require a minimum of 66 percent passing the 0.044 mm (No. 325) sieve.

Specific gravity: Although specific gravity does not directly affect concrete quality, it has value in identifying changes in other fly ash characteristics. It should be checked regularly as a quality control measure, and correlated to other characteristics of fly ash that may be fluctuating.

Chemical composition: The reactive aluminosilicate and calcium aluminosilicate components of fly ash are routinely represented in their oxide nomenclatures such as silicon dioxide, aluminium oxide and calcium oxide. The variability of the chemical composition is checked regularly as a quality control measure. The aluminosilicate components react with calcium hydroxide to produce additional cementitious materials. Fly ashes tend to contribute to concrete strength at a faster rate when these components are present in finer fractions of the fly ash.

Sulphur trioxide content is limited to five percent, as greater amounts have been shown to increase mortar bar expansion.

Available alkalis in most ashes are less than the specification limit of 1.5 per cent. Contents greater than this may contribute to alkali-aggregate expansion problems.

Carbon content. LOI is a measurement of unburned carbon remaining in the ash. It can range up to five percent per AASHTO and six percent per ASTM. The unburned carbon can absorb air entraining admixtures (AEAs) and increase water requirements. Also, some of the carbon in fly ash may be encapsulated in glass or otherwise be less active and, therefore, not affect the mix. Conversely, some fly ash with low LOI values may have a type of carbon with a very high surface area, which will increase the AEA dosages. Variations in LOI can contribute to fluctuations in air content and call for more careful field monitoring of entrained air in the concrete. Further, if the fly ash has a very high carbon content, the carbon particles may float to the top during the concrete finishing process and may produce dark-coloured surface streaks.

Two classes of fly ash are defined by ASTM C618: Class F fly ash and Class C fly ash. The chief difference between these classes is the amount of calcium, silica, alumina, and iron content in the ash. The chemical properties of the fly ash are largely influenced by the chemical content of the coal burned (i.e., anthracite, bituminous, and lignite).[7]

Not all fly ashes meet ASTM C618 requirements, although depending on the application, this may not be necessary. Ash used as a cement replacement must meet strict construction standards, but no standard environmental regulations have been established in the United States. 75% of the ash must have a fineness of 45 µm or less, and have carbon content, measured by the loss on ignition (LOI), of less than 4%. In the U.S., LOI needs to be under6%. The particle size distribution of raw fly ash is very often fluctuating constantly, due to changing performance of the coal mills and the boiler performance. This makes it necessary that, if fly ash is used in an optimal way to replace cement in concrete production, it needs to be processed using beneficiation methods like mechanical air classification. But if fly ash is used also as a filler to replace sand in concrete production, unbeneficiated fly ash with higher LOI can be also used. Especially important is the on-going quality verification. This is mainly expressed by quality control seals like the Bureau of Indian Standards mark or the DCL mark of the Dubai Municipality.

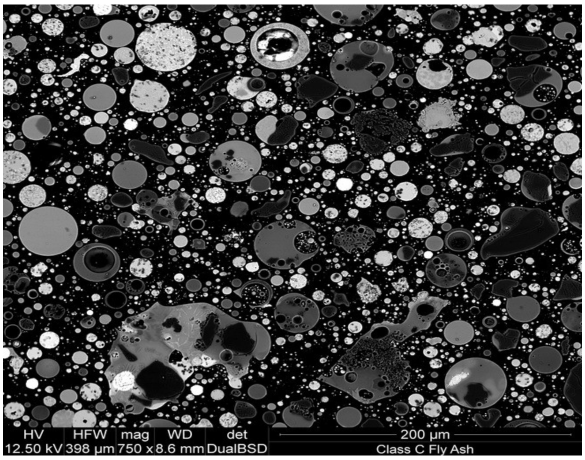

Fig 15: microscopic view of fly ash

3.2.2 Class F Fly Ash:

The burning of harder, older anthracite and bituminous coal typically produces Class F fly ash. This fly ash is puzzolanic in nature, and contains less than 20% lime (CaO). Possessing puzzolanic properties, the glassy silica and alumina of Class F fly ash requires a cementing agent, such as Portland cement, quicklime, or hydrated lime, with the presence of water in order to react and producecementious compounds. Alternatively, the additions of a chemical activator such as sodium silicate (water glass) to a Class F ash can lead to the formation of a geopolymer.

Fig 16: soil mixed with fly ash

3.2.3 Class C Fly Ash:

Fly ash produced from the burning of younger lignite or subbituminous coal, in addition to having pozzolanic properties, also has some self-cementing properties. In the presence of water, Class C fly ash will harden and gain strength over time. Class C fly ash generally contains more than 20% lime (CaO). Unlike Class F, self-cementing Class C fly ash does not require an activator. Alkali and sulphate (SO4) contents are generally higher in Class C fly ashes.

At least one US manufacturer has announced a fly ash brick containing up to 50% Class C fly ash. Testing shows the bricks meet or exceed the performance standards listed in ASTM C 216 for conventional clay brick; it is also within the allowable shrinkage limits for concrete brick in ASTM C 55, Standard Specification for Concrete Building Brick. It is estimated that the production method used in fly ash bricks will reduce the embodied energy of masonry construction by up to 90%.Bricks and pavers were expected to be available in commercial quantities before the end of 2009.

IV. Methodology

This includes the detailed description of stabilising the soil using fly ash which also includes the procedure of various laboratory tests that are behind the stabilisation. The following gives the step by step process of the things that are held in the laboratory.

4.1 Soil Treatment:-

After removing impurities like vegetation, stones etc. the soil was mixed with fly ash in varying proportion by volume. The Mixing was thoroughly carried out manually and the tests were conducted as per standard procedures.

Fly ash has been used in this project to improve the strength characteristics of soils. Fly ash can be used to stabilize bases or subgrades, to stabilize backfill to reduce lateral earth pressures and to stabilize embankments to improve slope stability. Typical stabilized soil depths are 15 to 46 centimetres (6 to 18 inches). The primary reason fly ash is used in soil stabilization applications is to improve the compressive and shearing strength of soils. The compressive strength of fly ash treated soils is dependent on:

1) In-situ soil properties
2) Delay time
3) Moisture content at time of compaction
4) Fly ash addition ratio

4.1.1 Delay Time:-

Delay time is the elapsed time measured between when the fly ash first comes into contact with water and final compaction of the soil, fly ash and water mixture. Compressive strength is highly dependent upon delay time. Both densities and strength are reduced with increasing delay to final compaction. Delay time is critical due to the rapid nature of the tricalcium aluminate (C3A) reaction that occurs when Class C fly ash is mixed with water. Densities and strengths are reduced because a portion of the compactive energy must be used to overcome the bonding of the soil particle by cementation and because a portion of the cementation potential is lost. Maximum strength in soil-fly ash mixtures is attained at no delay. Typically, a one-hour compaction delay is specified for construction purposes.

4.1.2 Moisture Content:-

The water content of the fly ash stabilized soil mixture affects the strength. The maximum strength realized in soil-fly ash mixtures generally occurs at moisture contents below optimum moisture content for density. For silt and clay soils the optimum moisture content for strength is generally four to eight percent below optimum for maximum density. For granular soils the optimum moisture content for maximum strength is generally one to three percent below optimum moisture for density. Therefore, it is crucial that moisture content be controlled during construction. Moisture content is usually measured using a nuclear density measurement device.

4.1.3 Addition Ratios:-

Typical fly ash addition rates are 8 percent to 16 percent based on dry weight of soil. The addition rate depends on the nature of the soil, the characteristics of the fly ash and the strength desired. The addition rate must be determined by laboratory mix design testing. In general the higher the addition rate the higher the realized compressive strength. Fly ashes for state department of transportation projects are usually specified to meet AASHTO M 295 (ASTM C 618), even though the requirements of this specification are not necessary for this application and may increase the ash supply costs. Increasingly non-AASHTO M 295 compliant materials are being successfully used. It should be noted that virtually any fly ash that has at least some self-cementitious properties can be engineered to perform in transportation projects.

4.1.4 Compaction Of Soil Fly Ash Mixtures:-

The density of soil with coal ashes is an important parameter since it controls the strength, Compressibility and permeability. The compacted unit weight of the material depends on the amount and method of energy application, grain size distribution, plasticity characteristics and moisture content at compaction. The variation of dry density with moisture content for fly ashes is less compared to that for a well-graded soil, both having the same grain size. The tendency for fly ash to be less sensitive to variation in moisture content than for soil is due to higher air void content of fly ash. The higher void content could tend to limit the buildup of pores pressures during compaction, thus allowing the fly ash to be compacted over a larger range of water content.

4.2 Soil Properties:-

The plasticity of soils treated with Class C or other high-calcium fly ash is influenced by the types of clay minerals present in the soil and their adsorbed water. Soils containing more than 10 percent sulphates have been prone to swell excessively in some applications. Also, organic soils are difficult to stabilize using fly ash.
The purpose of soil classification is to arrange various types of soils into groups according to their engineering or agricultural properties and various other characteristics. Soil possessing similar characteristics can be placed in the same group. Soil survey and soil classification are carried out by several agencies for different purposes. For example the agricultural departments undertake soil investigations from the point of view of the suitability or otherwise of the soil for crops and its fertility. However from engineering point of view the classification may

be done with the objective of finding the suitability of the soil for construction of dams, higher ways or foundations etc. For example engineering purposes soils may be classified by the following systems.
1) Particle size classification
2) Textural classification
3) Highway Research Board (HRB) classification
4) United soil classification and IS classification system.

The division of A-7 group on the basis of the demarcation line (Ip=WL-30) into A-7-5 and A-7-6 subgroups does not appear to divide the Indian Black Cotton soil into two distinct groups having maximum value of the group index as 20 only. Based on investigations carried out at the Central Road Research Laboratory, New Delhi (1953), a classification of black cotton soil into narrow sub-groups has been suggested extending the maximum value of group index from 20 to 50.

V. Laboratory Tests

Following laboratory tests have been carried out as per IS: 2720. The tests were carried out both on natural soil and stabilized soil with fly ash collected from Vijayawada Thermal Power Plant.

1) Grain Size Analysis
2) Atterberg Limit Test
3) Proctor Compaction Test
4) Unconfined Compression Test
5) Permeability Test

5.1 Grain size analysis:-

Fig 17: set of sieves

1) Take a representative oven dried sample of soil that weighs about 500 g. (this is normally used for soil samples the greatest particle size of which is 4.75 mm)
2) If soil particles are lumped or conglomerated crush the lumped and not the particles using the pestle and mortar.
3) Determine the mass of sample accurately. Wt (g)
4) Prepare a stack of sieves. Sieves having larger opening sizes (i.e lower numbers) are placed above the ones having smaller opening sizes (i.e higher numbers). The very last sieve is #200 and a pan is placed under it to collect the portion of soil passing #200 sieves. Here is a full set of sieves. (#s 4 and 200 should always be included)

Fig 18: sieve analysis test

5) Make sure sieves are clean; if many soil particles are stuck in the openings try to poke them out using brush.
6) Weigh all sieves and the pan separately. (Fill in column 3)
7) Pour the soil from step 3 into the stack of sieves from the top and place the cover, put the stack in the sieve shaker and fix the clamps, adjust the time on 10 to 15 minutes and get the shaker going.
8) Stop the sieve shaker and measure the mass of each sieve + retained soil.

Fig.19: Operation of motor in sieve analysis

Sieve size(mm)	Mass of soil retained(gm)	% on each sieve	Cumulative % retained	% fines
4.75	130	13.06	13.06	86.94
2.36	48	4.82	17.88	82.12
2.00	108	10.85	28.73	71.27
1.00	240	24.12	52.85	47.15
0.6	132	13.26	66.11	33.89
0.425	108	10.85	76.96	23.04
0.30	78	7.85	84.81	15.19
0.15	111	11.15	95.96	4.04
0.075	40	4.02	100	

Table.1: Sieve analysis

5.2 Atterberg limits:-

The Atterberg limits are a basic measure of the nature of a fine-grained soil. Depending on the water content of the soil, it may appear in four states: solid, semi-solid, plastic and liquid. In each state, the consistency and behaviour of a soil is different and consequently so are its engineering properties. Thus, the boundary between each state can be defined based on a change in the soil's behaviour. The Atterberg limits can be used to distinguish between silt and clay, and it can distinguish between different types of silts and clays. These limits were created by Albert Atterberg, a Swedish chemist.They were later refined by Arthur Casagrande. These distinctions in soil are used in assessing the soils that are to have structures built on. Soils when wet retain water and some expand in volume. The amount of expansion is related to the ability of the soil to take in water and its structural make-up (the type of atoms present). These tests are mainly used on clayey or silty soils since these are the soils that expand and shrink due to moisture content. Clays and silts react with the water and thus change sizes and have varying shear strengths. Thus these tests are used widely in the preliminary stages of designing any structure to ensure that the soil will have the correct amount of shear strength and not too much change in volume as it expands and shrinks with different moisture contents.

5.2.1 Plastic limit:

The plastic limit is determined by rolling out a thread of the fine portion of a soil on a flat, non-porous surface. The procedure is defined in ASTM Standard D 4318.

If the soil is plastic, this thread will retain its shape down to a very narrow diameter. The sample can then be remoulded and the test repeated.

As the moisture content falls due to evaporation, the thread will begin to break apart at larger diameters. The plastic limit is defined as the moisture content where the thread breaks apart at a diameter of 3 mm (about 1/8 inch).

A soil is considered non-plastic if a thread cannot be rolled out down to 3 mm at any moisture.

Procedure:

1) From the 20g sample select a 1.5 to 2 g specimen for testing.
2) Roll the test specimen between the palm or fingers on the ground glass plate to from a thread of uniform diameter.
3) Continue rolling the thread until it reaches a uniform diameter of 3.2mm or 1/8 in.
4) When the thread becomes a diameter of 1/8 in. reform it into a ball.
5) Knead the soil for a few minutes to reduce its water content slightly.
6) Repeat steps 2 to 5 until the thread crumbles when it reaches a uniform diameter of 1/8 in.
7) When the soil reaches the point where it will crumble, and when the thread is a uniform diameter of 1/8", it is at its plastic limit. Determine the water content of the soil.
8) Repeat this procedure three times to compute an average plastic limit for the sample.

Calculate the plasticity index as follows: PI = LL - PL where:

LL = liquid limit, and PL = plastic limit.

Number of container	9	14
Weight of empty container(w1)	35 g	38g
Weight of container+wet soil(w2)	49 g	58
Weight of container+dry soil(w3)	43 g	51
Water content(w2-w3/w3-w1)*100	75%	53%

Table.2: Plastic Limit

Fig 20: Casagrande liquid Limit Experiment

5.2.2 Liquid limit:

The importance of the liquid limit test is to classify soils. Different soils have varying liquid limits. Also, one must use the plastic limit to determine its plasticity index.

The liquid limit (LL) is the water content at which a soil changes from plastic to liquid behavior.The original liquid limit test of Atterberg's involved mixing a part of clay in a round-bottomed porcelain bowl of 10–12 cm diameter. A groove was cut through the pat of clay with a spatula, and the bowl was then struck many times against the palm of one hand.

Procedure:

1) Place a portion of the prepared sample in the cup of the liquid limit device at the point where the cup rests on the base and spread it so that it is 10mm deep at its deepest point. Form a horizontal surface over the soil. Take care to eliminate air bubbles from the soil specimen. Keep the unused portion of the specimen in the storage container.
2) Form a groove in the soil by drawing the grooving tool, bevelled edge forward, through the soil from the top of the cup to the bottom of the cup. When forming the groove, hold the tip of the grooving tool against the surface of the cup and keep the tool perpendicular to the surface of the cup.
3) Lift and drop the cup at a rate of 2 drops per second. Continue cranking until the two halves of the soil specimen meet each other at the bottom of the groove. The two halves must meet along a distance of 13mm (1/2 in).
4) Record the number of drops required to close the groove.
5) Remove a slice of soil and determine its water content, w.
6) Repeat steps 1 through 5 with a sample of soil at a slightly higher or lower water content. Whether water should be added or removed depends on the number of blows required to close the grove in the previous sample.

Fig 21: casagrande liquid limit apparatus

Note: The liquid limit is the water content at which it will takes 25 blows to close the groove over a distance of 13 mm. Run at least five tests increasing the water content each time. As the water content increases it will take less blows to close the groove.

Sample number	1	2	3
Container number	24	21	25
No of blows	17	28	36
Weight of container	44.9	46	44.6
Weight of container+wet soil	78.3	81.3	76.8
Weight of container+dry soil	70	75.3	74.1
Water content	33.07	20.30	10.0

Table.3: Liquid Limit

Plot the relationship between the water content, w, and the corresponding number of drops, N, of the cup on a semi- logarithmic graph with water content as the ordinates and arithmetical scale, and the number of drops on the abscissas on a logarithmic scale. Draw the best fit straight line through the five or more plotted points.

Take the water content corresponding to the intersection of the line with the 25 drop abscissa as the liquid limit, LL, of the soil.

5.3 Standard Compaction:-

The Proctor compaction test is a laboratory method of experimentally determining the optimal moisture content at which a given soil type will become most dense and achieve its maximum dry density. The term Proctor is in honour of R. R. Proctor, who in 1933 showed that the dry density of a soil for a given compactive effort depends on the amount of water the soil contains during soil compaction.[1] His original test is most commonly referred to as the standard Proctor compaction test; later on, his test was updated to create the modified Proctor compaction test.

These laboratory tests generally consist of compacting soil at known moisture content into a cylindrical mould of standard dimensions using a compactive effort of controlled magnitude. The soil is usually compacted into the mould to a certain amount of equal layers, each receiving a number blows from a standard weighted hammer at a specified height. This process is then repeated for various moisture contents and the dry densities are determined for each. The graphical relationship of the dry density to moisture content is then plotted to establish the compaction curve. The maximum dry density is finally obtained from the peak point of the compaction curve and its corresponding moisture content, also known as the optimal moisture content.

The testing described is generally consistent with the American Society for Testing and Materials (ASTM) standards, and are similar to the American Association of State Highway and Transportation Officials (AASHTO) standards. Currently, the procedures and equipment details for the standard Proctor compaction test is designated by ASTM D698 and AASHTO T99. Also, the modified Proctor compaction test is designated by ASTM D1557 and AASHTO T180.

Compaction is the process by which the bulk density of an aggregate of matter is increased by driving out air. For any soil, for a given amount of compactive effort, the density obtained depends on the moisture content. At very high moisture contents, the maximum dry density is achieved when the soil is compacted to nearly saturation, where (almost) all the air is driven out. At low moisture contents, the soil particles interfere with each other; addition of some moisture will allow greater bulk densities, with a peak density where this effect begins to be counteracted by the saturation of the soil.

Procedure:

After removing the foreign matter from the soil 3 kg of soil sample is taken by passing through 4.75 mm sieve and mixed with 8% of water i.e. 240ml. The soil is compacted in compaction mould in 3 layers with 25 blows per each layer. Small amount of soil sample is taken from the compacted mould for the water content determination.

Fig 22: Sample preparation for standard compaction test

The same procedure is repeated for 10%, 12%, 14%, 16%, and 18% of water to be mixed with soil. The water content is determined for all the samples and the corresponding dry density is evaluated.
A graph is plotted between water content and dry density to evaluate optimum moisture content and max dry density.

% of water added	10	12	14	16	18
Weight of empty mould(w)	2.209 kg	2.209 kg	2.209 kg	2.209 kg	2.209 kg
Volume of mould	944 cc	944 cc	944 cc	944 cc	944 cc
Weight of soil	1.862 kg	1.958 kg	2.019 kg	2.045 kg	2.018
Water content	0.052	0.047	0.068	0.067	0.090
Bulk density (g/cc)	1.972	2.074	2.138	2.166	2.137
Dry density(g/cc)	1.87	1.98	2.0	2.02	1.96

Table.4: Standard Compaction

Fig 23: Compaction of soil

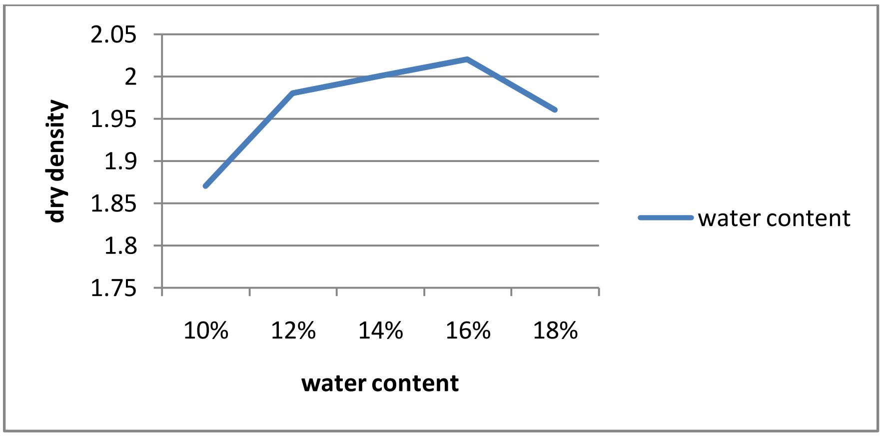

Fig 24: Chart representing OMC and MDD

The optimum moisture content and max dry density for black cotton soil is 16% and 2.02gm/cc.

5.4 California Bearing Ratio Test
Scope:

- This test method covers the determination of the CBR (California Bearing Ratio) of pavement subgrade, subbase, and base course materials from laboratory compacted specimens. The test method is primarily intended for (but not limited to) evaluating the strength of materials having maximum particle sizes less than 3 ⁄4 in. (19 mm).
- When materials having maximum particle sizes greater than 3 ⁄4 in. (19 mm) are to be tested, this test method provides for modifying the gradation of the material so that the material used for tests all passes the 3 ⁄4-in. sieve while the total gravel (+No. 4 to 3 in.) fraction remains the same. While traditionally this method of specimen preparation has been used to avoid the error inherent in testing materials containing large particles in the CBR test apparatus, the modified material may have significantly different strength properties than the original material. However, a large experience base has developed using this test method for materials for which the gradation has been modified, and satisfactory design methods are in use based on the results of tests using this procedure.
- Past practice has shown that CBR results for those materials having substantial percentages of particles retained on the No. 4 sieve are more variable than for finer materials. Consequently, more trials may be required for these materials to establish a reliable CBR.
- This test method provides for the determination of the CBR of a material at optimum water content or range of\ water content from a specified compaction test and a specified dry unit weight. The dry unit

weight is usually given as a percentage of maximum dry unit weight determined by Test Methods D698 or D1557.

- The agency requesting the test shall specify the water content or range of water content and the dry unit weight for which the CBR is desired.
- Unless specified otherwise by the requesting agency, or unless it has been shown to have no effect on test results for the material being tested, all specimens shall be soaked prior to penetration.
- For the determination of CBR of field compacted materials, see Test Method D4429.
- The values stated in inch-pound units are to be regarded as the standard. The SI equivalents shown in parentheses may be approximate.
- All observed and calculated values shall conform to the guidelines for significant digits and rounding established in Practice D6026.
- The procedures used to specify how data are collected, recorded or calculated in this standard are regarded as the industry standard. In addition they are representative of the significant digits that generally should be retained. The procedures used do not consider material variation, purpose for obtaining the data, special purpose studies, or any considerations for the user's objectives, and it is common practice to increase or reduce significant digits or reported data to be commensurate with these considerations. It is beyond the scope of this standard to consider significant digits used in analytical methods for engineering design.
- This standard does not purport to address all of the safety problems, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

The California Bearing Ratio Test (CBR Test) is a penetration test developed by California State Highway Department (U.S.A.) for evaluating the bearing capacity of sub grade soil for design of flexible pavement.

Tests are carried out on natural or compacted soils in water soaked or un-soaked conditions and the results so obtained are compared with the curves of standard test to have an idea of the soil strength of the sub grade soil.

Laboratory CBR Apparatus meets the essential requirements of IS : 2720 (Part XVI) with the mould as per of IS : 9669. The field CBR Apparatus meets the requirements of IS : 2720 (Part XXXI).

The CBR rating was developed for measuring the load-bearing capacity of soils used for building roads. The CBR can also be used for measuring the load-bearing capacity of unimproved airstrips or for soils under paved airstrips. The harder the surface, the higher the CBR rating. A CBR of 3 equates to tilled farmland, a CBR of 4.75 equates to turf or moist clay, while moist sand may have a CBR of 10. High quality crushed rock has a CBR over 80. The standard material for this test is crushed California limestone which has a value of 100.

$CBR = P/P_s$

CBR= CBR [%]

P= measured pressure for site soils [N/mm²]

P_s= pressure to achieve equal penetration on standard soil [N/mm²]

California Bearing Ratio (CBR) test was developed by the California Division of Highway as a method of classifying and evaluating soil-sub grade and base course materials for flexible pavements. CBR test, an empirical test, has been used to determine the material properties for pavement design. Empirical tests measure the strength of the material and are not a true representation of the resilient modulus. It is a penetration test wherein a standard piston, having an area of 3 in (or 50 mm diameter), is used to penetrate the soil at a standard rate of 1.25 mm/minute. The pressure up to a penetration of 12.5 mm and it's ratio to the bearing value of a standard crushed rock is termed as the CBR.

In most cases, CBR decreases as the penetration increases. The ratio at 2.5 mm penetration is used as the CBR. In some case, the ratio at 5 mm may be greater than that at 2.5 mm. If this occurs, the ratio at 5 mm should be used. The CBR is a measure of resistance of a material to penetration of standard plunger under controlled density and moisture conditions. The test procedure should be strictly adhered if high degree of reproducibility is desired. The CBR test may be conducted in re-moulded or undisturbed specimen in the laboratory. The test is simple and has been extensively investigated for field correlations of flexible pavement thickness requirement.

5.4.1 Un Soaked Condition:

Test Procedure:

The laboratory CBR apparatus consists of a mould 150 mm diameter with a base plate and a collar, a loading frame and dial gauges for measuring the penetration values and the expansion on soaking.

The specimen in the mould is soaked in water for four days and the swelling and water absorption values are noted. The surcharge weight is placed on the top of the specimen in the mould and the assembly is placed under the plunger of the loading frame.

Load is applied on the sample by a standard plunger with dia of 50 mm at the rate of 1.25 mm/min. A load penetration curve is drawn. The load values on standard crushed stones are 1370 kg and 2055 kg at 2.5 mm and 5.0 mm penetrations respectively.

CBR value is expressed as a percentage of the actual load causing the penetrations of 2.5 mm or 5.0 mm to the standard loads mentioned above. Therefore,

CBR = (load carried by specimen / load carried by standard specimen) * 100

Two values of CBR will be obtained. If the value of 2.5 mm is greater than that of 5.0 mm penetration, the former is adopted. If the CBR value obtained from test at 5.0 mm penetration is higher than that at 2.5 mm, then the test is to be repeated for checking. If the check test again gives similar results, then higher value obtained at 5.0 mm penetration is reported as the CBR value. The average CBR value of three test specimens is reported as the CBR value of the sample.

5.4.1(A) CBR for soil sample:

5 kg soil is mixed with OMC and is compacted in the CBR mould in 5 layers giving 55 blows for each layer. Then the sample along with mould is placed in CBR apparatus for the conduction of test. The proving ring readings are noted for every 50 divisions of the dial gauge reading. The CBR value is calculated as per the formulae given above. The value for 2.5 mm penetration is taken as CBR value of the sample.

1 proving ring division = 6.9532 kg

CBR = (test load/standard load)*100

Dial gauge reading	Proving ring reading
0	0
50	5
100	7
150	9.5
200	10.0
250	10.5
300	11.0
350	11.5
400	12.0
450	12.5
500	13.0
750	14.0
1000	15.0
1250	15.5

Table.5: Results for normal soil sample

CBR value for 2.5 mm penetration = 5.5 %

CBR value for 5 mm penetration = 4.39 %

Graph:

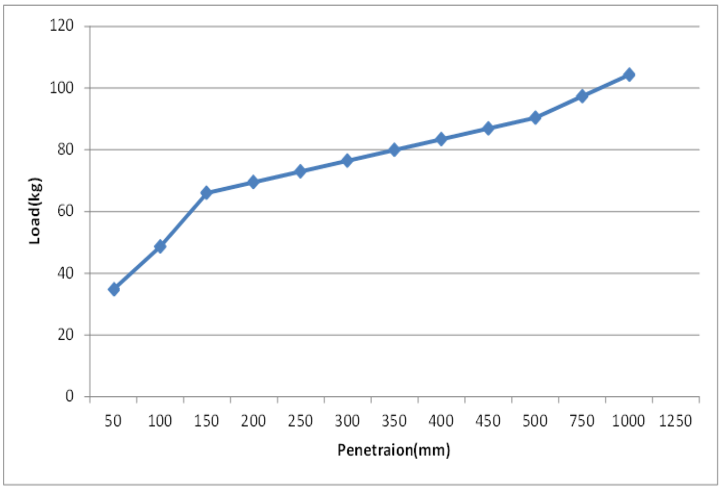

Fig 25: Load vs. penetration graph for normal soil sample

5.4.1(B) CBR for soil + 5% fly ash:

The soil is mixed with fly ash and CBR test is conducted to know the CBR value.5 kg soil sample passing 4.75 mm sieve is taken and 5 % 0f soil is replaced with fly ash and mixed with OMC of 16%. Then the soil sample is compacted in CBR mould in 5 layers with 55 Blows for each layer. The soil sample is tested using CBR apparatus to know the strength of the soil.

Fig 26: Soil sample mixed with 5% fly ash

Test results are as follows.

Dial gauge reading	Proving reading
0	0
50	4.5
100	9.5
150	10.5
200	14.0
250	15.0
300	15.5
350	17.0
400	17.5
450	18.0
500	19.0
750	20.0
1000	21.0
1250	22.0

Table.6: CBR results for soil mixed with 5% fly ash

CBR value for 2.5 mm penetration = 7.61 %

CBR value for 5mm penetration = 6.42 %

Graph:

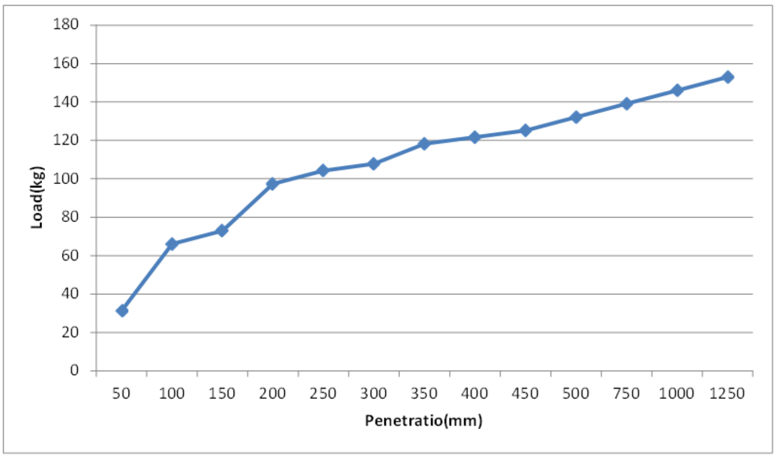

Fig 27: Load vs. penetration graph for soil+ 5% fly ash

5.4.1(C) CBR for soil + 10% fly ash:

The soil is mixed with 10 % fly ash and the same test procedure is repeated to know the CBR value. Test results are as follows.

Dial gauge reading	Proving ring reading
0	0
50	4
100	7
150	11.0
200	12.0
250	13.0
300	15.0
350	17.0
400	20.0
450	21.0
500	23.0
750	23.0
1000	24.0
1250	25.0

Table.7: CBR results for soil + 10% fly ash

CBR value for 2.5 mm penetration = 6.597%

CBR value for 5 mm penetration = 5.516%

Graph:

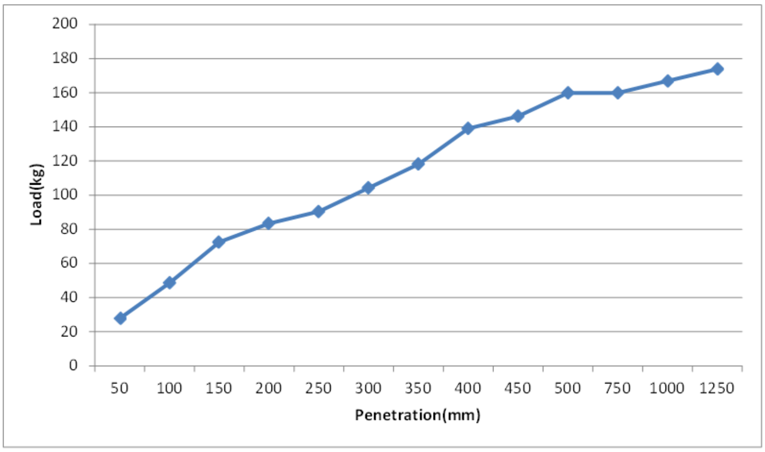

Fig 28: load vs. penetration graph for soil + 10% fly ash

Fig 29: Soil sample mixed with 10% fly ash

5.4.1(D) CBR for soil+20% flyash:

The soil is mixed with 20% fly ash and the same test procedure is repeated for the sample to know the cbr value.

The test values are as follows:

Dial guage reading	Proving ring reading

0	0
50	2
100	4
150	7
200	10
250	12
300	13
350	14
400	15
450	15
500	16
750	18
1000	19
1250	20

Table.8: CBR results for soil + 20% fly ash

CBR value for 2.5 mm penetration = 6.09%
CBR value for 5 mm penetration = 5.414%

Graph:

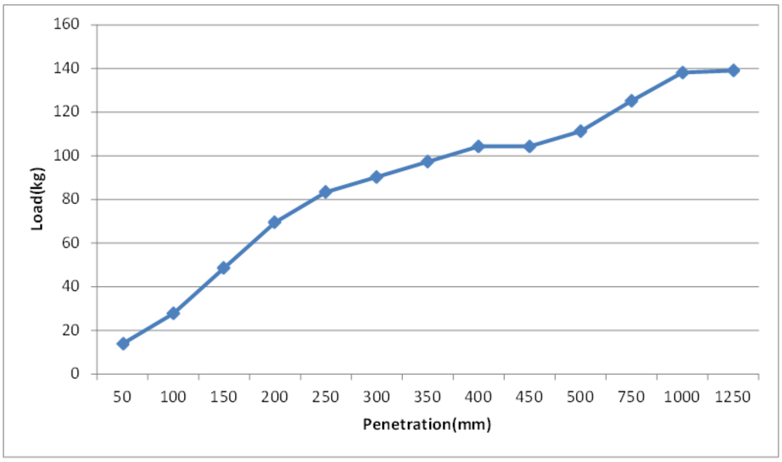

Fig 30: load vs. penetration graph for soil + 20% fly ash

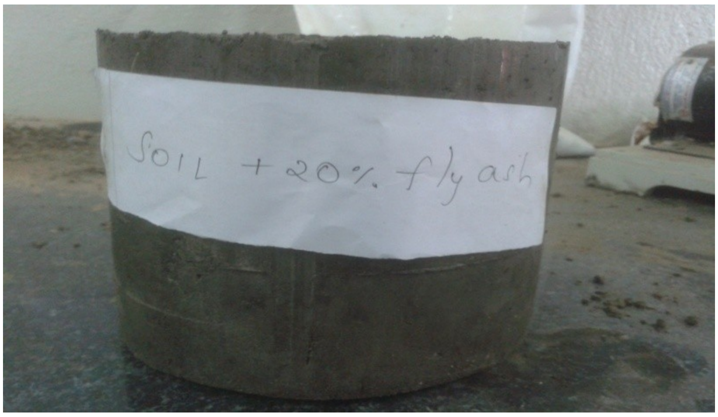

Fig 31: Soil sample mixed with 20% fly ash

From the above test we infer that

1) CBR value for the plain soil sample is 5.5
2) CBR value for the fly ash and soil mixture (5% fly ash) is 7.61
3) CBR value for the fly ash and soil mixture(10% fly ash) is 6.597
4) CBR value for the soil and fly ash mixture (20% fly ash) is 6.092

Depending upon on the composition of the flyash that is added to the soil its stability varies accordingly. Here we notice that stability of our soil sample increases with addition of 5% fly ash and then gradually decreases beyond it. Hence the optimum composition of fly ash to be considered is 5% by its weight.

5.4.2 Soaked Condition:

Apparatus:

- Mould
- Steel Cutting collar
- Spacer Disc
- Surcharge weight
- Dial gauges
- IS Sieves
- Penetration Plunger
- Loading Machine
- Miscellaneous Apparatus

Loading Machine—the loading machine shall be equipped with a movable head or base that travels at a uniform (not pulsating) rate of 0.05 in. (1.27 mm)/min for use in forcing the penetration piston into the specimen. The load rate of 0.05 in. (1.27 mm)/min shall be maintained within 620% over the range of loads developed during penetration. The minimum capacity of the loading machine shall be based on the requirements indicated in Table 1.

1. The machine shall be equipped with a load-indicating device matched to the anticipated maximum penetration load: 10 lbf (44 N) or less for a 10-kip (44.5-kN) capacity; 5 lbf (22 N) for 5-kip (22.3-kN) and 2 lbf (8.9 N) for 2.5-kip (11.2-kN).
2. Penetration measuring device (such as a mechanical dial indicator or electronic displacement transducer) that can be read to the nearest 0.001 in. (0.025 mm) and associated mounting hardware. A mounting assembly that connects the deformation measuring device to the penetrating piston and the edge of the mold will give accurate penetration measurements. However, mounting the deformation holder assembly to a stressed component of the load frame (such as tie rods) will introduce inaccuracies of penetration measurements.

Mould—the mould shall be a rigid metal cylinder with an inside diameter of 6 6 0.026 in. (152.4 6 0.66 mm) and a height of 7 6 0.018 in. (177.8 6 0.46 mm). It shall be provided with a metal extension collar at least 2.0 in. (50.8 mm) in height and a metal base plate having at least twenty eight 1⁄16-in. (1.59-mm) diameter holes uniformly spaced over the plate within the inside circumference of the mould. When assembled with spacer disc in place in the bottom of the mould, the mould shall have an internal volume (excluding extension collar) of 0.075 6 0.0009 ft3 (2124 6 25 cm3). Fig. 1 shows a satisfactory mould design. A calibration procedure should be used to confirm the actual volume of the mould with the spacer disk inserted. Suitable calibration procedures are contained in Test Methods D698 and D1557.

Spacer Disk—A circular metal spacer disc (see Fig. 1) having a minimum outside diameter of 515⁄16 in. (150.8 mm) but no greater than will allow the spacer disc to easily slip into the mould. The spacer disc shall be 2.416 6 0.005 in. (61.37 60.127 mm) in height.

Rammer—A rammer as specified in either Test Methods D698 or D1557 except that if a mechanical rammer is used it must be equipped with a circular foot, and when so equipped, must provide a means for distributing the rammer blows uniformly over the surface of the soil when compacting in a 6-in. (152.4-mm) diameter mould. The mechanical rammer must be calibrated and adjusted in accordance with Test Methods D2168.

Expansion-Measuring Apparatus— an adjustable metal stem and perforated metal plate, similar in configuration to that shown in Fig. 1. The perforated plate shall be 57⁄8 to 515⁄16 in.(149.23 to 150.81 mm) in diameter and have at least forty-two1⁄16-in. (1.59-mm) diameter holes uniformly spaced over theplate. A metal tripod to support the dial gauge for measuring the amount of swell during soaking is also required. The expansion measuring apparatus shall not weigh more than 2.8 lbf (1.27 kg).

Weights—One or two annular metal weights having a total mass of 4.54 6 0.02 kg and slotted metal weights each having masses of 2.27 6 0.02 kg. The annular weight shall be 57⁄8 to 515⁄16 in. (149.23 to 150.81 mm) in diameter and shall have a centre hole of approximately 21⁄8 in. (53.98 mm).

Penetration Piston—A metal piston 1.954 6 0.005 in.(49.63 6 0.13 mm) in diameter and not less than 4 in. (101.6 mm) long (see Fig. 1). If, from an operational standpoint, it is advantageous to use a piston of greater length, the longer piston may be used.

Swell Measurement Device—Generally mechanical dialindicators capable of reading to 0.001 in. (0.025 mm) with a range of 0.200-in. (5-mm) minimum.

Balance—a class GP5 balance meeting the requirements of Specifications D4753 for a balance of 1-g readability.

Drying Oven —Thermostatically controlled, preferably of a forced-draft type and capable of maintaining a uniform temperature of 230 6 9°F (110 6 5°˚C) throughout the drying chamber.

Sieves—3/4 in. (19 mm) and No. 4 (4.75 mm), conforming to the requirements of Specification E11.

Filter Paper—Fast filtering, high wet strength filter paper, 15-cm diameter.

Straightedge—a stiff metal straightedge of any convenient length but not less than 10 in. (254 mm). The total length of the straightedge shall be machined straight to a tolerance of 60.005 in. (60.1 mm). The scraping edge shall be bevelled if it is thicker than 1/8 in. (3 mm).
Soaking Tank or Pan—A tank or pan of sufficient depth and breadth to allow free water around and over the assembled mould. The tank or pan should have a bottom grating that allows free access of water to the perforations in the mould's base.
MixingTools — miscellaneous tools such as mixing pan, spoon, trowel, spatula,etc.,or a suitable mechanical device for thoroughly mixing the sample of soil with water.

Procedure:

If the CBR test specimen is to be soaked, take a representative sample of the material for the determination of water content in accordance with Test Method D2216. If the compaction process is conducted under reasonable controlled temperatures (65 to 75 F (18 to 24 C) and the processed soil is kept sealed during the compaction process, only one representative water content sample is required. However if the compaction process is being conducted in an uncontrolled environment take two water content samples one at the beginning of compaction and another sample of the remaining material after compaction. Use Test Method D2216 to determine the water contents and average the two values for reporting. The two samples should not differ more than 1.5 percentage points to assume reasonable uniformity of the compacted specimen's water content.

1. If the sample is not to be soaked, take a water content sample in accordance with Test Methods D698 or D1557 if the average water content is desired.
2. Clamp the mould (with extension collar attached) to the base plate with the hole for the extraction handle facing down. Insert the spacer disk over the base plate and place a disk of filter paper on top of the spacer disk. Compact the soil-water mixture into the mould.
3. Remove the extension collar and carefully trim the compacted soil even with the top of the mould by means of a straightedge. Patch with smaller size material any holes that may have developed in the surface by the removal of coarse material. Remove the perforated base plate and spacer disk, weigh, and record the mass of the mould plus compacted soil. Place a disk of coarse filter paper on the perforated base plate, invert the mould and compacted soil, and clamp the perforated base plate to the mould with compacted soil in contact with the filter paper.
4. Place the surcharge weights on the perforated plate and adjustable stem assembly and carefully lower onto the compacted soil specimen in the mould. Apply a surcharge equal to the weight of the base material and pavement within 5 lbf(2.27 kg), but in no case shall the total weight used be less than 10 lbf (4.54 kg). If no pavement weight is specified, use 10 lbf (4.54 kg).The mass of the Expansion Measuring Apparatus is ignored unless its mass is more than 2.8 lbf (1.27 kg). Immerse the mould and weights in water allowing free access of water to the top and bottom of the specimen. Take initial measurements for swell and allow the specimen to soak for 96 h. maintain a constant water level during this period. A shorter immersion period is permissible for fine grained soils or granular soils that take up moisture readily, if tests show that the shorter period does not affect the results. At the end of 96 h, take final swell measurements and calculate the swell as a percentage of the initial height of the specimen.
5. Remove the free water and allow the specimen to drain downward for 15 min. Take care not to disturb the surface of the specimen during the removal of the water. It may be necessary to tilt the specimen in order to remove the surface water. Remove the weights, perforated plate, and filter paper, and determine and record the mass. The user may find it convenient to set the mould's base on the rim of a shallow pan to provide the tilt and carefully using a bulb syringe and adsorbent towels to remove free water.

5.4.2 (A) Tabulation For Soaked Soil Sample:

Dial gauge reading	Proving ring reading
0	0
50	0.7
100	1.1
150	1.8
200	2.2
250	2.5
300	2.6
350	2.6
400	2.7
450	2.7
500	2.8
750	3
1000	3.1
1250	3.2

CBR value for soaked plain soil sample for 2.5 mm penetration= 1.24 %
CBR value for soaked plain soil sample for 5 mm penetration = 0.92 %

5.4.2 (B) Tabulation For Soil + 5% Fly Ash:

Dial gauge reading	Proving ring reading
0	0
50	1.8
100	2.6
150	3.7
200	4.9
250	5.8
300	6
350	6.2
400	6.4
450	6.4
500	6.6
750	6.8
1000	7
1250	7.1

CBR value for soaked plain soil sample for 2.5 mm penetration =2.92%
CBR value for soaked plain soil sample for 5 mm penetration = 2.23 %

5.5 Unconfined Compression Test:

The unconfined compressive strength of the fly ash samples has been determined using an unconfined compressive strength (UCS) test. The test specimens were 38 mm diameter and 76 mm height. A split mould was used to prepare the specimens. The moisture content and density used for the preparation of specimens conformed to the optimum moisture content and maximum dry density obtained from Standard Proctor compaction tests. The detailed procedure for preparation of specimen for the UCS test has been explained elsewhere (Ghosh and Subbarao, 2007). The UCS tests have been carried out in accordance with ASTM D2166 (2006).

Consistency	Q_u (lb/ft^2)
Very soft	0-500
Soft	500-1000
Medium	1000-2000
Stiff	2000-4000
Very stiff	4000-8000

Table.9: Range of consistency for different soils

$$S_t = \frac{q_{u(\text{undisturbed})}}{q_{u(\text{remolded})}}$$

Sensitivity, S_t	Description
1–2	Slightly sensitive
2–4	Medium sensitivity
4–8	Very sensitive
8–16	Slightly quick
16–32	Medium quick
32–64	Very quick
> 64	Extra quick

Fig 32: UCC apparatus

5.5.1 Significance:

For soils, the un drained shear strength (su) is necessary for the determination of the bearing capacity of foundations, dams, etc. The un drained shear strength (s_u) of clays is commonly determined from an unconfined compression test. The undrained shear strength (s_u) of a cohesive soil is equal to one-half the unconfined compressive strength (q_u) when the soil is under the f = 0 condition (f = the angle of internal friction). The most critical condition for the soil usually occurs immediately after construction, which represents un drained conditions, when the un drained shear strength is basically equal to the cohesion (c). this is expressed as:

$$S_u= c= q_u/2$$

Then, as time passes, the pore water in the soil slowly dissipates, and the Inter granular stress increases, so that the drained shear strength (s), given by s = c +s'tanf, must be used. Where s'= inter granular pressure acting perpendicular to the shear plane; and s' = (s - u), s = total pressure, and u = pore water pressure; c' and j' are drained shear strength parameters.

Fig 33: sample extractor

5.5.2 Procedure:

1) Calculate the volume of the mould
2) Calculate the weight of the soil by using dry density of the soil and volume of the mould.
3) Mix the soil at optimum moisture content, and compact the soil in the mould till the soil is perfectly compacted.
4) Using sample extractors extract the soil sample from the mould.
5) Now place the soil sample on the plate form of the U.C.C apparatus and apply the load axially.
6) Set the dial gauge reading to zero and note the readings when the load touches the top of the specimen.

Fig 34: UCC test sample

The results obtained are as follows:

Compression dial reading	Strain(e)	A= A_o/1-e(cm^2)	Proving ring reading(div.)	Axial load (kg)	Compressive stress(q_u) (kg/cm^2)	Shear stress(q_u/2) (kg/cm^2)
0.5	0.65	11.415	4	1.3356	0.117	0.052
1.0	1.31	11.502	7	2.337	0.203	0.1015
1.5	1.973	11.572	18	6.010	0.519	0.259
2.0	2.631	11.647	27	9.0153	0.774	0.387
2.5	3.289	11.726	37	12.354	1.05	0.525
3.0	3.947	11.807	45	15.025	1.272	0.636
3.5	4.605	11.888	50	16.695	1.404	0.702
4.0	5.263	11.971	54	18.030	1.506	0.753
4.5	5.921	12.054	55.5	18.531	1.537	0.7685
5.0	6.578	12.139	55.5	18.531	1.526	0.760

Table.10: Tabulation of UCC test values for normal soil

Unconfined compressive strength of the sample=0.768 kg/cm^2

5.5.3 Analysis:

(1) Convert the dial readings to the appropriate load and length units, and enter these values on the data sheet in the deformation and total load columns. (Confirm that the conversion is done correctly, particularly proving dial gage Reading's conversion into load)

(2) Compute the sample cross-sectional area A_o= 3.14/4(d^2).

(3) Compute the strain, e = change in length/original length (L_o)

(4) computed the corrected area, $A^1 = A_o/1\text{-e}$

(5) Using A', compute the specimen stress, $s_c = p/A^1$

(Be careful with unit conversions and use constant units).

(6) Compute the water content, w%.

(7) Plot the stress versus strain. Show q_u as the peak stress (or at 15% strain) of the test. Be sure that the strain is plotted on the abscissa. See example data.

(8) Draw Mohr's circle using qu from the last step and show the undrained shear strength, s_u = c (or cohesion) = q_u/2.

Fig 35: Arrangement of soil sample for UCC test

5.5.4 UCC for Soil + 5% fly ash:

Same procedure is repeated for the soil-fly ash mixture and the results are tabulated below

Compression dial reading	Strain(e)	A= Ao/1-e(cm2)	Proving ring reading(div.)	Axial load (kg)	Compressive stress(qu) (kg/cm2)	Shear stress(qu/2) (kg/cm2)
0.5	0.65	11.415	4	1.3356	0.117	0.0585
1.0	1.31	11.502	10	3.339	0.290	0.145
1.5	1.973	11.572	21	7.0119	0.605	0.3025
2.0	2.631	11.647	30	7.017	0.860	0.43
2.5	3.289	11.726	39	13.0221	1.11	0.555
3.0	3.147	11.807	59	17.028	1.442	0.721
3.5	4.605	11.888	55	18.36	1.544	0.772
4.0	5.263	11.971	55	18.36	1.533	0.665
4.5	5.921	12.054	50	16.695	1.385	0.6925

Table.11: Tabulation of UCC test values for soil + 5% fly ash

Unconfined compressive strength of the sample=0.772 kg/cm^2

5.5.5 UCC for Soil sample + 10% fly ash:
Same procedure is repeated for the soil-fly ash mixture and the results are tabulated below

Compression dial reading	Strain(e)	A= Ao/1-e (cm2)	Proving ring reading(div.)	Axial load (kg)	Compressive stress(qu) (kg/cm2)	Shear stress(qu/2) (kg/cm2)
0.5	0.65	11.415	4	1.3356	0.116	0.58
1	1.31	11.502	7	2.337	0.203	0.1015
1.5	1.973	11.572	19	6.345	0.548	0.274
2	2.631	11.647	26	8.681	0.754	0.3725
2.5	3.289	11.726	29	9.683	0.825	0.4125
3	3.947	11.807	31.5	10.517	0.890	0.445
3.5	4.605	11.888	33	11.018	0.926	0.463
4	5.263	11.971	33	11.018	0.920	0.46
4.5	5.921	12.054	32	10.684	0.914	0.457

Table.12: Tabulation of UCC test values for soil + 10% fly ash

Unconfined compressive strength of the sample=0.46 kg/cm^2

Fig 36: Arrangement of dial gauge

VI. Results And Discussion

Details Of Soil Sample:

The project area is on the black cotton soil, initially the soil consisted vegetative matter and some dust particles which were then removed from the soil to make it suitable for laboratory tests.

The tests were conducted according to the specifications and the results are tabulated for reference. The properties of the Black cotton soil observed are:

- Liquid limit of the soil is 23.5%.
- Plastic limit of the soil is 65%.
- Plasticity Index of the soil is 15% to 40%
- Optimum moisture content is 16%.
- Maximum dry density is 2.02gm/cc
- California bearing ratio value (C.B.R) is 5.5%.
- Compressive strength of soil is 0.768 kg/cm^2
- Shear strength is 0.384 kg/cm^2.
- Grain size distribution.

As a soil is expansive soil to avoid expansion and contraction when the soil encounters with water we have to stabilize the soil to increase the strength and other properties by adding fly ash to it.

It was observed that with the increase in water content the dry density decreases up to 20-30% moisture content and with further increase in water content the dry density decreases gradually. The maximum dry density is in the range of 1.35 g/cc for 95% soil and 5% fly ash mixture and lowest density was about 0.6g/cc for

70% soil and 30% fly ash mixture. This variation of density is primarily due to alteration of gradation of soil mixtures. The decrease of the maximum dry unit weight with the increase of the percentage of fly ash is mainly due to the lower specific gravity of the fly ash compared with expansive soil and the immediate formation of cemented products by hydration which reduces the density of soil. The decrease in dry density with increase in fine fly ash content is due alteration of gradation of soil mixtures. Whereas decrease in dry density with the increase in coarse fly ash mixture was attributed due to cat ion exchange between additives and expansive soil which decreases the thickness of electric double layer and promotes the flocculation.

- When the fly ash is added to the soil the properties of the soil changed considerably and shown an increase in the strength. The CBR value of the soil is 5.5 % which shows that the soil is clayeysoils (CL, CH) with good strength.
- When 5% of fly ash is added to the soil by its weight the stability of the soil increased. The CBR value of the soil is 7.67% which is greater than the plain soil.

Fig 37: Figure showing all soil samples with various % of fly ash

- When 10% of fly ash is added to the soil by its weight the stability of the soil decreased considerably than the original soil.The CBR value of the soil is 6.597%.
- When 20% of fly ash is added to the soil by its weight the stability of soil of the soil decreased considerably than the original soil and the above consecutive percentages of fly ash mixed with soil.
- When soaked CBR test is conducted on the soaked plain sample the CBR value is found to be 1.3 which indicates that the soil is medium soil.
- When 5% fly ash is added to the soil sample and soaked CBR test is conducted the CBR value increases to 2.7 % which shows that the soil is very good soil.
- By the increase in the % of fly ash the stability f the soil decreases that the soaked CBR value decreases.so the optimum percentage of fly ash that is to be added to the soil is 5% which gives more strength to the sub grade.

Typical Values of CBR

Material	CBR	Elastic Modulus (psi)
Crushed Stone (GW, GP, GM)	20 - 100	20,000 - 40,000
Sandy Soils (SW, SP, SM, SC)	5 - 40	7,000 - 30,000
Silty Soils (ML, MH)	3 - 15	5,000 - 20,000
Clayey Soils (CL, CH)	3 - 10	5,000 - 15,000
Organic Soils (OH, OL, PT)	1 - 5	< 5,000

Source: WSDOT Pavement Guide Interactive CD-ROM

Table.13

Typical Values of CBR

General Soil Type	USC Soil Type	CBR Range
Coarse-grained soils	GW	40 - 80
	GP	30 - 60
	GM	20 - 60
	GC	20 - 40
	SW	20 - 40
	SP	10 - 40
	SM	10 - 40
	SC	5 - 20
Fine-grained soils	ML	≤ 15
	CL	≤ 15
	OL	≤ 5
	MH	≤ 10
	CH	≤ 15
	OH	≤ 5

Source: WSDOT Pavement Guide Interactive CD-ROM

Table.14

Graph:

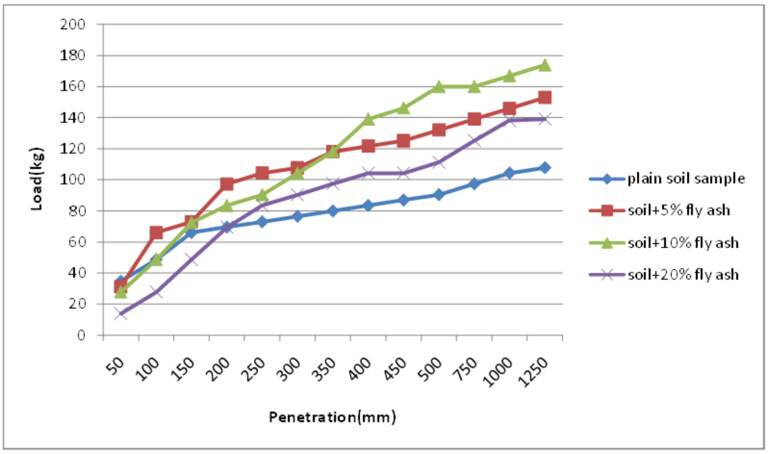

Fig 38: Comparison of soil samples

The UN confined compression test is conducted on soil by using the optimum moisture content; when the test is conducted on the original soil the UN confined compressive strength of the soil is found to be 0.348 kg/cm^2.

Now fly ash is mixed with soil to stabilize the soil in varying percentages, when 5% of fly ash is mixed with soil with optimum moisture content the compressive strength of soil is found to be 0.421 kg/cm2 which is greater than original soil without fly ash.

When soil is mixed with 10% of fly ash by its weight and UCC test is conducted on the sample the compressive strength of soil is found to be 0.23 kg/cm2 which shows that the strength is decreased by addition of fly ash more than 5%.

Fig 39: Operation of UCC apparatus

VII. Conclusion

The stability of the pavement solely depends on the stability of the embankment.In case we encounter an embankment consisting of the expansive soils; its tendency to imbibe water weakens the stability of pavement. Hence, instead of replacing the soil we are in turn stabilizing the existing the existing strength of the sub grade by which we are actually increasing the bearing capacity of the pavement.

With reference to all the laboratory tests that are performed on the soil, we conclude that the plain soil stability i. e the CBR value initially was 5.5 which by adding 5%, 10% and 20% fly ash are 7.61, 6.59 and 6.09 respectively by which we came to know that the optimum composition of the fly ash that should be added to the soil to increase its stability is 5% by its weight. Hence the compressive stress of the soil is found by adding 5% fly ash to the soil using unconfined compressive test.

Ultimately, the main aim of the project is to increase the characteristic strength of the soil by reducing the economy as much as possible. This purpose is achieved here by using the by-product from the thermal industry (Fly ash), which does not cost too much. Hence this way the project becomes efficient as well as economical.

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