Stabilization of swelling soils with Lime & stone Dust

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STABILIZATION OF BLACK COTTON SOIL WITH LIME AND STONE DUST

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By

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CERTIFICATE

This is to certify that the project report entitled "Stabilization of Black cotton soil with lime and stone dust" is a bona fide record of work carried out by Ankur Mudgal (02/GTE/2k10) under my guidance and supervision, during the session 2012 in partial fulfillment of the requirement for the degree of Master of Technology (Geotechnical Engineering) from Delhi Technological University, Delhi.

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other University/Institute for the award of any Degree or Diploma.

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ABSTRACT

Black cotton soil have the tendency to swell when their moisture content is increased and shrink when their moisture content is decreased. The moisture may come from rain flooding, leaking water or sewer lines or from reduction in surface evapotranspiration when an area is covered by a building or pavement. To achieve the economy and for proper performance of structures it is necessary to improve the geotechnical properties of expansive soil. Due to the for construction purpose, rubble and aggregates are high demanded rubble quarries and aggregate crushers are very common. Out of the different quarry wastes, quarry dust is one, which is produced in abundance. About 20–25% of the total production in each crusher unit is left out as the waste material-stone dust. Bulk utilization of this waste material is possible through geotechnical applications like embankments, back-fill material, sub-base material and the like. Lime treatment of soils is a proven method to save time and money on construction projects. Lime drying of wet soils minimizes weather-related construction delays and permits the return to work within hours. Lime modification chemically transforms clay soils into friable, workable, compactable material. Lime stabilization creates long-term chemical changes in unstable clay soils. For our project work we collected Black cotton soil sample from shivpuri (M.P). About 125 Kg soil sample was brought by us to soil mechanics lab for carrying out our project work. In this project the results of an experimental program undertaken to investigate the effect of stone dust & lime combined at different percentage with expansive soil, the test results such as X-ray diffraction analysis, scanning electronic microscopic, index properties, Proctors compaction, differential free swelling test and unconfined compression strength, California bearing ratio obtained on expansive clays mixed at different proportions of lime and stone dust admixture are presented and discussed. From the results, it is observed that at optimum percentages, i.e., 6% lime +25 % stone dust , it is found that the swelling of expansive clay is almost controlled and also noticed that there is a marked improvement in the other properties of soil. The conclusion drawn from this investigation is that the combination of equal proportion of stone dust and lime is more effective than the addition of stone dust/lime alone to the expansive soil in controlling the swelling behavior.

CHAPTER-1

INTRODUCTION

1.1 Expansive soils

The stability and bearing power of the soil is considerably improved by soil stabilization through controlled compaction, proportioning and the addition of suitable admixtures. Swelling soil is not suitable for the construction work on account of its volumetric changes.

It swells and shrinks excessively with change of water content. Such tendency of soil is due to the presence of fine clay particles which swell, when they come in contact with water, resulting in alternate swelling and shrinking of soil due to which differential settlement of structure takes place. Stabilization of black cotton soil has been done in this project work by using lime and stone dust as admixture.

1.2 Black Cotton soil

Expansive soils are soils or soft bedrock that increases in volume or expand as they get wet and shrink as they dry out. In India this Expansive soil is called ‘black cotton soil’. Color of this oil reddish brown to black and this helps for cultivation of cotton, so is called black cotton swelling soil covers about 30% of the land area in India. They are also commonly known as bentonite, expansive, or Black Cotton soil. In India Black Cotton soil also known as ‘regurs’ are found in extensive regions of Deccan Trap. They have variable thickness and are underlain by sticky material locally known as “Kali Mitti In terms of geotechnical Engineering, Black Cotton soil is one which when associated with as engineering structure and in presence of water will show a tendency to swell or shrink causing the structure to experience moments which are largely unrelated to the direct effect of loading by the structure.

These clays are characterized by

- Having a particle size, below 2 micron.
- A large specific surface area (SSA) and
- A high Cation Exchange Capacity (CEC).
- High liquid limit and plasticity index.

Black cotton soils are one of the most prevalent causes of damage to buildings and construction. This in turn can be an immense loss to a nation's economy. The damages that can be result from construction on swelling soil can include:

I. Severe structural damage,
II. Cracked driveways, sidewalks and basement floors,
III. Heaving of roads and highway structures,
IV. Condemnation of buildings.
V. Disruption of pipelines and sewer lines.

1.3 Distribution in India

In India, an area about one-six is occupied by black cotton soil. The area covers mostly the Deccan Trap plateau, between 73°80'East longitude and 15° to 24° north, latitude.

Thus, most of soil in and around Mumbai, Madras, Gwalior, bundelkhand Khandwa, Indore, Nagpur and even some on the river banks is Black cotton .That means these soils are predominant in Deccan trap plateau region, i.e., in states of Andhra Pradesh , Western Madhya Pradesh, Gujarat, Maharashtra, Northern Karnataka and Tamilnadu.

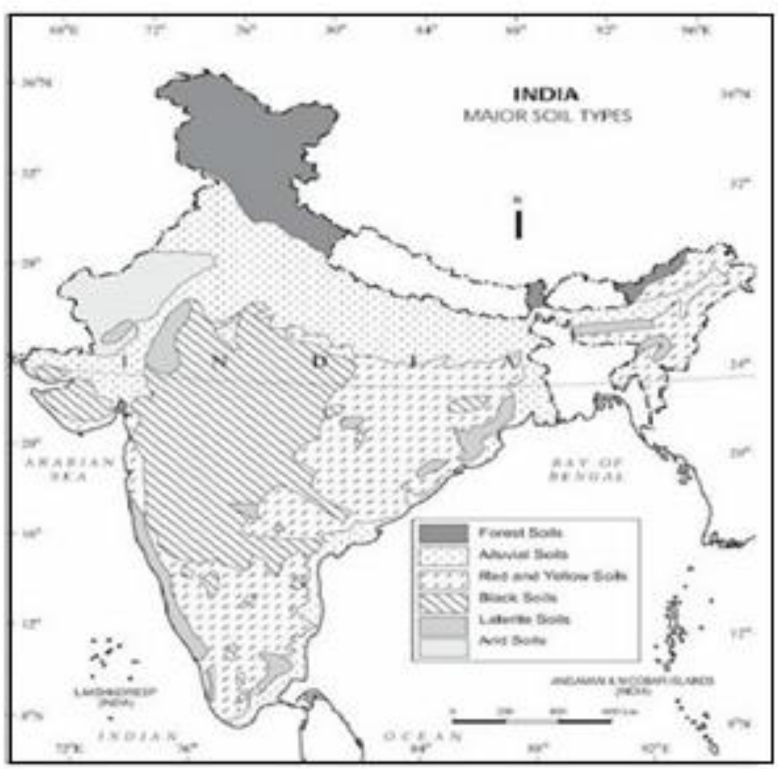


Figure.ure 1.1 Map of black cotton soil distributed in India

1.4 Typical Characteristics:

Black cotton soils are generally reddish brown to black in color and occur from 0.5m to 10m deep and have high compressibility. The generally observed characteristics of black cotton soils are recorded in table below:-

Table 1.1 Characteristics of Black Cotton Soils

S.No.	Property	Value
1.	Dry Density γ_d	1300 to 1800 kg/m^3
2.	Fines(<75μ)	70 to 100%

3.	2μ Fraction	20 to 60 %
4.	Atterberg Limits Liquid limit L.L.(%) Plastic Limit P.L.(%)	50 to 120 20 to 60
6.	Soil Classification	CH or MH
7.	Specific Gravity, G	2.60 to 2.75
8,	Proctor Density, Max Dry Density Optimum moisture content	1350 to 1600 kg/m^3 20 to 35 %
9.	Free swell Index	40 to 180 %
10.	Swelling Pressure	50 to 800 kN/m^2
11.	C.B.R (Soaked)	1.2 to 4.0
12.	Compression Index	0.2 to 0.5

1.5 Chemical Composition

Black cotton soil are made of varying properties of clay minerals like Montmorrilonite , Illite and Kaolinite, chemicals like iron oxide and calcium carbonate (in the form of kankar nodules) , and organic matter like humus. Montmorillonite is the predominant mineral of Black cotton soils. The swelling and shrinkage behavior of black cotton soil originate mainly from this mineral are hydrous silicates of aluminum and magnesium .They are made of sheets of silica (tetrahedral) and alumina (octahedral) stacked on above the other forming sheet like of flaky particle. Montmorillonite has a three-sheeted structure with expanding lattices. The structure carries negative charge, due to isomorphic substitution of some aluminum ions by magnesium ions and minerals becomes chemically active.

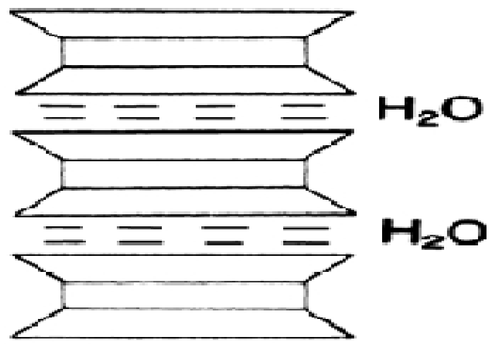


Fig1.2 Montmorillonite

The black cotton soils are found to have the following chemical composition

Table 1.2 Chemical Composition of Black Cotton Soil

S. No.	Property	Range
1.	**pH value**	>7(Alkaline)
2.	**Organic content**	0.4 to 204 %
3.	**$CaCO_3$**	5 to 15 %
4.	**SiO_2**	50 to 55 %
5.	**SiO_2 , Al_2O_3**	3 to 5 %
6.	**Montmorillinite mineral**	30 to 50 %

1.6 Problems associated with black cotton soil:

Black Cotton soils are problematic for engineers everywhere in the world, and more so in tropical countries like India because of wide temperature variations and because of distinct dry and wet seasons, leading to wide variations in moisture content of soils. The following problems generally occur in black cotton soil.

(a) High compressibility

Black Cotton soils are highly plastic and compressible, when they are saturated. Footing, resting on such soils under goes consolidation settlements of high magnitude.

(b)Swelling

A structure built in a dry season, when the natural water content is low shows differential movement as result of soils during subsequent wet season. This causes structures supported by such swelling soils to lift up and crack. Restriction on having developed swelling pressures making the structure suitable.

(c) Shrinkage

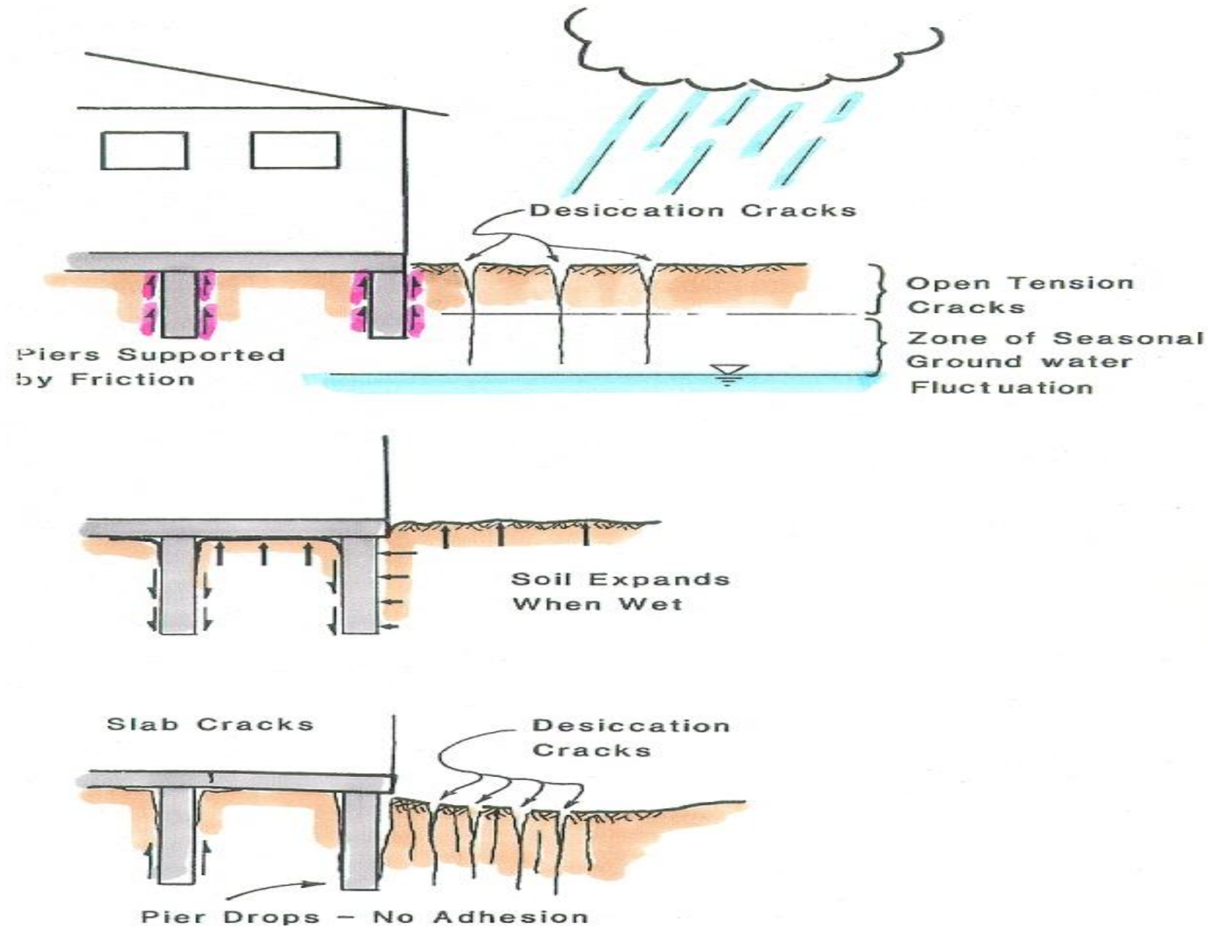
A structure built at the end of the wet season when the natural water content is high, shows settlement and shrinkage cracks during subsequent dry season.

Figure 1.4 Cracks Developed in black cotton soil after drying

1.7 Effects of black cotton soil on structures

Because of affinity of black cotton soil to the moisture, seasonal movement of moisture to the soil below the structure during wet and dry season respectively, creates a cumulative increase of moisture below the structure and produces a dome shaped heave at the central proportions of the structures thus leading to differential of level. Differential causes inclined cracks in structure flexible road pavements shows waviness and rigid pavement produces cracks. The pavement cracks in may even reach the de seated sub grade, which during subsequent wet season may be filled water and make the road unserviceable. Canals in the black cotton soil areas develop large transverse cracks, which during flooding open out, and cause heavy leakage and even failure of embankment. Cracks in any number do not really pose a threat to the safety of the structures, except when they are too wide or they continue to increase in width. Starting as hair cracks they go on widening.

Figure below shows various types of failures that is caused by swelling soil.



Courtsy :Robert Olshansky, and Robert B. Rogers

Figure.1.5 Damage to home supported on shallow piers. **(1)** At the beginning of the rainy season, the piers are still supported by friction with the soil. When it begins to rain, water enters deep into the soil through the cracks**. (2)** After 5 to 10 large storms, the soil swells, lifting the house and piers**. (3)** In the dry season, the groundwater table falls and the soil dries and contracts. As tension cracks grow around the pier, the skin friction is reduced and the effective stress of the soil increases (due to drying). When the building load exceeds the remaining skin friction, or the effective stress of the soil increases to an all-time high, adhesion is broken by this straining, and the pier sinks.

CHAPTER-2
LITERATURE REVIEW

Literature review: 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.

Chin (1975), Stated that expansive soils in Mississippi were recognized in mid 19^{th} century by E.W.HILGARD of soil science . He observed that certain clayey soil have tendency to cracks in dry seasons and form large surface cracks 2 to 3 inch wide, which were injurious to vegetation and building. Hilgard commented that most brick and stone building in Jackson, not secured by wall anchors or concrete foundations,developed cracks in all direction over time .

David Rogers, Robert Olshansky, and Robert B. Rogers J.(1998) ,Stated that Expansive soils in many parts of the United State pose a significant hazard to foundations for light buildings. Swelling clays derived from residual soils can exert uplift pressures of as much as 5,500 PSF, which can do considerable damage to lightly-loaded wood-frame structures. Insurance companies pay out millions of dollars yearly to repair homes distressed by expansive soils.

Expansive soils owe their characteristics to the presence of swelling clay minerals. As they get wet, the clay minerals absorb water molecules and expand; conversely, as they dry they shrink, leaving large voids in the soil. Swelling clays can control the behavior of virtually any type of soil if the percentage of clay is more than about 5 percent by weight. Soils with smectite clay minerals, such as montmorillonite, exhibit the most profound swelling properties.

Potentially expansive soils can typically be recognized in the lab by their plastic properties. Inorganic clays of high plasticity, generally those with liquid limits exceeding 50 percent and plasticity index over 30, usually have high inherent swelling capacity. Expansion of soils can also be measured in the lab directly, by immersing a remolded soil sample and measuring its volume change.

In the field, expansive clay soils can be easily recognized in the dry season by the deep cracks, in roughly polygonal patterns, in the ground surface The zone of seasonal moisture content fluctuation can extend from three to forty feet deep. This creates cyclic shrink/swell behavior in the upper portion of the soil column, and cracks can extend to much greater depths than imagined by most engineers.

Kumar Sabat (2012), Stated that Expansive soil is a type of clayey soil having montmorillonite mineral, which expands when comes in contact with water and shrinks when the water evaporates. This soil is generally found in arid and semi-arid regions of the world. A lot of damages occur on structures founded on this type of soil. The damages normally appear as cracks in, buildings, canal beds and linings, pavements, lifting of water supply pipeline and sewerage lines etc.

A number of innovative techniques are there for construction on this type of soil. Physical and chemical alteration of soil using solid wastes like fly ash, rice husk ash, marble dust, phosphor gypsum, granulated blast furnace slag, red mud, etc is one of them Utilization of solid wastes in this manner not only protects the environment from degradation but also improves the engineering properties of the expansive soil.

The quarry dust/ crusher dust is obtained as solid wastes, during crushing of stones to obtain aggregates. The annual production of quarry dust is roughly around 200 million tones(Soosan et al.2005).The disposal of which creates a lot of geo environmental problems. A limited research is available regarding the utilisation of this waste for stabilization of expansive soil. Gupta et al. (2002) made a study on the stabilization of black cotton soil using crusher dust a waste product from Bundelkhand region, India and optimal percentage of crusher dust was found to be 40%. Gulsah (2004) investigated the swelling potential of a synthetically prepared expansive soil (kaolinite and bentonite mixture), using aggregate waste (quarry dust) and lime.

There was reduction in the swelling potential along with improvement in other engineering properties and the reduction was increased with increasing percentage of stabilizers and days of curing. Sabat and Das(2009) had stabilized expansive soil using quarry dust and lime and found the stabilization effects with improvement in Unconfined compressive strength (UCS), soaked California bearing ratio (CBR) and reduction in swelling pressure etc.Ali and Koranne(2011) had studied the combined effects of stone dust(quarry dust) and fly ash (equal proportion of stone dust and fly ash) on swell and strength properties of an expansive soil along with other properties.

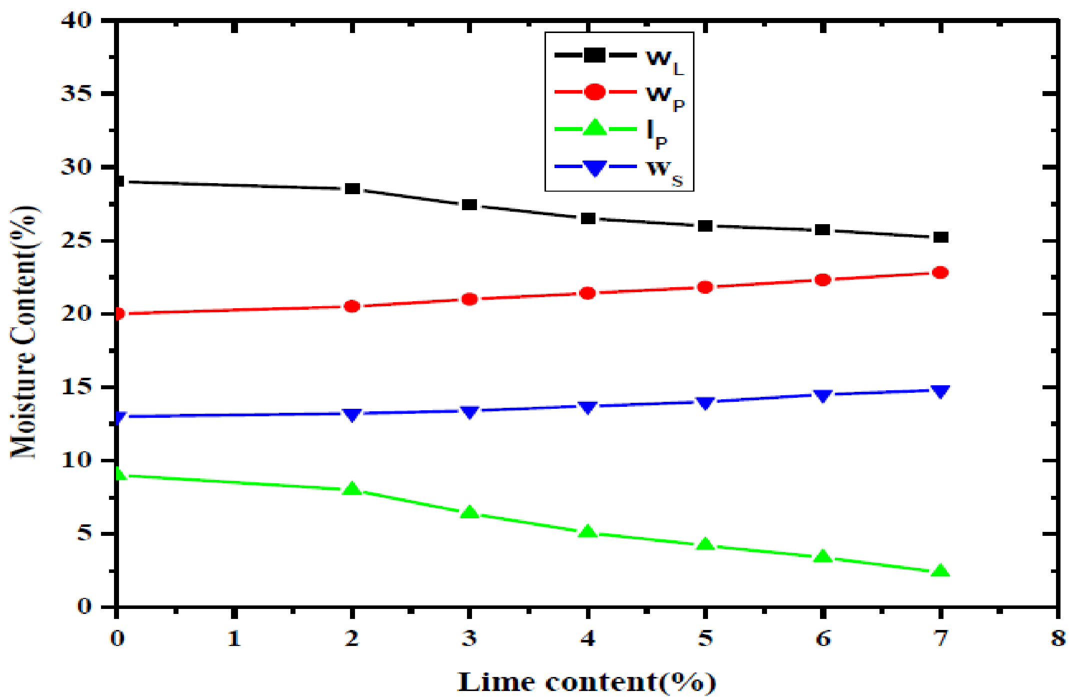


Figure. 2.1 Relations between moisture content and lime content

It was found that there was a maximum improvement in strength properties for the combination of fly ash and stone dust as compared to fly ash or stone dust, added separately. When quarry dust is added with expansive soil it is expected that it will, make it more porous, less durable, reduce cohesion etc. The addition of lime has been found to be a successful stabilizer of the expansive soil. The effect of lime on Atterberg's limit, compaction, shear strength and durability characteristics of quarry dust stabilized expansive soil has not been found in literature. The objective of this paper is to study the effects of lime on Atterberg's limit, compaction, shear strength and durability characteristics of an expansive soil stabilized with optimum percentage of quarry dust. The effect of 7 and 28 days of curing on shear strength are also studied.

Koranne & Ali (2011), Stated that the term expansive soil applies to soils that have the tendency to swell when their moisture content is increased and shrink when their moisture content is decreased. The moisture may come from rain flooding, leaking water or sewer lines or from reduction in surface evapotranspiration when an area is covered by a building or pavement.

To achieve the economy and for proper performance of structures it is necessary to improve the geotechnical properties of expansive soil. Due to the high demand for rubble and aggregates for construction purposes, rubble quarries and aggregate crushers are very common.

Out of the different quarry wastes, quarry dust is one, which is produced in abundance. About 20–25% of the total production in each crusher unit is left out as the waste material-stone dust. Bulk utilization of this waste material is possible through geotechnical applications like embankments, back-fill material, sub-base material and the like. Fly ash is a waste by product from thermal power plants, and consuming thousands hectares of precious land for its disposal and also causing severe health and environmental hazards.

The results of an experimental program undertaken to investigate the effect of stone dust & fly ash combine at different percentage on expansive soil, the test results such as index properties, Proctors compaction, swelling and unconfined compression strength obtained on expansive clays mixed at different proportions of fly ash and stone dust admixture are presented and discussed.

From the results, it is observed that at optimum percentages, i.e., 20 to 30% of admixture, it is found that the swelling of expansive clay is almost controlled and also noticed that there is a marked improvement in the other properties of soil. The conclusion drawn from this investigation is that the combination of equal proportion of stone dust and fly ash is more effective than the addition of stone dust/fly ash alone to the expansive soil in controlling the swelling nature.

Bhyravavajhala (2003), had observed volume changes due to changes in moisture content. In monsoon they imbibe water and swell, and in summer on evaporation of water, they shrink. This alternate swelling and shrinkage causes distress in many civil engineering structures like lightly loaded buildings, retaining walls, canal linings and pavements. This dual problem has been combated by researchers through innovative techniques. Different foundation techniques are adopted in practice to overcome the problems posed by expansive soils to various structures. While some special foundation techniques do not alter the nature of the soil, some change the nature of the soil. They are briefly discussed below.

While under-reamed piles and granular pile-anchors do not change the nature of the soil, they counteract the problem effectively. Under-reamed piles are bored cast in-situ piles with enlarged bases and connected at their top by plinth beams. Enormous uplift resistance is mobilized along the surface of the pile and reduces heave. In the case of multi under-reamed pile, frictional resistance is mobilized over a larger perimeter and reduces heave. They have been widely used in India. Granular pile-anchors (Phani Kumar, 1995) are highly effective in arresting the heave of foundations in expansive soils. In this technique, the foundation is anchored at the bottom of the granular pile to a mild steel plate with the help of a mild steel rod embedded in the foundation. Enormous shear resistance will be mobilized along the pile-soil interface and resists the uplift.

The swelling pressure acting in the lateral direction also confines the granular pile-anchor and contributes to the shear resistance. In mechanical alternation, the top layers of expansive soil are replaced with non-expansive material.

Sand cushion (Satyanarayana, 1966) and cohesive non swelling (CNS) layer method (Katti, 1978) are examples of this technique. In sand cushion technique, the entire depth of the expansive clay stratum if it is thin, or a part thereof, if it is deep enough, is removed and replaced by a sand cushion compacted to the desired density and thickness.

In monsoon, the saturated sand occupies less volume, accommodating some of the heave of underlying soil and in summer partially saturated sand bulks and occupies the extra space left by the shrinkage of the soil.

In CNS layer technique, about top 1m to 1.2m of the expansive soil is removed and replaced by a cohesive non-swelling soil layer. This technique has been effectively used in various field conditions. In chemical alteration, chemicals are added to the expansive clay to reduce heave by altering the nature of clay materials. Lime is the most effective and economical additive. Flocculation, pozzolanic reaction and carbonation improve the behavior of the lime-treated soil.

Calcium chloride has also been studied as a stabilizing agent of expansive soils. Calcium chloride is a hygroscopic material and hence is quite suited for stabilization of expansive soils, because it absorbs water from atmosphere and prevents shrinkage cracks in expansive soils during the summer.

Then advantage with calcium chloride, therefore, is its moisture-retaining property. It supplies cations in exchange of the ions on the clay particles which adsorb to the negatively charged clay particle reducing the inter granular electric repulsion which is chiefly responsible for reducing the swelling properties. Aggregation also immediately takes place and increases the strength. Plasticity index has also been found to decrease owing to the aggregation effect. This paper presents the details of an experimental work done to study the efficacy of calcium chloride in stabilizing expansive soils and to compare the strength of expansive soils treated with lime and calcium chloride.

Beni Lew (2010), stated that the influence of cyclic wetting and drying on the swelling behavior of natural expansive soils is well documented Expansive soils pose a problem where rapid urbanization and development are occurring. As development extends into these areas, identification and quantification of the soil properties that define shrink-swell potential are essential to properly evaluate the stability of a soil as a foundation material.

The swelling potential (swell percent and swell pressure) of each sample was measured using the "Load-Swell Method" and the "Constant Volume Method" proposed by Al-Rawas (1993) in undisturbed samples. Mineral composition was determined by x-ray diffraction (XRD), scanning electron microscopy (SEM) and energy dispersive x-ray method (EDX). Clay minerals were identified with a Scintag XDS 2000 x-ray diffractometer with Cu-K_ radiation. After conducting the tests, the diffraction pattern for each sample was matched with the standard patterns prepared by the Joint Committee of Powder Diffraction Data Service (JCPDS) for a both qualitative and quantitative evaluation. The SEM analysis was carried out using a JEOL (JSM-

840), which performs morphological and micro-structural assessment and gives a full elemental description using the EDX analyzer.

Expansive soils are well known to contained expansive clay minerals such as montmorillonite, vermiculite, hlloysite, and sepiolite . moreover, beside the existence of these minerals two alternate seasons are necessary through out the year a wet and dry one , like the one in Cuiaba.

Mineralogy composition of sample was energy dispersive x-ray technique (EDX), x-ray diffraction analysis (XRD), scanning electron microscopic (SEM).

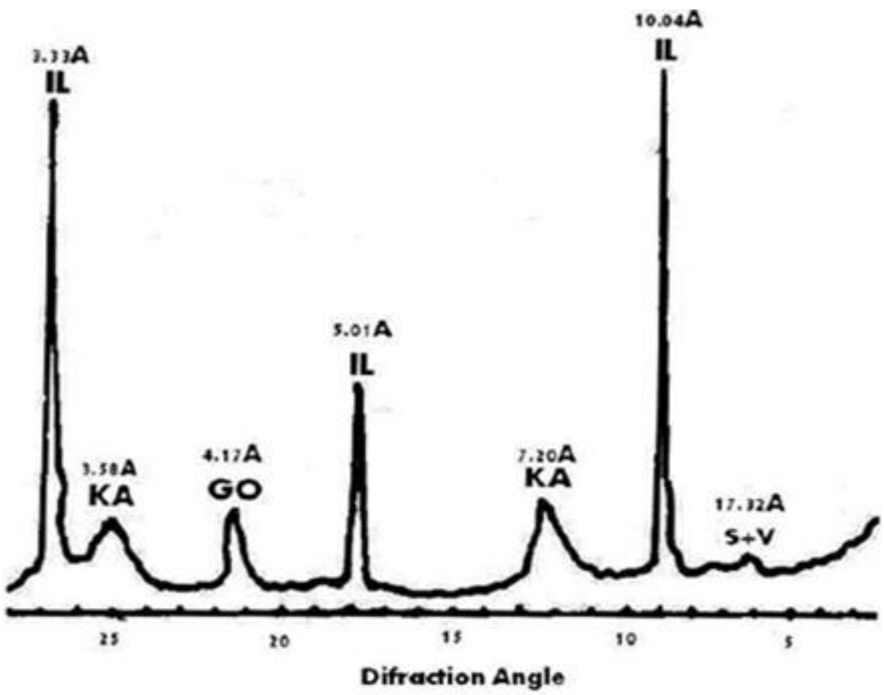


Figure. 2.2 X-ray Diffraction results of soil sample .Illite(IL).kaolinite (KA)SG(smectite group)

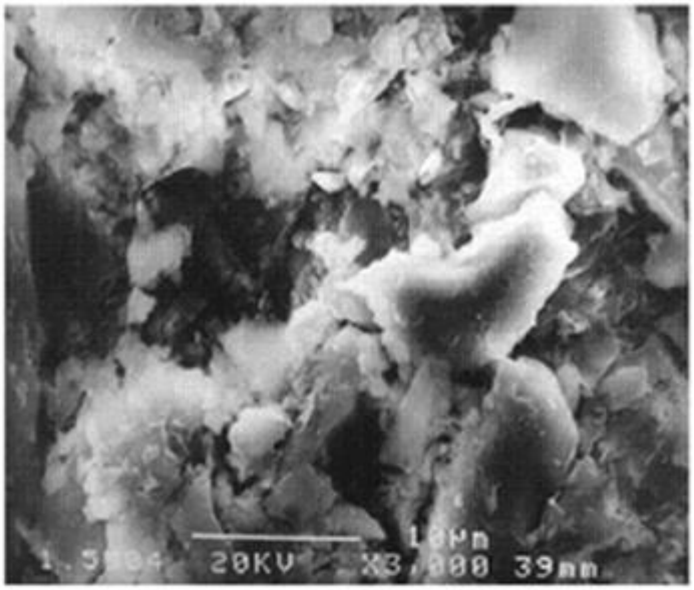


Figure. 2.3 scanning electron microscopic of smectite group

Ameta and purohit(2003),Stated that The expansive soils, which are spread over extensive areas of India in states, like Rajasthan, Madhya Pradesh, Gujarat, Andhra Pradesh, Karnataka and Tamilnadu posed serious problems for buildings and roads. In our country the typical example of expansive soils are black cotton soil, bentonite, mar and kabar Bentonite is a highly expansive soil. It exhibits a tendency of swelling on coming in contact with water and shrinks on removal of water which may result in structural damage to engineering construction.

In case of bentonite, free swelling is up to ten times to fifteen times to its original volume. Since expansive soils have a tendency to change their volume to a large extent, they cause heavy distress to engineering constructions. The lightweight structures are severely affected due to high swelling pressure exerted by these soils. Such type of large scale distress, due to expansive shrinking nature of expansive soil, can be prevented by either obstructing the soil movement and reducing the swelling pressure of soil or making the structure sufficiently resistant to damage from soil movement.

Limestone belts are spread in 26 districts. Rajasthan has about 7,000 million tonnes of all grades of limestone. It is the third largest producer of limestone in the country. The districts with large deposits of limestone are Ajmer, Banswara, Bundi, Pali, Sirohi, Chittaurgarh, Jaisalmer, Jodhpur, Nagaur, Sawai Madhopur and Udaipur districts. Rajasthan contributes 95 per cent of the country's gypsum production. Gypsum is widely distributed throughout the northeastern part of the state, particularly in the districts of Bikaner, Ganganagar, Nagaur, Churu, Barmer, Jaisalmer and Pali .

The locally available materials lime and gypsum are used for investigations. As a result of extensive research work carried out in India and abroad on stabilization of soil with lime and gypsum, it has been established that all soils do not respond favorable to lime-gypsum treatment. Only reactive soils respond favorably to lime-gypsum treatment to form chemical compounds of well-developed crystalline structures under suitable sets of conditions.

We found that following mixtures of lime-gypsum have reduced plasticity and swelling of bentonite soil: (8% Lime + 2% Gypsum); (4% Lime + 3% Gypsum); (6% Lime + 3% Gypsum); (8% Lime + 3% Gypsum); (2% Lime + 4% Gypsum); (4% Lime + 4% Gypsum); (6% Lime + 4% Gypsum); (8% Lime + 4% Gypsum). A comparison graph of decrease in liquid limit and plastic limit is shown in Figure.

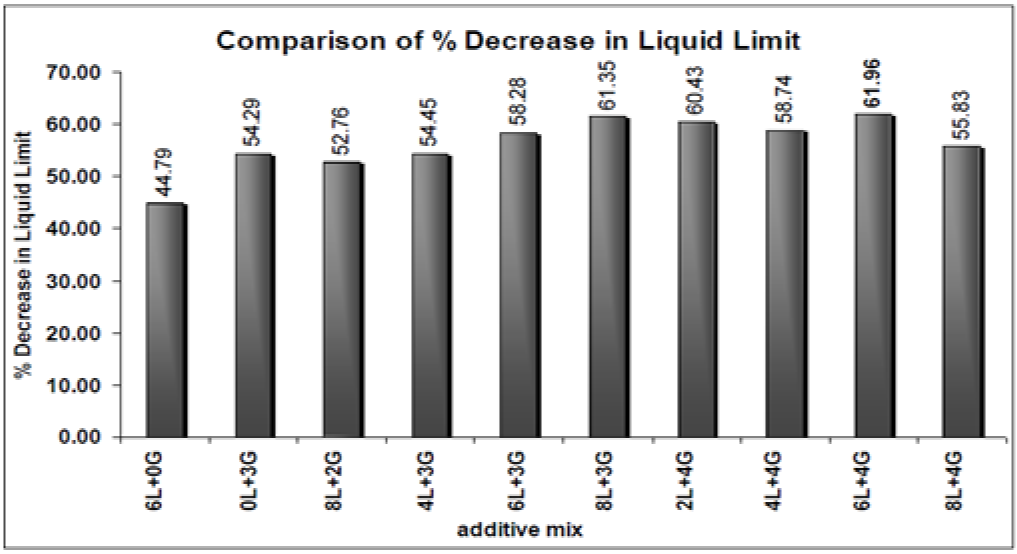


Figure. 2.4 comparison % Decrease in liquid limit

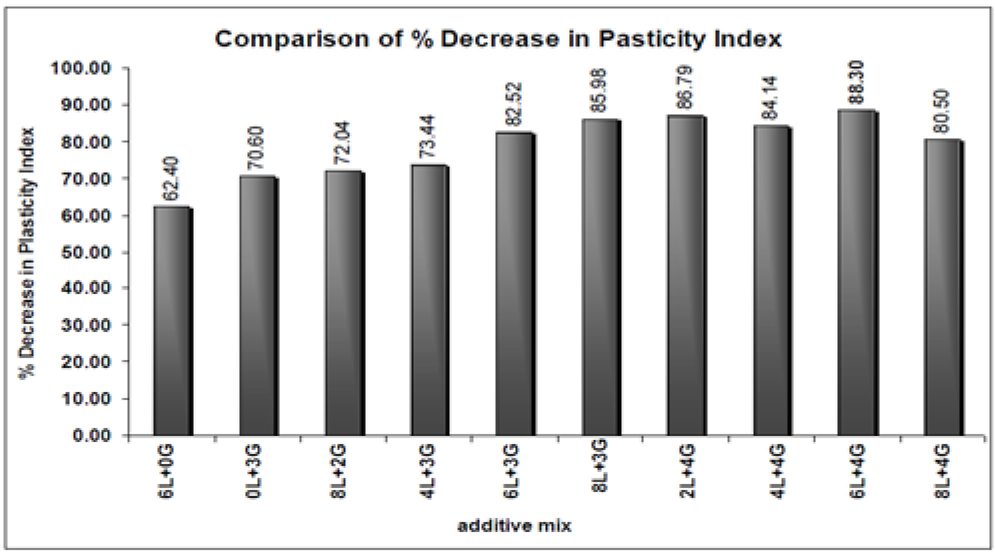


Figure. 2.5 comparison % Decrease in Plastic limit

CHAPTER-3

EXPERIMENTAL STUDIES

3.1 x-ray diffraction analysis:

An electron in an alternating electromagnetic field will oscillate with the same frequency as the field. When an X-ray beam hits an atom, the electrons around the atom start to oscillate with the same frequency as the incoming beam. In almost all directions we will have destructive interference, that is, the combining waves are out of phase and there is no resultant energy leaving the solid sample. However the atoms in a crystal are arranged in a regular pattern, and in a very few directions we will have constructive interference.

The waves will be in phase and there will be well defined

X-ray beams leaving the sample at various directions. Hence, a diffracted beam may be described as a beam composed of a large number of scattered rays mutually reinforcing one another. This model is complex to handle mathematically, and in day to day work we talk about X-ray reflections from a series of parallel planes inside the crystal. The orientation and inter planar spacing of these planes are defined by the three integers h, k, l called indices.

A given set of planes with indices h, k , l cut the a-axis of the unit cell in h sections, the b axis in k sections and the c axis in l sections. A zero indicates that the planes are parallel to the corresponding axis. E.g. the 2, 2, 0 planes cut the a– and the b– axes in half, but are parallel to the c– axis.

If we use the three dimensional diffraction grating as a mathematical model, the three indices h, k, l become the order of diffraction along the unit cell axes a, b and c respectively. It should now be clear that, depending on what mathematical model we have in mind, we use the terms X-ray reflection and X-ray diffraction as synonyms.

Let us consider an X-ray beam incident on a pair of parallel planes P1 and P2, separated by an inter planar spacing d.

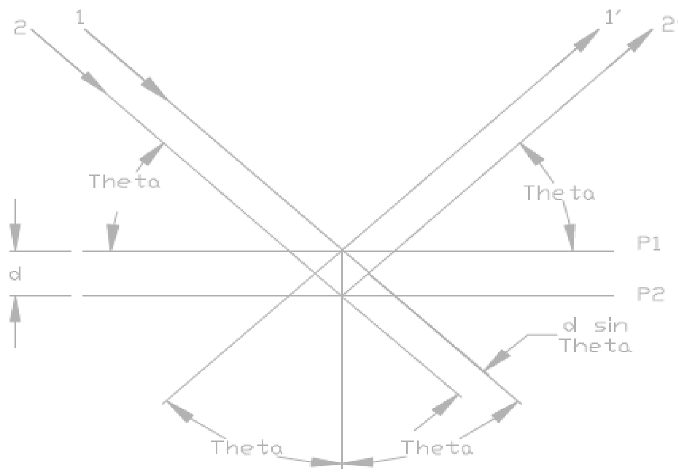


Figure.3.1 x-ray diffrecto metreCourtsy: www.scintag.com

The two parallel incident rays 1 and 2 make an angle (THETA) with these planes. A reflected beam of maximum intensity will result if the waves represented by 1' and 2' are in phase. The difference in path length between 1 to 1' and 2 to 2' must then be an integral number of wavelengths, (LAMBDA). We can express this relationship mathematically in Bragg's law

$$\mathbf{2d\sin\theta = n\lambda}$$

Fig 3.2 x-ray diffracto meter

3.2 Scanning Electron Microscope (SEM):

A scanning electron microscope (SEM) is a type of electron microscope that images a sample by scanning it with a high-energy beam of electrons in araster scan pattern. The electrons interact with the atoms that make up the sample producing signals that contain information about the sample's surfacetopography, composition, and other properties such as electrical conductivity.

The types of signals produced by an SEM include secondary electrons, back-scattered electrons (BSE), characteristic X-rays, light (cathodoluminescence), specimen current and transmitted electrons. Secondary electron detectors are common in all SEMs, but it is rare that a single machine would have detectors for all possible signals.

The signals result from interactions of the electron beam with atoms at or near the surface of the sample. In the most common or standard detection mode, secondary electron imaging or SEI, the SEM can produce very high-resolution images of a sample surface, revealing details less than 1 nm in size. Due to the very narrow electron beam, SEM micrographs have a large depth of field yielding a characteristic three-dimensional appearance useful for understanding the surface structure of a sample.

This is exemplified by the micrograph of pollen shown to the right. A wide range of magnifications is possible, from about 10 times (about equivalent to that of a powerful hand-lens) to more than 500,000 times, about 250 times the magnification limit of the best light microscopes. Back-scattered electrons (BSE) are beam electrons that are reflected from the sample by elastic scattering. BSE are often used in analytical SEM along with the spectra made from the characteristic X-rays. Because the intensity of the BSE signal is strongly related to the atomic number (Z) of the specimen, BSE images can provide information about the distribution of different elements in the sample. For the same reason, BSE imaging can image colloidal gold immuno-labels of 5 or 10 nm diameter, which would otherwise be difficult or impossible to detect in secondary electron images in biological specimens. Characteristic X-rays are emitted when the electron beam removes an inner shell electron from the sample, causing a higher energy electron to fill the shell and release energy. These characteristic X-rays are used to identify the composition and measure the abundance of elements in the sample.

Figure.3.3Scanning electron microscopic

3.3 Liquid and plastic limit tests:

Object: - To determine liquid limit

To determine plastic limit

Apparatus: -

- Casagrande liquid limit device
- ASTM and BS grooving tool
- Glass plate 20 x 15 cm
- 425 micron I.S. Sieve
- 3 mm diameter rod
- Balance (0.01gm sensitivity)
- Drying oven
- Distilled water
- Measuring cylinder

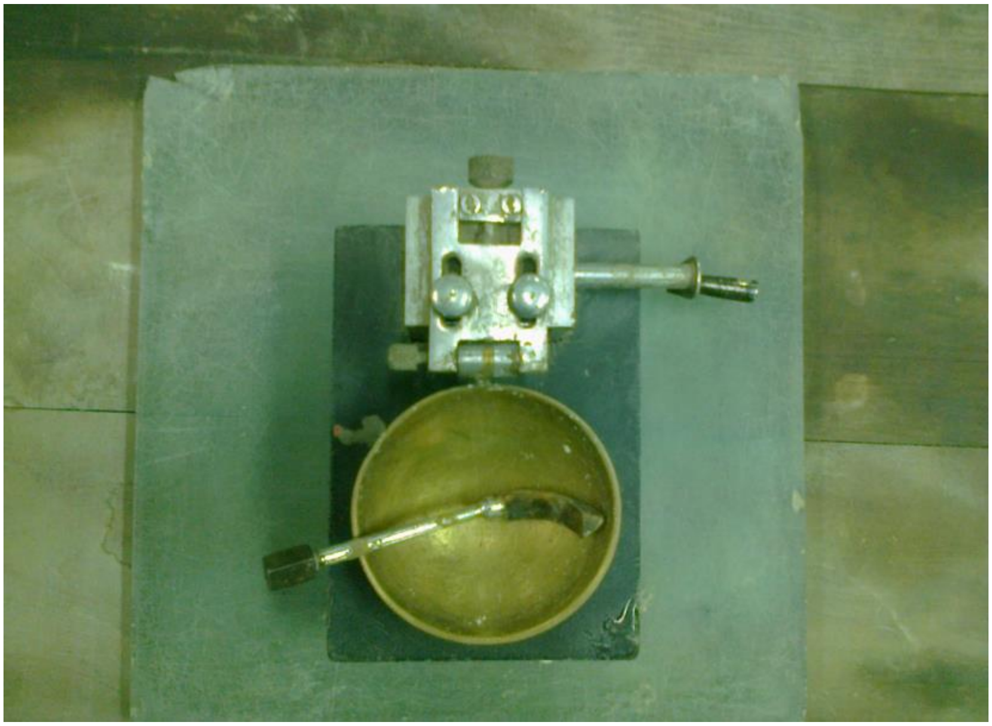


Figure.3.4 Casagrande liquid limit device

Theory and Applications:-

Liquid limit is the water content at which soil passes from zero strength to an infinitesimal strength, hence the true value of liquid limit can not be determined. For determining purpose liquid limit is that water content at which apart of soil, cut by a groove of standard dimensions, will flow together for a distance of 1.25 cm under an impact of 25 blows in a standard Liquid limit apparatus. The soil at the water content has some strength which is about 0.17 N/cm2 (17.6 g/ cm2). At this water content soil just passes from liquid state to plastic state.

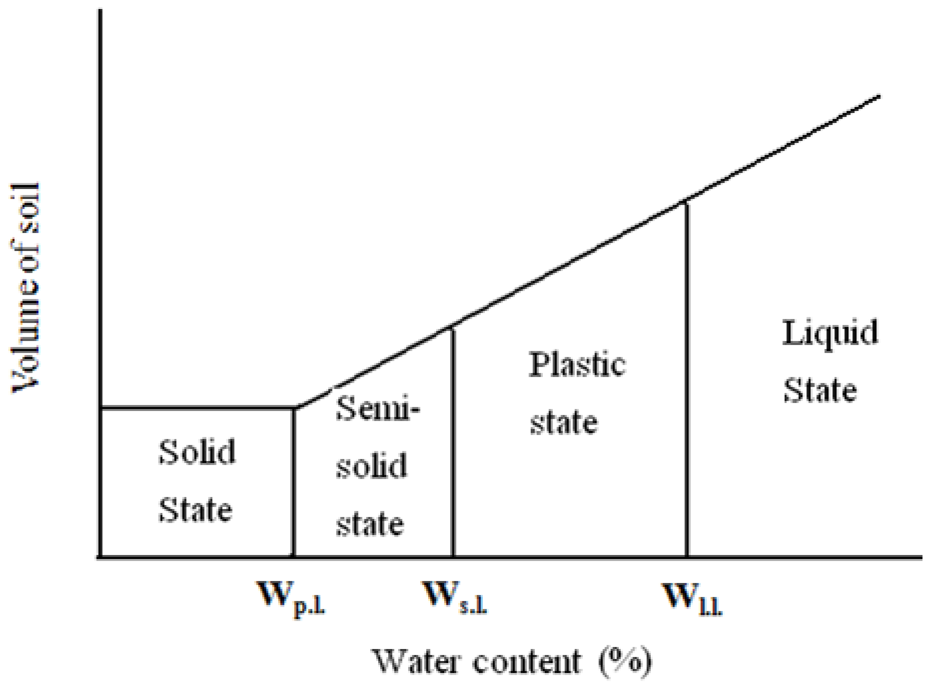


Figure.3.5 Relation between soil volume and moisture content

The moisture content at which soil has the smallest plasticity is called the plastic limit. Just after plastic limit the soil displays the properties of a semi solid.

For determination of purpose, the plastic limit is defined as the water content at which a soil will just begin to crumble when rolled in to a thread of 3mm in diameter.

The difference in moisture content or interval between “liquid and plastic limits is termed the plasticity index. knowing the liquid limit and plasticity index. Soil may be classified with the help of plasticity. In the plasticity chart following symbols are used

CL = Clay of low compressibility
CI = Clay of medium compressibility
CH = Clay of high compressibility
ML = Silt of low compressibility
MI = Silt of medium compressibility
MH = Silt of high compressibility
OL = Organic soil of low compressibility
OI = Organic soil of medium compressibility
OH = Organic soil of high compressibility

Standard value: Table 3.1

Plasticity index	**Soil type**	**Degree of Plasticity**	**Degree of Cohesiveness**
0	Sand	Non Plastic	Non Cohesive
<7	Silt	Low plastic	Partly Cohesive
7-17	Silt Clay	Medium Plastic	Cohesive
>17	Clay	High Plastic	Cohesive

Precautions:

Use distilled water in order to minimize the possibility of iron exchange between the soil and any impurities in the water.

(1) Soil used for liquid and plastic limit determinations should not be oven dried prior to testing.

(2) In liquid limit test, the groove should be closed by a flow of the soil and not by slippage between the soil and the cup.

(3) After mixing distilled water to the soil sample, sufficient time should be given to permeate the water throughout the soil mass.

(4) Wet soil taken in the container for moisture content determinations should not be left in the air even for some time, the containers with soil samples should either be placed in desiccators or immediately be weighed.

(5) For each test, cup and grooving tool, should be clean.

3.4 Compaction test (standard proctor):

Object:-

(1) To determine the optimum moisture content and maximum dry density of a soil by proctor test

(2) To plot the curve of zero air void.

Apparatus:-

(1) Cylinder mould
(Capacity 1000 c.c., internal dia.100 mm, effective ht. 127.3 mm)

(2) Rammer for light compaction (face dia. 50 mm., mass of 2.6 kg, free drop 310 mm)

Or

Rammer for heavy compaction (face dia50 mm, mass4.89 kg, free drop 450mm)

(3) Mould accessories (detachable base plate removal collar)

(4) I.S. Sieves (20 mm,4.75 mm)

(5) Balance (capacity 10 kg, sensitivity 1 gm)

(6) Balance (Capacity 200 gm sensitivity 0.01 gm)

(7) Drying oven (temperature 105°C to 11°C)

(8) Desiccators

(9) Graduated jars

(10) Straight edge

(11) Spatula

(12) Scoop

Figure.3.6 Proctor test

Theory and Applications:-

Compaction is the process of densification of soil mass by reducing air voids. This process should not be confused with consolidation which is also a process of densification of soil mass but by the expulsion of water under the action of continuously acting static over a long period.

The degree of compaction of a soil is measured in terms of its dry density. The degree of compaction mainly depends upon its moisture content, compaction energy and type of soil. For a given compaction energy every soil attains the maximum dry density at a particular water content which is known as optimum moisture content.

In the dry side, water acts as a lubricant and helps in the closer packing of soil grains. In the wet side water starts to occupy the space of soil grains and hinders in the closer packing of grains

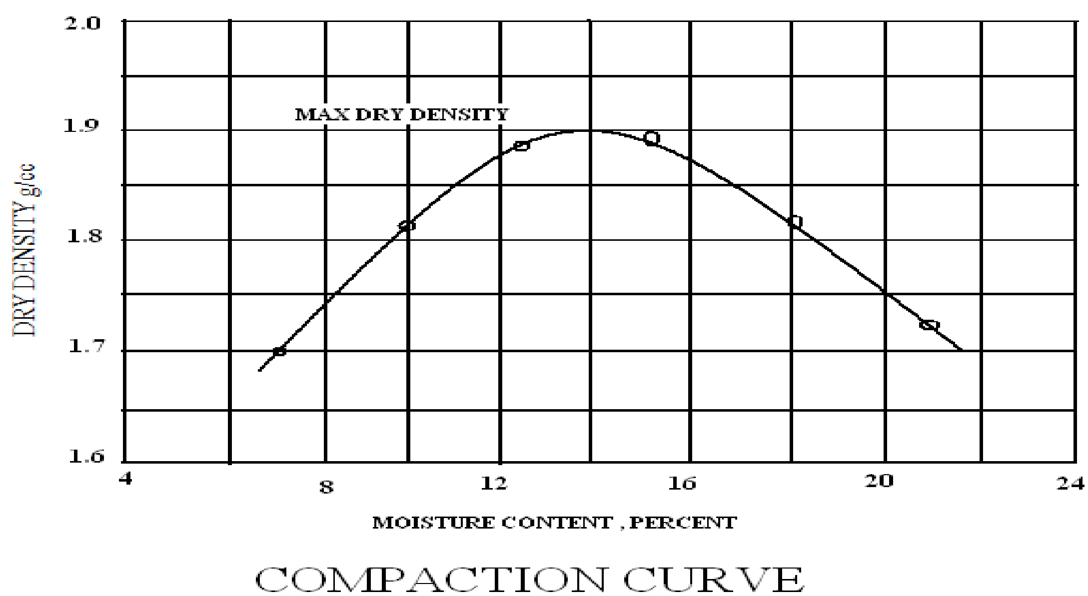


Figure.3.7 compaction curve

Application:-

Compaction of soils increases their density, shear strength, bearing capacity but reduces their void ratio porosity, permeability and settlements. The results of this test are useful in the stability of field problems like earthen dams, embankments, road and airfields. In such constructions, the soils are compacted. The moisture content at which the soils are compacted in the field is controlled by the value of optimum moisture content determined by the laboratory proctor compaction test. The compaction energy to be given by the field compaction unit is also controlled by the maximum dry density determination in the laboratory. In other words, the laboratory compaction tests results are used to write the compaction specification for field compaction of soils

Precautions:-

1. Adequate period is allowed for mixing the water with soil before compaction.
2. The blows should be uniformly distributed over the surface of each layer.
3. Each layer of compacted soil is scored with spatula before placing the soil for the succeeding layer.
4. The amount of soil used should be just sufficient to fill the mould i.e." at the end of compacting the last layer the surface of the soil should be slightly (5 mm) above the top rim of the mould.
5. Mould should be placed on a solid foundation during compaction.

3.5 CBR test:

The 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. Just after World War II, the US corps of Engineers adopted the CBR test for use in designing base course for airfield pavements. The test is empirical and results cannot be related accurately with any fundamental property of the material. The method of test has been standardized by the ISI also.

The CBR is a measure of a material to penetration of standard plunger under controlled density and moisture conditions. The test procedure should be strictly adhered if high degree of re productivity is required. The CBR test may be conducted in remolded or undisturbed specimen in the laboratory. US corps of engineers have also recommended a test procedure for in situ test. Many methods exist today which utilize mainly CBR test values for designing pavement structure .The test is simple and has been extensively investigated for field correction of flexible pavement thickness requirement.

The test consists of cylindrical plunger of 50 mm diameter to penetrate a pavement component material at 1.25 mm/ min. The loads for 2.5 mm and 5 mm are recorded. This is expressed as a percentage of standard load value at a representive deformation level to obtain CBR value. The standard load values were obtained from the average of a large number of tests on different crushed stones. Are as:-

Table 3.1Standard Load values on Crushed Stones for Different Penetration Value:-

Penetration mm	Standard load, kg	Unit standard load, kg/cm^2
2.5	1370	70
5.0	2055	105
7.5	2630	134
10.0	3180	162
12.5	3600	183

Apparatus:-

(a) **Loading Machine: -** Any compression machine can operate at a constant rate of 1.25 mm per minute can be used for this purpose. If such machine is not available then a calibrated hydraulic press with proving ring to measure load can be used. In fig a metal penetration piston or plunger of diameter 50 mm is attached to the loading machine.

(b) **Cylindrical mould:-** Moulds of 150 mm diameter and 175 mm height provide with a collar of about 50 mm length and detachable perforated base are used for this purpose. A spacer disc of 148 mm diameter and 47.7 mm thickness is used to obtain a specimen of exactly 127.3 mm height.

(c) **Compaction Rammer: -** The material is usually compacted as specified for the work, either by dynamic compaction or by static compaction. The detail for dynamic compaction suggested by the ISI are as:-

(d) **Adjustable Stem, perforated plate, tripod and dial gauge:-**

The standard procedure requires that the soil sample before testing should be soaked in water to swelling .For this purpose the above listed accessories are required.

(e) **Annular weight: - Annular weight: -** In order to simulate the effect of the overlying pavement weight, annular weights each of 2.5 kg and 147 mm dia. Are placed on the top of the specimen, both at the time of soaking and testing the samples, as surcharge.

Figure. 3.8CBR test apparatus

3.6 Grain size analysis (by hydrometer):

- This process describes the quantitative determination of the distribution of particle sizes in soils. The distribution of particle sizes larger than 75 um is determined by a sedimentation process, by means of a hydrometer to secure the essential data.
- Dispersing agent - prepare a solution of sodium hexametaphosphate (sometimes called sodium metaphosphate) in distilled or demineralised water. 33 g of sodium hexametaphosphate/litre and 7 g sodium carbonate is used in the solution.
- About 200 g of black cotton soil taken and added with water and sodium hexametaphosphate and put in the mechanical stir cup. String process occurs for a period of 15 mins. After that it is poured into the hydrometer flask
- After 20 s the Hydrometer is inserted gently to a depth slightly below its floating position.
- Hydrometer readings are taken in the interval of 1/2, 1, 2, and 4 minutes. After that it was taken out and rinse with distilled water.
- The hydrometer was re-inserted in the suspension and readings are taken after periods of 8, 15, and 30 minutes; 1, 2 and 4 hours after shaking. The hydrometer is removed and rinsed with water after each reading.

Figure.3.9 Hydrometer analysis

3.7 Differential Free Swell Test:

Two sample of dried soil weighing 10g each , passing through 425 micron sieve are taken . one is put in a 50cc graduated glass cylinder containing kerosene. Other sample is put is a similar cylinder containing distilled water. Both the samples are left undisturbed for 24 hours and then there volumes are noted.

The differential free swell is expressed by

$$DFS = \frac{\text{Soil volume in water} - \text{soil volume in kerosene}}{\text{Soil volume in kerosene}} \times 100\%$$

Chapter-4

Identification of black cotton soil

4.1 Collection of soil sample

Soil stabilization is carried out for weak soils having low strength and poor engineering properties, the mostly available black cotton soil has low strength and stability to resist load coming on it and also it has high settlement characteristics.

For our project work we collected Black cotton soil sample from Bundhelkhand region .About 125 Kg kg soil sample was brought by us to soil mechanics lab for carrying out our project work.

Black cotton soil has been identified on the basis of following results

4.2 X-Ray Diffraction analysis of black cotton soil

The clay minerals of smectite groups occur in the form of very small crystals with frequently defective structure. Theories about their structural constitution come from the hypothesis that their structure is compounded of 2:1 layers, between which are exchangeable hydrated ions. The 2:1 layer is created when two tetrahedral sheets (with opposite polarity) are connected with one octahedral sheet in the middle - Si.Al.Si.Si.Al.Si-. This latticed structure may lightly expand and bond molecules on the exterior and also interior Water molecules in the interlayer space of smectites and vermiculites can be displaced by many polar organic molecules. Neutral organic ligands can form complexes with the interlayer cations. One of the most significant and widespread minerals of dioctahedral smectite is montmorillonite. It is characterized by no or very small substitution of Al^{3+} for Si^{4+} on the tetrahedron position. The charge of the 2:1 layers is produced primarily by octahedron substitution. Variability in chemical composition leads to their division into types: Wyoming, Chambers, Otay, Fe-montmorillonite. Monovalent (especially Na^+ , K^+) and divalent cations could appear in the interlayer of smectite. These compensate the negative charge of 2:1 layers

Mineralogy composition of black cotton soil sample was determined by x- ray diffraction technique(XRD). Results of XRD test showed that presence of kaolinite, illite ,smectite (a mineral form the montmorillonite group), in all samples. A typical results of a sample is shown in Figure. XRD peak was matched from JCPDS data,With the help of JCPDS data three peaks of smectite group , two peaks of illite, and one peak of kaolite was matched.

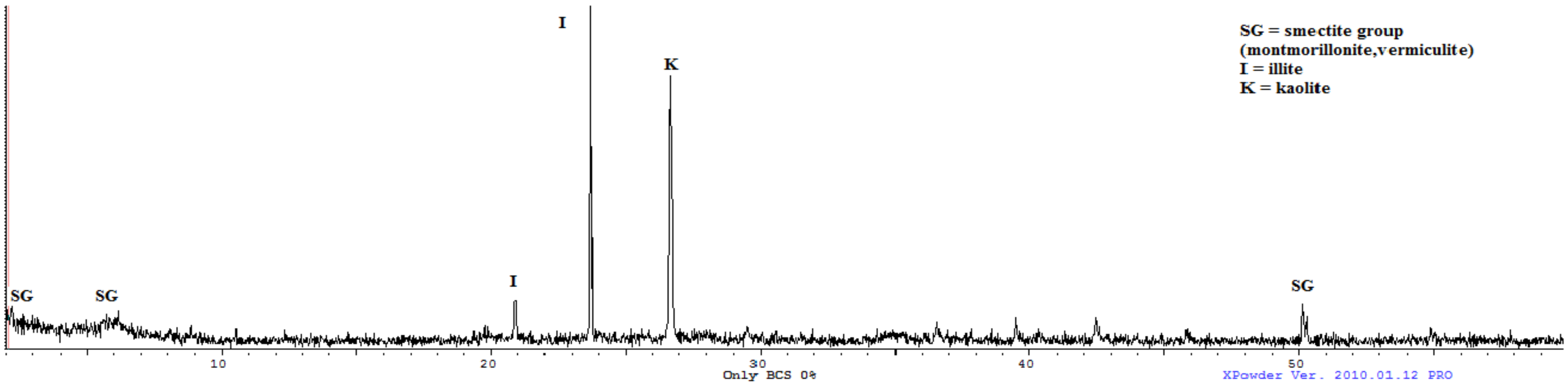


Fig 4.1: X- Ray Diffractogram of Black Cotton Soil containing Smectite, Illite and Kaolite minerals.

4.3 Scanning electronic microscopic Analysis of black cotton soil:

The smectite content was confirmed by the SEM analysis. Images of black cotton soils was taken at 500,300.100 and 50 micron Figure shows the SEM micrographic depicts thin sheets of smectite stacked on one to another, which is supported by the crenulated morphology of the specimen. The whole structure of clay appears as a thin webby crust.

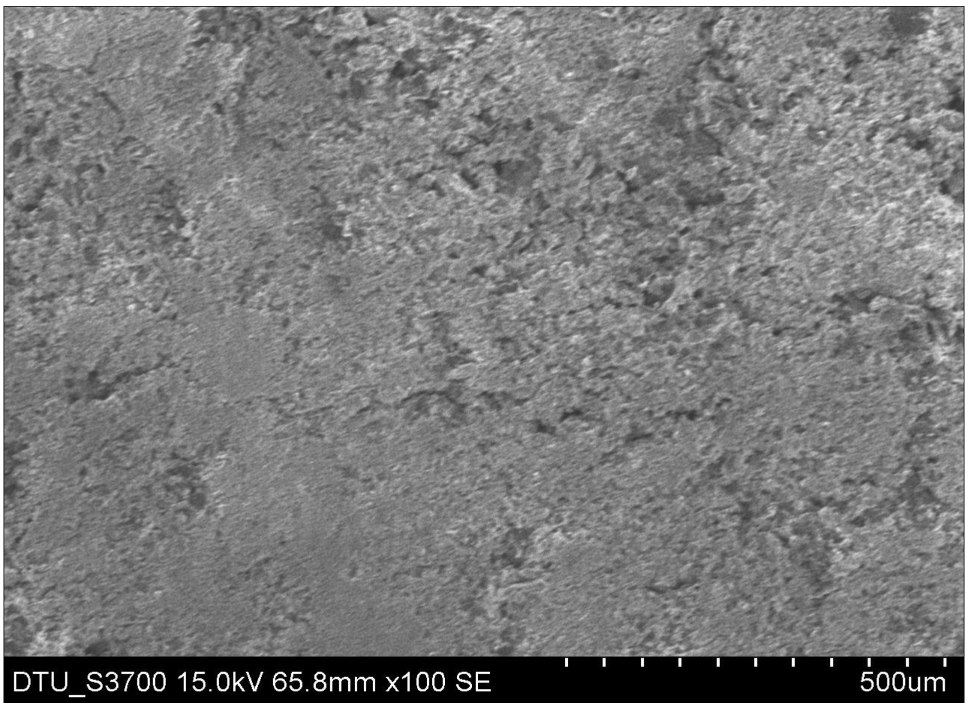


Figure.4.2 Scanning electron micrograph of smectite montmoriilonite mineral

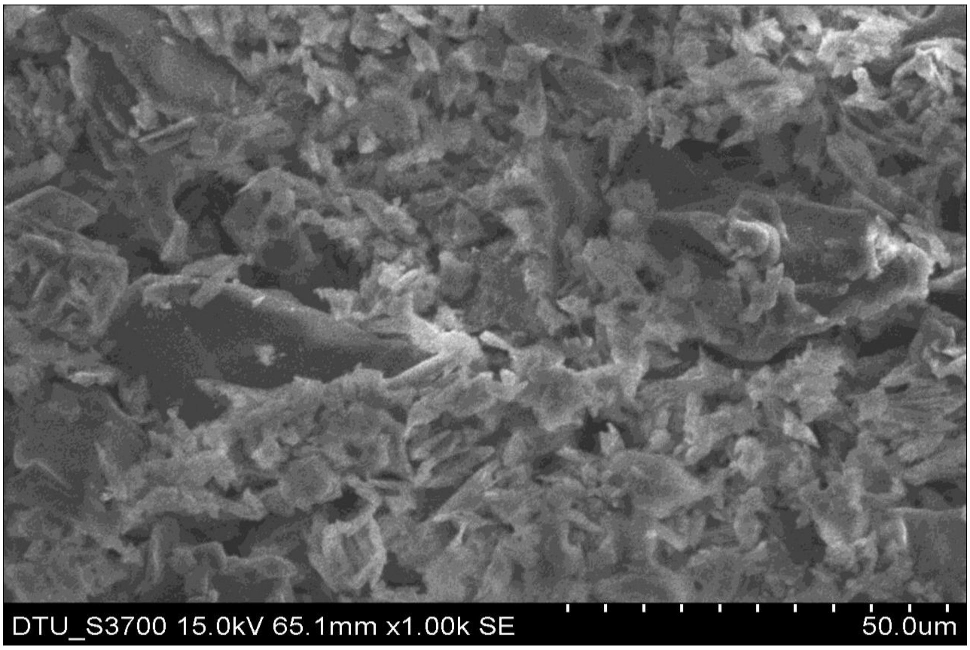


Figure.4.3 Scanning electron micrograph of smectite montmoriilonite mineral

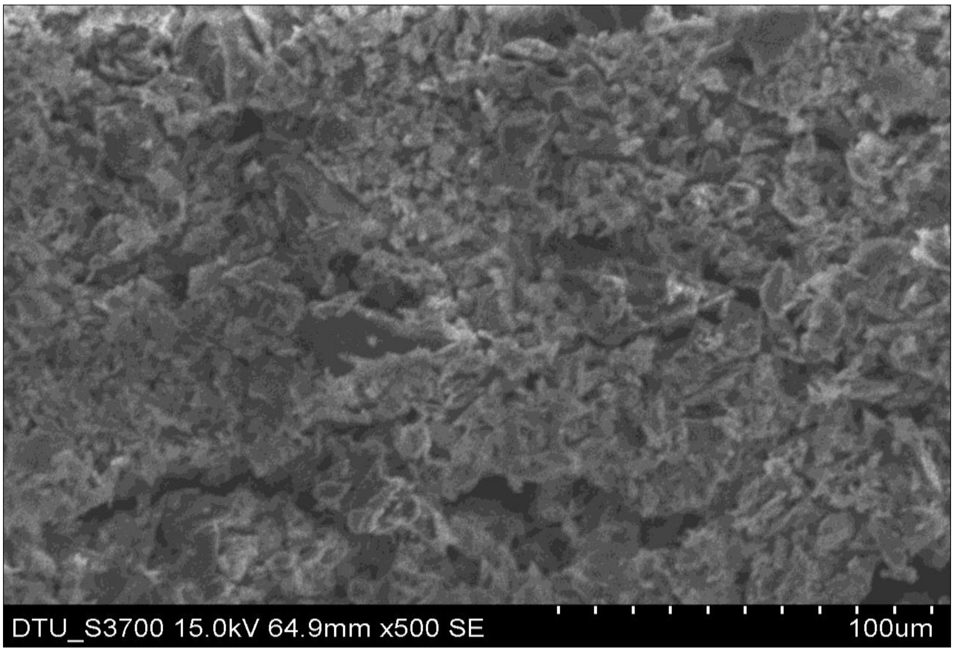


Figure.4.4 Scanning electron micrograph of smectite montmoriilonite mineral

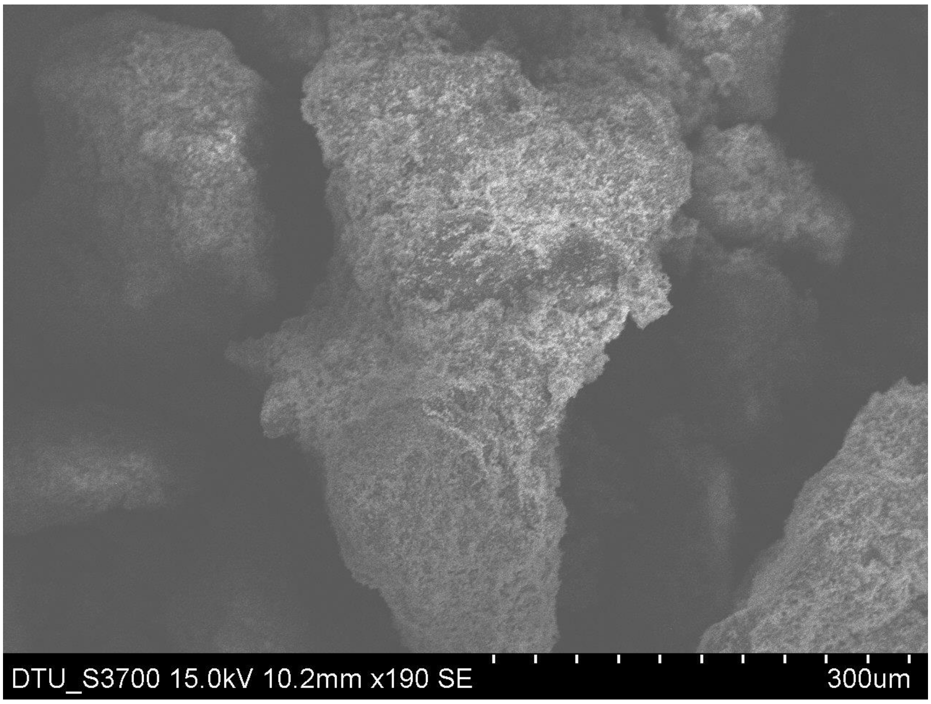


Figure.4.5 Scanning electron micrograph of smectite montmoriilonite mineral

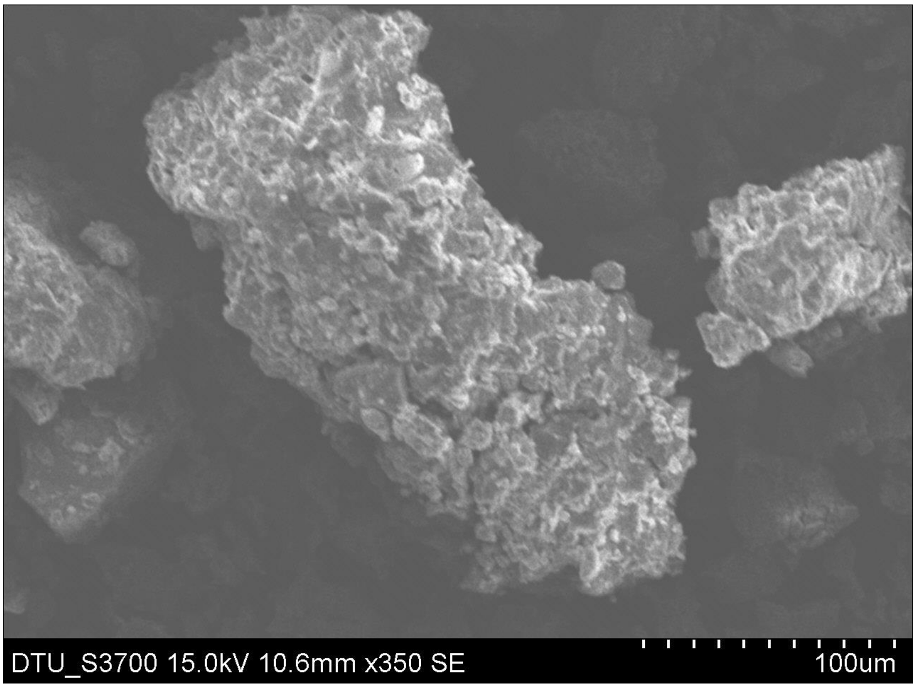


Figure.4.6 Scanning electron micrograph of smectite montmoriilonite mineral

4.4 Particle Size Distribution (Hydrometer analysis):

Clay soils are those soils which have particle size below 2 micron so by the hydrometer analysis we have seen about 40% particles are below 2 micron.

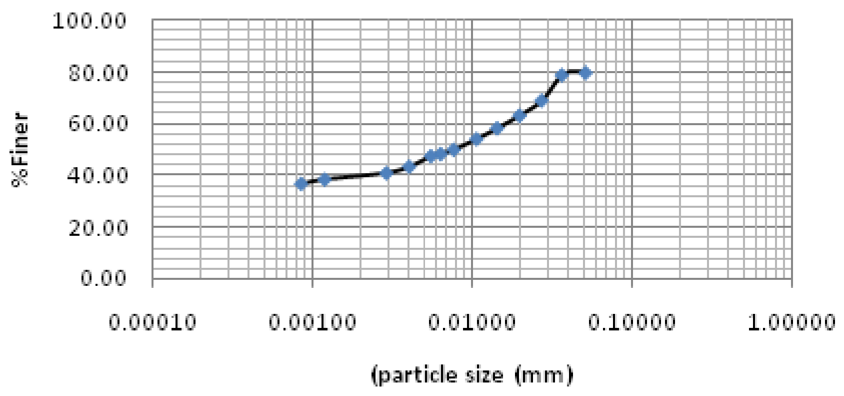


Figure.5.7 particle size distribution curve of black cotton soil

4.5 Specific gravity:

Specific gravity of black cotton soil was found by density bottle: 2.61

Figure.4.8 Density bottle

4.6 Atterbergs Limits Black Cotton Soil

Liquid Limit = 57%

Plastic limit = 30.43%

Plasticity Index = 26.57%

Plasticity of black cotton soil was found 26.57%. So according to unified soil classification system (USCS) clay is classified as inorganic clays of high plasticity (CH)

Figure. 4.8 Liquid Limit curve of Black Cotton soil

4.7 Unconfined compression strength test

Figure 4.9 shows the relation between stress and strain of black cotton soil samlple .

From the figure UCS has been found 0.1662 N/mm^2

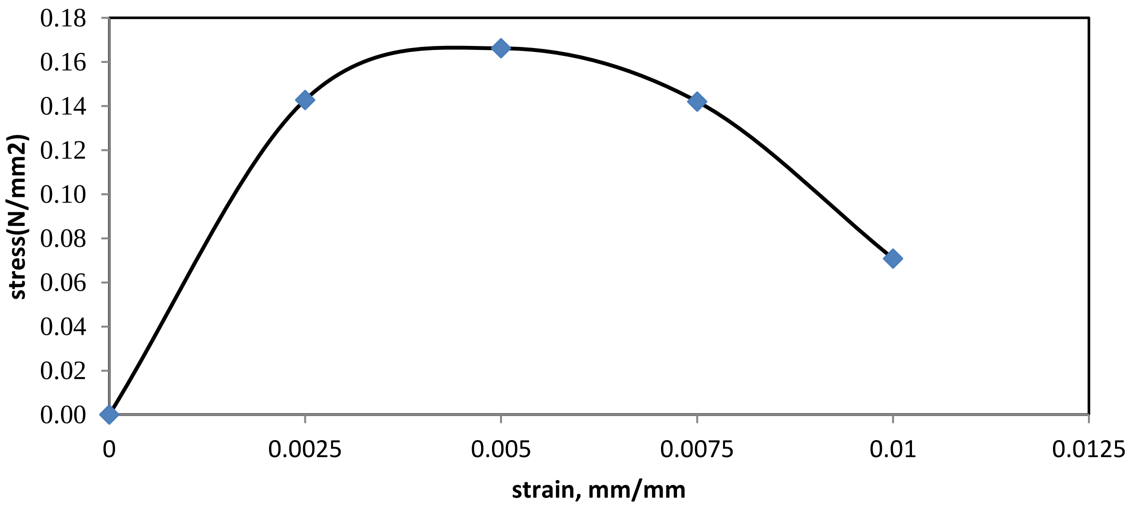


Figure.4.9 UCS of Black Cotton Soil. UCS = 0.1662 N/mm^2

4.8 Compaction Test (Standard Proctor Test)

Figure 4.10 shows the relation between water content and dry density the optimum moisture content of black cotton soil sample has been found W_0 = 18.29% and max. Dry density has been found $(\rho_d)_{max}$ = 1.6 g/cc.

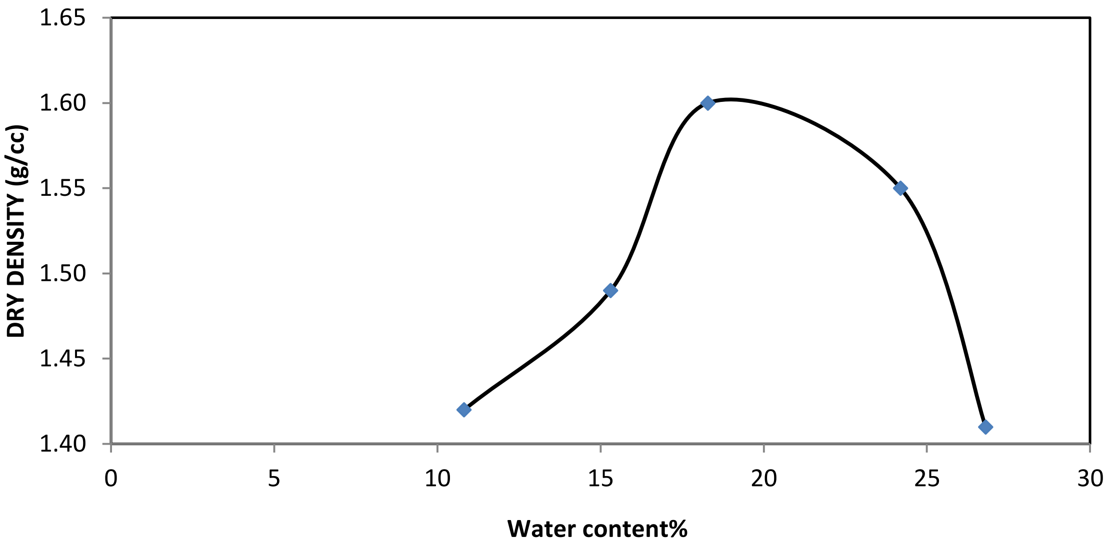


Figure.4.10 compaction curve of Black cotton Soil

4.9 Differential Free Swell Test

Differential free swell test was performed in 100 ml cylindrical jar with 10 g soil sample.

DFS of Black cotton soil = 41 %

Figure. 4.11 Differential free swell test of Black cotton soil sample

4.10 CBR test of Black cotton soil

CBR of Black cotton soil was performed at optimum mosture content, figure 4.12 shows relation between load and penetration. CBR value of black cotton soil was found 1.70 at 2.5 mm penetration .

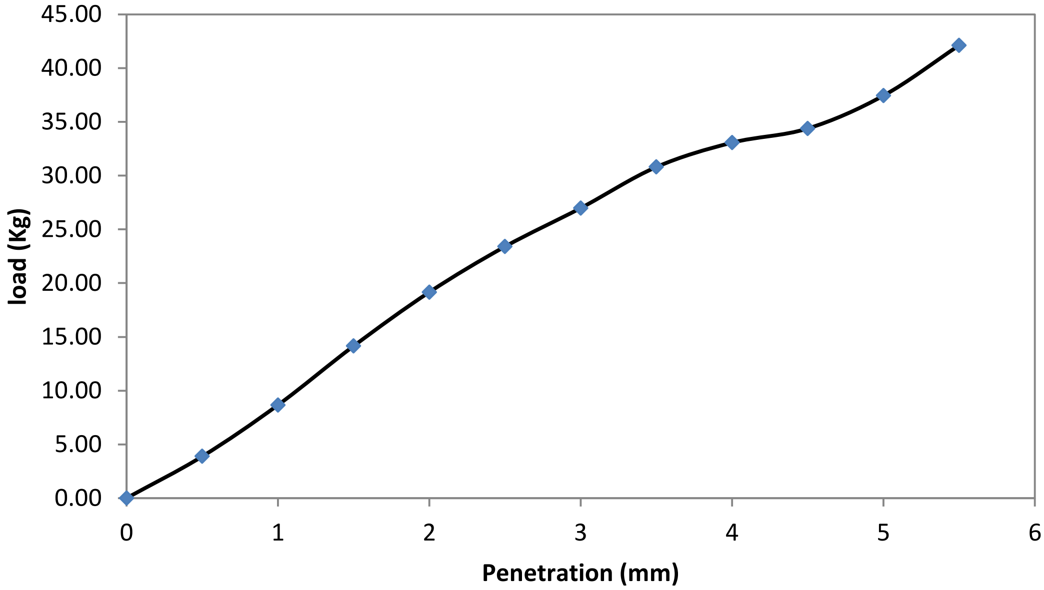


Figure 4.12 Load penetration curve of Black cotton soil

CHAPTER-5

GEO TECHNICAL PROPERTIES OF VARIOUS ADMIXTURES

5.1 Geo technical properties of lime

Add of lime to clay cause several phenomenon's in soil like; plasticity index, raising of efficiency, resistance on survival in soil. Lime will show some chemical reaction with clay that causes some cemented minerals like: calcium silicate and calcium aluminates came to existence. The quantity or ratio of those minerals increases during time; and also the resistance of soil rises.

When quicklime chemically combines with water, it can be used very effectively to dry any type of wet soil. Heat from this reaction further dries wet soils. The reaction with water occurs even if the soils do not contain significant clay fractions. When clays are present, lime’s chemical reactions with clays increase the moisture-holding capacity of the soil, which reduces free liquids and causes further drying. The net effect is that drying occurs quickly, within a matter of hours, enabling more rapid site access and soil compaction than by waiting for the soil to dry through natural evaporation. “Dry-up” of wet soil at construction sites is one of the widest uses of lime for soil treatment.

In contrast to lime modification, lime stabilization creates long-lasting changes in soil characteristics that provide structural benefits. Lime is used in stabilizing and strengthening sub grades (or sub bases) and bases below pavements. Non pavement applications for lime treatment include building foundations and embankment stabilization.

Lime stabilization chemically changes most clay soils:

1. Markedly reduces shrinkage and swell characteristics of clay soils.
2. Increases unconfined compressive strength by as much as 40 times.
3. Substantially increases load-bearing values as measured

4. Creates a water-resistant barrier. Impedes migration of surface water from above and capillary moisture from below; thus helping to maintain foundation strength

When adequate quantities of lime and water are added, the pH of the soil quickly increases to above 10.5, which enables the clay particles to break down. Silica and alumina are released and react with calcium from the lime to form calcium-silicatehydrates (CSH) and calcium-aluminate- hydrates (CAH). These compounds form the matrix that contributes to the strength of lime-stabilized soil layers. As this matrix forms, the soil is transformed from its highly expansive, undesirable natural state to a more granular, relatively impermeable material that can be compacted into a layer with significant load bearing capacity. In a properly designed system, days of mellowing and curing produce years of performance. The controlled pozzolanic reaction creates a new material that is permanent, durable, resistant to cracking, and significantly impermeable. The structural layer that forms is both strong and flexible.

5.2 Geo technical properties and characterization of stone dust:

The object of our present studies is to improve the various properties of black cotton soil by mixing locally available material, hence stone dust, which is locally and easily available material, selected to mix with black cotton soil in different proportions The admixture is collected from aman,vihar nithari road delhi . The raw material is Granite stone, which comes from leased open cast mines located nearby. The unit generally operates in one shift a day. The basic details are given below.

Type & No. of screens :Vibratory with Bucket Elevator 1 No.

Type of Raw Material : Granite stone

Table 5.1 Product size analysis

Products Size in mm	**Local name**
12-20	Aggregate
8-12	Grit
4-8	Dust

Process Flow Diagram

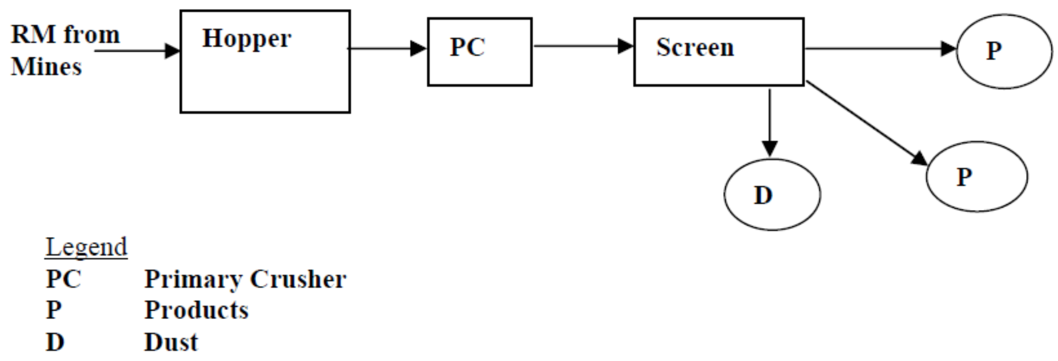


Fig 5.1 Production of stone dust

5.2.1 XRD analysis of stone dust

Mineralogy composition of stone dust sample was determined by x- ray diffraction technique (XRD). Results of XRD test showed that presence of three peaks in all samples. A typical result of a sample is shown in Figure.

In the Figure.5.2 two major peaks were obtained at 100 and 80 intensity it shows whewellite and carbonatehyroxylapatite minerals are present in stone dust. Chemical composition of whewelite is $Ca(C_20_4).(H_20)$.

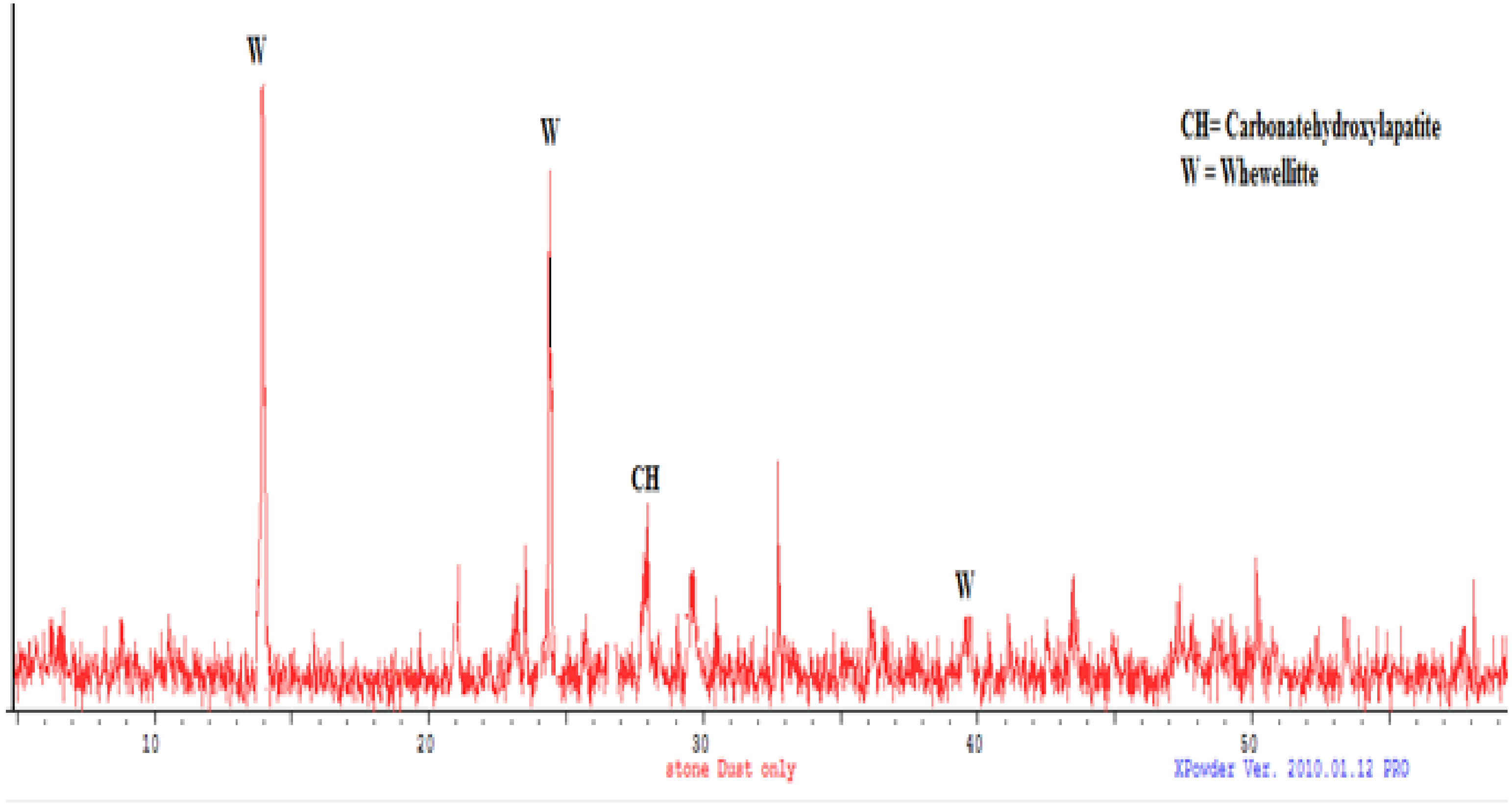


Figure. 5.2 X-RD analysis of stone dust

5.2.2 SEM analysis of stone dust

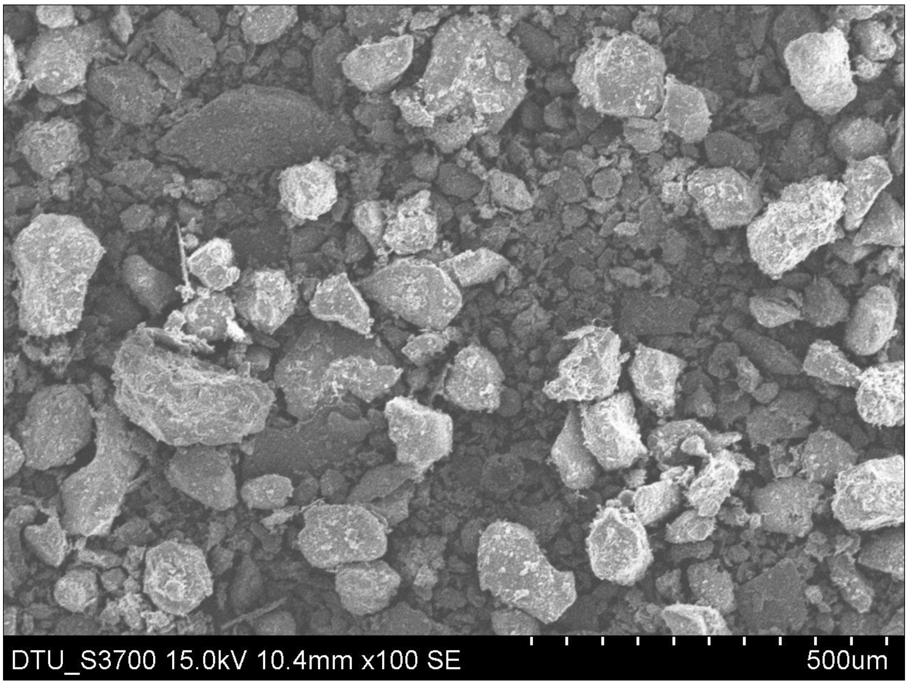


Figure.5.3 Scanning electronic microscopic of mineral whewellite and carbonatehyroxylapatite minerals

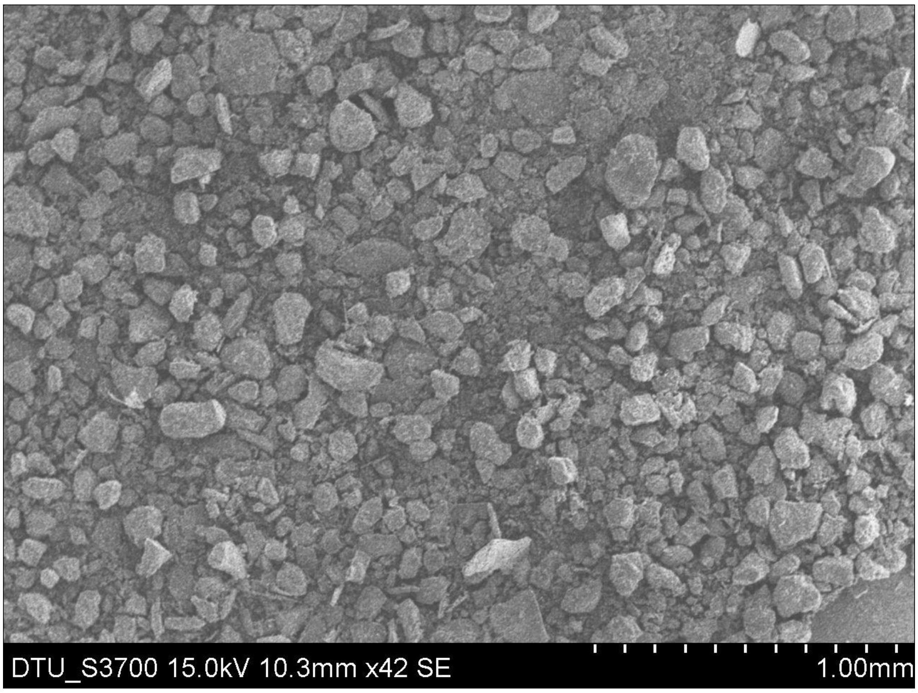


Figure.5.4Scanning electronic microscopic of mineral whewellite and carbonatehyroxylapatite minerals

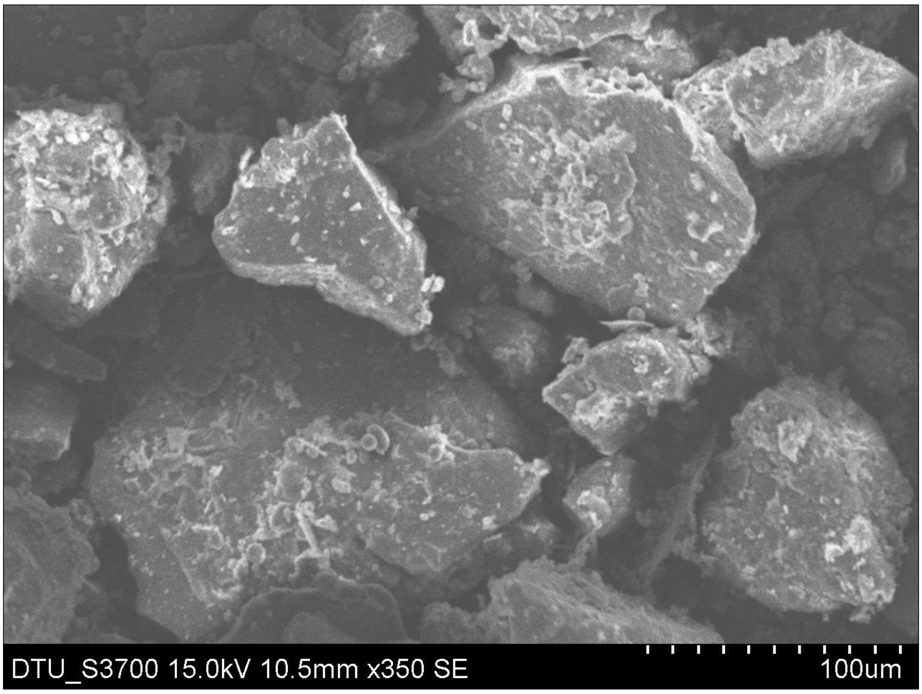


Figure.5.3 Scanning electronic microscopic of mineral whewellite and carbonatehyroxylapatite minerals

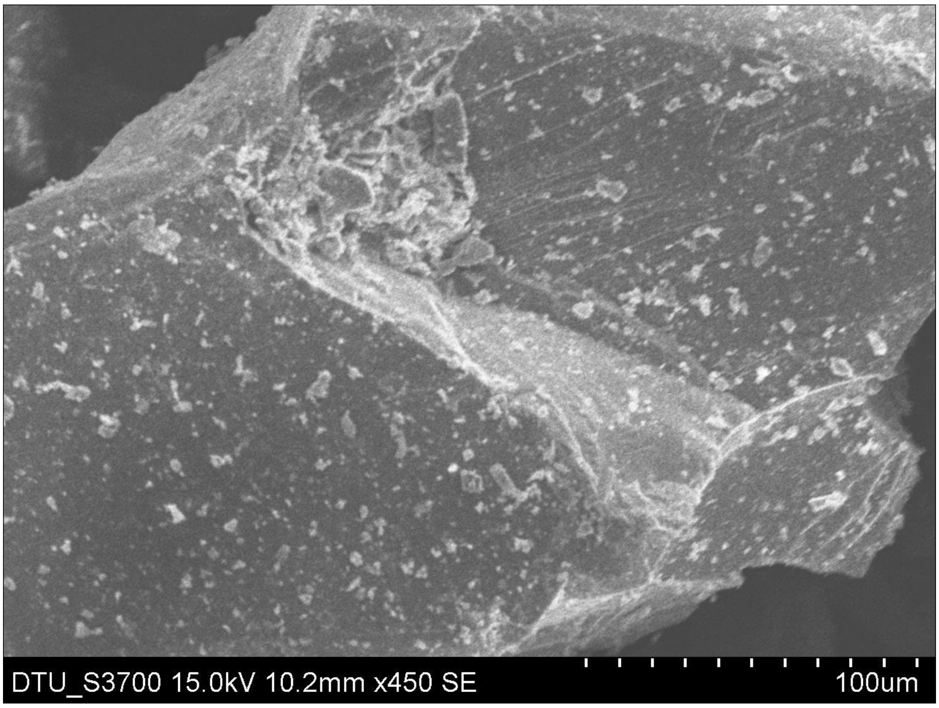


Figure.5.3 Scanning electronic microscopic of mineral whewellite and carbonatehyroxylapatite minerals

5.2.3 Grain size analysis of stone dust

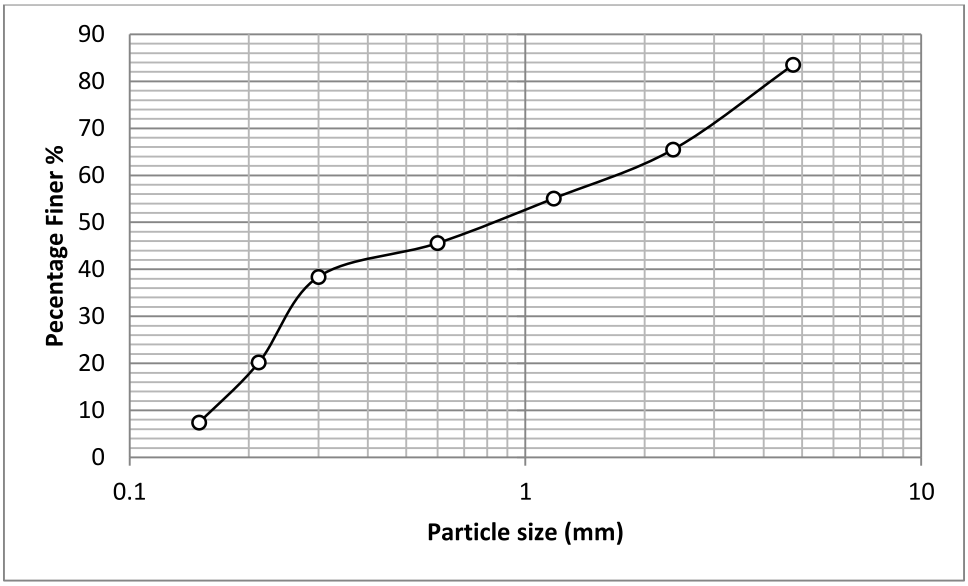


fig 5.3 Grain size distribution curve of stone dust

5.2.4 Standard Proctor test of stone dust

From the graph it has been observed that maximum dry density of stone dust was found 1.83 g/cc and optimum moisture content of stone dust was found 13.6 %.

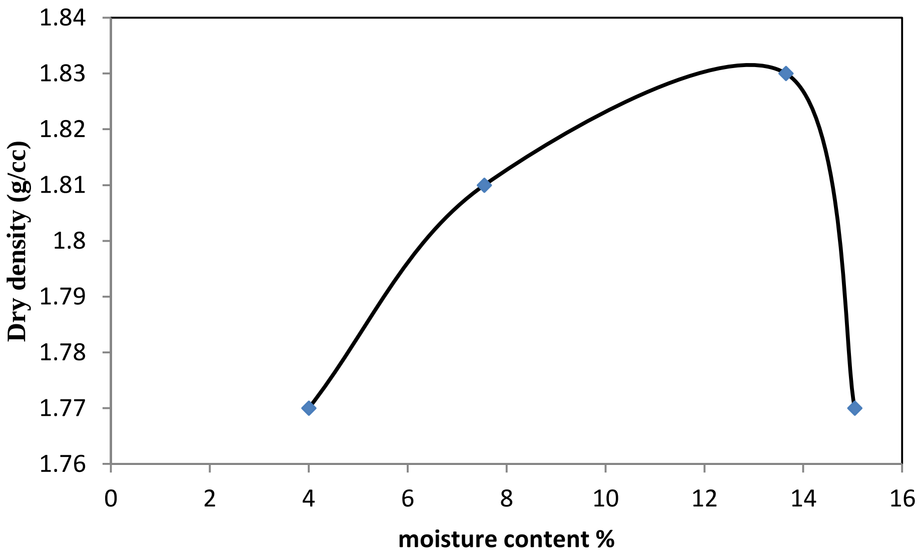


Fig 5.4 compaction curve of stone dust

5.2.5 Specific gravity of stone dust: The specific gravity of stone dust was found by density bottle. It was found 2.75

CHAPTER -6

EXPERIMENTAL RESULTS AND DESCRIPTIONS

6.1 Lime stabilization:

Lime stabilization helps in increasing the strength, durability and also minimizes the moisture variations in the soil. Lime must be well compacted for obtaining sufficient strength and durability by maintaining OMC and the same assumption is made in the experimental determination of the required lime proportion. Quality of lime to be added depends upon the specific surface area of soil particles.

Therefore we have mixed lime with soil in 3%, 6%, 9%, and 12% by weight of soil and investigated the change in engineering properties of black cotton soil.

In this stabilization is due to reduction in plasticity and formation of matrix enclosing small clay lumps. By increasing percentage of lime there is increase in strength and decrease in volume change, moisture and plasticity. In lime and stabilization we have taken another black cotton soil samples which have following properties.

Liquid Limit = 48%

Plastic Limit = 25.3%

Differential Free Swelling index=32%

Maximum Dry Density= 1.63 g/cc

Optimum moisture content = 19.4%

CBR= 1.9

Fig 6.1 Mixing of Black cotton soil with lime

6.1.1 Atterbergs Limits:

6.1.1.1 Black Cotton Soil+ 3 % Lime

Liquid limit = 46%
Plastic limit = 30.43%
Plasticity Index = 15.57%

It has been observed that by addition of 3% lime in Black cotton soil, rate in decrease in liquid limit was found 4.16 % and rate in increasing in plastic limit was found 21%.

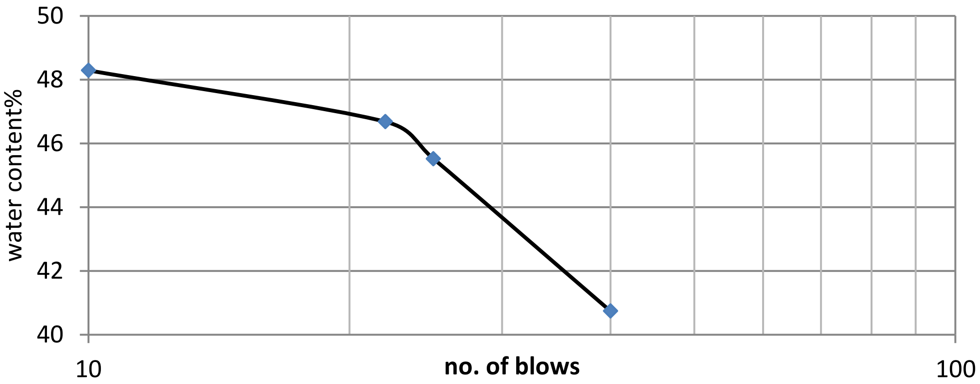


Fig 6.2 Liquid limit curve of Black Cotton Soil+ 3 % Lime

6.1.1.2 Black Cotton Soil+ 6 % Lime

Liquid Limit = 42%

Plastic limit = 32.47%

Plasticity index = 9.53%

It has been observed that by addition of 6% lime in Black cotton soil, rate of decrease in liquid limit was found 12.5 % and rate in increasing in plastic limit was found 20%.

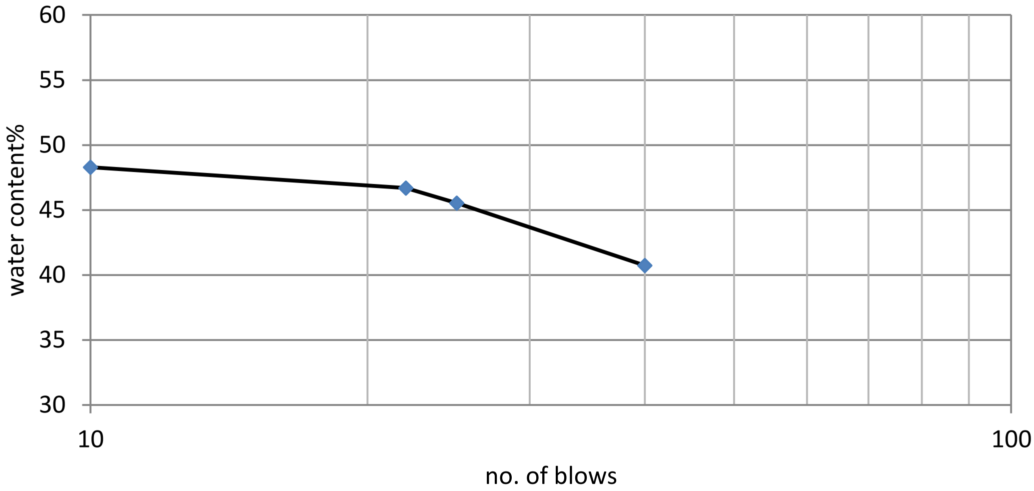


Fig 6.3 Liquid limit test Black Cotton Soil+ 6 % Lime

6.1.1.3 Black Cotton Soil+ 9 % Lime

Liquid Limit = 38.5%

Plastic limit = 32.45%

Plasticity Index = 6.05%

It has been observed that by addition of 6% lime in Black cotton soil rate, in decreasing in percentage of liquid limit was found 12.5 % and rate in increasing in plastic limit was found 20%.

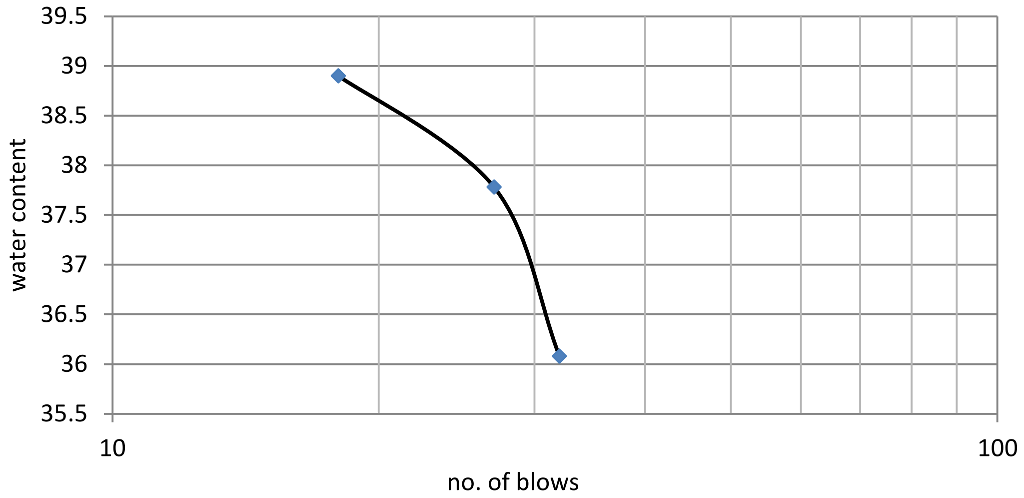


Fig 6.4 Liquid limit test Black Cotton Soil+ 9 % Lime

6.1.1.4 Black Cotton Soil+ 12% Lime

Liquid Limit = 46%

Plastic limit = 38.08%

Plasticity Index = 7.92%

It has been observed that by addition of 12% lime in Black cotton soil, rate in decreasing in percentage of liquid limit was found 4.16 % so rate in decreasing in liquid limit is decreased. and rate in decreasing in plastic limit was found 52.32%.

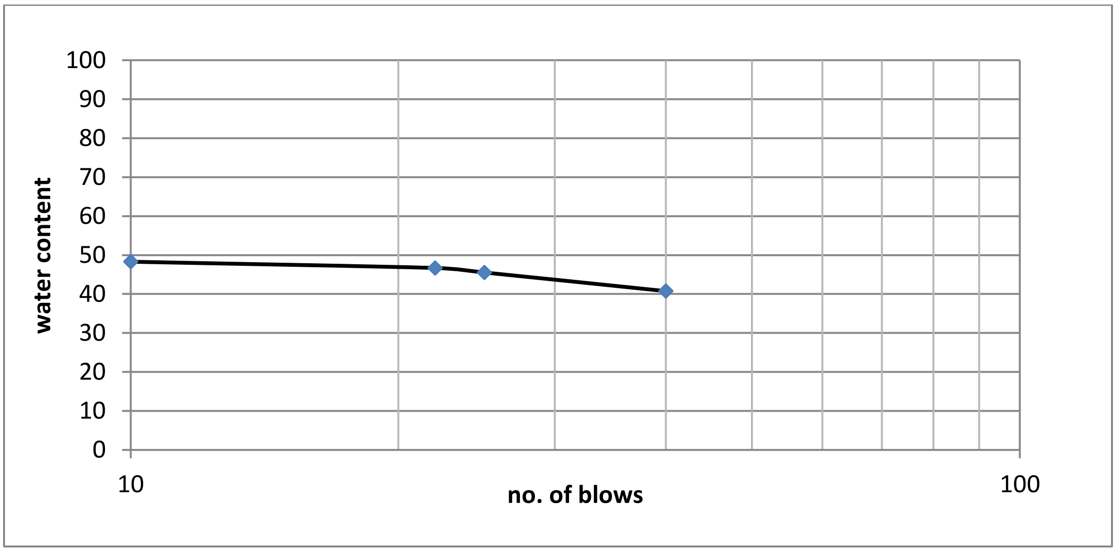


Fig 6.5 Liquid limit test

6.1.2. Differential Free Swell Test:

6.1.2.1 Black cotton soil +3% lime

Differential free swelling of soil sample was found 3.42% by addition of 3 % lime and it has been observed that rate of decreasing in percentage in Differential free swelling was found 81.6 %.

6.1.2.2 Black cotton soil +6% lime

Differential free swelling of soil sample was found 3.38% by addition of 6 % lime and it has been observed that rate of decreasing in percentage in Differential free swelling was found 89.4 %.

6.1.2.3 Black cotton soil +9% lime

Differential free swelling of soil sample was found 8.76% by addition of 9 % lime and it has been observed that rate of decreasing in percentage in Differential free swelling was found 72.62 %.

6.1.2.4Black cotton soil +12% lime DFS= 7.14%

Differential free swelling of soil sample was found 7.14% by addition of 12 % lime and it has been observed that rate of decreasing in percentage in Differential free swelling was found 77.68 %.

6.1.3 Compaction Test (Standard Proctor Test):

6.1.3.1 Black cotton soil+ 3% Lime

From the graph it has been observed that maximum dry density of Black cotton soil was increased by 1.84% and optimum moisture content of Black cotton soil was decreased by 2.06% by addition of 3% lime.

Optimum moisture content W = 19%

Max. Dry density $(\rho_d)_{max}$ = 1.66 g/cc

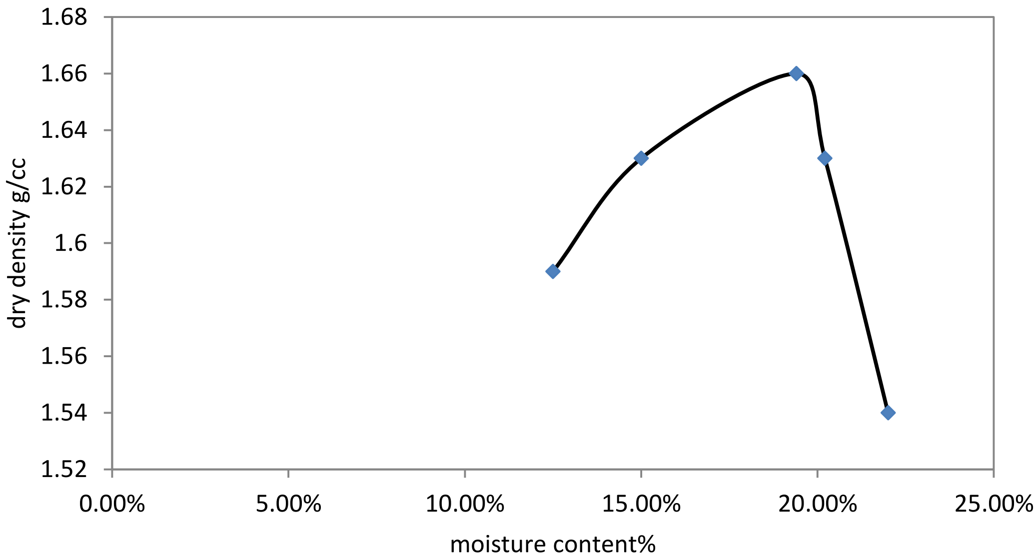


Fig 6.6 compaction curve of Black cotton soil+ 3% Lime

6.1.3.2 Black cotton soil + 6% Lime

From the graph it has been observed that maximum dry density of Black cotton soil was increased by 0.61%% and optimum moisture content of Black cotton soil was decreased by 12.37% by addition of 6% lime. So it was observed that percent increment of maximum dry density of black cotton soil is decreased

Optimum moisture content W = 17%

Max. Dry density $(\rho_d)_{max}$ = 1.64 g/c

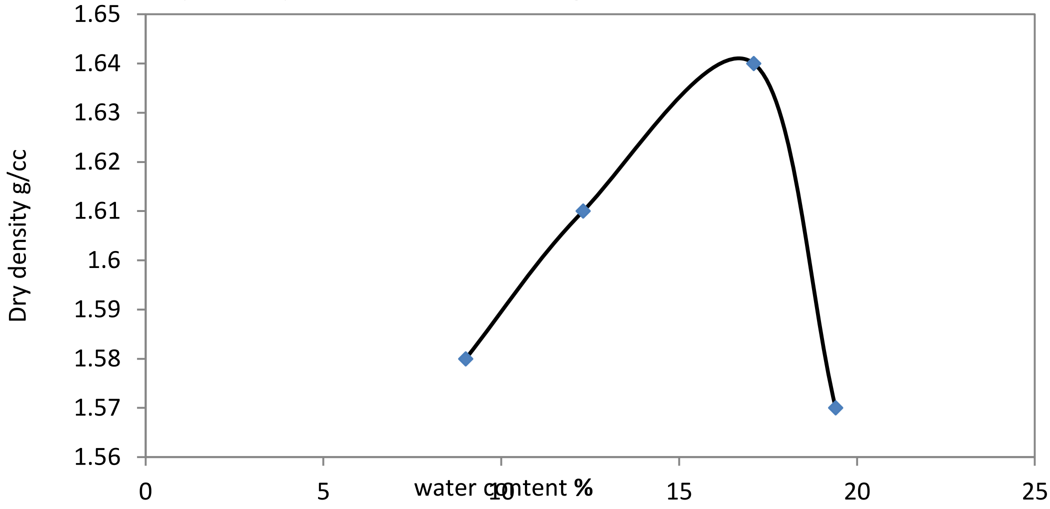


Fig 6.7 compaction curve of Black cotton soil+ 6% Lime

6.1.3.3 Black cotton soil + 9% Lime

From the graph it has been observed that maximum dry density of Black cotton soil was increased by 1.84% and optimum moisture content of Black cotton soil was decreased by 25.25% by addition of 9% lime. So it was observed that percent increment of maximum dry density of black cotton soil is decreased.

Moisture content W_0 = 14.5%

. Dry density $(\rho_d)_{max}$ = 1.66g/cc

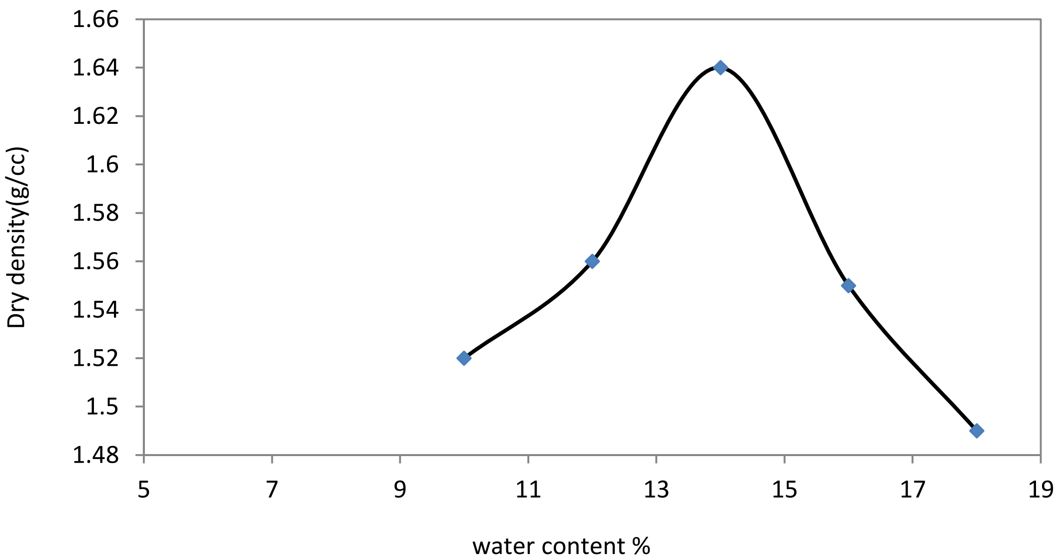


Fig 6.8 compaction curve of Black cotton soil+ 9% Lime

6.1.3.4 Black cotton soil + 12% Lime

Optimum moisture content W_0 = 18%

Max. Dry density $(\rho_d)_{max}$ = 1.60 g/cc

From the graph it has been observed that maximum dry density of Black cotton soil was decreased by 6% and optimum moisture content of Black cotton soil was decreased by 7.2%% by addition of 12% lime.

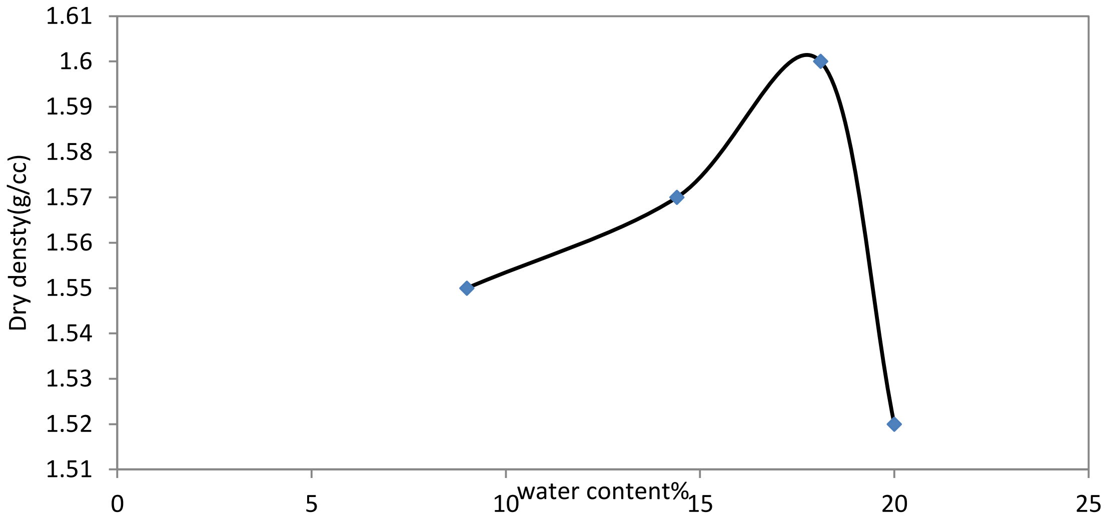


Fig 6.9 compaction curve of Black cotton soil+ 12% Lime

6.1.4 California Bearing Ratio:

6.1.4.1 Black Cotton Soil With 3% Lime

OMC= 19 %, MDD= 1.66 g/cc

CBR Value at 2.5mm= 11.2

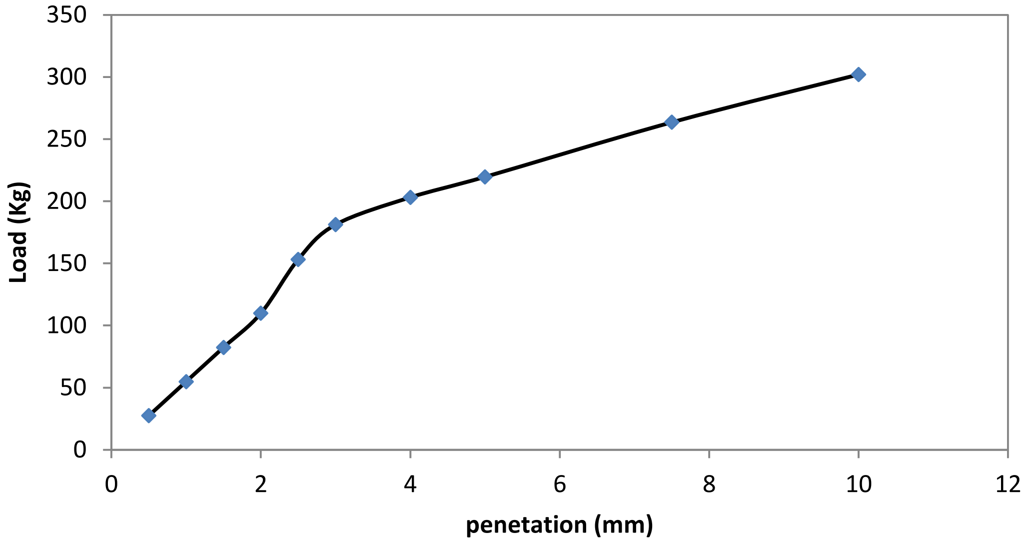


Fig 6.10 CBR test Load penetration curve of Black Cotton Soil With 3% Lime

6.1.4.2 Black Cotton Soil With 6% Lime

OMC= 17%, MDD= 1.64 g/cc

CBR Value at 2.5mm= 15.2

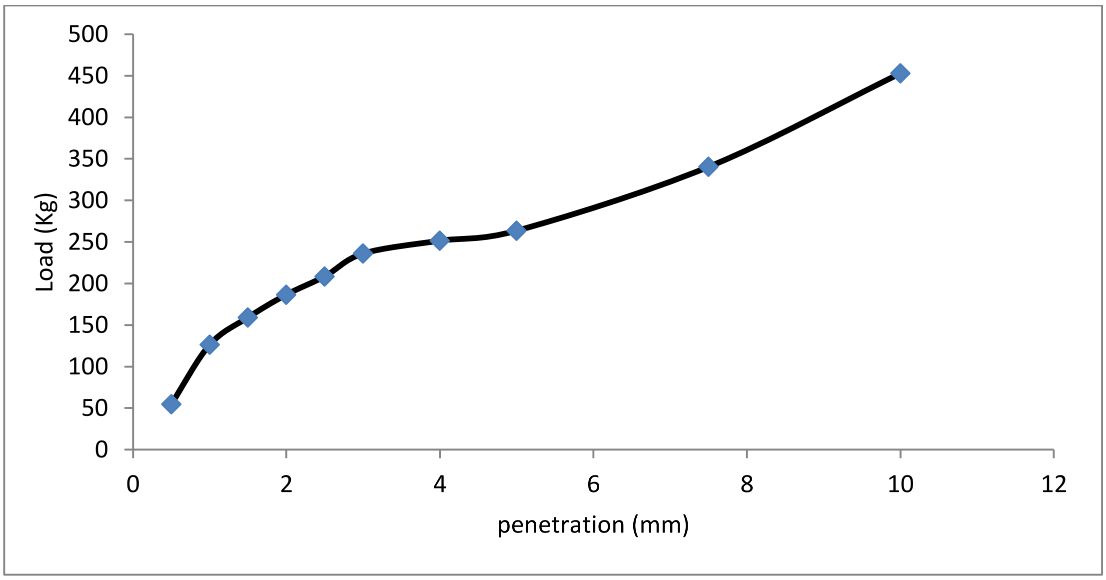


Fig 6.11 Load penetration curve of Black Cotton Soil With 6% Lime

6.2 Stabilization of Black cotton soil with Stone Dust:

6.2.1 Atterberg's Limits

6.2.1.1 Black Cotton soil + 5% Stone Dust

Liquid limit= 45.4 %

Plastic Limit = 24 %

Liquid limit of Black cotton soil was decreased with 5.41% and plastic limit was decreased with 12.1%by addition of 5% stone dust.

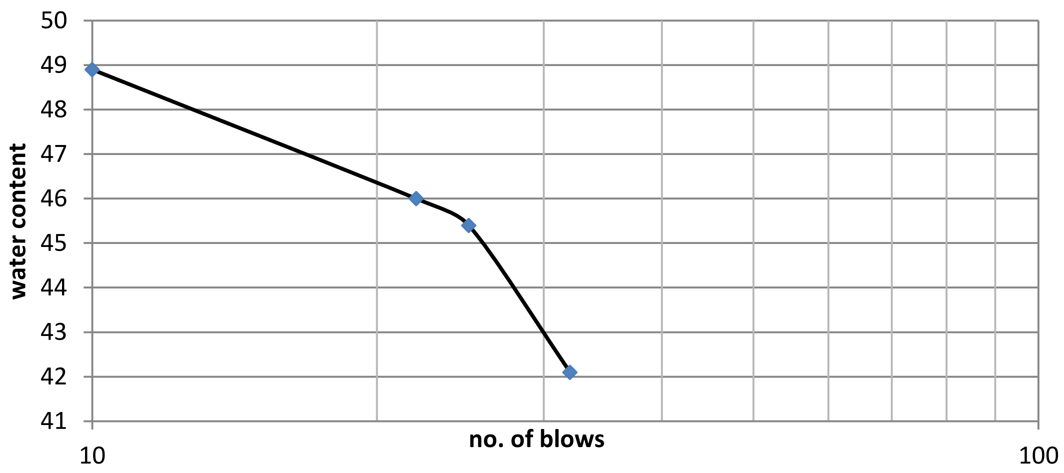


Figure. 6.12 Liquid limit test Cotton soil + 5% Stone Dust

6.2.1.2 Black cotton soil +10% Stone Dust

Liquid limit= 43%

Plastic Limit=23%

Liquid limit of Black cotton soil was decreased with 10.41% and plastic limit was decreased with 17.85%by addition of 10% stone dust.

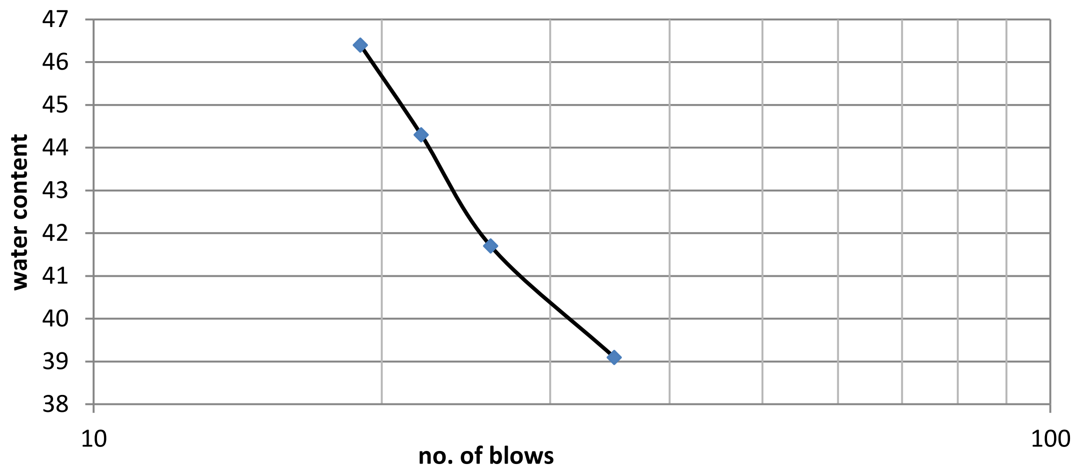


Figure. 6.13 Liquid limit test Cotton soil + 10% Stone Dust

6.2.2 Compaction Test (Standard Proctor Test)

6.2.2.1 Black Cotton soil + 5% Stone Dust

Optimum moisture content W_0 = 18 %

MDD= 1.66 g/cc

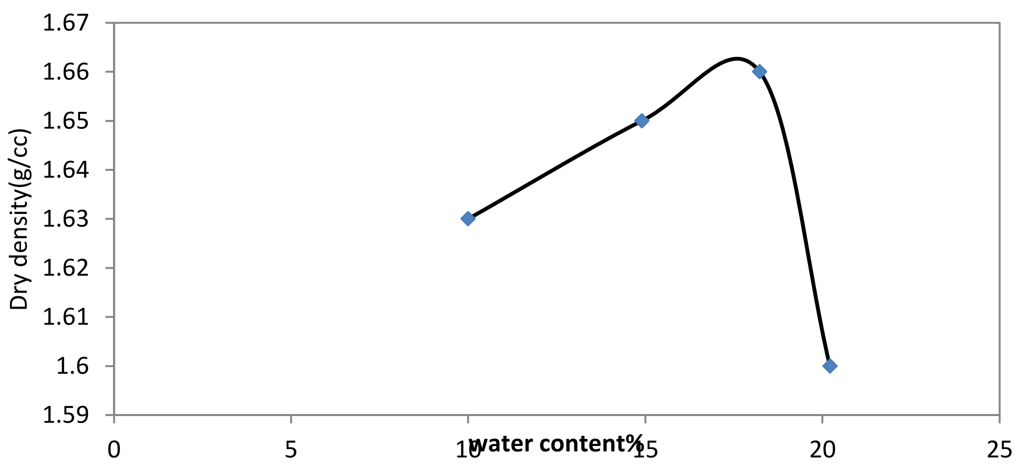


Fig 6.14 compaction curve of Black Cotton soil + 5% Stone Dust

6.2.2.2 Black cotton soil +10% Stone Dust:

Optimum moisture content W = 11.4 %

MDD $(\rho_d)_{max}$ = 1.72 g/cc

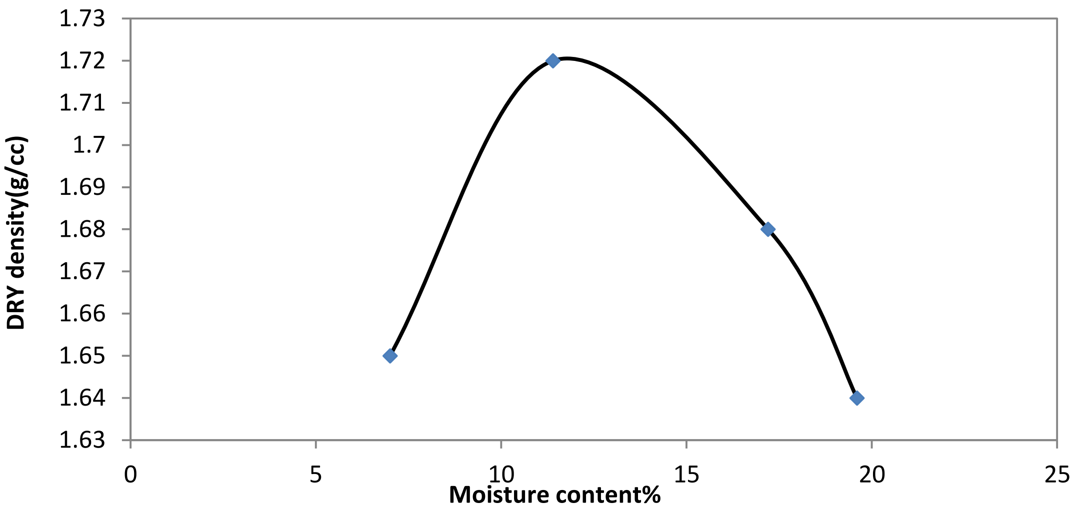


Figure 6.15 compaction curve of Black Cotton soil + 10% Stone Dust

6.3 Stabilization of black cotton soil with stone dust and lime

It has been seen in lime stabilization maximum strengths has been found at 3,6and 9% lime.

And it has been also seen in stone dust stabilization that maximum strength was found at 5 and 10% stone dust.

Course of plan:

Following proportions of stone dust & lime are to be used for all the tests.

1. 3% lime with 5% stone dust
2. 3% lime with 10% stone dust
3. 6% lime with 5% stone dust
4. 6% lime with 10% stone dust
5. 6% lime with 15% stone dust
6. 6% lime with 20% stone dust
7. 6% lime with 25% stone dust
8. 9% lime with 5% stone dust
9. 9% lime with 10% stone dust
10. 9% lime with 15% stone dust
11. 9% lime with 20% stone dust
12. 9% lime with2 5% stone dust

6.3.1 Atterbergs limits:

6.3.1.1 Black cotton soil with 3% lime + 5% stone dust

Liquid limit= 50%

Plastic limit= 32.92

Plasticity index= 17.01

Liquid limit of Black cotton soil was decreased with 12.28%, and plastic limit of black cotton soil was increased with 9.73%¸by adding 3% lime + 5% stone dust.

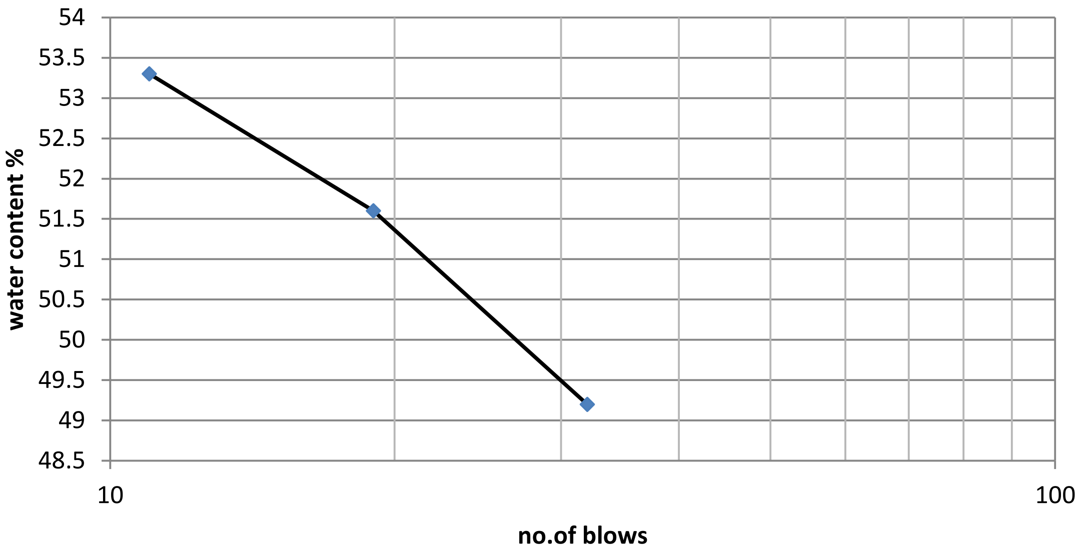


Figure 6.16Liquid limit test3% lime + 5% stone dust

6.3.1.2 Black cotton soil with 3% lime + 10% stone dust

Liquid limit= 47%

Plastic limit= 32.27

Plasticity index= 14.73

Liquid limit of Black cotton soil was decreased with 17.54%, and plastic limit of black cotton soil was increased with 7.56%¸by adding 3% lime + 10% stone dust.

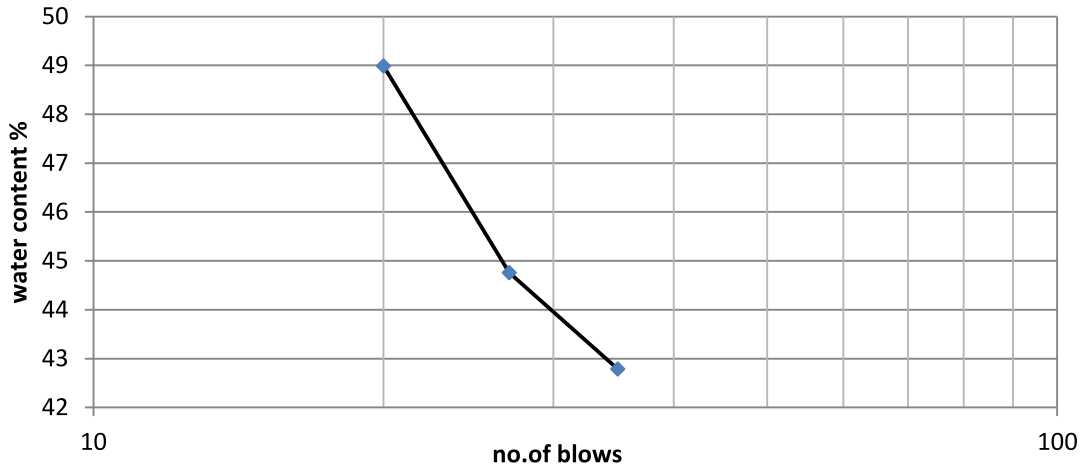


Fig 6.17 Liquid limit test

6.3.1.3 Black cotton soil with 6% lime + 5% stone dust

Liquid limit= 46 %

Plastic limit= 31.51

Plasticity index= 14.49

It has been found that rate of decrease in liquid limit was 19.29%, and plastic limit of black cotton soil was increased with 5.03%¸by adding 6% lime + 5% stone dust.

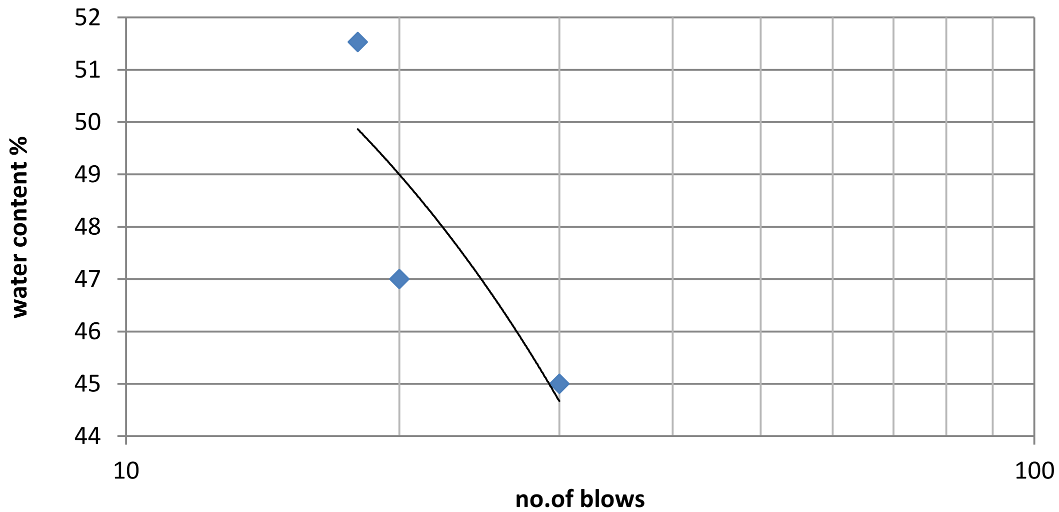


Figure 6.18 Liquid limit test 6% lime + 5% stone dust

6.3.1.4Black cotton soil with 6% lime + 10% stone dust

Liquid limit= 42 %

Plastic limit= 31.68

Plasticity index= 14.46

It has been found that rate of decrease in liquid limit was 26.31%, and plastic limit of black cotton soil was increased with 5.6%¸by adding 6% lime + 10% stone dust.

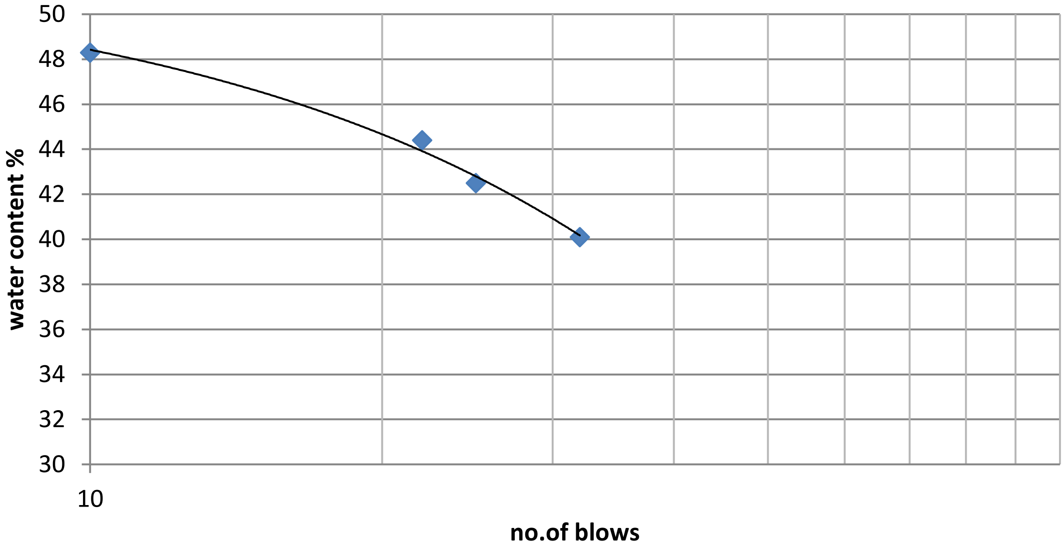


Figure 6.19 Liquid limit test 6% lime + 10% stone dust

6.3.1.5 Black cotton soil with 6% lime + 15% stone dust

Liquid limit= 40 %

Plastic limit= 29.40

Plasticity index= 10.9

Liquid limit of Black cotton soil was decreased with rate 29.82%, and plastic limit of black cotton soil was decreased with rate 2%¸by adding 6% lime + 15% stone dust.

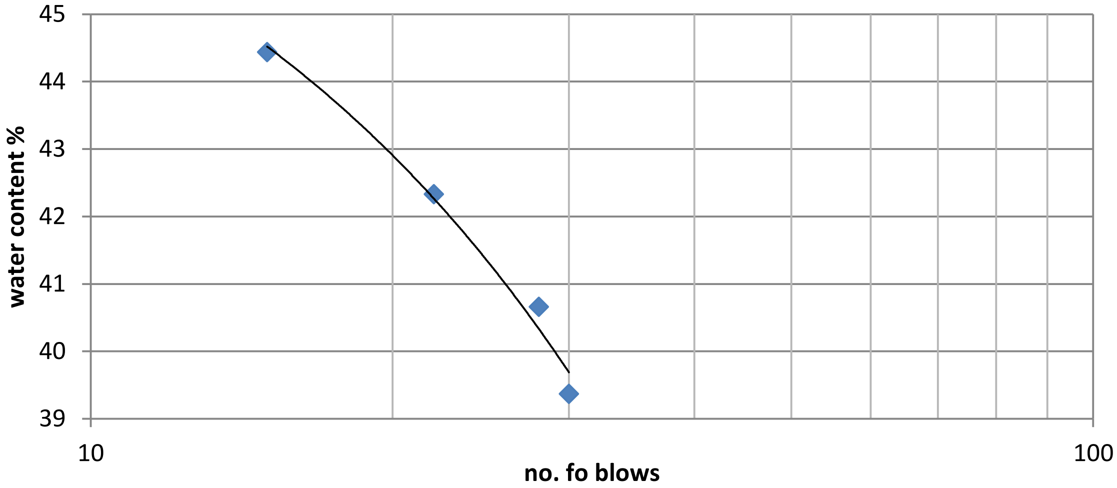


Figure 6.20 Liquid limit test6% lime + 15% stone dust

6.3.1.6 Black cotton soil with 6% lime + 20% stone dust

Liquid limit= 41 %

Plastic limit= NP

Plasticity index= NP

Liquid limit of Black cotton soil was decreased with 28.08%, and Black cotton soil was found non plastic by adding 6% lime + 20% stone dust.

Figure 6.21 Liquid limit test 6% lime + 20% stone dust

6.3.1.7 Black cotton soil with 6% lime + 25% stone dust

Liquid limit = 37.5%

Plastic limit= NP

Plasticity index= NP

Liquid limit of Black cotton soil was decreased with 34.21%, and Black cotton soil was found non plastic because quantity of silt size was increased in Black cotton soil by adding 6% lime + 25% stone dust.

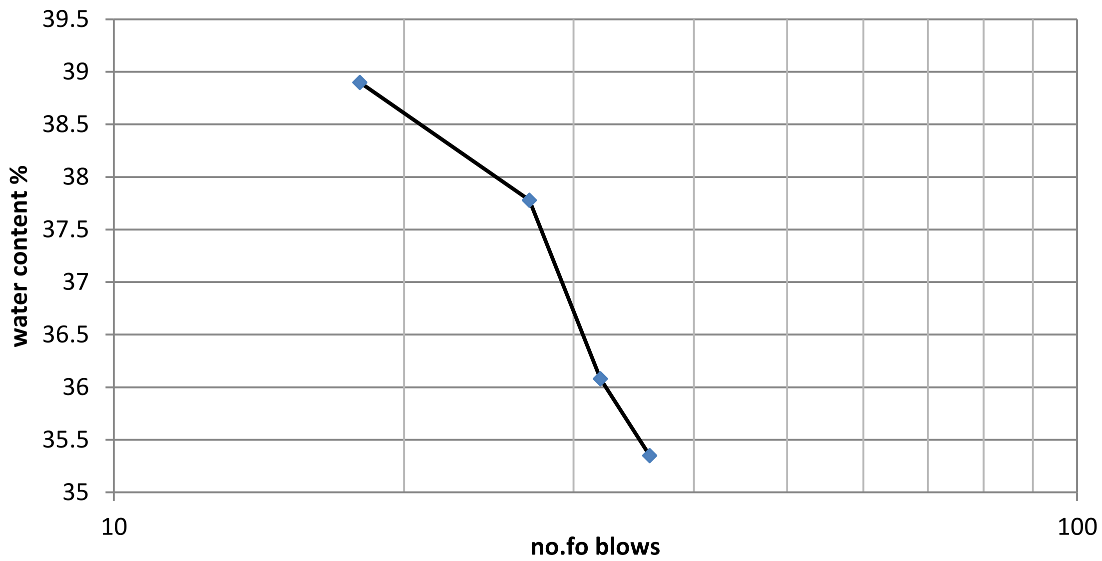


Figure 6.22Liquid limit test 6% lime + 25% stone dust

6.3.1.8 Black cotton soil with 9% lime + 5% stone dust

Liquid limit= 48 %

Plastic limit= 25.96%

Plasticity index= 22.04

Liquid limit of Black cotton soil was decreased with rate 15.78%,and plastic limit of Black cotton soil was decreased with rate 15.56% by adding 9% lime + 5% stone dust.

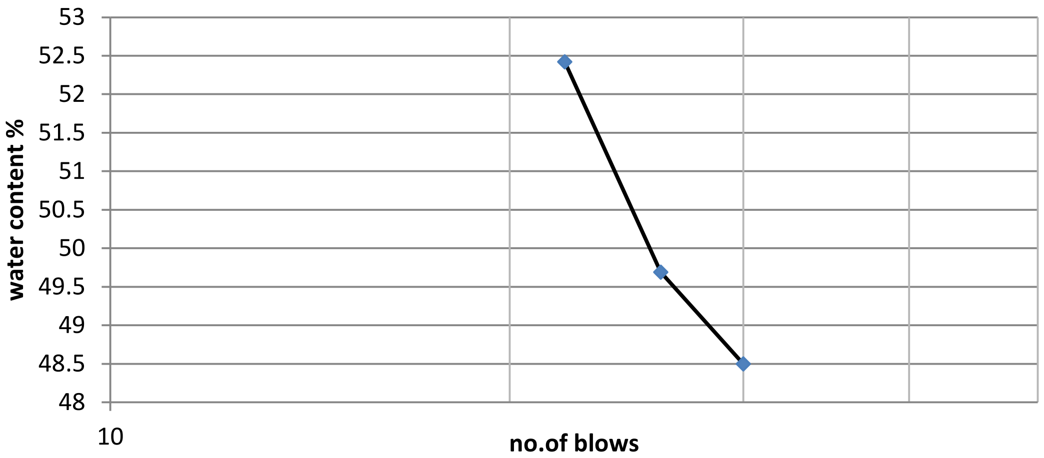


Figure 6.23 Liquid limit test9% lime + 5% stone dust

6.3.1.9 Black cotton soil with 9% lime + 10% stone dust

Liquid limit= 43.2

Plastic limit= NP

Plasticity index= NP

Liquid limit of Black cotton soil was decreased with rate 24.21%,and Black cotton soil was found non plastic by adding 9% lime + 10% stone dust.

Figure. 6.24 Liquid limit test 9% lime + 10% stone dust

6.3.1.10 Black cotton soil with 9% lime + 15% stone dust

Liquid limit= 39 %

Plastic limit= NP

Plasticity index= NP

Liquid limit of Black cotton soil was decreased with 31.57%, and Black cotton soil was found non plastic by adding 9% lime + 15% stone dust.

Figure. 6.25 Liquid limit test 9% lime + 15% stone dust

6.3.1.11 Black cotton soil with 9% lime + 20% stone dust

Liquid limit= 41%

Plastic limit= NP

Plasticity index= NP

Liquid limit of Black cotton soil was decreased with 28%,and Black cotton soil was found non plastic by adding 9% lime + 15% stone dust.

Figure 6.26 Liquid limit test

6.3.1.12 Black cotton soil with 9% lime + 25% stone dust

Liquid limit= 39%

Plastic limit= NP

Plasticity index= NP

Liquid limit of Black cotton soil was decreased with 31.57%,and Black cotton soil was found non plastic by adding 9% lime + 25% stone dust.

Figure 6.27 Liquid limit test

6.3.2 Compaction test (standard proctor test)

6.3.2.1 Black cotton soil + 3% lime + 5% stone dust

Optimum moisture content W = 16.58 %, $(\rho_d)_{max}$ = 1.63 g/cc

From the graph it has been observed that maximum dry density of Black cotton soil was increased with 1.87% and optimum moisture content of Black cotton soil was decreased by 9.17% by addition of 3% lime + 5% stone dust. Maximum dry density of Black cotton soil was increased because a new bond ettringite was created with addition of lime in Black cotton soil. Optimum moisture content of Black cotton soil was decreased because

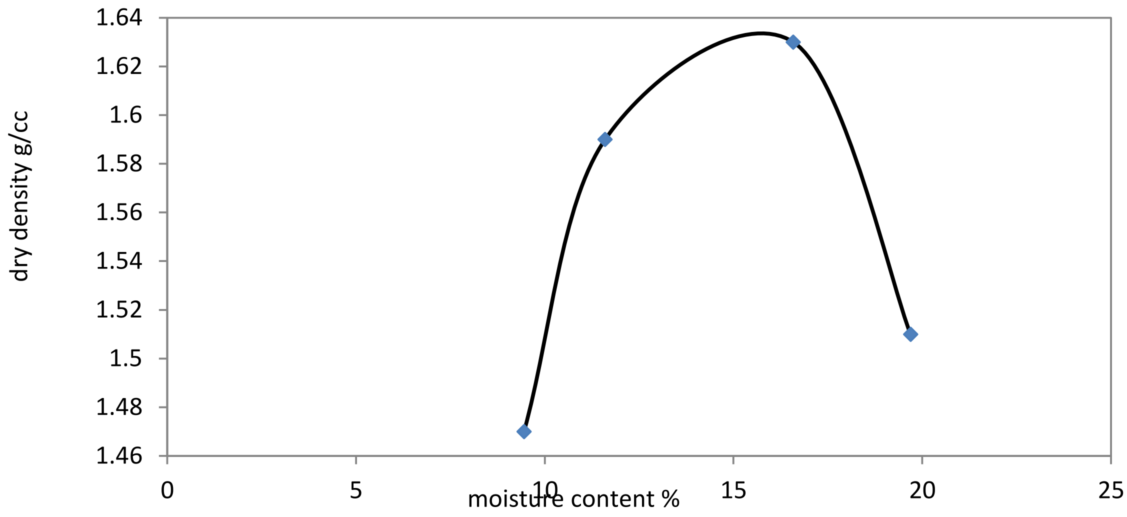


Figure 6.28 compaction curve of Black cotton soil + 3% lime + 5% stone dust

6.3.2.2 Black cotton soil + 3% lime + 10% stone dust

Maximum dry density of Black cotton soil was increased with rate 1.87% and optimum moisture content of Black cotton soil was decreased with rate 12.95% by addition of 3% lime + 10% stone dust.

Optimum moisture content W = 15.92 %

Max. Dry density $(\rho_d)_{max}$ = 1.63 g/cc

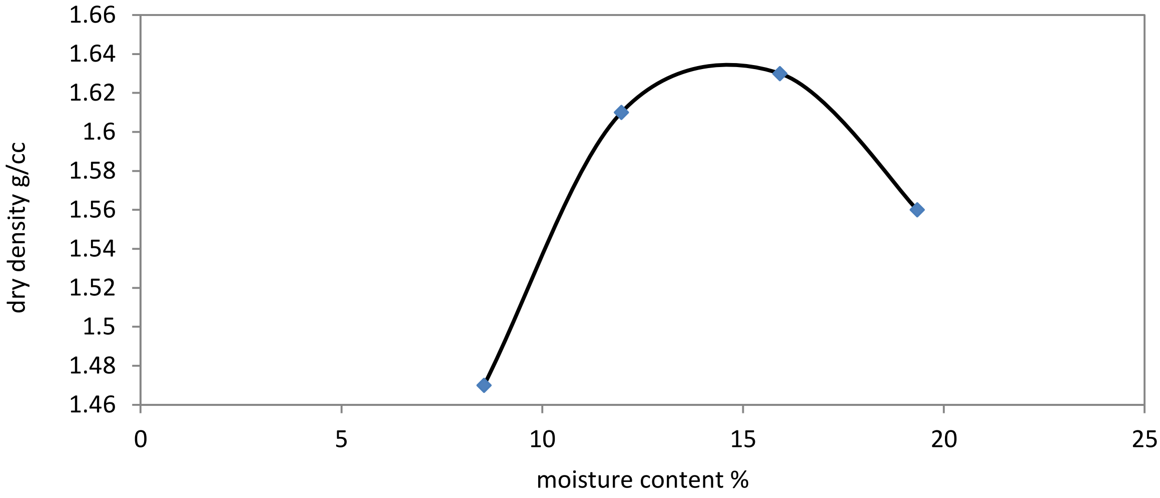


Figure 6.29 compaction curve of Black cotton soil + 3% lime + 10% stone dust

6.3.2.3 Black cotton soil + 6% lime + 5% stone dust

Optimum moisture content W = 15.39 %

Max. Dry density $(\rho_d)_{max}$ = 1.647g/cc

Maximum dry density of Black cotton soil was increased with rate 2.93% and optimum moisture content of Black cotton soil was decreased with rate 15.85% by addition of 6% lime + 5% stone dust.

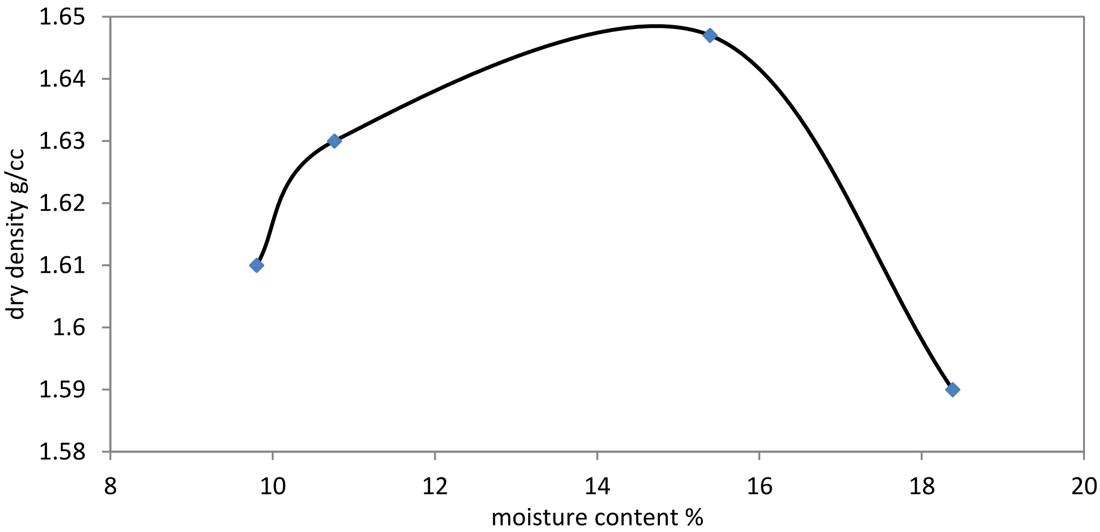


Figure 6.30 compaction curve of Black cotton soil + 6% lime + 5% stone dust

6.3.2.4 Black cotton soil + 6% lime + 10% stone dust

Optimum moisture content W = 14.48 %

Max. Dry density $(\rho_d)_{max}$ = 1.68g/cc

Maximum dry density of Black cotton soil was increased with rate 5% and optimum moisture content of Black cotton soil was decreased with rate 22.27% by addition of 6% lime + 10% stone dust.

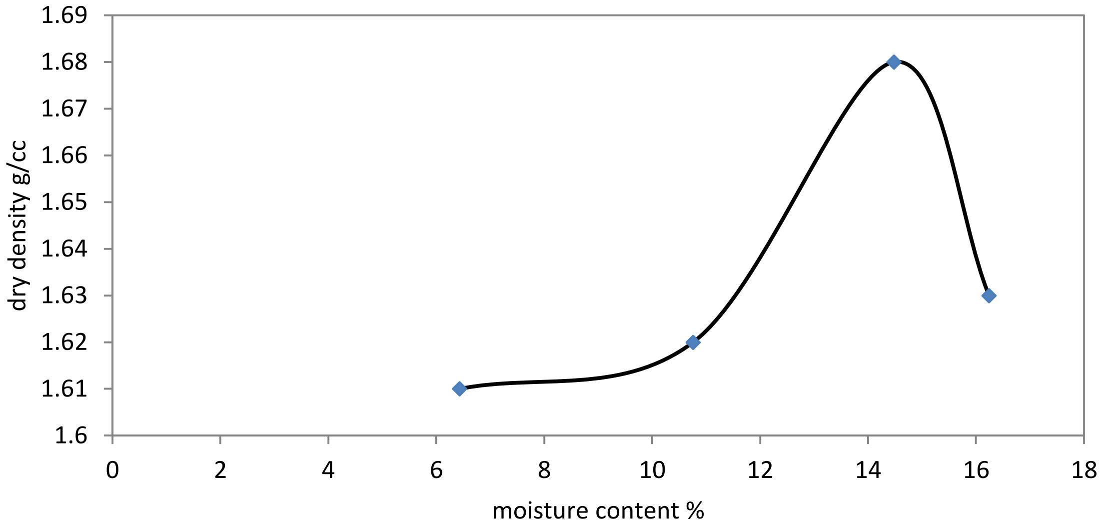


Figure 31 compaction curve of Black cotton soil + 6% lime + 10% stone dust

6.3.2.5 Black cotton soils + 6% lime + 15% stone dust

Maximum dry density of Black cotton soil was increased with rate 5% and optimum moisture content of Black cotton soil was decreased with rate 22.68% by addition of 6% lime + 15% stone dust.

Optimum moisture content W = 15.99%

Max. Dry density $(\rho_d)_{max}$ = 1.68g/cc

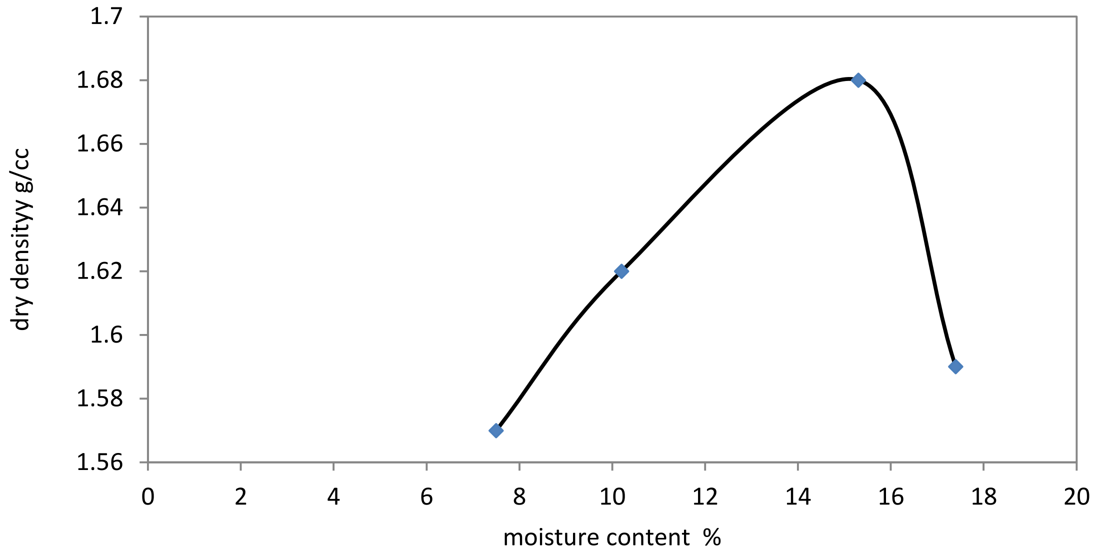


Figure. 6.32 Figure compaction curve of Black cotton soil + 6% lime + 10% stone dust

6.3.2.6 Black cotton soils + 6% lime + 20% stone dust

Maximum dry density of Black cotton soil was increased with rate 5.625% and optimum moisture content of Black cotton soil was decreased with rate 20.61% by addition of 6% lime + 20% stone dust.

Optimum moisture content W = 14.79%

Max. Dry density $(\rho_d)_{max}$ = 1.69g/cc

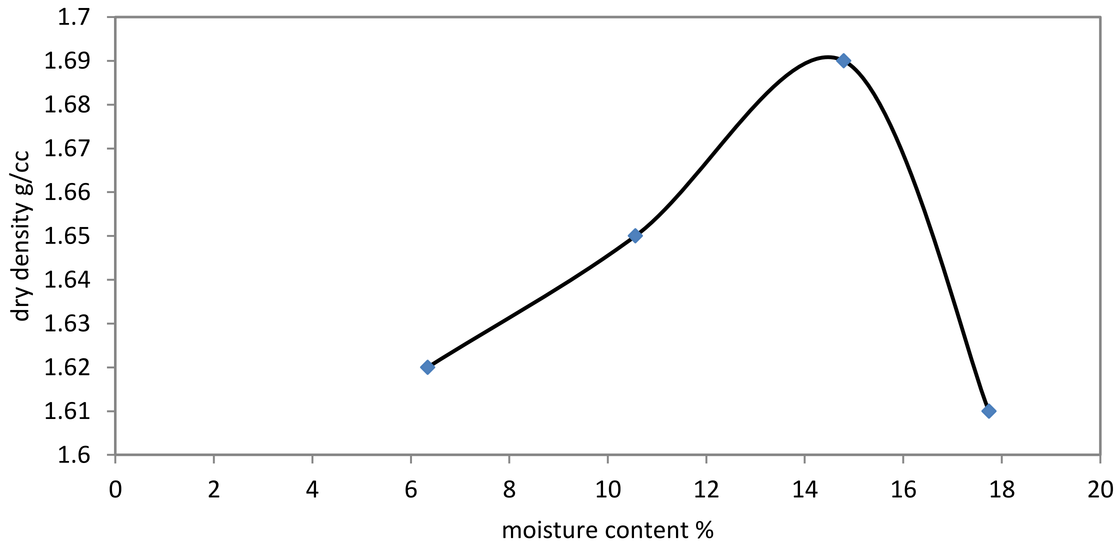
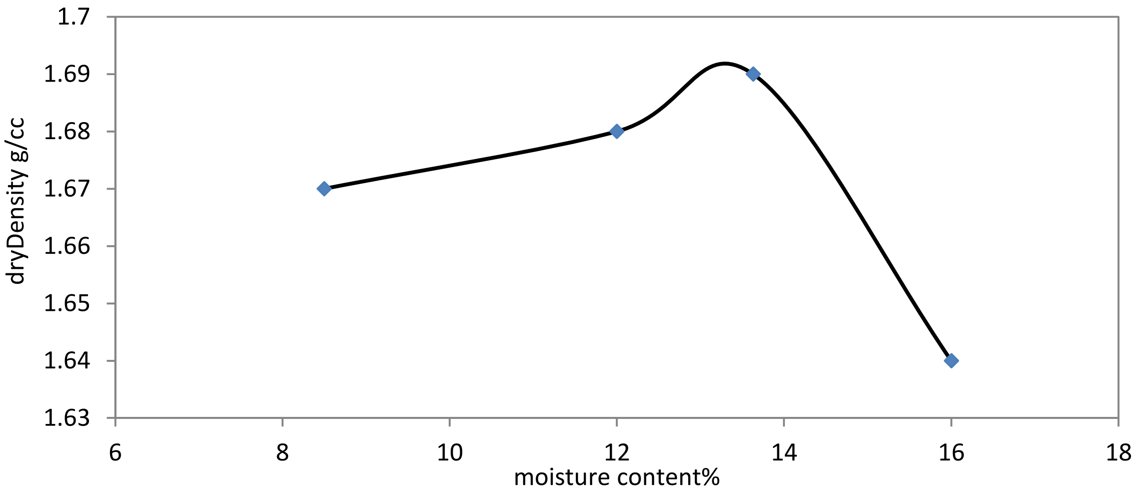


Figure 6.33 compaction curve of Black cotton soil + 6% lime + 20% stone dust

6.3.2.7 Black cotton soils + 6% lime + 25% stone dust

Maximum dry density of Black cotton soil was increased with rate 5.62% and optimum moisture content of Black cotton soil was decreased with rate 26.51% by addition of 6% lime + 25% stone dust.

Optimum moisture content W = 13.69 %



Max. Dry density $(\rho_d)_{max}$ = 1.69g/cc

Figure 6.34 compaction curve of Black cotton soil + 6% lime + 25% stone dust

6.3.2.8Black cotton soil + 9% lime + 5% stone dust

Maximum dry density of Black cotton soil was increased with 3.75% and optimum moisture content of Black cotton soil was decreased with 12.50% by addition of 9% lime + 5% stone dust.

Optimum moisture content W = 16.3%

Max. Dry density $(\rho_d)_{max}$ = 1.66g/cc

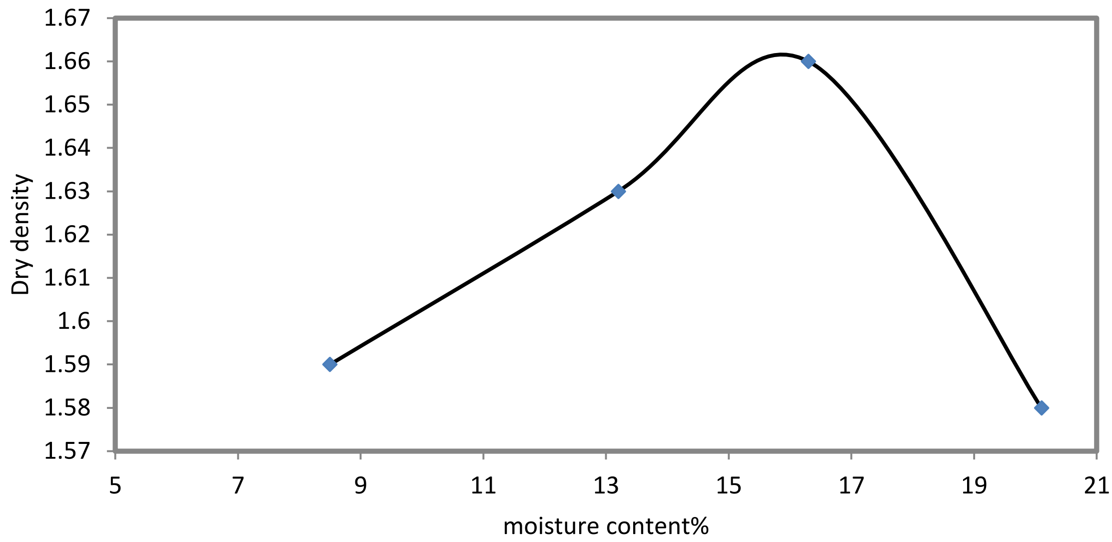


Figure 6.35 compaction curve of Black cotton soil + 9% lime + 5% stone dust

6.3.2.9Black cotton soil + 9% lime + 10% stone dust

Maximum dry density of Black cotton soil was increased with 3.15% and optimum moisture content of Black cotton soil was decreased b with 14.44% by addition of 9% lime + 10% stone dust.

Optimum moisture content W = 15.94%

Max. Dry density $(\rho_d)_{max}$ = 1.65g/cc

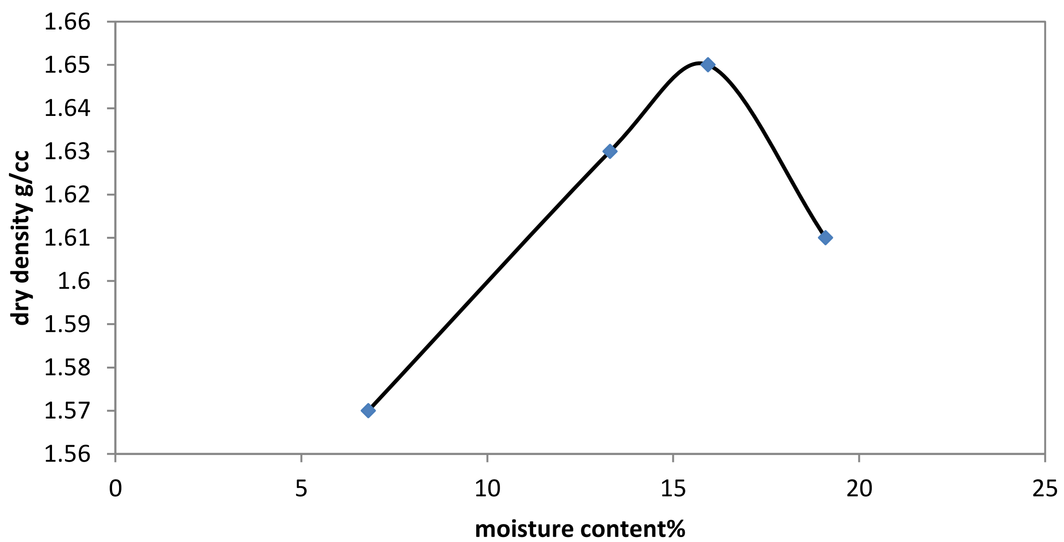


Figure 6.36 compaction curve of Black cotton soil + 9% lime + 10% stone dust

6.3.2.10 Black cotton soil + 9% lime + 15% stone dust

Maximum dry density of Black cotton soil was increased with 3.15% and optimum moisture content of Black cotton soil was decreased with 19.43% by addition of 9% lime + 10% stone dust.

Optimum moisture content W = 15.01%

Max. Dry density $(\rho_d)_{max}$ = 1.68g/cc

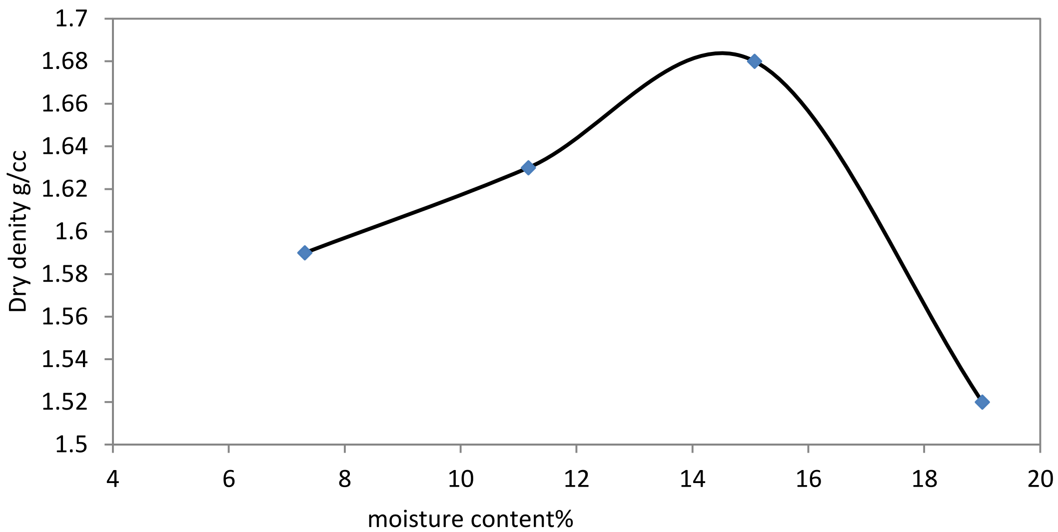


Figure 6.37 compaction curve of Black cotton soil + 9% lime + 15% stone dust

6.3.2.11 Black cotton soil + 9% lime + 20% stone dust\

Maximum dry density of Black cotton soil was increased with 5.88% and optimum moisture content of Black cotton soil was decreased with 27.53% by addition of 9% lime + 20% stone dust.

Optimum moisture content W = 13.5%

Max. Dry density $(\rho_d)_{max}$ = 1.70g/cc

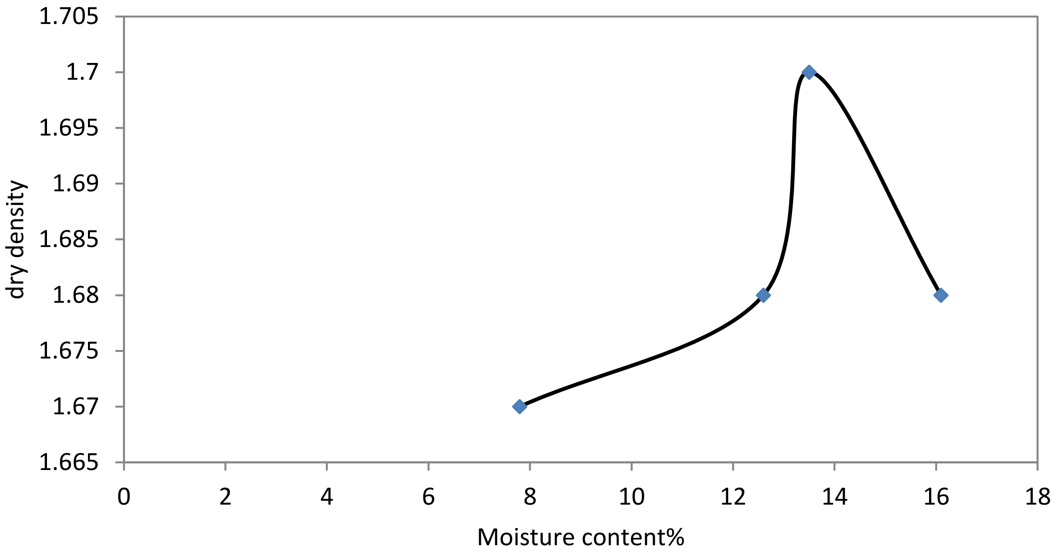


Figure 6.38 compaction curve of Black cotton soil + 9% lime + 20% stone dust

6.3.2.12 Black cotton soil + 9% lime + 25% stone dust

Maximum dry density of Black cotton soil was increased with 3.15% and optimum moisture content of Black cotton soil was decreased with 14.44% by addition of 9% lime + 10% stone dust.

Optimum moisture content W = 14.1%

Max. Dry density $(\rho_d)_{max}$ = 1.68g/cc

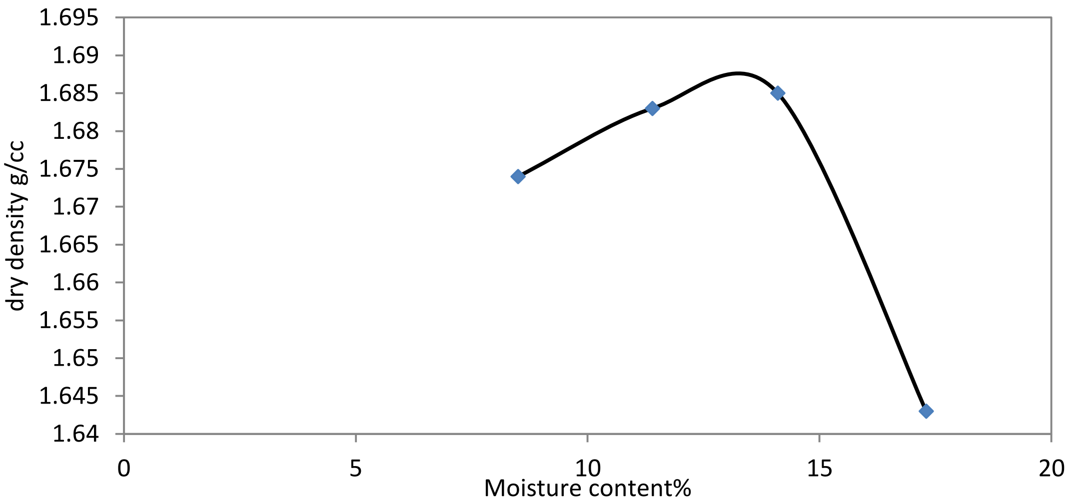


Figure 6.39 compaction curve of Black cotton soil + 9% lime + 25% stone dust

6.3.3 Unconfined Compression Strength Test

6.3.3.1 Black Cotton Soil With 3% Lime +5% Stone Dust

UCS value was found at optimum moisture content

From graph: UCS =.1645 N/mm^2

UCS value of Black cotton soil was decreased with 1.02% by addition with 3% Lime +5% Stone Dust.

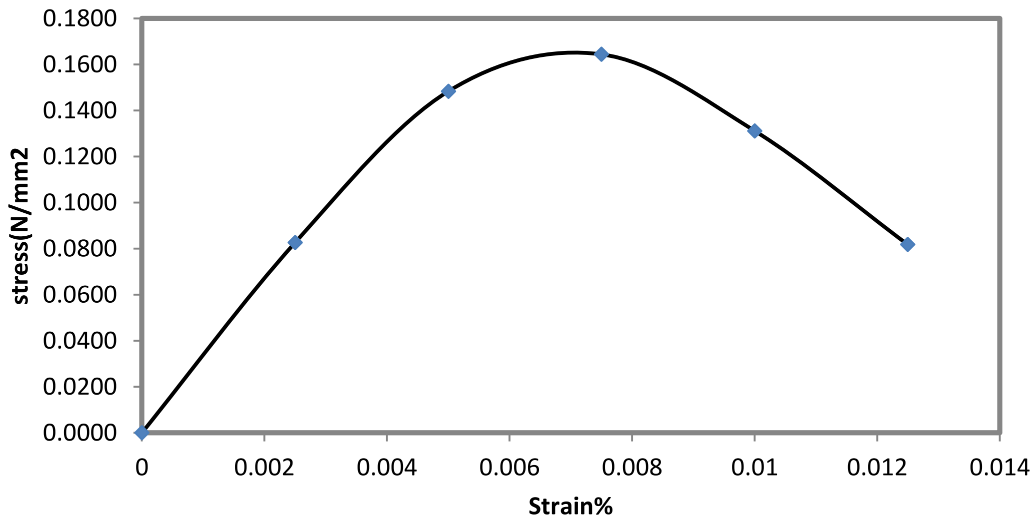


Figure 6.40 UCS curve of Black cotton soil + 3% lime + 5% stone dust

6.3.3.2Black Cotton Soil With 3% Lime +10% Stone Dust

From graph: UCS =.1662 N/mm^2

UCS value of Black cotton soil was same found by addition 3% Lime +10% Stone Dust as UCS value of Black cotton soil. UCS value was increased because lime reacted to soil and provided a binding property with stone dust to Black cotton soil.

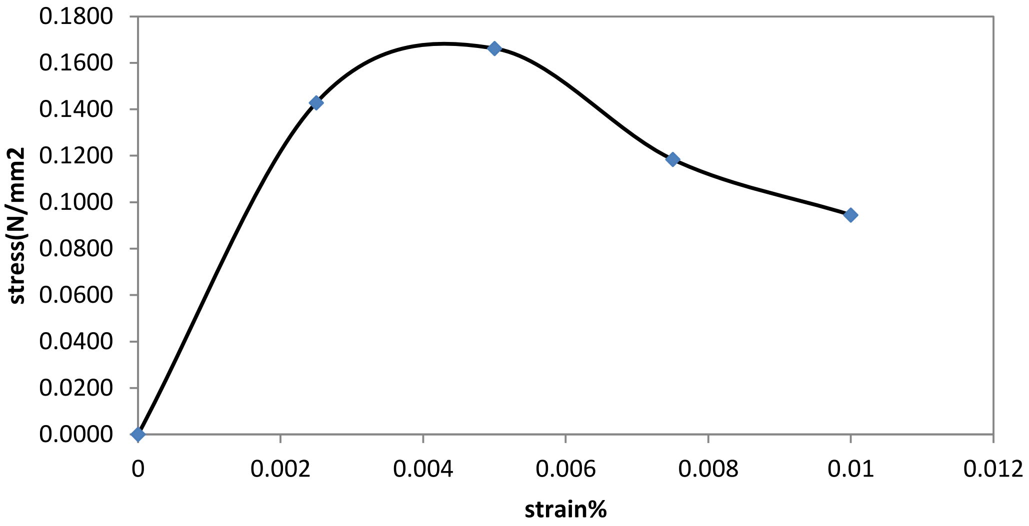


Figure 6.41 UCS curve 3% Lime +10% Stone Dust

6.3.3.3 Black Cotton Soil With 6% Lime +5% Stone Dust

From graph: UCS =.1895 N/mm^2

With addition of 6% Lime +5% Stone Dust in Black cotton soil UCS value is further increased with 11.16%.

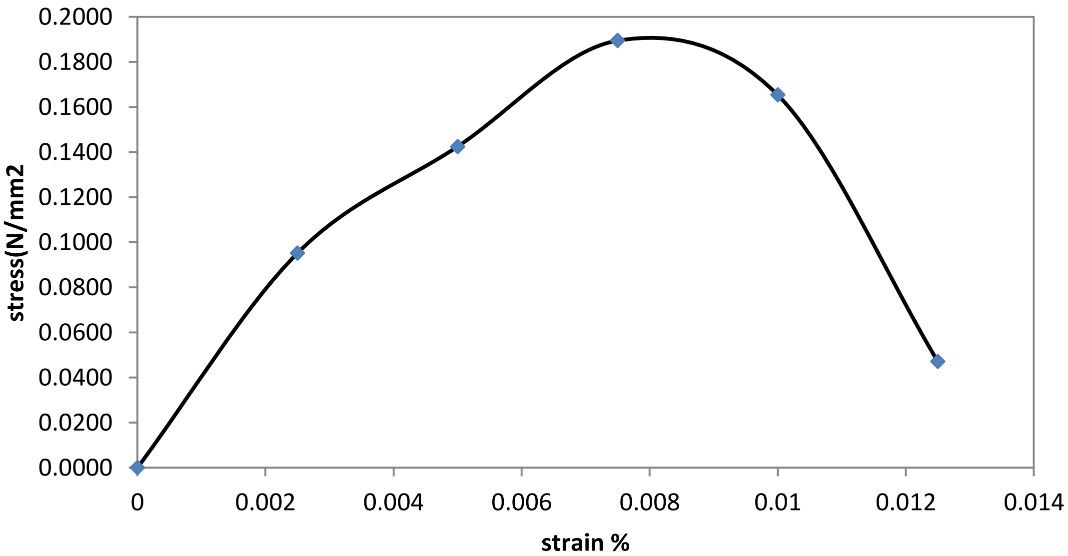


Figure 6.42 UCS curve 6% Lime +5% Stone Dust

6.3.3.4Black Cotton Soil With 6% Lime +10% Stone Dust

From graph: UCS =**.1900**N/mm^2

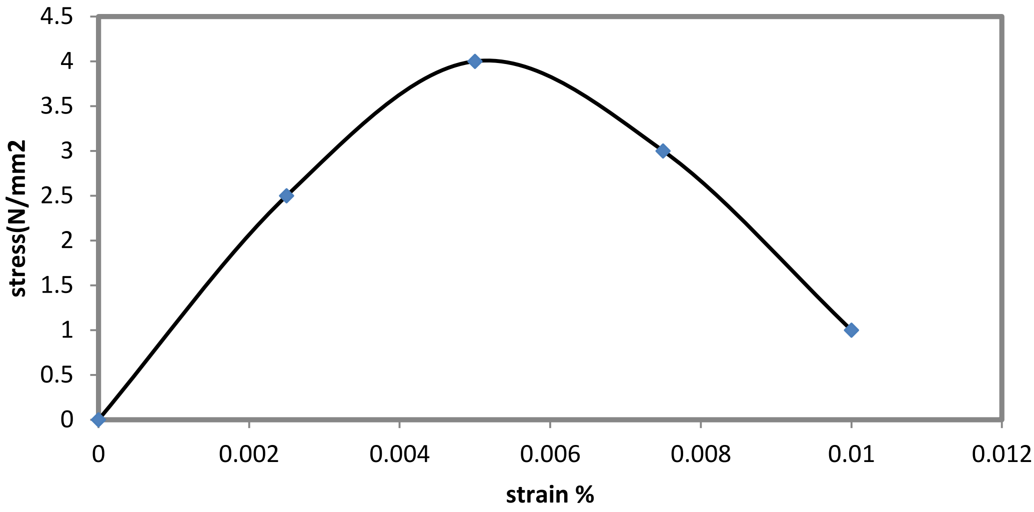


Figure 6.43 UCS curve 6% Lime +10% Stone Dust

At this stage UCS was increased with 14.3% by addition of % Lime +10% Stone Dust in Black cotton soil.

6.3.3.5 Black Cotton Soil With 6% Lime +15% Stone Dust UCS= .2363N/mm2

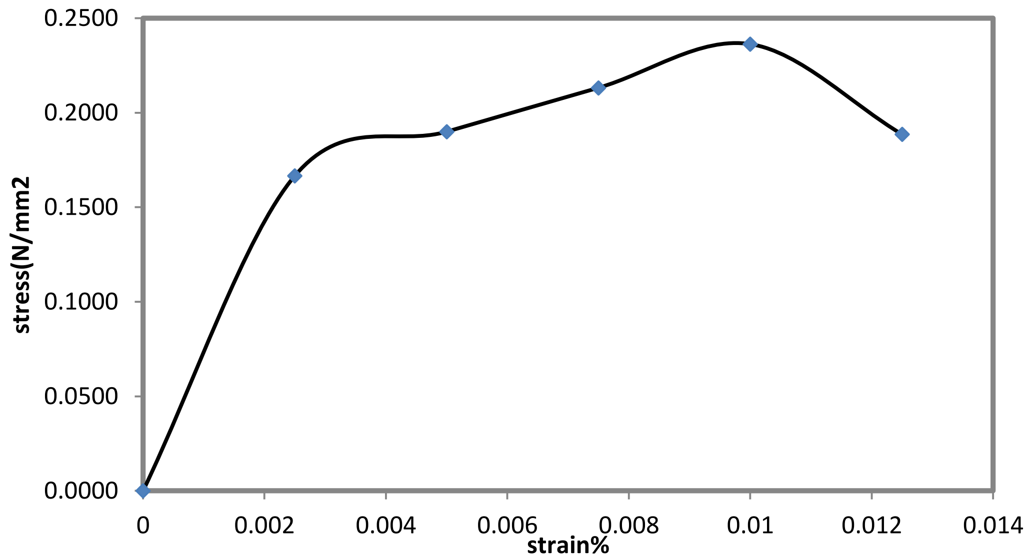


Figure 6.44 UCS curve 6% Lime +15% Stone Dust

UCS was increased with increment of 42% by addition of 6% lime and 15% stone dust in Black cotton soil.

6.3.3.6Black Cotton Soil With 6% Lime +20% Stone Dust:

From graph: UCS =**.2132**N/mm^2

At this stage UCS value was further decrease with 28.27% by addition of With 6% Lime +20% Stone Dust in Black cotton soil.

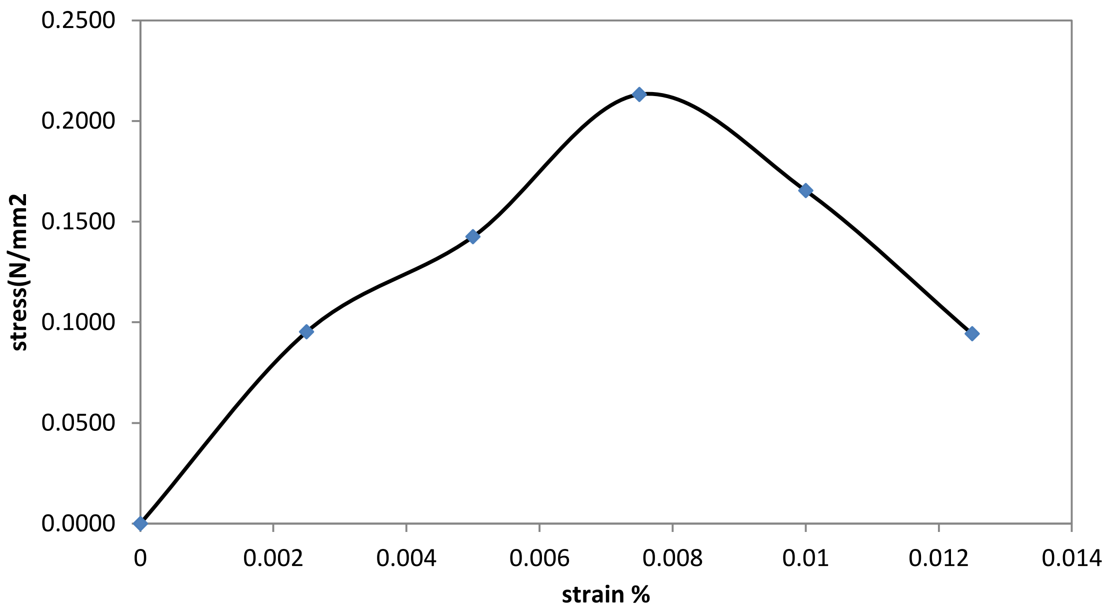


Figure 7.45 UCS curve 6% Lime +20% Stone Dust

6.3.3.7Black Cotton Soil With 6% Lime +25% Stone Dust

From graph: UCS =**.**2842N/mm^2

After addition of 6% Lime +25% Stone Dust further increment of UCS was found in Black cotton soil.

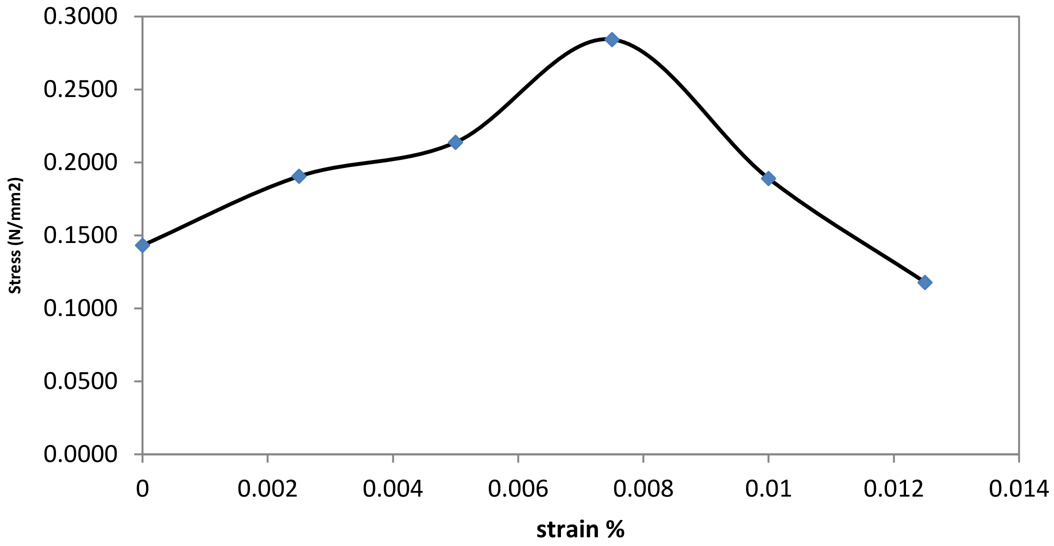


Figure. 6.46 UCS curve 6% Lime +25% Stone Dust

6.3.3.8 Cotton Soil With 9% Lime +5% Stone Dust

From graph: UCS =**.2137**N/mm^2

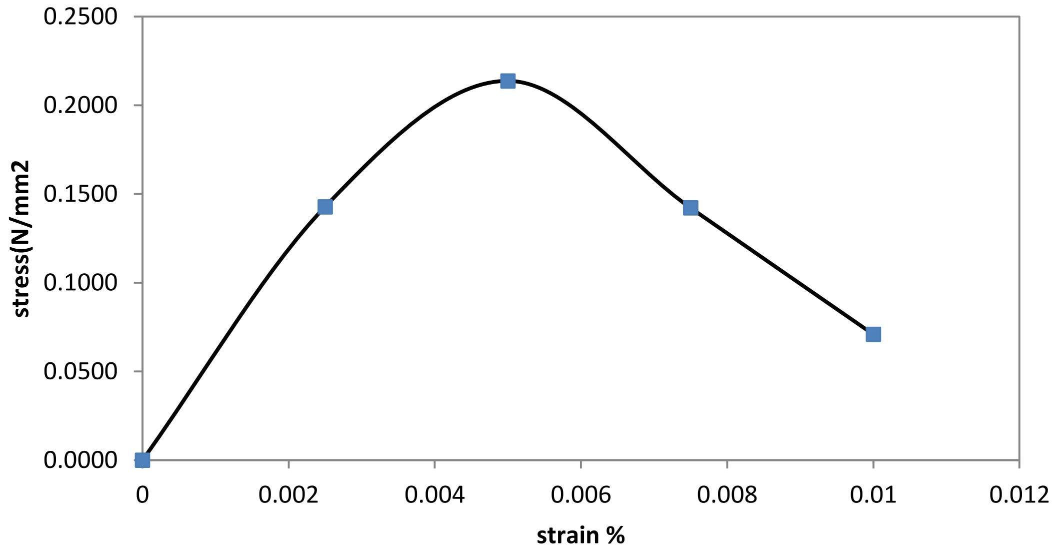


Figure 6.47 UCS curve 9% Lime +5% Stone Dust

By addition of 9% Lime +5% Stone Dust in Black cotton soil UCS value of Black cotton soil was decreased with 28.58%.

6.3.3.9Black Cotton Soil With 9% Lime +10% Stone Dust

From graph: UCS =**.2606**N/mm^2

After addition of 9% Lime +10% Stone Dust further increment of UCS was found in Black cotton soil with 56.7%.

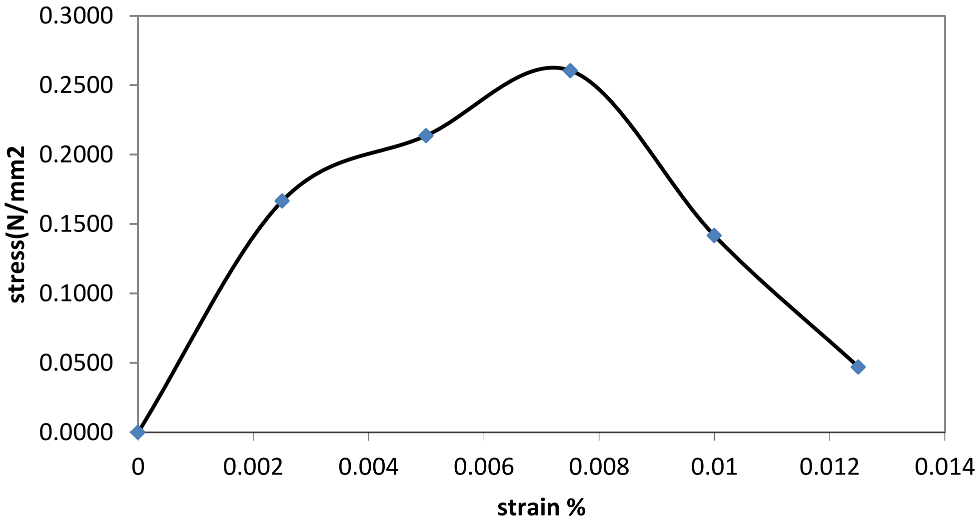


Figure 6.48 UCS curve 9% Lime +10% Stone Dust

6.3.3.10Black Cotton Soil With 9% Lime +15% Stone Dust

From graph: UCS =**.3072**N/mm^2

After addition of 9% Lime +15% Stone Dust further increment of UCS was found in Black cotton soil with 84.83%.

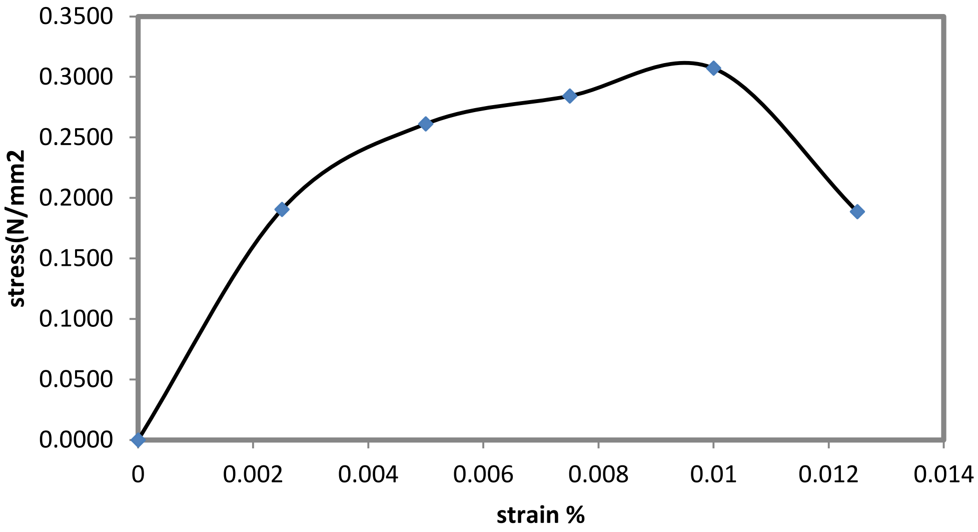


Figure 6.49 UCS curve 9% Lime +15% Stone Dust

6.3.3.11Black Cotton Soil With 9% Lime +20% Stone Dust

From graph: UCS =**.3544**N/mm^2

After addition of 9% Lime +15% Stone Dust further increment of UCS was found in Black cotton soil.

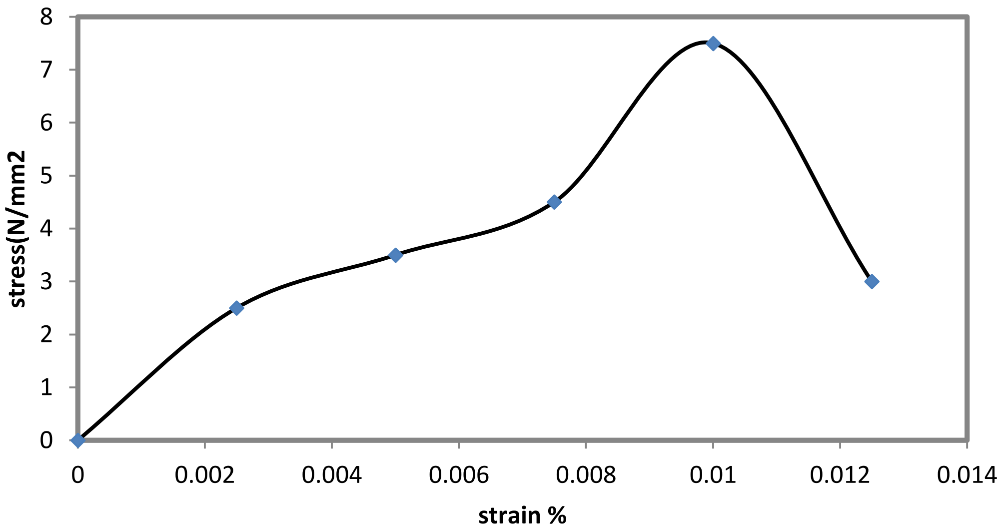


Figure 6.50 UCS curve 9% Lime +20% Stone Dust

6.3.3.12Black Cotton Soil With 9% Lime +25% Stone Dust

From graph: UCS =**.3117**N/mm^2

By addition of 9% Lime +5% Stone Dust in Black cotton soil UCS value of Black cotton soil was decreased with . Reduction in UCS occurs due to reduction in cohesion because of the reduction in Black cotton soil content.

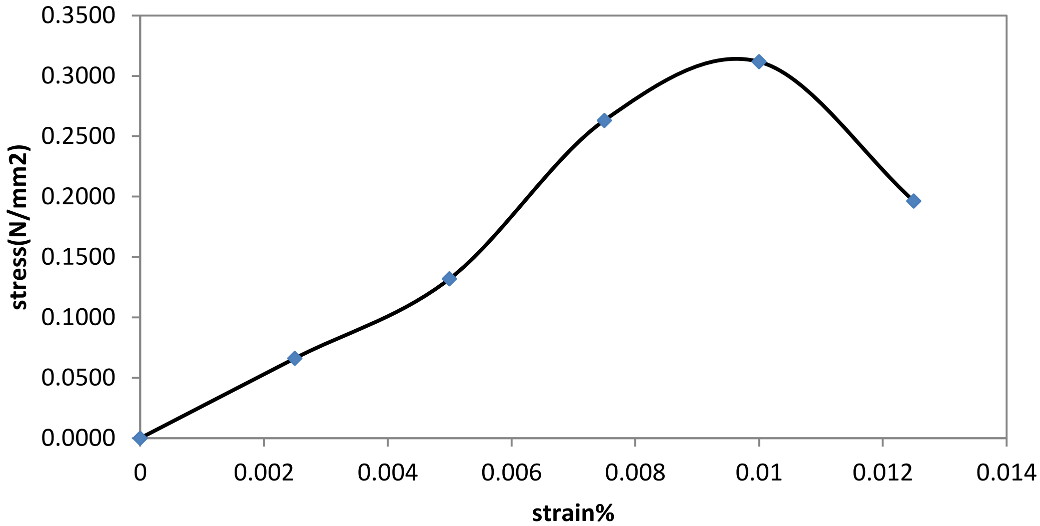


Figure. 6.51 UCS curve 9% Lime +25% Stone Dust

6.3.4 California Bearing Ratio

6.3.4.1 Black Cotton Soil With 3% Lime +5% Stone Dust

OMC= 16.58 %, MDD= 1.60 g/cc

CBR Value at 2.5mm= 7.0

CBR value of Black cotton soil was increased with increment of 311% by addition of 3% Lime +5% Stone Dust. CBR was increased because stone dust percentage was increased with lime. The increase in the CBR strength with addition of lime and stone dust

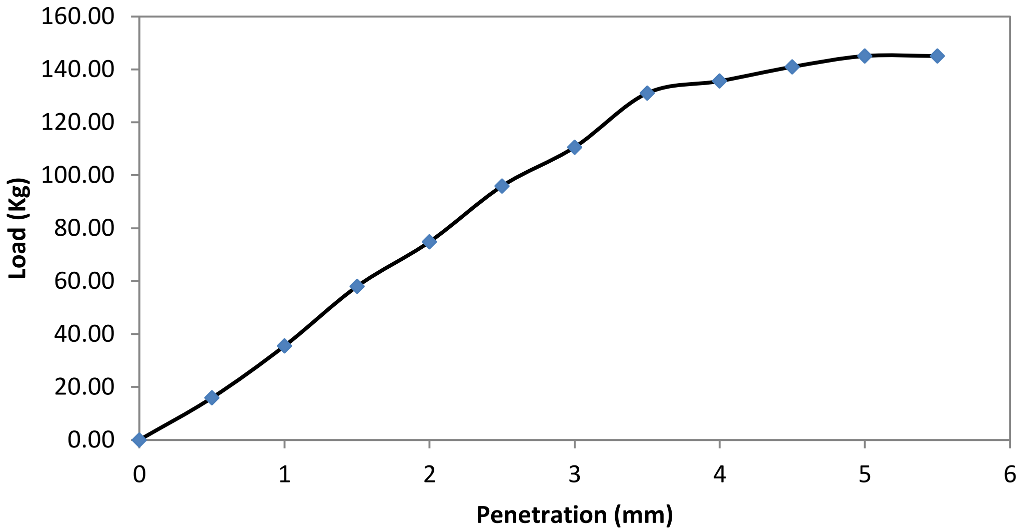


Figure 6.52 CBR curve 3% Lime +5% Stone Dust

6.3.4.2Black Cotton Soil With 3% Lime +10% Stone Dust

OMC= 15.92 %, MDD= 1.63 g/cc

CBR Value at 2.5mm= 9.56

CBR value of Black cotton soil was increased with increment of 462% by addition of 3% Lime +10% Stone Dust in Black cotton soil.

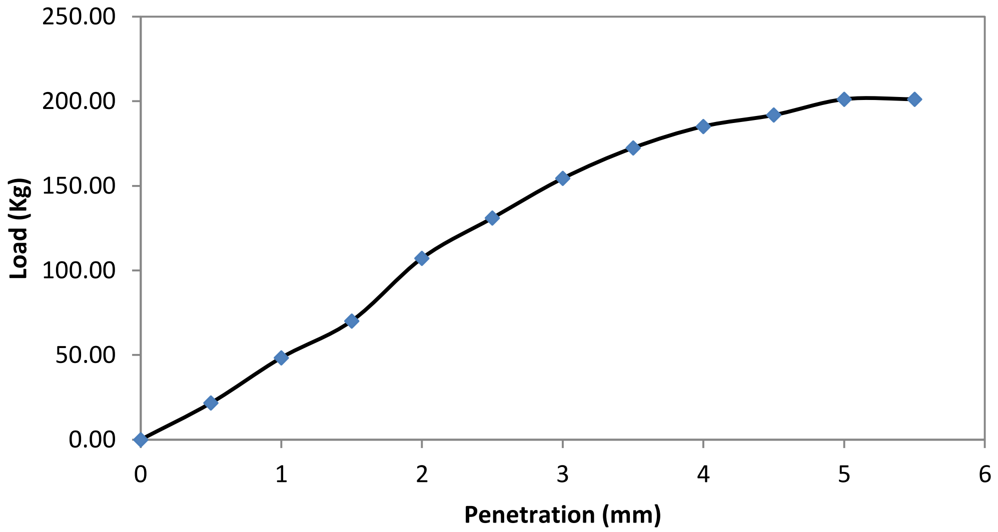


Figure 6.53 CBR curve 3% Lime +5% Stone Dust

6.3.4.3Black Cotton Soil With 6% Lime +5% Stone Dust

OMC= 15.39 %, MDD= 1.63g/cc

CBR Value at 2.5mm= 7.36

CBR value was further decreased by addition of 6% Lime +5% Stone Dust in Blacl cotton soil

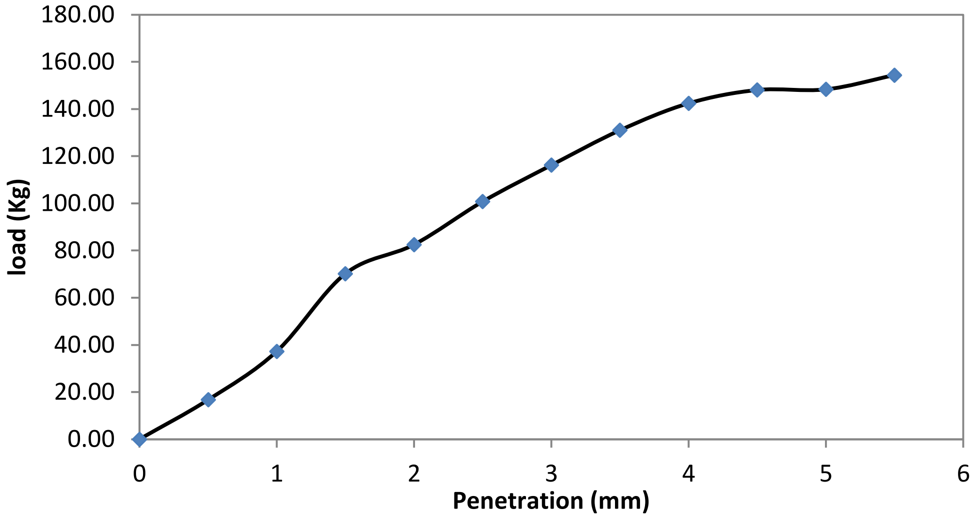


Figure. 6.54 CBR curve6% Lime +5% Stone Dust

6.3.4.4Black Cotton Soil With 6% Lime +10% Stone Dust

OMC= 15.39 %, MDD= 1.647 g/cc

CBR Value at 2.5mm= 8.54

Further increment was found in CBR with 402% by addition of 6% Lime +10% Stone Dust in Black cotton soil.

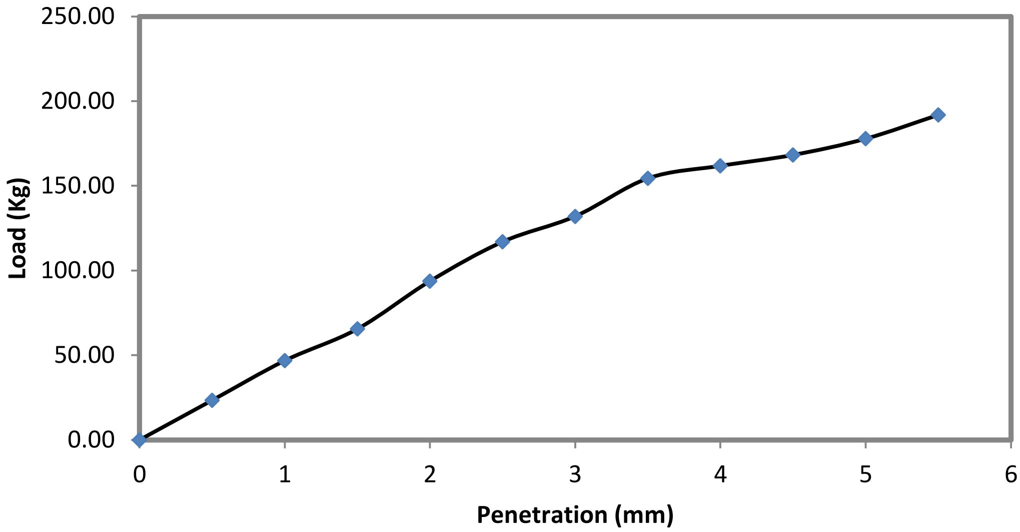


Figure. 6.55 CBR curve 6% Lime +10% Stone Dust

6.3.4.5Black Cotton Soil With 6% Lime +15% Stone Dust

OMC= 15.39 %, MDD= 1.68 g/cc

CBR Value at 2.5mm= 13.70

CBR value of Black cotton soil was increased with increment of 705% by addition of 6% Lime +15% Stone Dust in Black cotton soil.

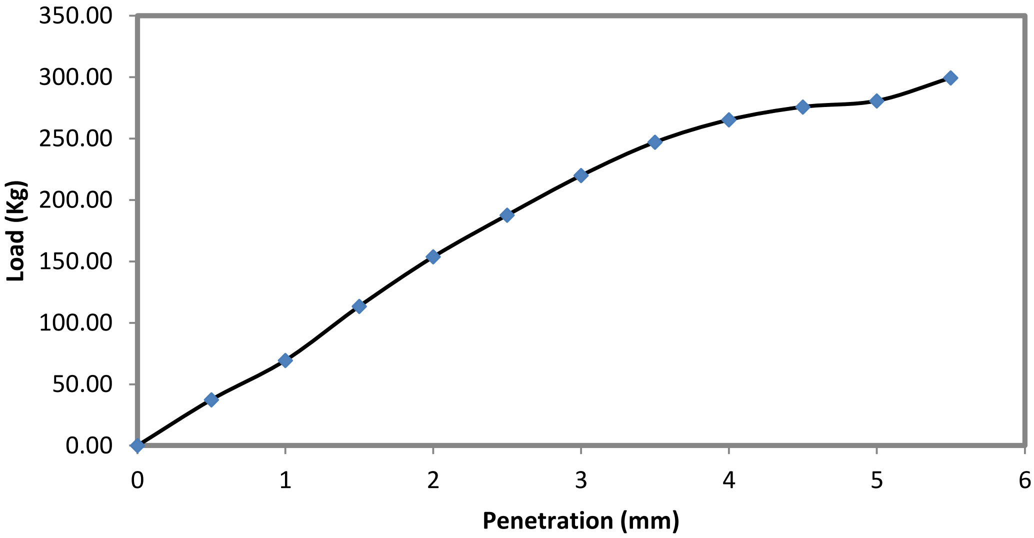


Figure. 6.56 CBR curve 6% Lime +15% Stone Dust

6.3.4.6Black Cotton Soil With 6% Lime +20% Stone Dust

OMC= 13.63 %, MDD= 1.69g/cc

CBR Value at 2.5mm= 20.76

CBR value of Black cotton soil was increased with increment of 1121% by addition of 6% Lime +20% Stone Dust in Black cotton soil.

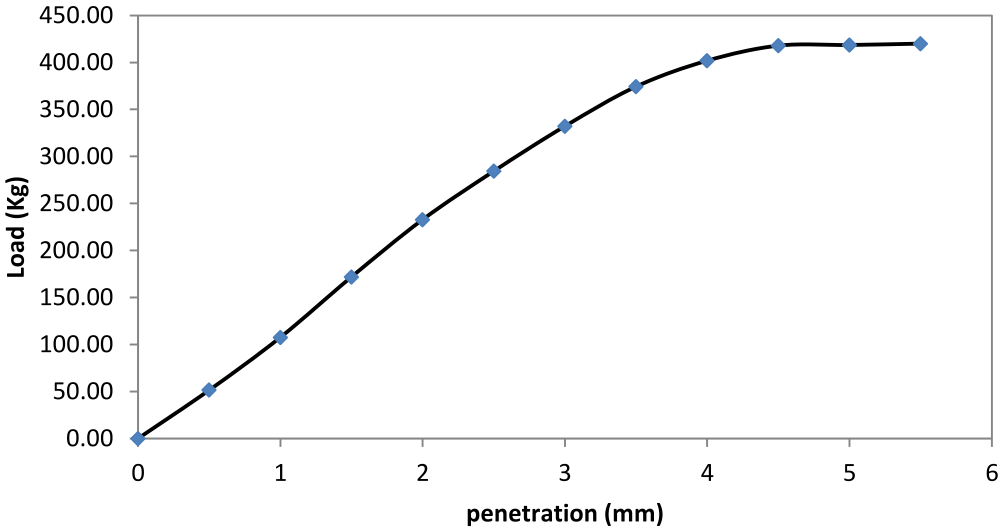


Figure. 6.57 CBR curve 6% Lime +20% Stone Dust

6.3.4.7Black Cotton Soil With 6% Lime +25% Stone Dust

OMC= 14.79 %, MDD= 1.69 g/cc

CBR Value at 2.5mm= 19.13

At this stage UCS vale was again decreased with 7.85% by addition of 6% Lime +25% Stone Dust in Black cotton soil.

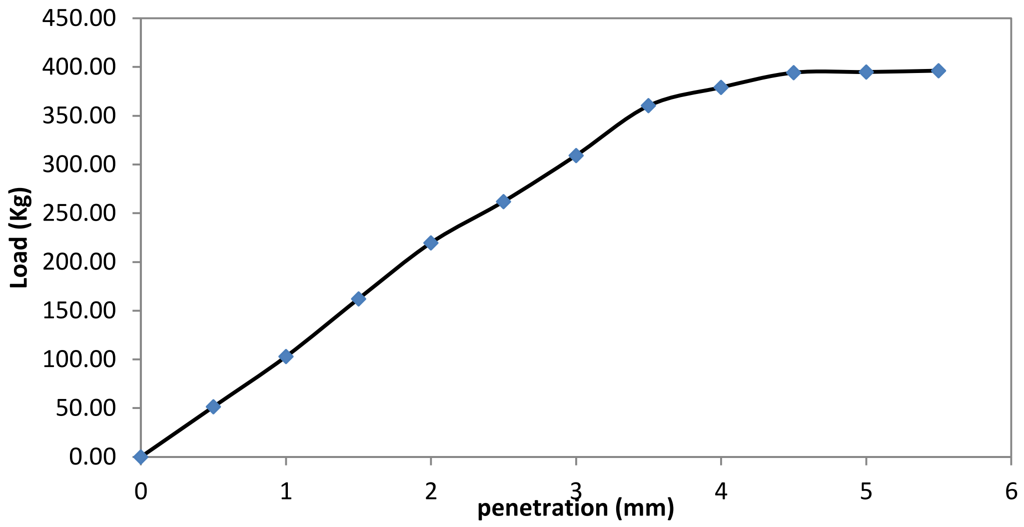


Figure. 6.58 CBR curve 6% Lime +25% Stone Dust

6.3.4.8Black Cotton Soil With 9% Lime +5% Stone Dust

OMC= 16.3 %, MDD= 1.66g/cc

CBR Value at 2.5mm= 7.36

Increment of CBR in Black cotton soil was found with 332% by addition of 9% Lime +5% Stone Dust in Black cotton soil.

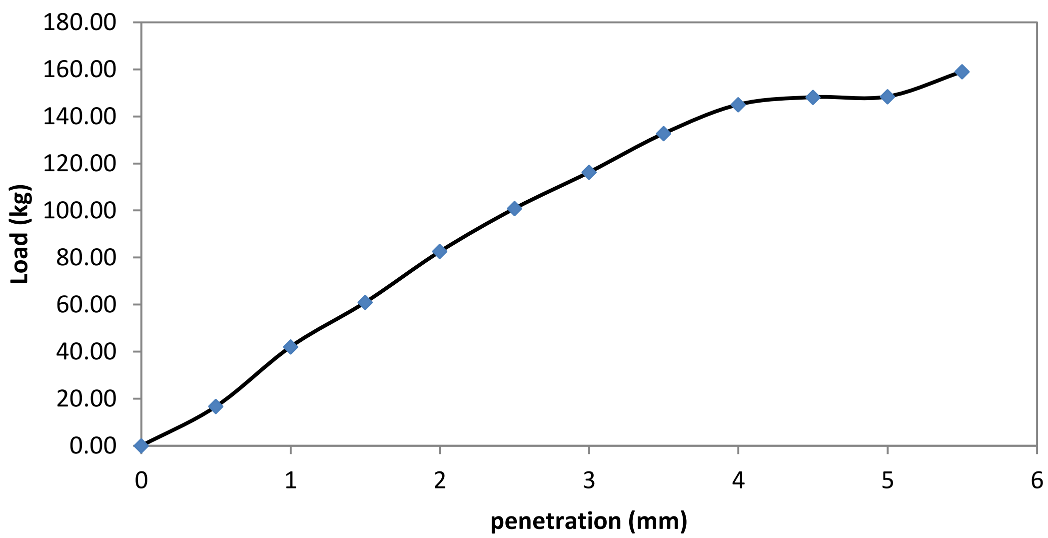


Figure. 6.59 CBR curve 9% Lime +5% Stone Dust

6.3.4.9Black Cotton Soil With 9% Lime +10% Stone Dust

OMC= 15.94%, MDD= 1.65 g/cc

CBR Value at 2.5mm= 8.36

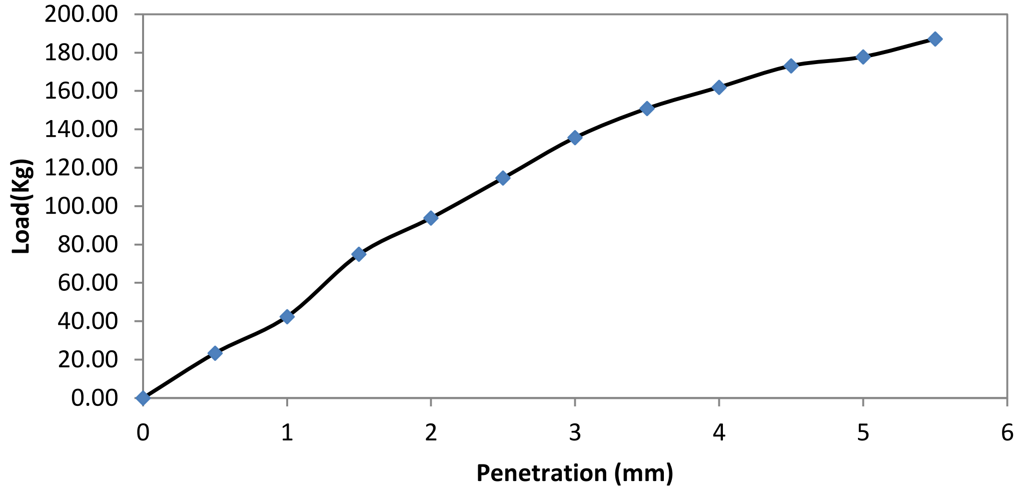


Figure. 60 CBR curve 9% Lime +10% Stone Dust

6.3.4.10 Cotton Soil With 9% Lime +15% Stone Dust

OMC= 15.07 %, MDD= 1.68g/cc

CBR Value at 2.5mm= 17.08

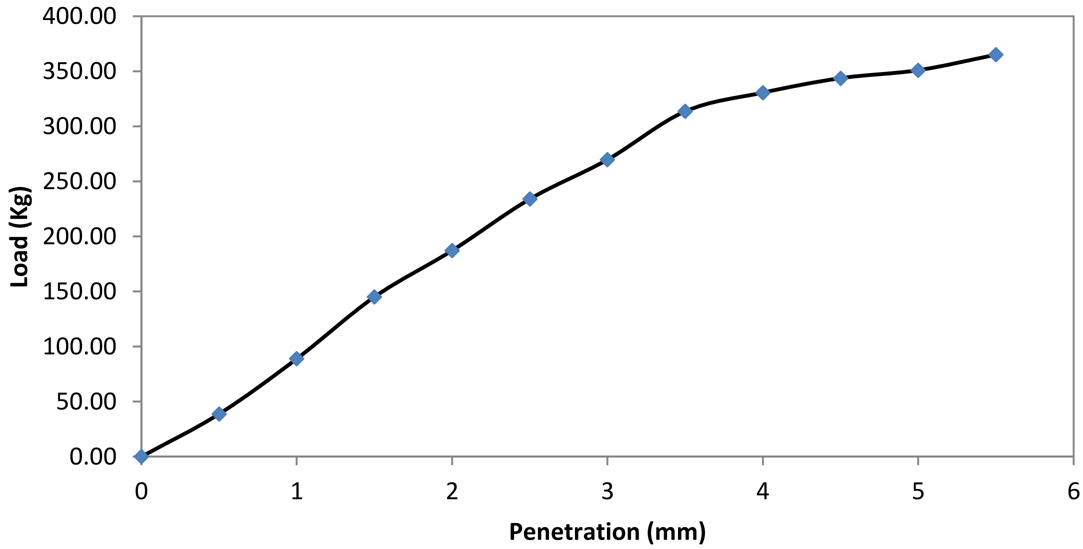


Figure. 6.61 CBR curve 9% Lime +15% Stone Dust

6.3.4.11Black Cotton Soil With 9% Lime +20% Stone Dust

OMC= 13.5%, MDD= 1.7 g/cc

CBR Value at 2.5mm= 21.89

Increment of CBR in Black cotton soil was found with 1187% by addition of 9% Lime +10% Stone Dust in Black cotton soil.

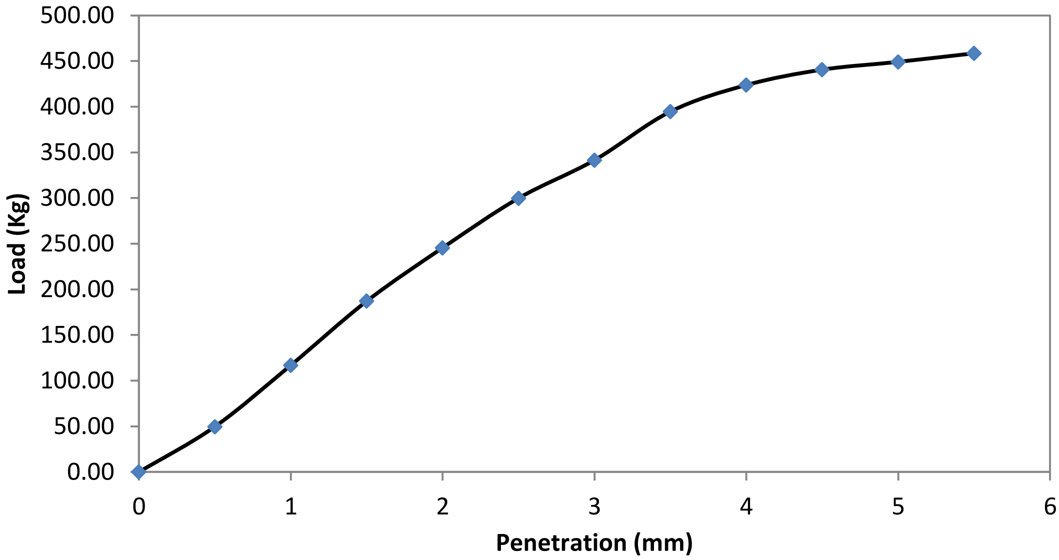


Figure. 6.62CBR curve

6.3.4.12 Cotton Soil With 9% Lime +25% Stone Dust

OMC= 14.1 %, MDD= 1.68 g/cc

CBR Value at 2.5mm= 21.76

CBR value was slightly decreased with0.59% by addition of 9% Lime +25% Stone Dust

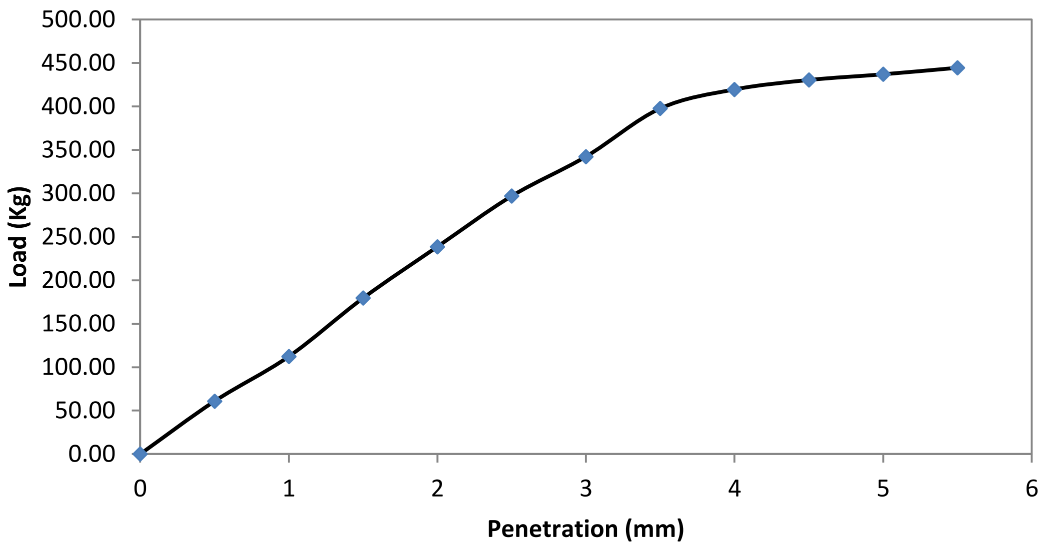


Figure 6.63 CBR curve

6.3.5 Differential Free Swell Test:

6.3.5.1Black cotton soil +3% lime +5% stone dust **DFS= 19 %**

Differential free swelling index was decreased because silt size percentage was increased in Black cotton soil. and DFS was decreased with rate 54.76% by addition of 3% lime +5% stone dust in Black cotton soil, swelling was decreased because with increasing in quantity of calcite in soil with addition of lime.

6.3.5.2 Black cotton soil +3% lime +10% stone dust **DFS=16.25 %**

DFS was decreased with 61.30% by addition of 3% lime +10% stone dust in Black cotton soil.

6.3.5.3Black cotton soil +6% lime +5 % stone dust **DFS= 14 %**

DFS was decreased with 66.66% by addition of 6% lime +5% stone dust in Black cotton soil.

6.3.5.4Black cotton soil +6% lime +10 % stone dust **DFS= 3.47**

DFS was decreased with 91.73% by addition of 6% lime +10% stone dust in Black cotton soil.

6.3.5.5Black cotton soil +6% lime +15% stone dust **DFS= 7.69**

DFS was slightly increased by addition of 6% lime +15% stone dust in Black cotton soil.

6.3.5.6Black cotton soil +6% lime +20% stone dust **DFS= 3.17**

DFS was further decreased with by addition of 6% lime +20% stone dust in Black cotton soil.

6.3.5.7Black cotton soil +6% lime +25% stone dust **DFS= 10.6**

DFS was slightly increased by addition of 6% lime +25% stone dust in Black cotton soil.

6.3.5.8Black cotton soil +9% lime +5% stone dust **DFS= 9.5**

DFS was decreased with 77.37% by addition of 3% lime +5% stone dust in Black cotton soil.

6.3.5.9Black cotton soil +9% lime +10% stone dust **DFS= 9.5**

DFS was increased in same amount as addition of 9% lime +10% stone dust while addition of 3% lime +5% stone dust in Black cotton soil.

6.3.1.0Black cotton soil +9% lime +15% stone dust **DFS= 9.21**

DFS was decreased with 78.07% by addition of 9% lime +15% stone dust in Black cotton soil.

6.3.1.1Black cotton soil +9% lime +20% stone dust **DFS= 11**

DFS was slightly increased by addition of 9% lime +20% stone dust in Black cotton soil

6.3.1.2Black cotton soil +9% lime +25% stone dust **DFS= 9.1**

DFS was slightly decreased by addition of 6% lime +25% stone dust in Black cotton soil.

DISCUSSIONS

1. The results of UCS tests on Black cotton soil treated with different percentage of lime and stone dust are shown in above figures. By increasing the percentages of lime and stone dust, UCS of soil increases up to a limit at addition of 9% lime and 20% stone dust, further addition of admixture decreases the UCS of the expansive soil. The UCS of Black cotton soil increases to 0.3554N/mm^2 from 0.1645 N/mm^2, when 9% lime and 20% stone dust was added. This is because of the additional frictional resistance. Reduction in UCS occurs due to reduction in cohesion because of the reduction in expansive soil content.
2. The results of soaked CBR tests on black cotton soil with lime and stone dust are shown in above figures. It is observed that by addition of lime and stone dust at different percentage rate of increases in the soaked CBR of soil increases to 1187% from 311% up to addition of 9% lime and 20% stone dust, further addition of admixtures slightly decreases the soaked CBR of the soil. The soaked CBR of the soil decreases to 0.59% when 9%lime and 25 %stone dust was added to Black cotton soil. The soaked CBR attains the highest value when the percentage of 9%lime and 20%stone dust was added. There is a 1187% increase in Soaked CBR of the virgin soil by the combined effect of lime and stone dust. The reason of this effect is the pozzolanic reactions of lime with the amorphous silica and Alumina present in soil and stone dust. After addition of 9% lime and 20% stone dust the strength decreases because of the availability of extra Lime to react with the insufficient amorphous silica and Alumina present in soil and stone dust which results in carbonation reaction and thus strength decreases.

3. The results of differential free swelling tests on lime and stone dust stabilized expansive soil treated with different percentage has been shown in above Figures. It is observed that by addition of lime and stone dust, the differential free swelling index of soil decreases to 3.47%. The reason of which is the decrease in plasticity characteristics of soil due to reduction in clay content of soil because of replacement of clay with stone dust, decreases to zero from 42% when 6%lime and 25% stone dust was added %. This is Because of the pozzolanic reaction of lime with the amorphous silica and Alumina present in soil and stone dust a strong inter particle bond develops, this cementing bond offers great resistance to swelling and also does not allow the water to escape from soil to induce shrinkage.
4. It is observed that maximum dry density of Black cotton soil was increased up to addition of 6% lime and 25% stone dust . This is because of the frictional resistance from stone dust in addition to the cohesion from Black cotton soil and lime gives the binding property to soil.
5. Liquid limit of Black cotton soil was decreased by addition of lime and stone dust at different percentages. This is because when quicklime chemically combines with water, it can be used very effectively to dry any type of wet soil. Heat from this reaction further dries wet soils. The reaction with water occurs even if the soils do not contain significant clay fractions. When clays are present, lime's chemical reactions with clays increase the moisture-holding capacity of the soil, which reduces free liquids and decreases in liquid limit and plastic limit decreases because clay particles are reduces by addition of stone dust in black cotton soil.

CHAPTER – 7

COMPARISON OF RESULTS

7.1 Differential Free Swelling results

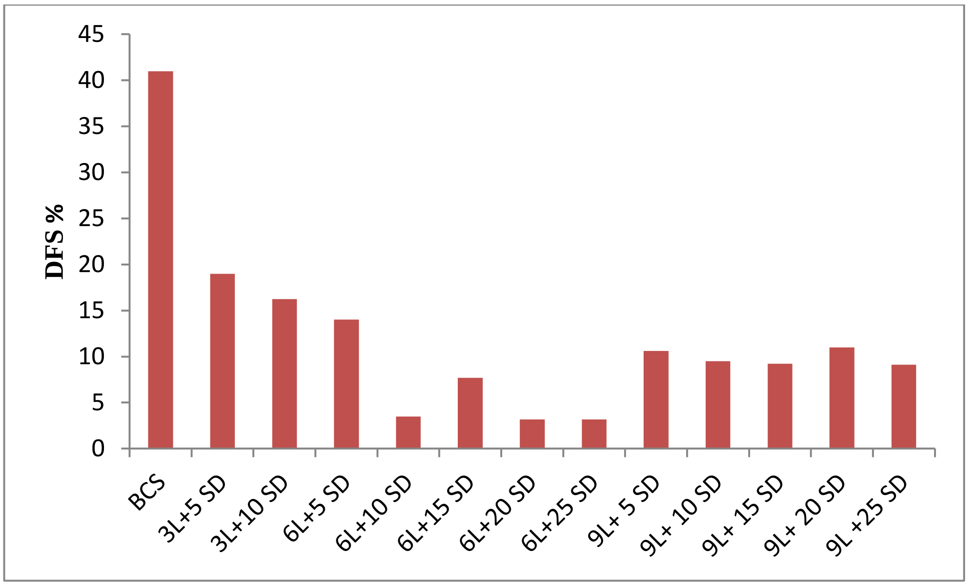


Figure7.1: Variation in Differential Free Swelling Results.

Figure 7.1 shows variation in differential free swelling test, it shows that maximum reduction in Differential Free Swelling was found with addition of 6% lime and 25% stone dust in black cotton soil.

7.2 Liquid limit Results

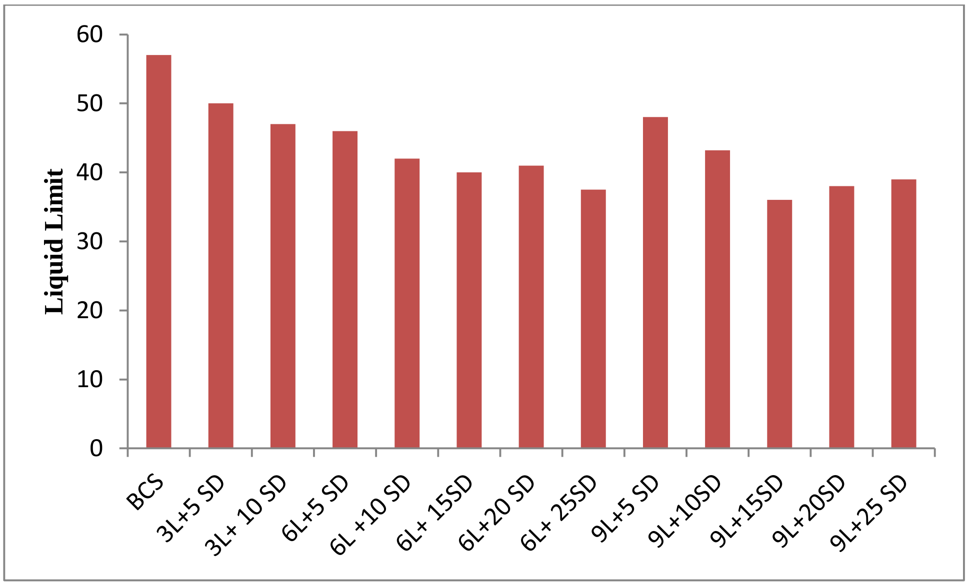


Figure7.2: Variation in Liquid Limit

Figure 7.2 shows variation in liquid limit of black cotton soil , it shows that maximum reduction in liquid limit was found with addition of 6% lime and 25% stone dust in black cotton soil.

7.8 Plastic Limit Results

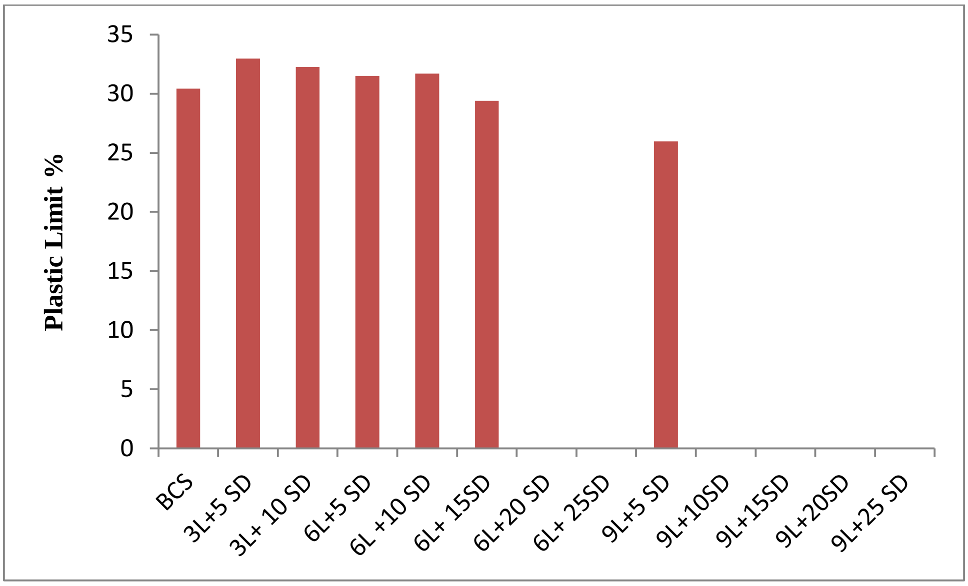


Figure 7.3 Variation in plastic limit

Figure 7.3 shows variation in plastic limit of black cotton soil, it shows that maximum reduction in plastic limit was found with addition of 6% lime and 25% stone dust in black cotton soil.

7.9 Maximum Dry Density Results

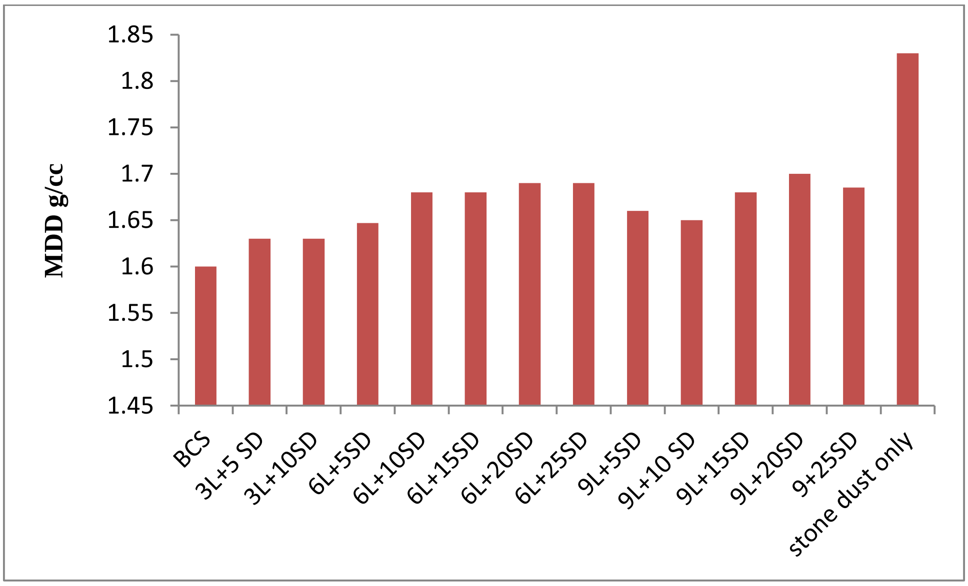


Figure7.4: Variation in Maximum dry density.

Figure 7.4 shows variation in maximum dry density of black cotton soil, it shows that maximum dry density is increased with addition of 6% lime and 25% stone dust in black cotton soil.

7.10 Optimum Moisture Content Results:

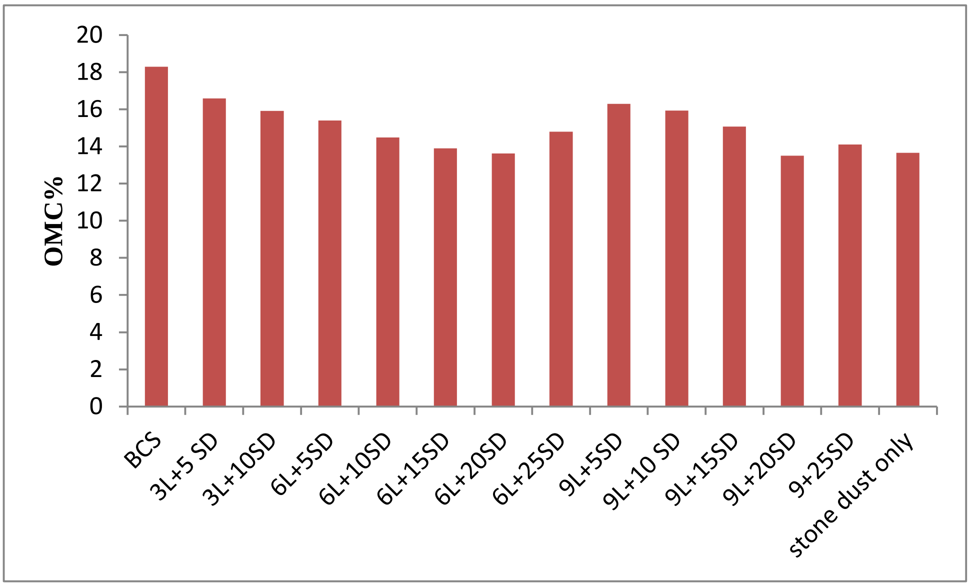


Figure7.5: Variation in Optimum Moisture Content.

Figure 7.5 shows variation optimum moisture content of black cotton soil, it shows that maximum reduction in OMC was found with addition of 6% lime and 20% stone dust in black cotton soil

7.11 California Bearing Ratio Results

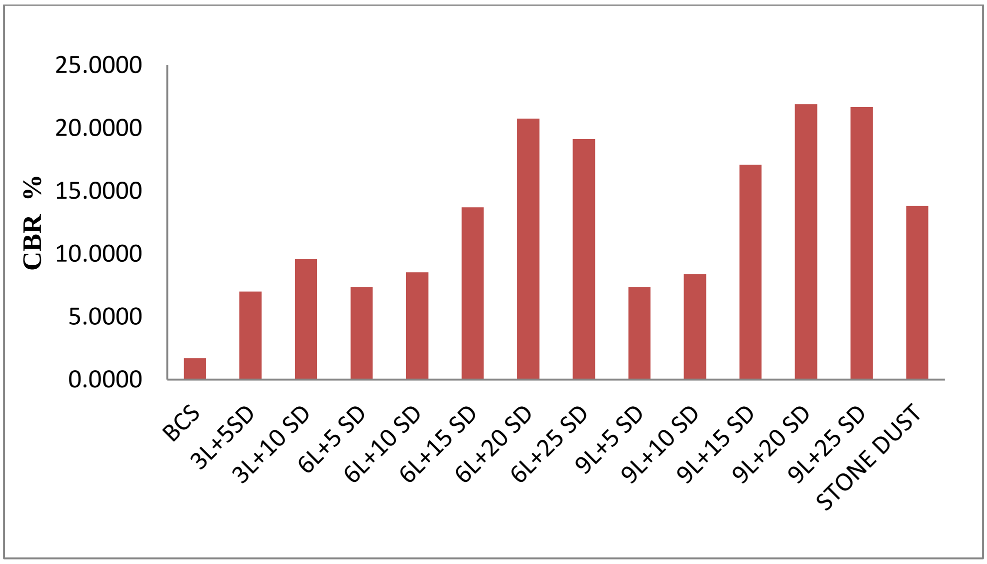


Figure7.6: CBR Results.

Figure 7.6 shows variation in CBR of black cotton soil, it shows that maximum CBR value was found with addition of 9% lime and 20% stone dust in black cotton soil

7.12 Unconfined Compression Strength Test Results

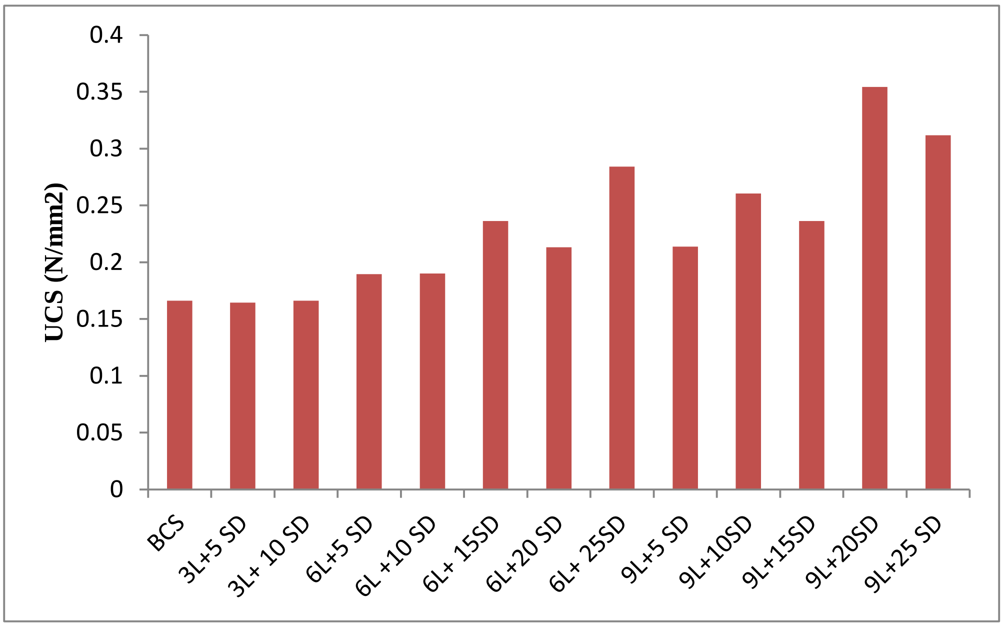


Figure7.7: UCS Test Results

Figure 7.7 shows variation in UCSof black cotton soil, it shows that maximum UCS value was found with addition of 9% lime and 20% stone dust in black cotton soil.

CHAPTER -8

CONCLUSIONS

CONCLUSIONS

The present study can serve as an effective method to utilize stone dust and lime in the stabilization of expansive soil. The conclusions are based on the tests carried out on various clay-stone dust and lime mixes selected for the same.

1. It has been seen that differential free swelling index and liquid limit decreases by adding lime and stone dust up to 6%lime & 25% stone dust, whereas further addition of admixtures increases it.
2. The optimum value of maximum dry density and unconfined compressive strength was found at 6% lime & 25 % stone dust.
3. Optimum moisture content was found gradually decreasing by adding admixtures and maximum reduction in OMC was found at 6% lime & 25 % stone dust.
4. Increase in plastic limit was very less up to addition of 3% lime & 5% stone dust further addition of admixtures plastic limit was gradually decreased up to 6% lime & stone dust and after addition soil was found non plastic.
5. Maximum CBR value was found at addition of 9% lime & 20% stone dust.
6. It was found that there is a maximum improvement in strength properties for the combination of lime and stone dust as compared to lime/stone dust individually. This helps to find an application for industrial waste to improve the properties of expansive soil both in embankments and pavement constructions.

So the optimum percentages of lime and stone dust were observed at 6% lime and 25 % stone dust for improving the properties of expansive soil. Stone dust and lime has good potential for use in geotechnical application of soils is a proven method to save time and money on construction projects. Lime drying of wet soils minimizes weather-related construction delays and permits the return to work within hours. Lime modification chemically transforms clay soils into friable, workable, compactable material. Stone dust and lime stabilization creates long-term chemical changes in unstable clay.

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