A Sustainable Methodology to Ameliorate Stabilization of Fine-grained Soils

Thesis

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Priyanka Agarwal

Pantnagar
January, 2021

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Authoress

CERTIFICATE

This is to certify that the thesis entitled **"A SUSTAINABLE METHODOLOGY TO AMELIORATE STABILIZATION OF FINE-GRAINED SOILS"** submitted in partial fulfilment of the requirements for the degree of **Master of Technology (Civil Engineering)** with major in **Soil Mechanics & Foundation Engineering** of the College of Post Graduate Studies, G.B. Pant University of Agriculture & Technology, Pantnagar, is a record of *bonafide* work carried out by **Ms. Priyanka Agarwal**, Id. No. **53892** under my supervision and no part of the thesis has been submitted for any other degree or diploma.

The assistance and help received during the course of this investigation have been acknowledged.

Pantnagar
January, 2021

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CERTIFICATE

We, the undersigned, member of the Advisory Committee of **Ms. Priyanka Agarwal** Id. No. **53892,** a candidate for the degree of **Master of Technology (Civil Engineering)** with major in **Soil Mechanics & Foundation Engineering** agree that the thesis entitled **"A SUSTAINABLE METHODOLOGY TO AMELIORATE STABILIZATION OF FINE-GRAINED SOILS"** may be submitted in partial fulfilment of the requirements for the degree.

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LIST OF ABBREVIATIONS

MDD	Maximum Dry Density
OMC	Optimum Moisture Content
CBR	California Bearing Ratio
UCS	Unconfined Compressive Strength
IS	Indian Standards
SD	Stone Dust
G	Specific Gravity
HDPE	High Density Polyethylene Fibre
MH	Inorganic Silts of High Plasticity
CH	Inorganic Clays of High Plasticity
SP	Poorly Graded Sands
CI	Clays of Medium Plasticity
CL	Clays of Low Plasticity
ML	Inorganic Silts of low Plasticity
SC	Clayey Sands
LL	Liquid Limit
PL	Plastic Limit
PI	Plasticity Index

Introduction

Chapter 1 INTRODUCTION

1.1 GENERAL

Due to rapid growth in urbanization and industrialization there has been a phenomenal increase in civic amenities and communication networks. But there is a shortage of suitable land for civil engineering activities because the soil available may be clayey, loamy and black cotton soil. It is also not necessary that the site conditions available may be satisfactory and strong enough to bear the load coming on it. As, the fine grained soils are the most problematic soils because different types of volumetric changes are observed, when acted as supported material below foundations. Hence, it is always a challenge for the designers to plan the structures on such soils. So, many researchers try to find solution to overcome this issue. And one such economic solution is soil stabilization, *i.e.*, to enhance the soil properties by introducing different additives in it.

The fine grained soils are stiff when they are dry but when they get saturated they give up their stiffness. Hence, soil stabilization is done when the exigencies of the poor quality of soil demanded it. Usually stabilization using fly ash and lime stones is selected but it is not favorable as these additives possess pozolanic property. The major drawback with these additives is that they are not coarser than the soil; therefore some of the geotechnical properties are left to be modified. But simply mixing coarser soil in such poor soil is also not a solution. Hence we need such additives which possess both pozolanic nature as well as coarser particles in them so that desirable strength is achieved in the existing soil.

Stone dust is such a material which possesses both pozolanic characteristics as well as coarser particles in it. Stone dust is obtained from stone crushing industries as a by-product. This is available to a generous extent of 200 million tonnes per annum. Since, this waste is disposed off without treatment therefore creates environmental problems.

Generous amount of waste is produced every year is India, disposal of which is very difficult due to high costs involved in its treatment. Theses wastes are dumped untreated at open sites or discharged into the rivers. If these wastes are not utilized they get accumulated with time, which can lead to many new problems. Therefore, utilization of these wastes becomes necessary. With the advancement in technology, researchers utilize these wastes to ameliorate the geotechnical properties of soil to make it suitable for construction activities. Utilization of waste materials such as rice husk ash, sawdust ash, fly ash, coir fibre, agro waste, rubber tyre waste, stone dust, brick ash dust, *etc*., are suggested by many researchers to enhance the soil properties to make it suitable for construction.

Many researchers in the past have studied modifications in geotechnical properties of soil and suitability of poor soils when mixed with stone dust. Various methodologies and testing techniques have been proposed for finding suitability and modification in characteristics of different types of soil when mixed with stone dust. Chetia *et al*. (2018) studied the effect on compaction characteristics of clay and suggested that these quarry dust can replace sand in construction activities and found MDD increases and OMC decreases with increase in quarry dust content of any gradation. Many researchers find the suitability of this additive in soil sub grade or sub base in flexible pavements by conducting tests on CBR of various soil samples. Abdulrasool (2015) and Visalakshi *et al.* (2018) study the strength properties of different soil when stone dust was added. Similarly many other researchers work on different properties like OMC, MDD, liquid and plastic limits of such soil when stone dust was used as additive. Besides, many other researchers also focused on improvisation of geotechnical properties of various soils when mixed with stone dust and other additives such as rice ask ash, fly ash, lime, polypropylene fibres *etc*. which are presented in chapter 2.

After studying various previous studies, it was observed that all such studies have focused only on some geotechnical properties like consistency limits, compaction characteristics, CBR and unconfined compressive strength and very little has been done so far on other geotechnical properties like permeability, shear strength parameters when fine grained soil is mixed with stone dust. The present study emphasizes on geotechnical properties such as compaction characteristics, permeability, shear strength parameters, CBR and unconfined compressive strength of clayey soil when mixed with stone dust.

1.2 STONE DUST: AN OVERVIEW

Stone dust is a waste material which is obtained enormously from crusher plants in India. Each crusher unit is estimated to produce 15-20% stone dust (Mishra *et al*. 2019). The disposal of this waste causes many geo environmental problems. A critical evaluation has been done by many researchers for the effective utilization of this waste alone or with some other additives with an intention to stabilize soil properties as the mineral composition of stone dust is comparable with sand and inertness of coarse grained structure (Chetia *et al*., 2018).

Stone dust possesses both pozzolanic and cementations properties. It is characterized by enhanced hardness and low expansion. Stone dust fulfils the primary function to provide strength to soil and consequently improving deformation characteristics of weak soil regions. It has also been identified to have high shear resistance and is beneficial to use it as a stabilizing material. It improves the density of soil as they consist of angular particles which have more interlocking strength when mixed with soil. Therefore, its applications are major in construction of sub base for roads, backfill, capping layer, *etc*.

Many researchers have reported its suitability in sub base of flexible pavements, as a replacement of sand, as a replacement of clay layer in core of earth dam, at RCC dam to replace filler, at urban waste site to prevent waste leachate seepage, as a replacement of fine aggregate in concrete, *etc*.

1.3 SCOPE OF THE STUDY

The main objective of this study is to analyze the modifications in the geotechnical properties of soil (clayey with low compressibility) when stabilized using stone dust (poorly graded sand). This study attempts to explain the effect of stone dust on selected soil when mixed in percentages varying from 0 to 50% (*i.e*., 0%, 10%, 20%, 30%, 40% and 50%) in order to predict the transformations in its strength, permeability, compaction characteristics and consistency.

In view of the above, the present study has been chosen, planned and carried out involving an extensive laboratory investigation, in order to specifically achieve the following objectives:

- A comprehensive review of previous studies on modification in geotechnical properties of soil using stone dust only and a brief review on the stabilization when soil mixed with stone dust and other additives like polypropylene fibres, rice husk ash, fly ash *etc*.
- Laboratory investigation of the index properties such as specific gravity, grain size distribution (with coefficient of uniformity and coefficient of curvature), consistency limits, *i.e*., liquid limit, plastic limit and plasticity index of both soil and stone dust and their mix samples.
- An evaluation of the classification of soil and stone dust samples based on the results obtained from laboratory investigation of the index properties.
- Laboratory study of the compaction characteristics of the samples using proctor tests.
- To study the effect on permeability of the soil when stone dust was mixed in varying percentages.
- A thorough experimental investigation of the shear strength parameters using direct shear test apparatus.
- A series of unconfined compression tests were performed to study the compressive strength of various samples prepared.
- California bearing ratio of soil samples were obtained by performing a series of CBR tests in the laboratory.

1.4 ORGANISATION OF THE THESIS

The thesis is divided into 5 chapters. The present chapter is intended to provide a general introduction to the research problem and structure of the thesis.

Chapter 2 presents an extensive overview of the various aspects relevant to the present study, with emphasis on the modifications in the geotechnical properties of soil when stone dust is added and also when stone dust is combined with other additives.

Chapter 3 describes the sampling areas of soil and stone dust, the specimen preparation, testing program and procedures involved.

Chapter 4 is devoted to the discussion of the experimental results obtained from various laboratory tests like particle size distribution test, consistency limit test, compaction test, permeability test, direct shear test, unconfined compression test and California bearing ratio test.

Finally the conclusions drawn on the basis of the results of previous sections and suggestions for further work are made in chapter 5, followed by the list of references.

Review of Literature

Chapter 2 REVIEW OF LITERATURE

2.1 GENERAL

Appropriate soil sites available for carrying out construction activities are very seldom occur in nature. This is because of the tremendous increase in civil engineering activities and the soil available may be clayey, loamy or black cotton soils and working on these soils is always puzzling for engineers as they show large volume changes when acted as supporting material below the foundations. Therefore, engineers find an economical solution to overcome this issue and hence suggest the idea of soil stabilization with an objective of modification in the geotechnical properties of weak soil deposits.

Many investigators have studied the effect of stone dust alone and with other additives on different geotechnical properties of soils (with different nature). Many geotechnical tests like consistency limit tests, proctor compaction tests, direct shear test, unconfined compressive strength test and California bearing ratio tests were performed but less emphasis was given on other soil properties like permeability and shear strength characteristics.

A critical review of literature concerning important aspects of effect of stone dust alone and with other additives on some geotechnical properties of soil is carried out. The various factors affecting compaction, strength, consistency, permeability and index properties of soil are reviewed to orient the present study on the effect of stone dust on weak soil in the light of conclusions drawn from them.

The present review of literature is concerned with the work done by researchers on effect of stone dust alone and with other admixtures on geotechnical properties of soil as listed below:

- Liquid limit
- Plastic limit
- Plasticity index
- Compaction characteristics

- Unconfined compressive strength
- California bearing ratio
- Effect on values of cohesion and angle of internal friction
- Permeability

2.2 EFFECT ON VARIOUS GEOTECHNICAL PROPERTIES OF SOILS BY ADDING STONE DUST ALONE AND WITH OTHER ADDITIVES

2.2.1 ALTERATIONS IN LIQUID LIMIT

Liquid limit is the minimum water content at which the soil is in liquid state, but has negligible shearing strength. Most research carried out till date reflects that liquid limit will show a decrement trend with increase in the percentage of stone dust.

Muley *et al*. (2010) reported that the value of liquid limit decreases as 41%, 36%, 29% and 28% respectively with increase in stone dust percentage to 0%, 10%, 20% and 30%, when mixed with soil (SC). Similarly, Muley and Jain (2013) in a study on effect of stone dust on different soils like black cotton soil, yellow soil and red murrum observed that the value of liquid limit was showing a declining trend when stone dust was mixed in percentages 0%, 10%, 20% and 30% with three different soils. Black cotton soil classified as MH shows a decreasing trend of liquid limit (56%, 48%, 42% and 30%) when stone dust was used as additive mixed in different percentages (0%, 10%, 20% and 30%) in a study done by Bshara *et al*. (2014). LL values were reduced by adding 0, 5, 10, 15 and 20 percentage of quarry dust respectively according to a study done by Visalakshi *et al*. (2018) on expansive soils (CH) when treated with quarry dust. Rana *et al*. (2018) also observed a decreasing trend in liquid limit when low plastic to low compressible soils were treated with crusher dust (SP). Al Rawi *et al*. (2018) observe a decreasing trend when sand additive was mixed in percentages 0, 2, 5, 10 and 20% with the fine grained soil. When bentonite soil was treated with stone dust in percentages 5, 10, 15, 20 and 25%, the value of liquid limit was observed to decrease by Jat and Purohit (2017). Indiramma and Sudharani (2016) observed a decreasing trend in LL value when CI soil was mixed with quarry dust. According to one more study on expansive soil treated with stone dust, Dixit and Patil (2016) found a decreasing trend in the value of liquid limit as 51, 44, 37, 30 and 26 when stone dust was added in 10, 20, 30, 40 and 50% respectively. Agarwal (2015) on the basis of his experimental study observed a decreasing trend in the value of liquid limit when CL soil was mixed with stone dust (SP). Venkateswarlu *et al*. (2015) in his study on expansive soil (classified as CH) treated with quarry dust showed a decreasing trend in the liquid limit values when quarry dust was mixed in percentages 0, 5, 10 and 15%.

2.2.2 MODIFICATION IN VALUES OF PLASTIC LIMIT

The water content at which a soil will begin to crumble when rolled into a thread of approximately 3 mm in diameter is referred as plastic limit. Moreover, plastic limit also shows a decreasing trend after reviewing various studies done by many researchers in the past.

Muley *et al*. (2010) examined that the value of plastic limit decreases as 19%, 19%, 18% and 17% relatively with increase in stone dust percentage 0%, 10%, 20% and 30%, when mixed with soil (SC). Similarly, Muley and Jain (2013) in a study on effect of stone dust on different soils like black cotton soil, yellow soil and red murrum observed that the value of plastic limit was showing a decreasing trend when stone dust was mixed in percentages 0%, 10%, 20% and 30% with three different soils. Black cotton soil classified as MH showed a decreasing trend of plastic limit when stone dust was used as additive mixed in different percentages (0%, 10%, 20% and 30%) in a study done by Bshara *et al*. (2014). PL values were reduced by adding 0, 5, 10, 15 and 20 percentage of quarry dust respectively according to a study done by Visalakshi *et al*. (2018) on expansive soils (CH) when treated with quarry dust. Rana *et al*. (2018) also observed a decreasing trend in plastic limit when low plastic to low compressible soils were treated with crusher dust (SP). Al Rawi *et al*. (2018) observed a decreasing trend of plastic limit when sand additive was mixed in percentages 0, 2, 5, 10 and 20% with the fine grained soil. When bentonite soil was treated with stone dust in percentages 5, 10, 15, 20 and 25%, the value of plastic limit was decreased as 22, 20, 17, 15 and 13% respectively (Jat and Purohit 2017). Indiramma and Sudharani (2016) observed a decreasing trend in PL value when CI soil was mixed with quarry dust. According to one more study on expansive soil treated with stone dust Dixit and Patil (2016) observed a decreasing trend in the value of plastic limit as 25, 23, 19, 17 and 15 when stone dust was added in 10, 20, 30, 40 and 50% respectively. Agarwal (2015) after conducting his experimental work observed a decreasing trend in the value of plastic limit when CL soil was mixed with stone dust (SP). Venkateswarlu *et al*. (2015) in his study on expansive soil (classified as CH) treated with quarry dust also showed a decreasing trend in the plastic limit value when quarry dust was mixed in percentages 0, 5, 10 and 15%.

2.2.3 EFFECT ON PLASTICITY INDEX

When liquid limit and plastic limit of soil decreases plasticity index also decreases as it is the numeric difference between liquid and plastic limit of soil. As studied in the previous sections liquid and plastic limit of soil shows a decreasing trend, hence, their plasticity index values will also decreases. A brief review of plasticity index done by the same researchers in above sections is presented below.

Muley *et al*. (2010) examined that the value of plasticity index decreases with increase in stone dust percentage from 0%, 10%, 20% and 30%, when mixed with soil (SC). Similarly, Muley and Jain (2013) in a study on effect of stone dust on different soils like black cotton soil, yellow soil and red murrum observed that the value of plasticity index was showing a decreasing trend when stone dust was mixed in percentages 0%, 10%, 20% and 30% with three different soil. Black cotton soil classified as MH shows a decreasing trend of plasticity index when stone dust was used as additive mixed in different percentages (0%, 10%, 20% and 30%) in a study done by Bshara *et al*. (2014). Rana *et al*. (2018) also observed a decreasing trend in plasticity index when low plastic to low compressible soils were treated with crusher dust (SP). Al Rawi *et al*. (2018) observed a decreasing trend of plasticity index when sand additive was mixed in percentages 0, 2, 5, 10 and 20% with the fine grained soil. When bentonite soil was treated with stone dust in percentages 5, 10, 15, 20 and 25%, the value of plasticity index was decreased as 76, 35, 28, 25 and 26% respectively (Jat and Purohit 2017). Indiramma and Sudharani (2016) observed a decreasing trend in PI value when CI soil was mixed with quarry dust. According to one more study on expansive soil treated with stone dust Dixit and Patil (2016) observed a decreasing trend in the value of plasticity index when stone dust was added in 10, 20, 30, 40 and 50% respectively. Agarwal (2015) after conducting his experimental work observed a decreasing trend in the value of plasticity index when CL soil was mixed with stone dust (SP).

2.2.4 IMPACT ON COMPACTION CHARACTERSTICS

For reducing the porosity of soil compaction is carried out. Compaction helps to increase the shear strength of soil and decrease its consolidation and permeability characteristics. Optimum moisture content and maximum dry density are such consolidation parameters whose practical importance is considered in densification of soil, such as in embankments, highway and railway subgrades and in foundations. Extensive work has been done by previous researchers on compaction characteristics of soil treated with stone dust.

Chetia *et al*. (2018) investigated the influence of quarry dust addition of different gradations on the compaction behavior of clay of different compressibility and concluded that for all clay- sand mixtures, all clay quarry dust mixes and bentonite quarry dust mixes, OMC decreases while MDD increases despite of the fact that the soil used was of intermediate or high compressible and the quarry dust used was of different gradations and also suggest that the quarry dust can replace sands in construction engineering activities and also evaluate the optimum range of quarry dust content to be 40-60%. After mixing soil (SC) with stone dust Muley *et al*. (2010) observed that the value of OMC decreases and the value of MDD increases as the stone dust were mixed as 0, 10, 20 and 30% respectively. Muley and Jain (2013) in their another study on mixing stone dust with three different soils black cotton soil (MH), yellow soil (MH) and red murrum observed that as the percentage of stone dust increases the value of OMC decreases while the value of MDD increases with stone dust mixed as 0, 10, 20 and 30%. Bshara *et al*. (2014) in her study on effect of stone dust on poor soil (MH) evaluated that the value of OMC decreases while the value of MDD increases from 1.45 to 1.80 g/cc with increases in the percentage of stone dust from 0 to 30%. Ganie *et al*. (2017) observed a decreasing trend in OMC while an increasing trend in MDD of soil (CL) when stone dust mixed in percentage varying from 0 to 50% in increment of 10. Jat and Purohit (2017) concluded that OMC decreases while MDD increases when stone dust was added in bentonite soil in percentages varying from 5 to 25% in increment of 5. When stone dust was added in murrum in percentages 10, 15, 20, 25 and 30% Mahent and Joshi (2015) observed that the value of MDD increases while OMC decreases. Addition of crusher dust also increases the value of MDD even when mixed in high plastic gravel soils as studied by Satyanarayana *et al*. (2013). Mishra *et al*. (2019) in his study done on different soils classified as CL, ML, CL-ML stabilized using stone dust and coarse aggregate observed that OMC decreases while MDD increases when stone dust mixed from 10 to 30% alone and also with coarse aggregates. Purushottam and Malviya (2019) observed decrement in OMC and increment in MDD when black cotton soil was treated using stone dust and sisal fibre. Addition of different ratio of stone dust to the black cotton soil stabilized with optimum ratio of rice husk ash, the MDD goes on increasing and OMC goes on decreasing was observed by Manjunath (2015). Phonsa and Singh (2019) examined that MDD increases when clayey soil was stabilized using stone dust and plastic bottle strips. Sabat (2012) concluded that MDD increases when expansive soils were treated using lime and quarry dust mixes. Ramdas *et al*. (2010) and Rajput *et al*. (2019) both studied that MDD increases and OMC decreases when expansive soils were treated using stone dust and fly ash.

2.2.5 VARIATIONS IN UNCONFINED COMPRESSIVE STRENGTH

The unconfined compression test is one of the simplest and quickest tests used for the determination of shear strength of cohesive soil. Immense work has been done by many researchers on UCS property in the past.

Visalakshi *et al*. (2018) observed that the UCS value for untreated expansive soil is 130 kPa and increased by 138 kPa for 1 day, 142 kPa for 7 days, 144 kPa for 14 days and 146 kPa for 28 days curing respectively at 10% quarry dust and then its value decreases when percentage of quarry dust increases up to 20%. Deepak and Bishnoi (2019) observed that by adding Kota stone dust UCS value increases from 0.213 to 0.274 MPa but after more than 4% stone dust added then strength did not increase. Sand additives showed a general increase in the UCS, however, higher ratios of sand additives showed a marginal increase in the values (Al Rawi *et al*. 2018). Indiramma and Sudharani (2016) studied that the value of UCS increases when quarry dust was mixed with expansive soils. UCS increases both in soaked and unsoaked conditions when stone powder was mixed in percentages 1, 3 and 5% in clayey soil, as studied by Abdulrasool (2015). Manjunath (2015) observed that UCS improved nearly by 103% of the virgin soil after adding 15% stone dust, adding more stone dust reduces the UCS of the black cotton soil. Sabat (2012) observed a decrease in UCS at 5% lime addition.

2.2.6 EFFECT ON CALIFORNIA BEARING RATIO

CBR is an important parameter which is needed in construction stage of a pavement in order to check the quality of the component pavement layers. Effect of stone dust on CBR studied by many researchers is presented below.

Muley *et al*. (2010) observed an increment in CBR from 10.94% to 25.56% when stone dust was added from 0 to 30% to murrum. Muley and Jain (2013) in their another study found that CBR increases when stone dust was added with three different soils i.e. black cotton soil, yellow soil and red murrum. Bshara *et al*. (2014) after studying effect of stone dust (mixed in 0, 10, 20 and 30%) on black cotton soil concluded that CBR increases with increase in the percentage of stone dust and found that it increased almost twice to its initial value and hence quality of poor soil for road construction can be improved by using stone dust as additive. Soil classified as CL when mixed with stone dust, Ganie *et al*. (2017) observed that CBR improves as percentage of stone dust increases but it happens only till 30% addition of stone dust and after the value of CBR decreases. When cusher dust was mixed with gravely soils, CBR increases with addition of crusher dust, Rana *et al*. (2018) also found that high plastic soils and intermediate compressible soils require high percentage (30-35%) of crushed dust while medium to low plastic and low compressible soils require 15-25% of crushed dust to meet the specifications of MORTH as a sub-base material. Satyanarayan *et al*. (2013) and Mahent and Joshi (2015) analysed that CBR improves when gravely soils were mixed with increased percentage of stone dust. When expansive soils were treated with quarry dust, CBR improves as the mixing percentage of stone dust increases, observed by Venkateswarlu *et al*. (2015), Dixit and Patil (2016) and Visalakshi *et al*. (2018). Jat and Purohit (2017) in a study on stabilization of bentonite soil with stone dust evaluated that on addition of stone dust, the value of unsoaked CBR increased 20% addition of stone dust and afterwards its value decreases with addition of more stone dust in soil. Mishra *et al*. (2019) observed that the CBR value significantly and the desired value of minimum CBR of 8% under soaked condition was obtained by adding to the soil 30% of stone dust, or 20% of 10mm size coarse aggregates, or with both 10% stone dust + 10% coarse aggregate and 10% stone dust + 20% coarse aggregates. Purushottam and Malviya (2019) observed in their study that the CBR value of black cotton soil improved when stone dust and sisal fibre were added. Patidar and Mahiyar (2014) in their study on stabilization of black cotton soil using additive mixture of HDPE wastage fibres, stone dust and lime observed that CBR was maximum with combination of fibre (1%), stone dust (10%) and lime (6%). Nabil Al-Joulani (2012) studied the effect on fine soils when stone powder and lime was used as additive and concluded that the addition of 30% stone powder and lime has increased the CBR value from 5.2% to 16% and 18% respectively and also evaluated that the increase in CBR values due to stone powder and lime caused a reduction in the flexible pavement thickness by 47% and 55% respectively. Ramdas *et al.* (2010) and Rajput *et al.* (2019) after studying effect of stone dust and fly ash on expansive soils found improvement in the value of CBR. Suresh *et al*. (2009) and Tiwari *et al*. (2016) found that the value of CBR increases when black cotton soil was treated using stone dust and polypropylene fibres. Pandey and Saw (2019) and Dutta and Sarda (2007) found improvement in CBR with addition of stone dust and waste plastic strips.

2.2.7 TRANSFORMATIONS IN SHEAR STRENGTH PARAMETERS

The stability of slopes depends upon the shearing resistance offered by the soil, hence it is necessary to find the shear strength parameters *i.e.* angle of internal friction and cohesion of the soil. As our present thesis deals with the stabilization of soil using stone dust, hence a review has been done on effect on shear strength properties of soil when mixed with stone dust by some past researchers. However, only a little work has been done on shear strength parameters of soils which are discussed below.

Al Rawi *et al*. (2018) evaluated that after mixing sand additive in fine grained soils from 0 to 20%, cohesion decreases while angle of internal friction increases. Mixing expansive soil with quarry dust, Venkateswarlu *et al*. (2015) reported that the value of cohesion goes on decreasing when the percentage of quarry dust increases from 0 to 15%. Nabil Al-Joulani (2012) found that value of angle of internal friction increased by 50% and cohesion decreased by about 64% when 30% stone powder was mixed and also observed that the addition of 30% of lime has decreased the friction angle and cohesion about 57% and 28% respectively when he stabilized fine soils using stone powder and lime. Pandey and Saw (2019) after working on improvement in soil properties using stone dust and plastic wastes observed that both the value of cohesion and angle of internal friction increases.

2.2.8 CHANGES IN COEFFICIENT OF PERMEABILITY

Permeability may be defined as the ability of water to pass through the soil. It is one of the important parameter considered during the designing of hydraulic structures. However limited work has been done on it which is discussed below.

Al Rawi *et al*. (2018) observed that the coefficient of permeability increases from 2.17×10^{-7} to 1.24×10^{-6} cm/sec when sand was mixed in fine grained soil from 2 to 20%. Patidar and Mahiyar (2014) observed that k decreases when stone dust was added from 5 to 15% and also concluded that k decreases when lime was added but increases when HDPE waste fibres were added. When coarse grained soil was treated using cement kiln dust, silica fumes and lime Saidik (2018) observed that the coefficient of permeability decreases with increase in percentage of additives. Islam *et al*. (2018) observed that permeability decreases with addition of fly ash and lime when mixed with two different types of soil like clay of low plasticity and fine sand.

2.3 CONCLUDING REMARKS

Based on the comprehensive review and analysis of the literature available on similar studies, the following may be concluded:

- Very finite study is available on permeability characteristics of fine grained soil, when treated with stone dust. Hence more evaluation is required in this analysis in order to verify the suitability of stone dust for hydraulic structures.
- Limited evaluation has been done on shear strength parameters of soil when mixed with stone dust. Shear strength parameters are important in checking the stability of slopes, hence more work is required to analyse the effect of stone dust on these parameters also.

- The above analysis revealed that with addition of stone dust liquid limit decreases, plastic limit decreases and plasticity index also decreases.
- Stone dust addition helps to achieve high MDD and hence compaction increases and thus strength of soil increases.
- Above literature also reflects the improvement in CBR with addition of stone dust and also suggest that stone dust can be used in soil sub grades in flexible pavements.
- The overall conclusion from the literature review is that addition of stone dust is advantageous as many geotechnical properties of soil are improved on its addition and can be used as a replacement in place of sand for fine grained soil and its utilization in civil engineering properties is economical.

Materials and Methods

Chapter 3 MATERIALS AND METHODS

3.1 GENERAL

This chapter includes the brief description of the materials and methodology used to achieve the research objectives. As stated earlier, this study emphasizes the effect of adding different percentages of stone dust in local soil in order to find out the modifications in its geotechnical properties. The following points are covered in this chapter.

- Soil preparation and identification of its geotechnical properties.
- Testing of stone dust to analyze its properties.
- Mixing of stone dust to the soil for conducting various tests, such as Specific Gravity test, Consistency limit test, Proctor Compaction test, Direct Shear test, Permeability test, Unconfined Compression test and CBR test.

3.2 MATERIALS USED

Soil sample, stone dusts were collected from the specified sources which were used as the main materials for conducting the study.

3.2.1 SOIL

As a part of this investigation, the soil was acquired from the campus of G.B Pant University of Agriculture & Technology, Pantnagar, Uttarakhand, India. Initially, the topmost layer of the soil was removed up to 60cm depth. The soil thus obtained was carried to the laboratory in sacks and then various tests were performed, such as liquid limit, plastic limit, standard proctor test, CBR test, permeability test, UCS test. Fig. 3.1 shows a typical sample of this material.

Fig. 3.1 Sample of Soil used in the study

3.2.2 STONE DUST

It is a material which is obtained from crusher plants as waste. For the purpose of investigation in this study, the stone dust was obtained from Pal stone crusher Pvt. Limited Lalkuan (Uttarakhand). The physical and geotechnical characteristics of stone dust was obtained after performing various tests like specific gravity test, grain size distribution, consistency limit tests, compaction characteristics, soaked CBR and shear strength parameters. As per IS: 2720 (Part 4)-1985, it was observed that the stone dust comes in the category of SP. Fig. 3.2 shows a typical sample of this material. For present investigation, the different proportions of stone dust were used (10%, 20%, 30%, 40% & 50%) on soil modification.

Fig. 3.2 Sample of Stone Dust used in the study

3.3 METHODOLOGY ADOPTED

To evaluate the effect of stone dust as a stabilizing additive in soil, series of tests were conducted. In this study stone dust was taken in various percentages as 10%, 20%, 30%, 40% and 50%. The Indian Standard codes were followed during the conduction of the following experiments:

- Standard proctor test – IS : 2720 (Part 7) - 1980
- Unconfined compressive strength (UCS) test – IS : 2720 (Part 10) - 1991
- California bearing ratio (CBR) test – IS : 2720 (Part 16) - 1987

- Liquid & Plastic limit test – IS 2720 (Part 5) – 1985
- Direct shear test- IS: 2720 (Part 13)-1986
- Permeability test- IS 2720- (Part 17)-1986
- Particle size distribution- IS: 2720 (Part 4)-1985
- Classification of soil- IS: 1498-1970

The overall testing programme was carried out in three phases.

In the first phase tests were conducted on the soil. In the second phase a series of tests were performed on the stone dust. In the third and last phase, different percentages of stone dust were added in virgin soil.

The various tests performed in all the three phases are listed in Table 3.1

3.3.1 DETERMINATION OF SPECIFIC GRAVITY

The specific gravity of soil, stone dust and soil-stone dust mixes were determined as per IS: 2720 (Part 3)-1980 using a pycnometer of one litre capacity with a conical brass cap screwed at its top. Specific gravity was calculated from the following equation:-

$$\boldsymbol{G} = \frac{[\boldsymbol{M_2} - \boldsymbol{M_1}]}{[(\boldsymbol{M_2} - \boldsymbol{M_1}) - (\boldsymbol{M_3} - \boldsymbol{M_4})]}$$

Where,

M_1= Mass of pycnometer

M_2= Mass of pycnometer and dry soil

M_3= Mass of pycnometer, dry soil and water

M_4= Mass of pycnometer and water

Table 3.1 Experimental Programme

S.No.	Sample	Tests conducted
1	Soil	Particle size distribution test Atterberg limit test Specific gravity test Proctor compaction test California bearing ratio test Unconfined compressive strength test Permeability test
2	Stone Dust	Particle size distribution test Atterberg limit test Specific gravity test Proctor compaction test California bearing ratio test Unconfined compressive strength test
3	Soil + 10% SD Soil + 20% SD Soil + 30% SD Soil + 40% SD Soil + 50% SD	Atterberg limit test Specific gravity test Proctor compaction test California bearing ratio test Unconfined compressive strength test Permeability test

3.3.2 PARTICLE SIZE DISTRIBUTION ANALYSIS

As per IS: 2720 (Part 4)-1985, grain size analysis was performed on soil and stone dust. The grain size distribution of coarse gained soil and stone dust is determined through sieve analysis using a set of sieves consisting of 4.75mm, 2mm, 1mm, 600micron, 425micron, 300micron, 150micron, 75micron (Fig. 3.3). An oven dried sample of soil is taken. The various sieves are arranged one over the other in the order of their mesh openings. A receiver or pan is kept at the bottom and a cover plate is kept at the top of the whole assembly. The soil sample is put on the whole sieve and the whole assembly is subjected to mechanical shaking. At least 10min shaking is desirable. The portion of the soil retained on each sieve is weighed. The grain size distribution of the fine grained soil is determined through hydrometer analysis. A fraction of fine grained soil was mixed with distilled water to make 1000ml of suspension and a hydrometer is used to measure the density of the soil-water suspension at different time intervals. The time density data, recorded over a few days, is translated into grain size and percentage finer than different sizes.

Fig. 3.3 Sieve Set for Particle Size Distribution Analysis

3.3.3 DETERMINATION OF CONSISTENCY LIMITS

Liquid limit and Plastic limit tests were conducted with reference to IS: 2720 (Part 5)-1985. Standard liquid limit apparatus (Fig. 3.4) consists of a hard rubber base over which a brass cup drops through a desired height. The height of fall of the cup can be adjusted with the help of adjusting screws.

Fig. 3.4 Casagrande Liquid Limit Apparatus

The liquid limit (LL) is defined as the water content in percent at which a part of soil in a standard cup and cut by a grooving tool of standard dimensions will flow together at the base of the groove for a distance of 13mm when subjected to 25 number of blows from the cup being dropped 10mm in a standard liquid limit apparatus operated at a rate of two blows per second. The plastic limit (PL) is the water content in percent at which a soil can no longer be deformed by rolling into approximately 3.0mm diameter threads without crumbling. Plasticity index is defined as the range of moisture content over which a soil possesses plasticity. On the basis of plasticity index various soils are classified and is presented in Table 3.2. It can be determined by following relation:

$$I_p = w_l - w_p$$

Where,

w_l= liquid limit in percentage

w_p= plastic limit in percentage

I_p= plasticity index in percentage

Table 3.2 Types of Soil on the basis of Plasticity Index

Plasticity Index	Soil Description
0	Non-plastic
1-5	Slight plastic
5-10	Low plastic
10-20	Medium plastic
20-40	Highly plastic
More than 40	Very high plastic

3.3.4 PROCEDURE FOR STANDARD PROCTOR COMPACTION TEST

To find the relationship between the moisture content and maximum dry density of samples Proctor Compaction test was performed. For this, Standard Proctor test was adopted form IS: 2720 (Part 7)-1980. The test is performed using 3 kg of the soil sample, compacted in three equal layers in 1000ml volume mould. Each layer was given 25 uniformly distributed blows using 2.6 kg rammer falling through a height of 310 mm. Fig. 3.5 shows the sample preparation for compaction test.

Fig. 3.5 Proctor Mould with Rammer

3.3.5 DIRECT SHEAR TEST PROCEDURE

This test is used to determine shear strength parameters of the soil samples at known density and moisture content. The shear strength parameters were determined according to IS: 2720 (Part 13)-1986. The samples were prepared in the size of 60mmx 60mm x 25mm and sheared at the rate of 1.25mm/minute.

The samples were compacted at the maximum dry density and optimum moisture content in the specimen cutter and then transferred into the shear box after fixing the two halves of the shear box together by means of the fixing screws. The shear box with the specimen was fitted into the position in load frame. The required normal stress was applied and the rate of shear stress applications so adjusted that no drainage could occur in the sample during the test. The test was conducted by applying horizontal shear load up till failure. The shear load readings indicated by the proving ring assembly and the corresponding longitudinal displacements were noted at regular intervals. Fig. 3.6 shows the apparatus for direct shear test.

Fig. 3.6 Standard Direct Shear Test Equipment

3.3.6 PROCEDURE FOR OBTAINING UNCONFINED COMPRESSIVE STRENGTH

The UCS test is the most popular and fastest method for determining soil shear strength. UCS is defined as the ratio of failure load to cross sectional area of the soil sample. This test is undrained, as the rate of applying load is so fast that no pore water is drained out and the pore water pressure do not dissipate.

The standard procedure was adopted from IS: 2720 (Part 10)-1991. The cylindrical specimen was prepared at OMC in the metallic split mould of dimension 38mm (diameter) x 76mm (height). These specimens were tested with strain rate of 0.5 to 2% per minute. Fig. 3.7 shows the unified compression testing machine with specimen.

Fig. 3.7 Standard Unconfined Shear Test Machine

3.3.7 DETERMINATION OF CALIFORNIA BEARING RATIO

The standard procedure for the CBR test was adopted from IS: 2720 (Part XVI)-1979. It is essential to determine during designing of pavement thickness.

Soaked CBR tests were conducted on soil, stone dust and soil-stone dust mixes, the samples of which were prepared at the optimum moisture content. The displacer disc of 148mm diameter was placed in the mould and the sample was filled in three layers with 55 blows given to each layer uniformly by a hammer of 2.5kg. The displacer disc was removed after the compaction and a surcharge weight of 2.5 kg was kept on the soil after reversing the mould. The mould with sample was immersed in water for 4 days. Mould with sample was tested in the CBR testing machine. The load was recorded using a mechanical loading machine, equipped with a base moving at a uniform rate of 1.25mm/min and a calibrated proving ring. A piston attached with the proving ring is allowed to penetrate in the compacted specimen. Diameter of piston is 50mm. The stress and strain dial gauges were set to zero and then the load was applied. The load was recorded up to a penetration of 7.5mm with a dial gauge of least cont 0.01mm.Fig. 3.8 shows the CBR testing machine.

Fig. 3.8 CBR Apparatus with Mould

3.3.8 PROCEDURE FOR DETERMINING COEFFICIENT OF PERMEABILITY

Hydraulic conductivity or permeability (k) is a crucial characteristic of soil in relation to drainage function. It is a function of several parameters such as the type of soil, its gradation, chemical composition, degree of saturation and the stress distribution pattern within the soil.

Permeability test in the laboratory can be carried out either by falling head method or by constant head method for fine and coarse soils respectively. As per IS: 2720 (Part 17)-1986, determination of permeability is done. Fig. 3.9 shows permeability test apparatus.

Fig. 3.9 Permeability Test Apparatus

As the soil in this case is fine in nature, hence the permeability has been determined by using falling head test and is determined by using standard formula i.e. Coefficient of permeability,

$$\boldsymbol{k = \frac{aL}{At} \log_e \frac{h_1}{h_2}}$$

Here,

a= cross sectional area of stand pipe

L= effective length of sample

A= cross sectional area of sample

t= time interval to fall the head from h_1 to h_2

h_1= initial height of water in the pipe above the outlet

h_2= final height of water in the pipe above the outlet

Typical values of hydraulic conductivity for various soils is shown in the Table 3.3

Table3.3 Coefficient of Permeability for Various Types of soil

Type of Soil	Coefficient of Permeability, k (cm/sec)	Remarks
Gravel	k> 1.00 x 10^{-01}	Very High
Coarse Sand, Fine Sand	1.00 x 10^{-03} <k< 1.00 x 10^{-01}	High to Medium
Silty Sand	1.00 x 10^{-05} <k< 1.00 x 10^{-03}	Low
Silt, Silty Clay	1.00 x 10^{-07} <k< 1.00 x 10^{-05}	Very Low
Clay	k< 1.00 x 10^{-07}	Tends to Impermeable

3.4 CONCLUDING REMARKS

The experiments so discussed in this chapter is carried out in accordance with IS procedures in order to fulfil the objective of this investigation. The results so obtained by conducting different types of tests on the soil and soil mixed with different percentages of stone dust are presented and discussed in the following chapter.

Results & Discussion

4.1 GENERAL

This chapter presents the results of laboratory tests such as specific gravity, consistency limits, standard compaction test, direct shear tests, unconfined compressive strength tests, California bearing ratio test and permeability tests, which were carried on the soil mixed with different percentages of stone dust. Further, these results are summarized in tables and have been discussed for drawing conclusions.

4.2 RESULTS ON GEOTECHNICAL PROPERTIES OF STONE DUST

4.2.1 SPECIFIC GRAVITY

It was determined according to IS: 2720 (Part III/Sec I)-1980. Specific gravity is reported by taking an average of three samples. The specific gravity of stone dust comes out to be 2.6.

4.2.2 GRAIN SIZE ANALYSIS

According to IS: 2720 (Part IV) 1985, grain size analysis was carried out. Stone dust comprises of 20.3% fine sand, 52.6% medium sand and 23.4% coarse sand. The coefficient of uniformity (C_u) of stone dust is 7.5 and coefficient of curvature (C_c) is found as 0.867. As C_u is greater than 6 and C_c is less than 1, it is concluded that the stone dust used is poorly graded sand (SP). Particle size distribution of stone dust is shown in Fig. 4.1.

4.2.3 CONSISTENCY LIMITS

The consistency limits were performed as per IS: 2720 (Part V) 1985. It is not possible to determine liquid and plastic limit of stone dust as it does not show any plastic behaviour. Hence, the plasticity index (I_p) of stone dust is reported as non-plastic (NP).

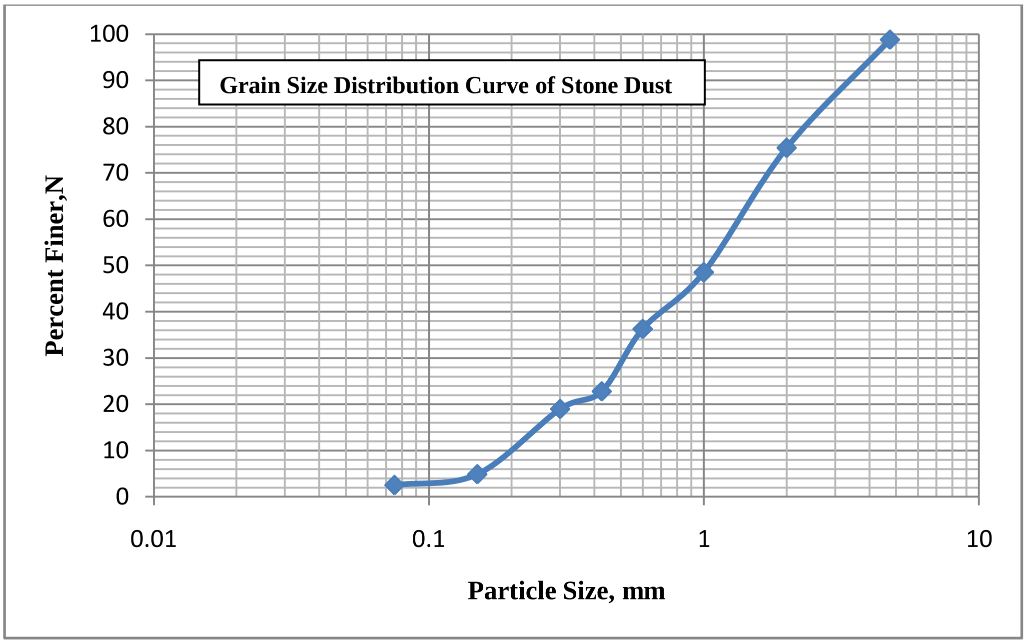

Fig. 4.1 Grain Size Distribution of Stone Dust

4.2.4 OMC AND MDD OF STONE DUST

Standard proctor test was conducted as per IS: 2720 (Part VII) 1980 to determine OMC and MDD of stone dust. From the test the optimum moisture content (OMC) and maximum dry density (MDD) was found as 12% and 1.87g/cm^3 respectively. The graph of compaction curve is shown in Fig. 4.2

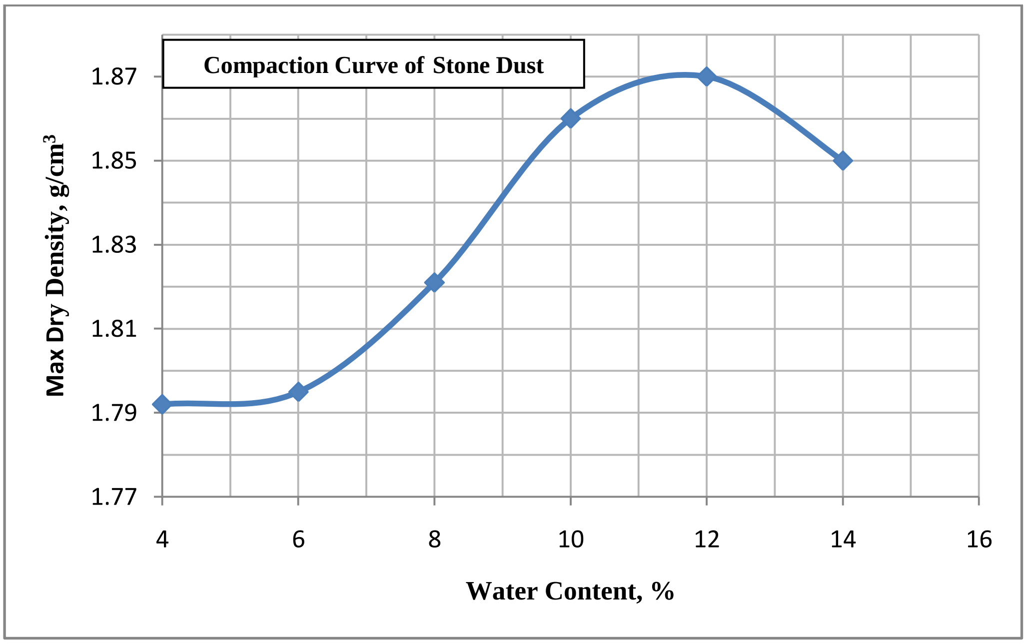

Fig. 4.2 Compaction Characteristics of Stone Dust

4.2.5 COHESION AND ANGLE OF INTERNAL FRICTION

Direct shear test was carried out. The angle of internal friction and cohesion value of stone dust was determined to be 32.94° and 0.068 respectively. The curve for direct shear test is shown in Fig. 4.3.

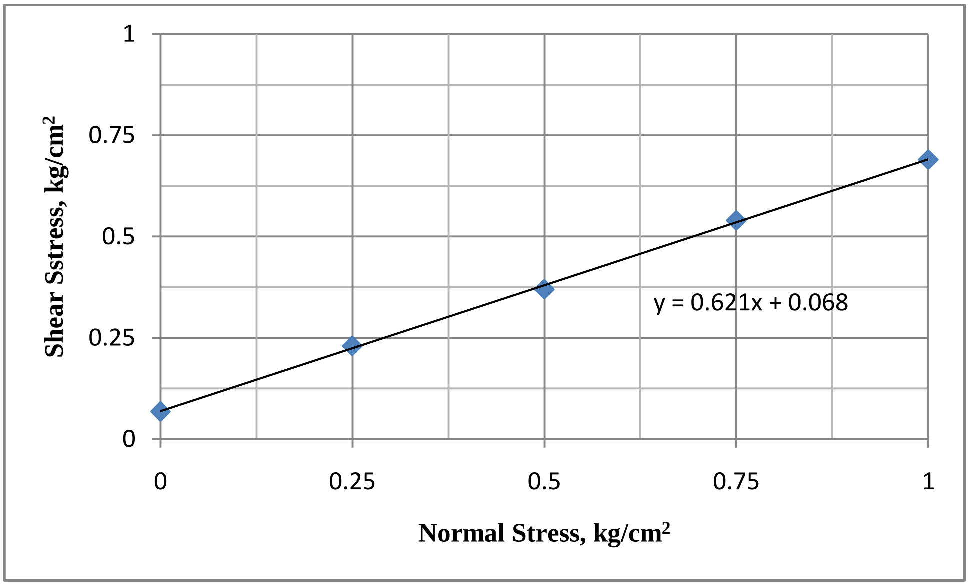

Fig. 4.3 Normal stress and Shear stress plot of stone dust

4.2.6 CALIFORNIA BEARING RATIO

The soaked value of CBR value of stone dust was determined to be 9.16. The plot for load dial reading v/s penetration graph for soaked CBR is shown in Fig. 4.4.

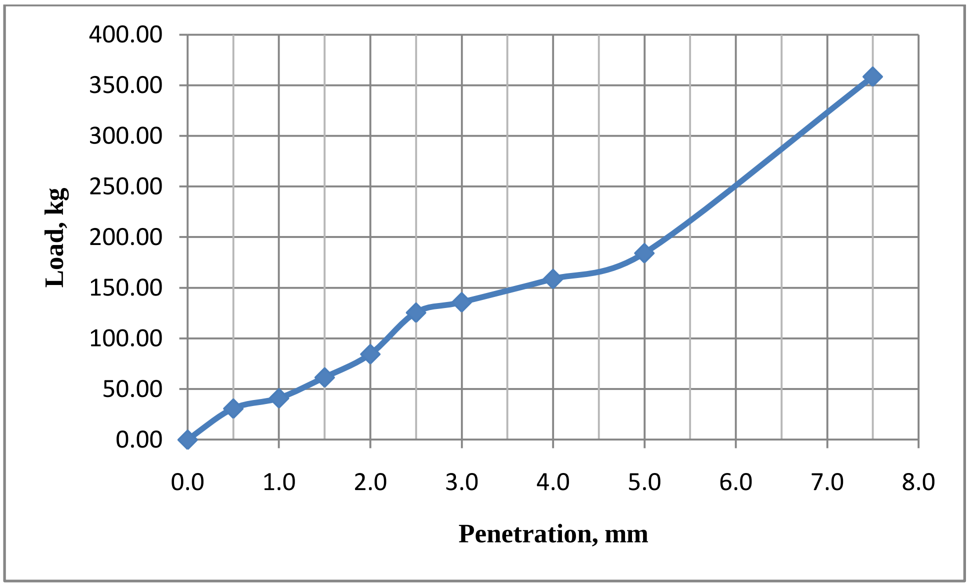

Fig. 4.4 Soaked CBR of Stone Dust

The index properties of stone dust are presented in Table 4.1.

Table 4.1: Index Properties of Stone Dust

Parameter	Value
Specific Gravity (G)	2.6
Particle size analysis Coarse Sand (%) Medium Sand (%) Fine Sand (%)	 23.4 52.6 20.3
IS Classification	SP
Liquid Limit (%)	NP
Plastic Limit (%)	NP
Plasticity Index (%)	NP
Optimum Moisture Content (%)	12
Maximum Dry Density (g/cm^3)	1.87
Angle of Internal Friction (degree)	32.94
Cohesion (kg/cm^2)	0.068
Soaked CBR (%)	9.16

4.3 DETERMINATION OF GEOTECHNICAL PROPERTIES OF SOIL

4.3.1 SPECIFIC GRAVITY

It was determined according to IS: 2720 (Part III/Sec I)-1980. Specific gravity is reported by taking an average of three samples. The specific gravity of soil comes out to be 2.33.

4.3.2 GRAIN SIZE ANALYSIS

According to IS: 2720 (Part 4) 1985, grain size analysis was conducted. The soil was classified according to Indian Standard Soil Classification System as Inorganic

Clay of Medium to Low Plasticity (CL) which comprises 24% sand, 60% silt and 16% clay. Fig. 4.5 shows the particle size ranges of the soil.

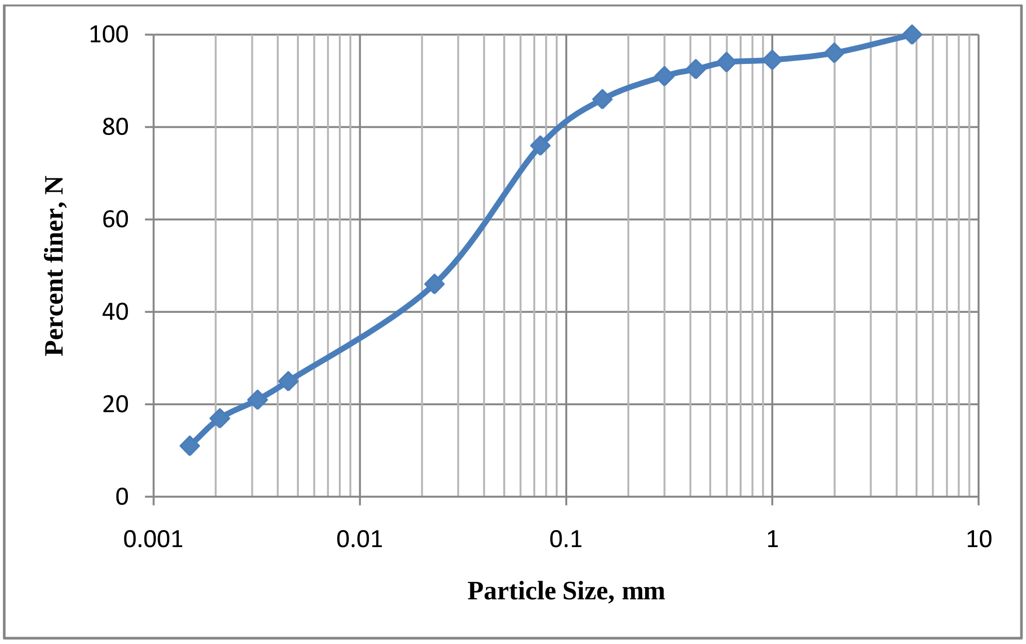

Fig. 4.5 Grain Size Distribution Curve of Soil

4.3.3 CONSISTENCY LIMITS

The liquid limit, plastic limit and plasticity index of soil were found as 20.29%, 7.51% and 12.78% respectively. The flow curve for the soil is shown in Fig. 4.6.

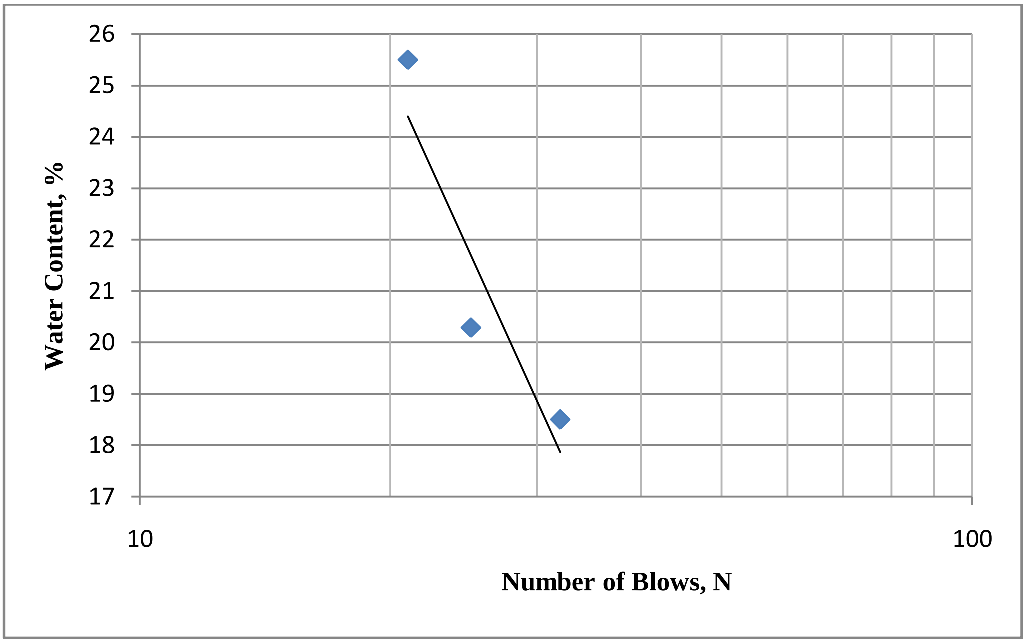

Fig. 4.6 Flow Curve of Soil

4.3.4 OMC AND MDD OF SOIL

Standard proctor test was conducted as per IS: 2720 (Part VII) 1980 to determine OMC and MDD of soil. From the test the optimum moisture content (OMC) and maximum dry density (MDD) was found as 16.75% and 1.742g/cm^3 respectively. The graph of compaction curve is shown in Fig. 4.7.

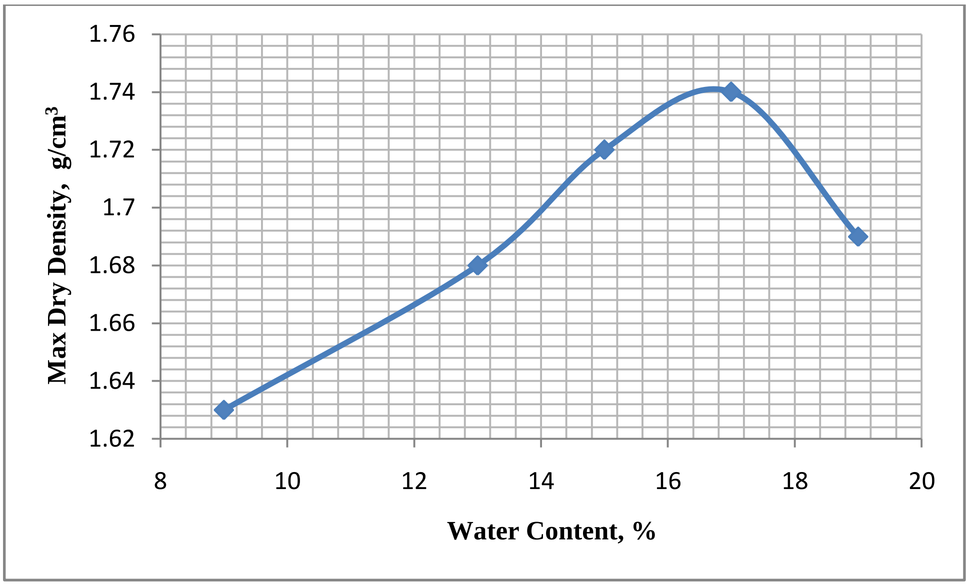

Fig. 4.7 Compaction Curve of Soil

4.3.5 CALIFORNIA BEARING RATIO

The CBR value of soil was determined to be 1.86. The graph of CBR test is shown in Fig. 4.8.

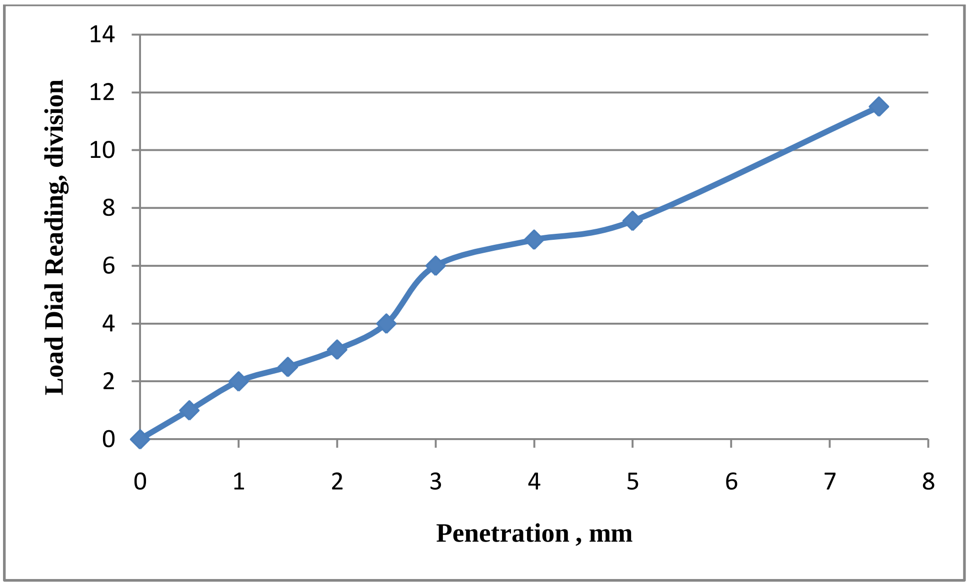

Fig. 4.8 CBR of Virgin Soil

Index properties of soil are summarized in Table 4.2**.**

Table4.2 Index Properties of Soil

Parameter	**Value**
Specific Gravity (G)	2.33
Particle Size Distribution Sand (%) Silt (%) Clay (%)	 24 60 16
IS Classification	CL
Liquid Limit (%)	20.29
Plastic Limit (%)	7.51
Plasticity Index (%)	12.78
OMC (%)	16.75
MDD (g/cm^3)	1.742
CBR (%)	1.86

4.4 OUTCOMES OF GEOTECHNICAL PROPERTIES OF SOIL MIXES

4.4.1 SPECIFIC GRAVITY

It was carried out on soil mixed with different percentages of stone dust (*i.e.*, 10, 20, 30, 40 and 50%). Average value of three tests samples were taken as specific gravity of soil mixtures. Hence in total 3x6=18 specific tests were conducted. The variation of specific gravity with varying percentages of stone dust is shown in Fig. 4.9. Table 4.3 shows the values of specific gravity of mixes.

Table 4.3 Variation of Specific Gravity with Stone Dust Mixes

S. No.	Material	G
1	Soil + 0% Stone Dust mix	2.33
2	Soil + 10% Stone Dust mix	2.5
3	Soil + 20% Stone Dust mix	2.81
4	Soil + 30% Stone Dust mix	2.96
5	Soil + 40% Stone Dust mix	2.93
6	Soil + 50% Stone Dust mix	2.83

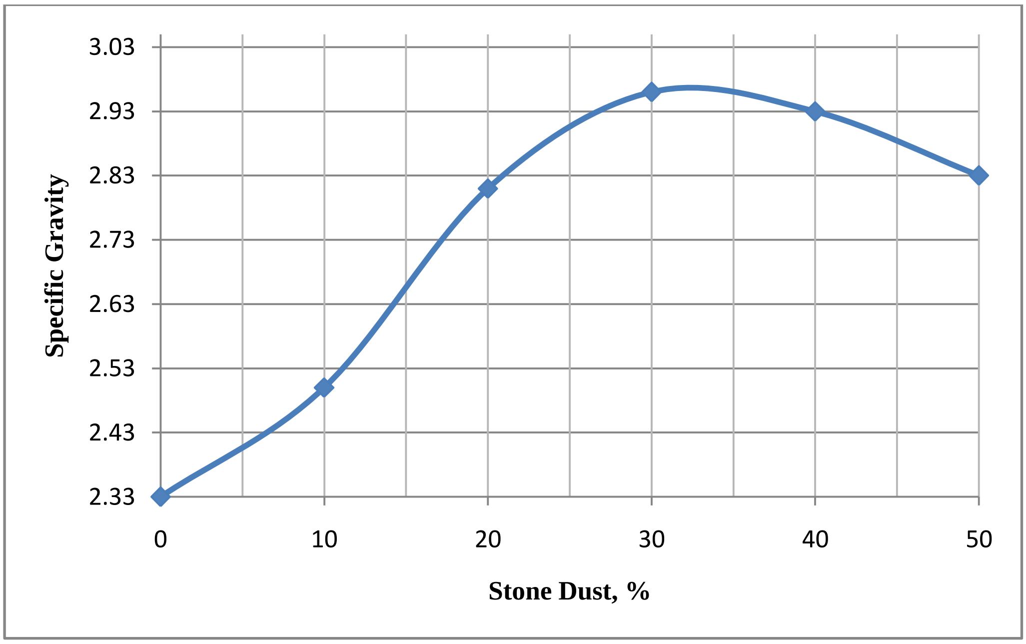

Fig. 4.9 Variation of Specific Gravity with Increasing Percentage of Stone Dust

From the above results of specific gravity it can be concluded that the specific gravity increases from 2.35 to 2.95 on increasing the stone dust percentage from 0% to 30% and thereafter it began to decrease as the percentage of stone dust increases to 50%. The decrement in the specific gravity may be due to the reduction of plasticity character in soil.

4.4.2 CONSISTENCY LIMITS

The term consistency is used to indicate the degree of firmness of cohesive soils. The consistency limits are the water contents at which the soil mass passes from one state to another state, *i.e*., from solid to semi solid to plastic and finally to liquid state and vice a versa. Liquid limit and plastic limit are most useful consistency limits for engineering purpose. These limits are expressed as percent water content.

The consistency limit tests were performed on soil samples with percentage of stone dust varying from 0-50% (*i.e*., 0%, 10%, 20%, 30%, 40% and 50%). The consistency limit test values i.e. for liquid limit, plastic limit and plasticity index values are presented in Table 4.4 for the discussed soil samples. The typical plots of liquid limits for these soil samples are presented from Fig. 4.10 to Fig. 4.14. The graph for variation in plastic limit with varying percentage of stone dust is shown in Fig. 4.15 and the variation of plasticity index with varying percentage of stone dust is presented in Fig. 4.16.

Table 4.4 Variation of Consistency Limit with Stone Dust Mixes

S.No.	Material	LL (%)	PL (%)	PI (%)
1	Soil+0% Stone Dust mix	20.29	7.51	12.78
2	Soil+10% Stone Dust mix	16.28	6.82	9.46
3	Soil+20% Stone Dust mix	12.31	5.1	7.21
4	Soil+30% Stone Dust mix	8.64	3.62	5.02
5	Soil+40% Stone Dust mix	5.31	3.2	2.11
6	Soil+50% Stone Dust mix	NP	NP	NP

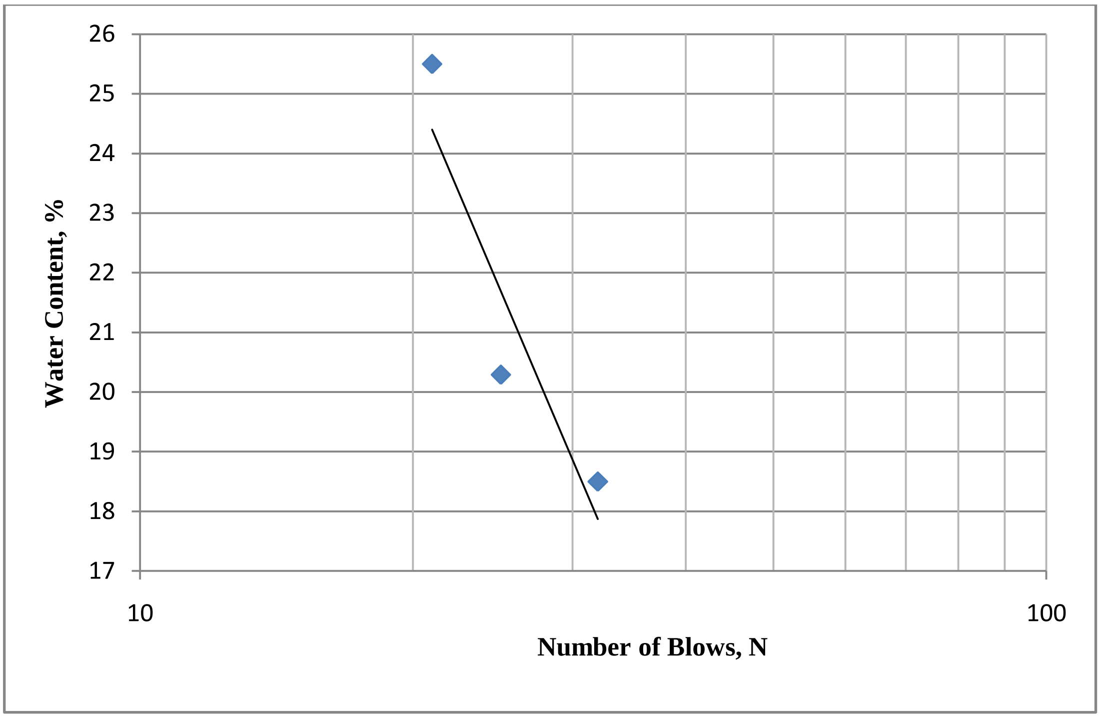

Fig. 4.10 Liquid Limit of Soil + 0% Stone Dust

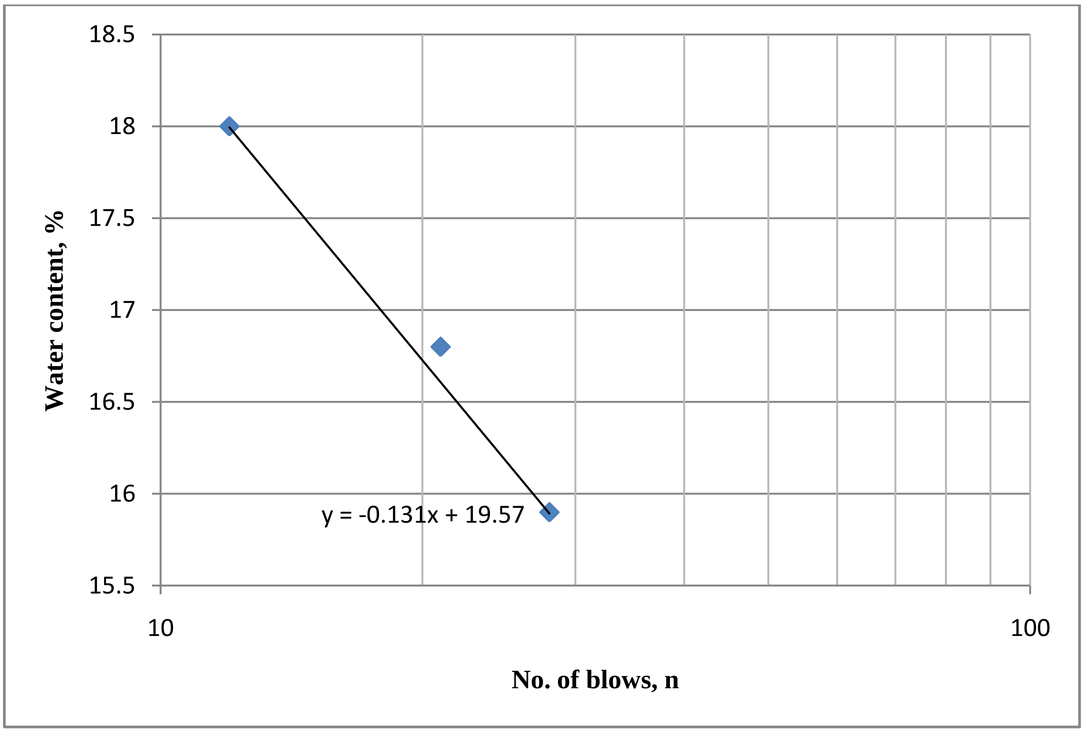

Fig. 4.11 Liquid Limit of Soil + 10% Stone Dust

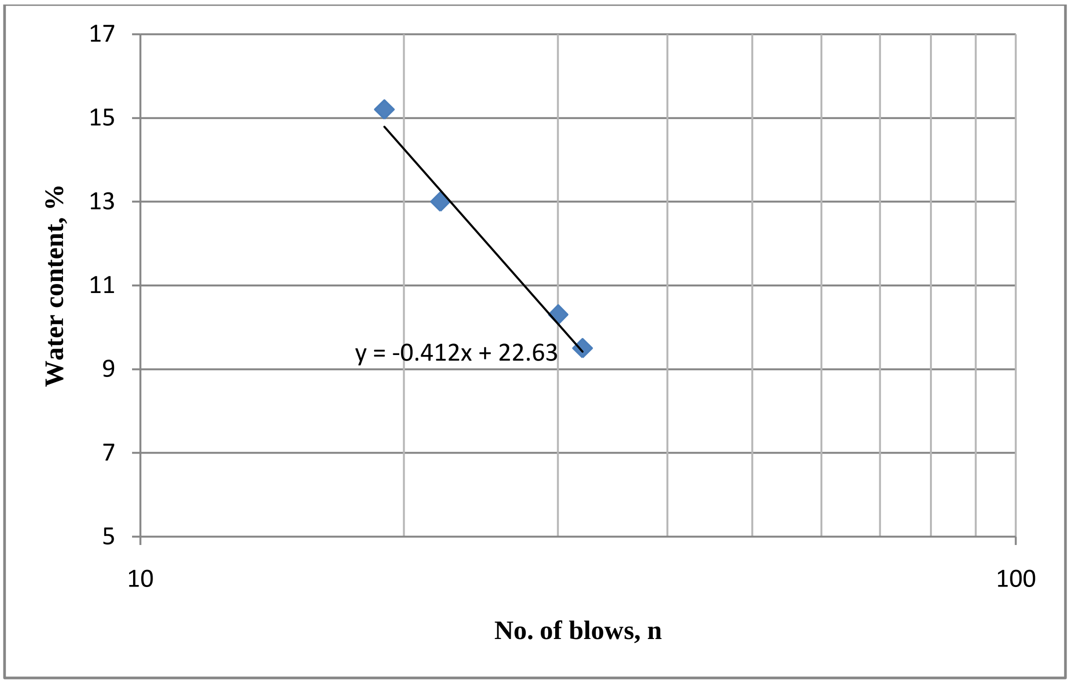

Fig. 4.12 Liquid Limit of Soil + 20% Stone Dust

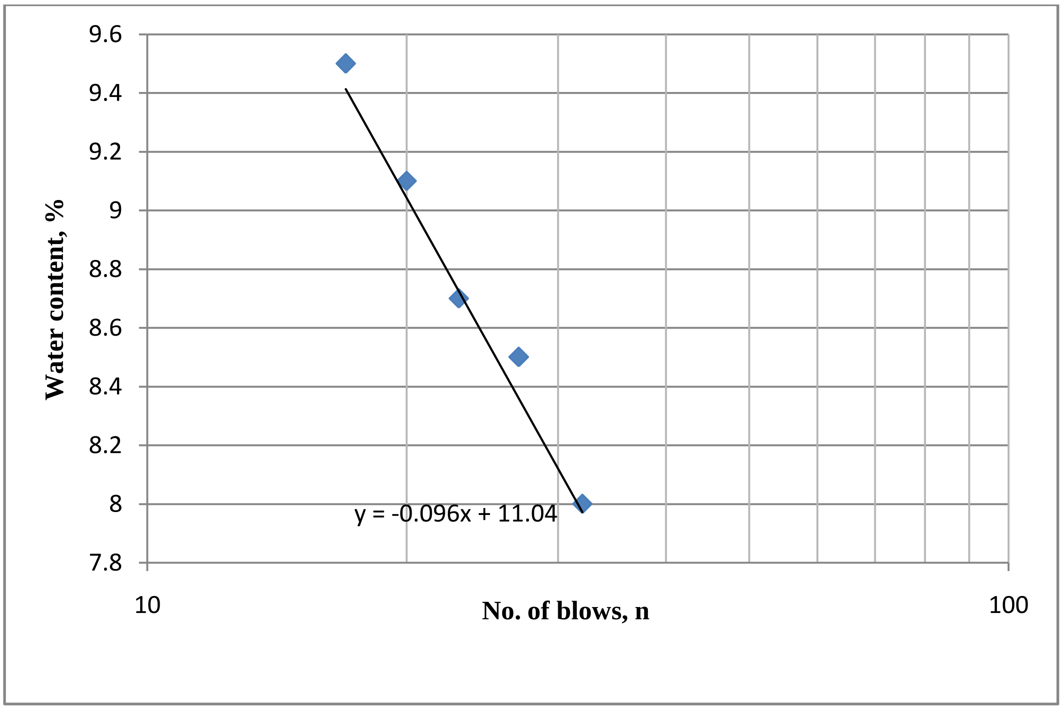

Fig. 4.13 Liquid Limit of Soil + 30% Stone Dust

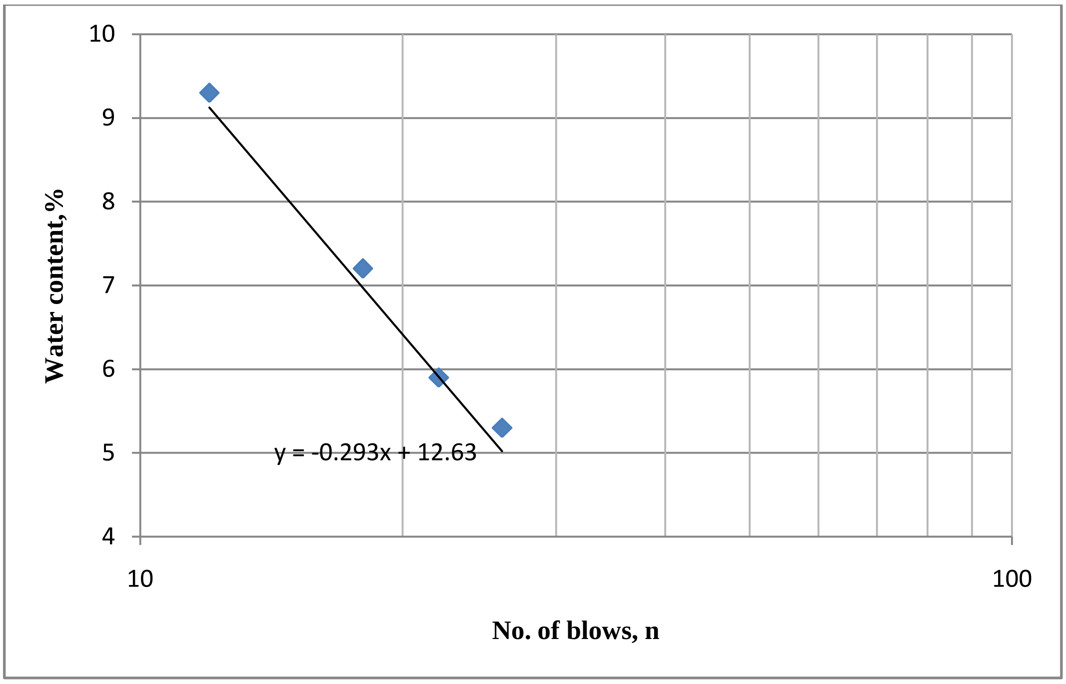

Fig. 4.14 Liquid Limit of Soil + 40% Stone Dust

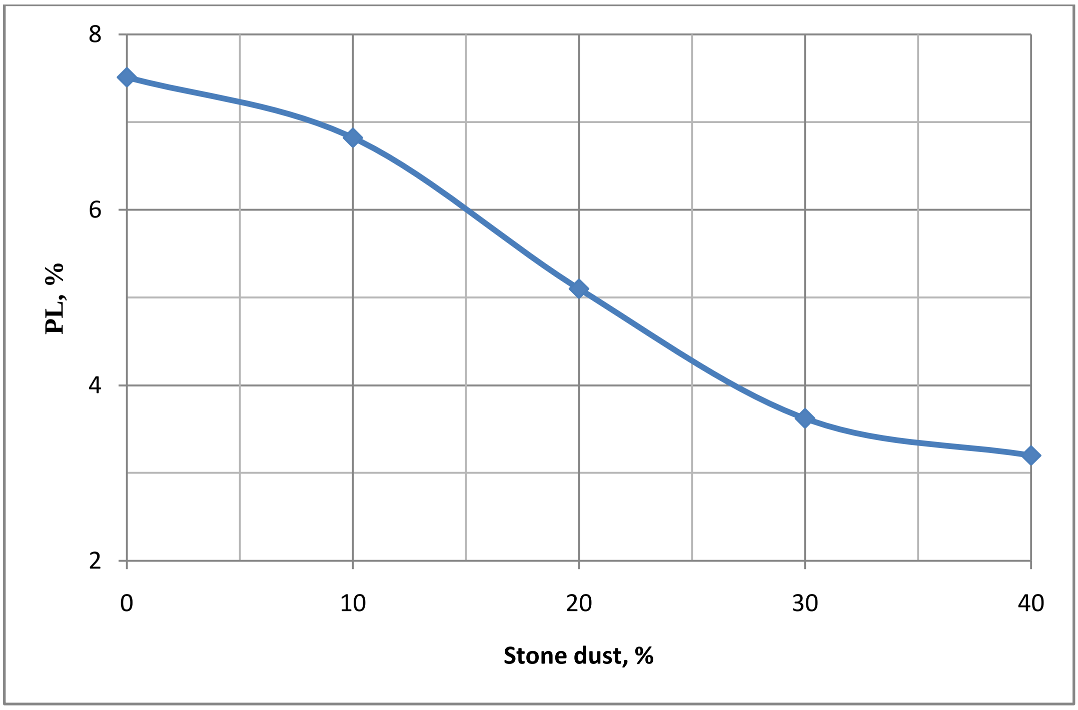

Fig. 4.15 Variation of Plastic Limit + Stone Dust mixes

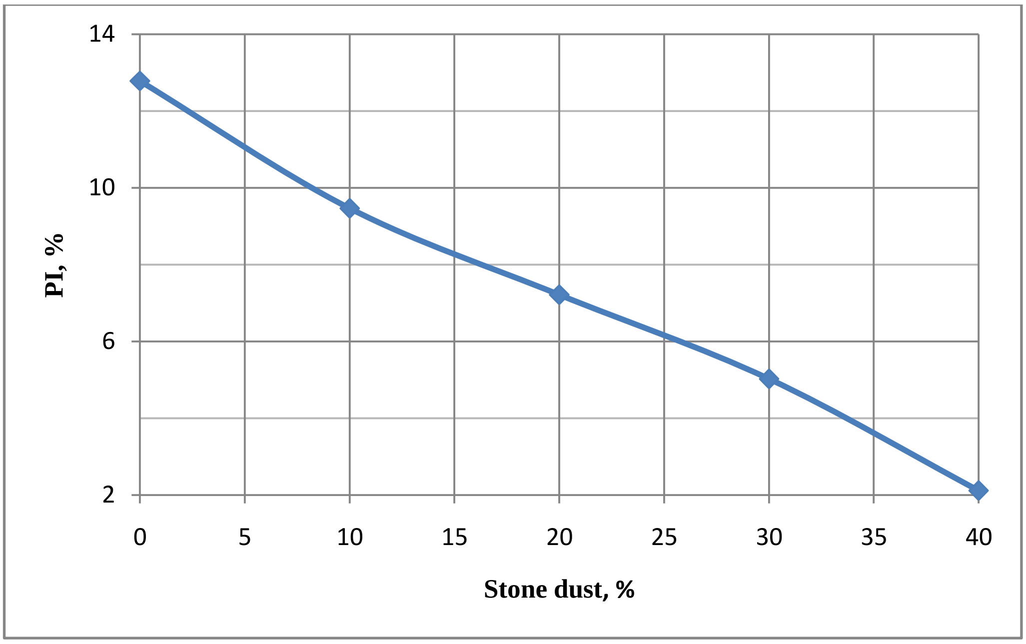

Fig. 4.16 Variation of Plasticity Index + Stone Dust mixes

From the above results of consistency limits, it can be concluded that the liquid limit shows a decreasing trend i.e. it decreases from 20.29% to 5.31% on increasing the percentage of stone dust from 0-40%, beyond this a true non-plastic behaviour of soil is observed. Further, the typical plot of plastic limit v/s stone dust mixes (Fig. 4.15) shows a decreasing trend. The plastic limit decreases from 7.51% to 3.2% on increasing the stone dust from 0% to 40% while on increasing the stone dust content to 50% plastic limit could not be determined as a true non-plastic behaviour was observed. Fig.4.16 shows the decreasing trend of plasticity index with the increasing percentage of stone dust. PI reduced from 12.78% to 2.11% with increase in percentage of stone dust from 0% to 40% while on increasing the stone dust percentage to 50%, plasticity index could not be determined due to non-plastic behaviour of soil. The drop in the consistency limits is thought to come about because of the negligible value of liquid and plastic limit of stone dust. The plasticity characteristics of soil mainly depend on the percentage of fines present in the soil. In this mixes used in the present study, the percentage of fines are less and the main contribution for development of plasticity characteristics are due to clay content is also less. The above results show that the liquid limit becomes less than 25 (w_l<25) and plasticity index is less than 6 (I_p<6) which confirms that the soil becomes low compressible and attains low to non plastic characteristics. This type of soil is useful as subgrde material in highway construction as soils with high ranges of plasticity are not considered good for bearing moving load.

4.4.3 COMPACTION CHARACTERISTICS

Compaction is a mechanical process, which helps to pack the soil particles closely which in turn decreases the porosity of soil and thus increases its dry density, unit weight which ultimately increases the strength and bearing capacity of soil. It also reduces permeability and settlement of soil. Compaction results play a significant role in the stability of various field problems like in construction of earthen dam, embankments, roads etc. as the durability and stability of a structure is achieved only when the soil is properly compacted. Standard Proctor test is done in order to find a definite relationship between soil water content and degree of dry density to which a soil might be compacted.

The compaction test was done on soil samples with percentages of stone dust varying from 0-50% (*i.e.*, 0%, 10%, 20%, 30%, 40% and 50%). The results of these tests, *i.e.*, Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) are presented in Table 4.5. The relationship between dry density and moisture content of typical soil samples are shown on Fig.4.17 to Fig. 4.22.

Table 4.5 Optimum Moisture Content and Maximum Dry Density of Soil variation with Stone Dust mixes

S.No.	Material	OMC, %	MDD, g/cm^3
1	Soil + 0% Stone dust mixes	16.75	1.74
2	Soil + 10% Stone dust mixes	16	1.83
3	Soil + 20% Stone dust mixes	15	1.85
4	Soil + 30% Stone dust mixes	15	1.86
5	Soil + 40% Stone dust mixes	14	1.88
6	Soil + 50% Stone dust mixes	13.25	1.89

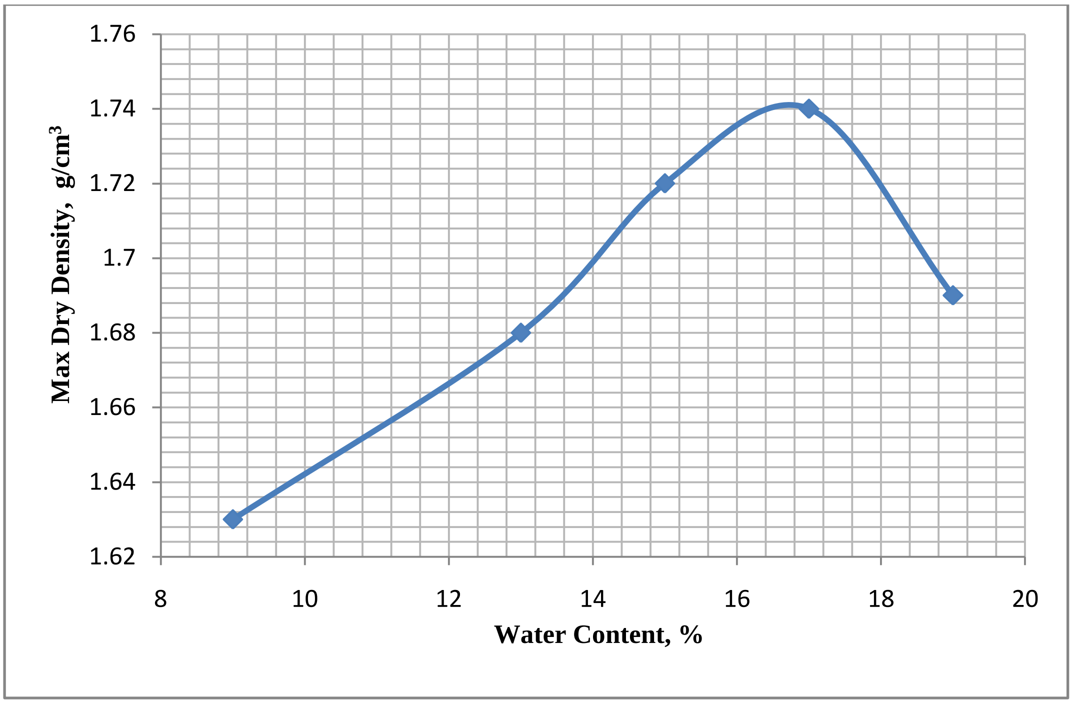

Fig. 4.17 Compaction Characteristic of Soil + 0% Stone Dust

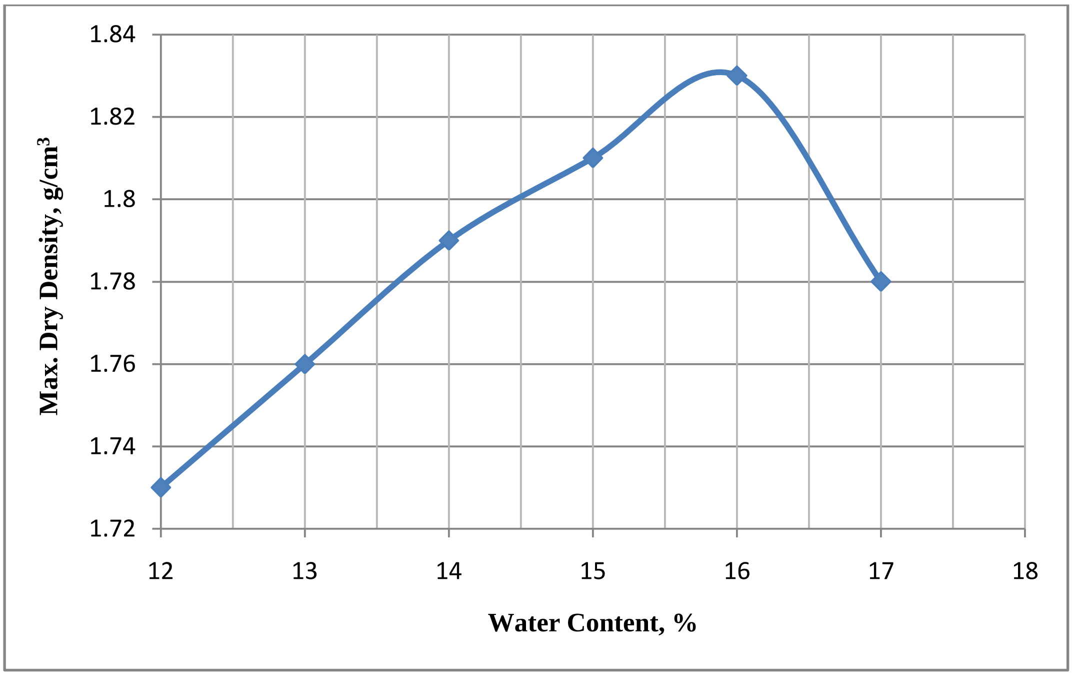

Fig. 4.18 Compaction Characteristic of Soil + 10% Stone Dust

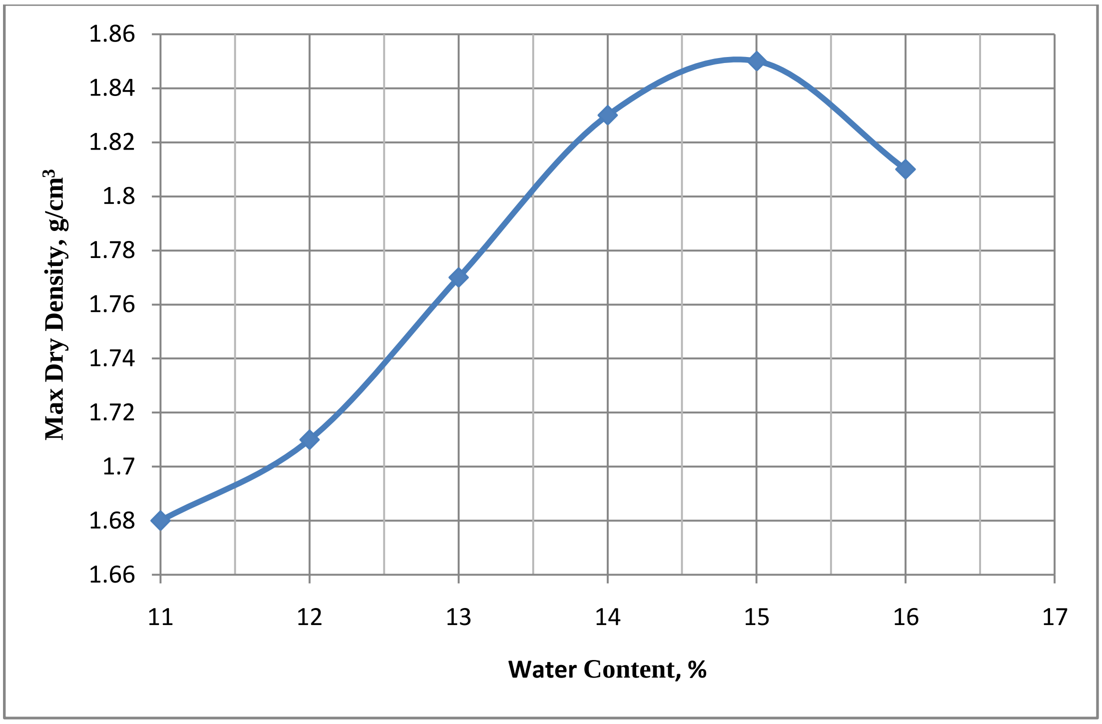

Fig. 4.19 Compaction Characteristic of Soil + 20% Stone Dust

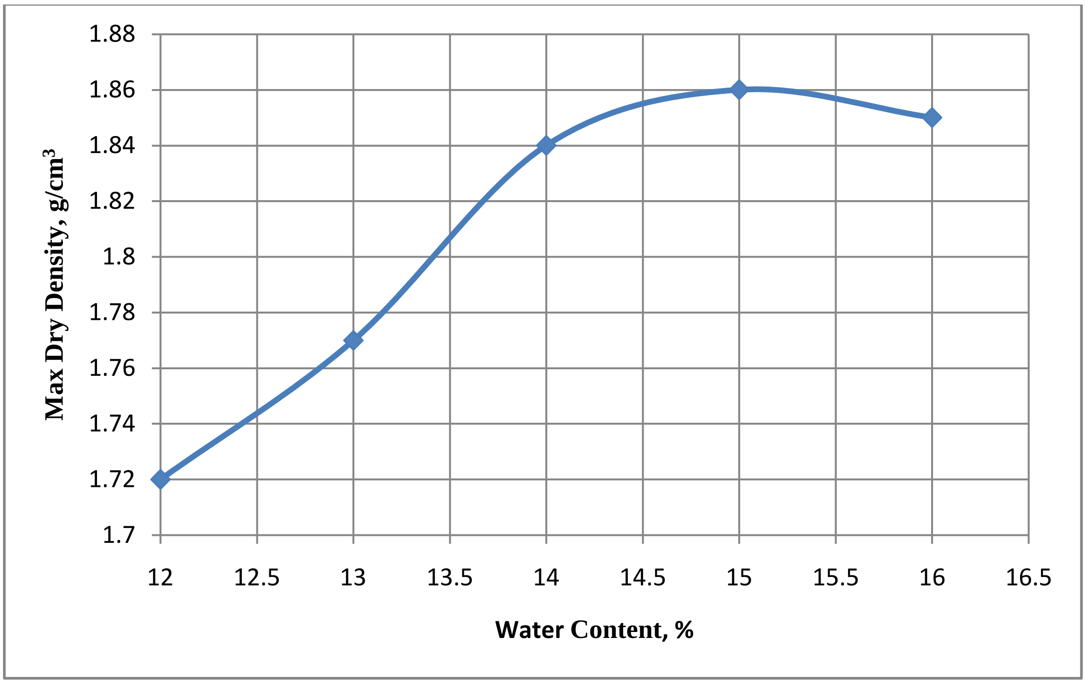

Fig. 4.20 Compaction Characteristic of Soil + 30% Stone Dust

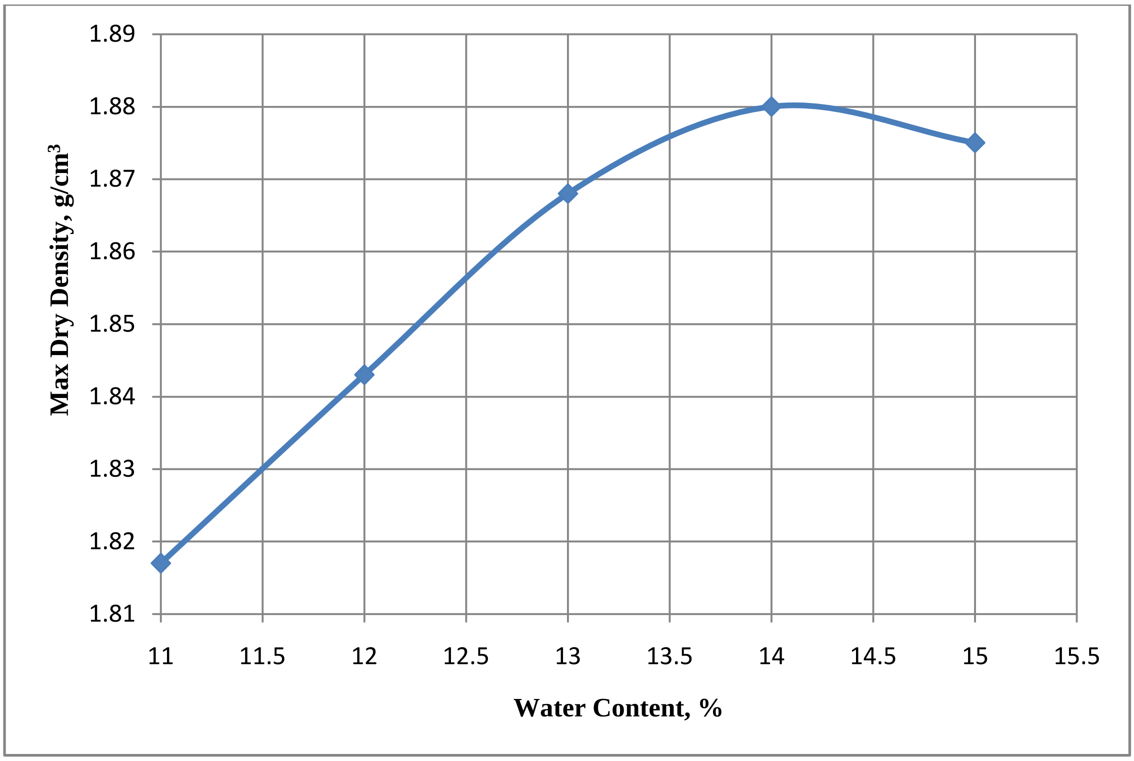

Fig. 4.21 Compaction Characteristic of Soil + 40% Stone Dust

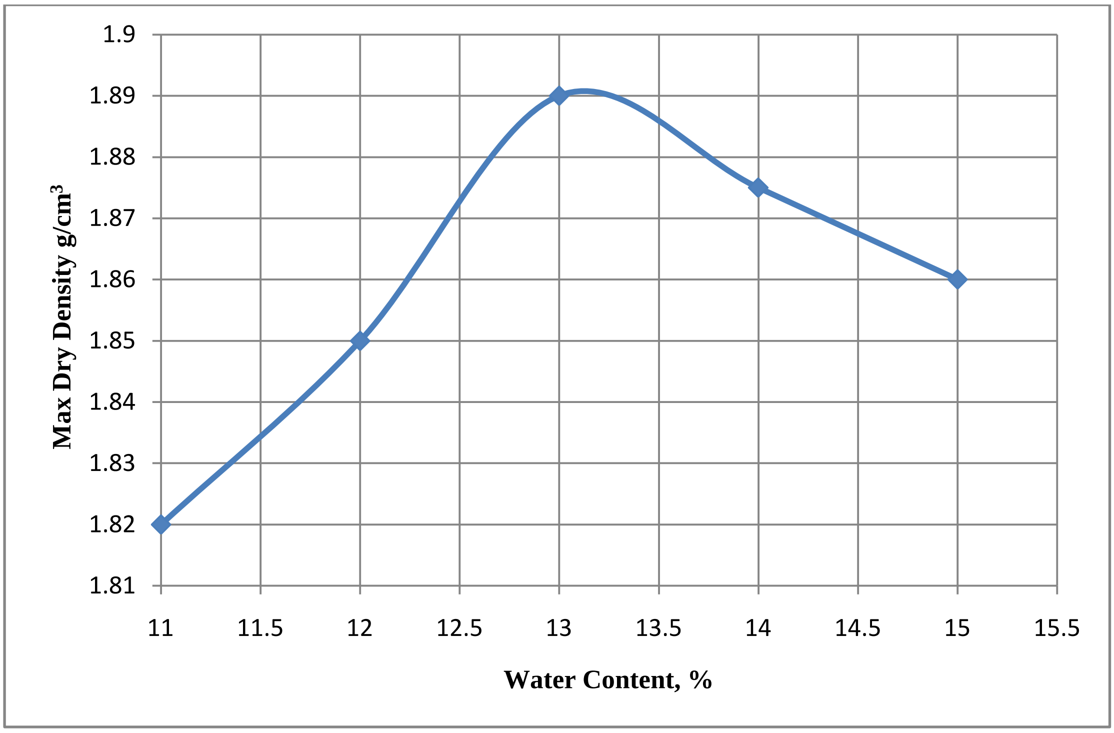

Fig. 4.22 Compaction Characteristic of Soil + 50% Stone Dust

The data summarized in Table 4.5, showed that the optimum moisture content of soil mixes decreases from 16.75% to 13.25% while maximum dry density increases from 1.74 to 1.89g/cm^3 on increasing the stone dust content from 0% to 50%. This increment in MDD is thought to come about because of the engrossment of more solids as compared to interaction of stone dust with finer soil particles. Otherwise speaking, MDD increases due to the reduction of voids as the stone dust fulfil the space between the soil particles. And OMC decreases due to replacement of silt and clay particles by stone dust which reduces intake of moisture.

4.4.4 DIRECT SHEAR TEST

Shear parameters are required for calculating the bearing capacity of foundations which are further used in the design of earthen dams and embankments.

The direct shear test was done on soil samples with percentages of stone dust varying from 0-50% (*i.e.*, 0%, 10%, 20%, 30%, 40% and 50%). Typical plots of shear stress v/s normal stress are drawn to determine shear strength parameters (*i.e.*, c and Φ). The values of cohesion and angle of internal friction are presented in table4.6. The plots of shear and normal stress are drawn in Fig. 4.23 to Fig. 4.28.

Table 4.6 Variation of Cohesion and Angle of Internal Friction of soil with stone dust mixes

S.No.	Material	Cohesion, kg/cm^2	Angle of Internal Friction, degree
1	Soil + 0% Stone Dust Mixes	0.27	24.23
2	Soil + 10% Stone Dust Mixes	0.18	25.6
3	Soil + 20% Stone Dust Mixes	0.13	26.75
4	Soil + 30% Stone Dust Mixes	0.11	27.47
5	Soil + 40% Stone Dust Mixes	0.1	28
6	Soil + 50% Stone Dust Mixes	0.05	32.6

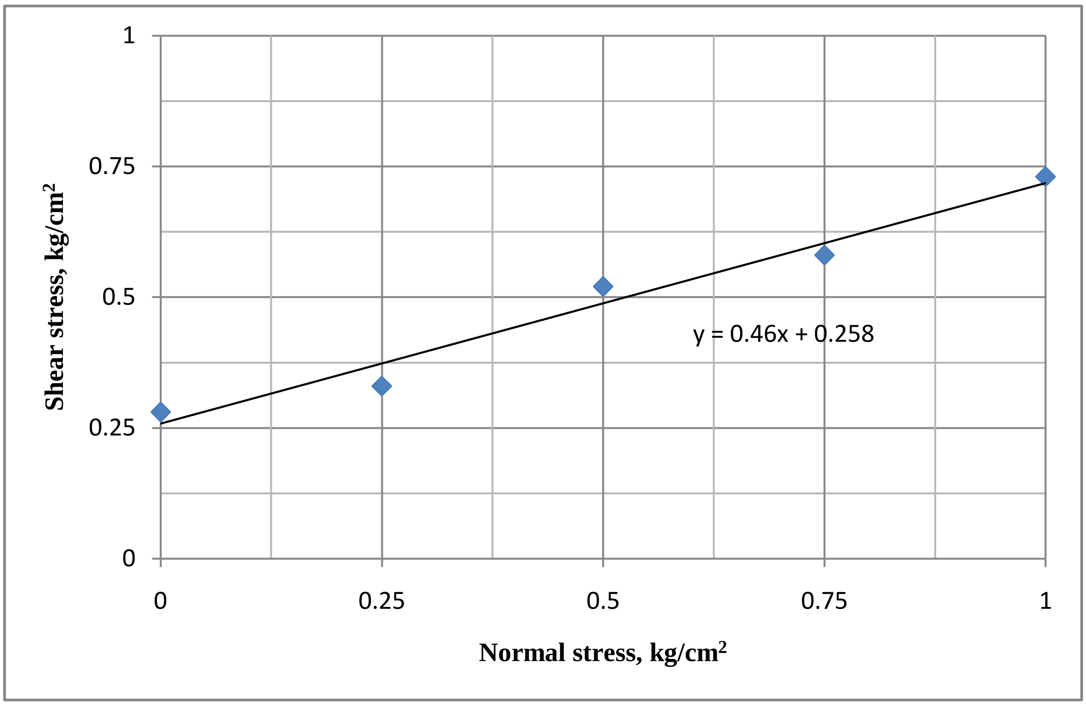

Fig. 4.23 Normal Stress v/s Shear Stress plot for soil + 0% Stone Dust

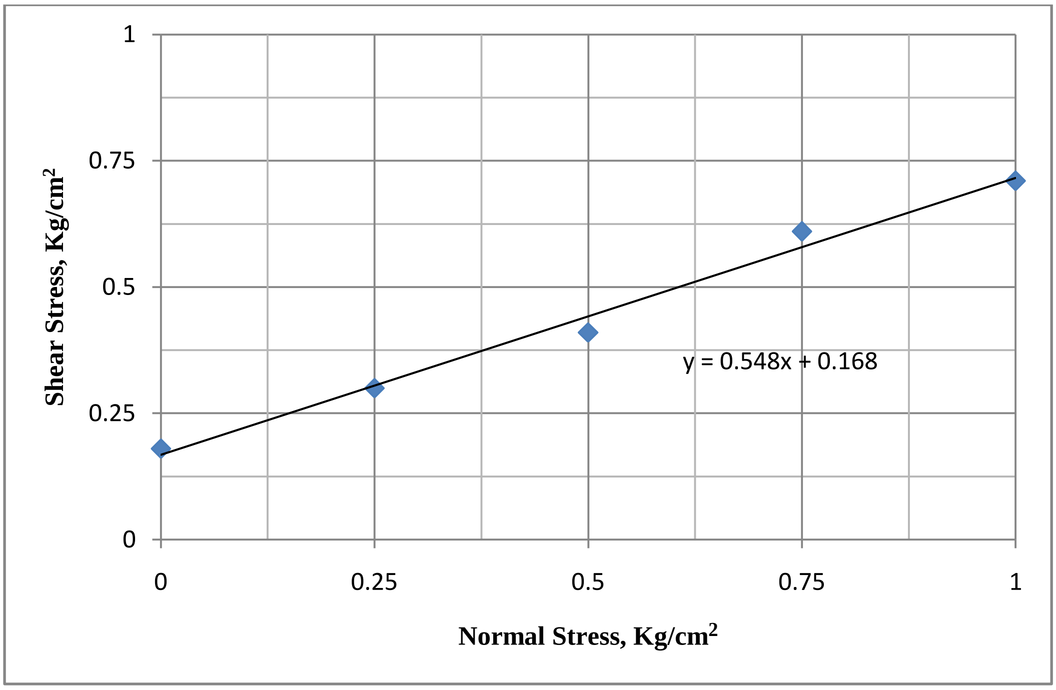

Fig. 4.24 Normal Stress v/s Shear Stress plot for soil + 10% Stone Dust

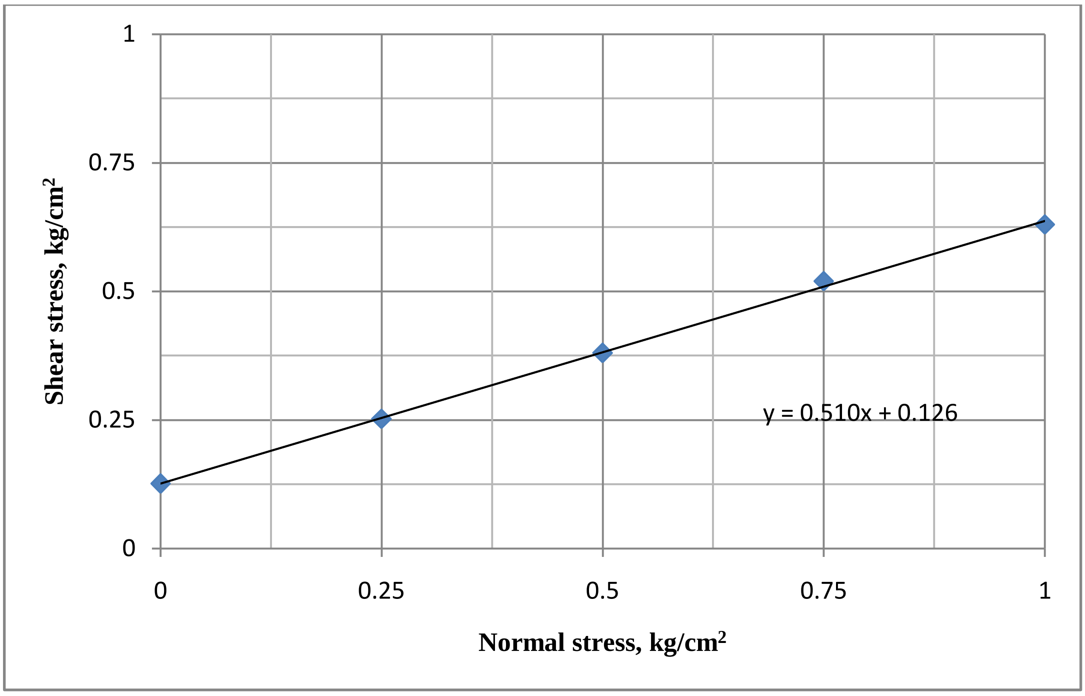

Fig. 4.25 Normal Stress v/s Shear Stress plot for soil + 20% Stone Dust

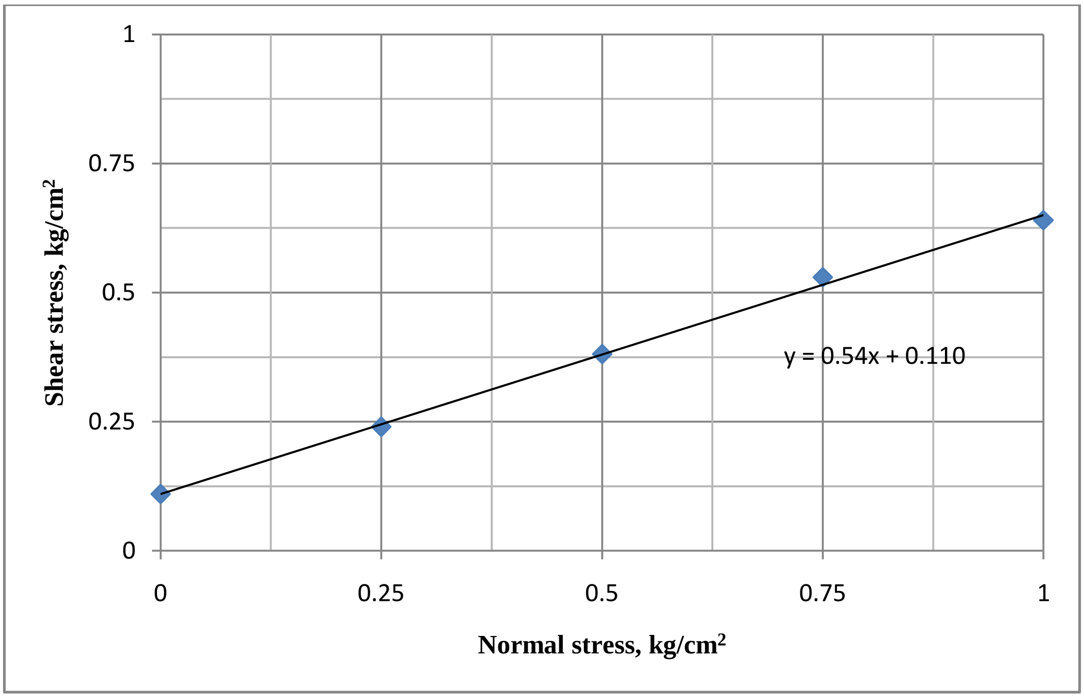

Fig. 4.26 Normal Stress v/s Shear Stress plot for soil + 30% Stone Dust

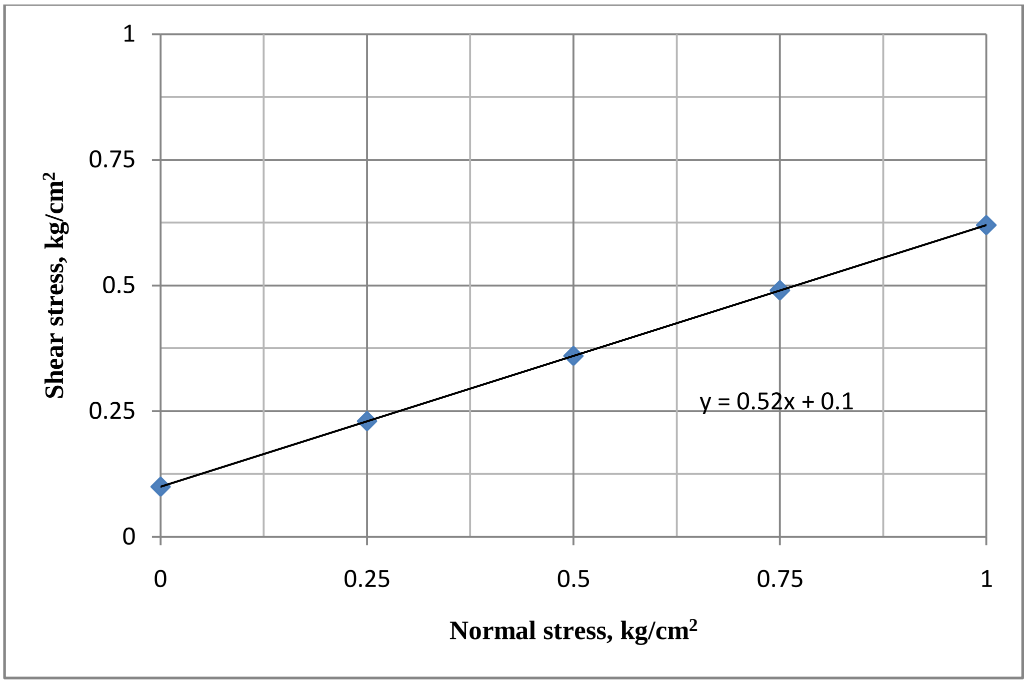

Fig. 4.27 Normal Stress v/s Shear Stress plot for soil + 40% Stone Dust

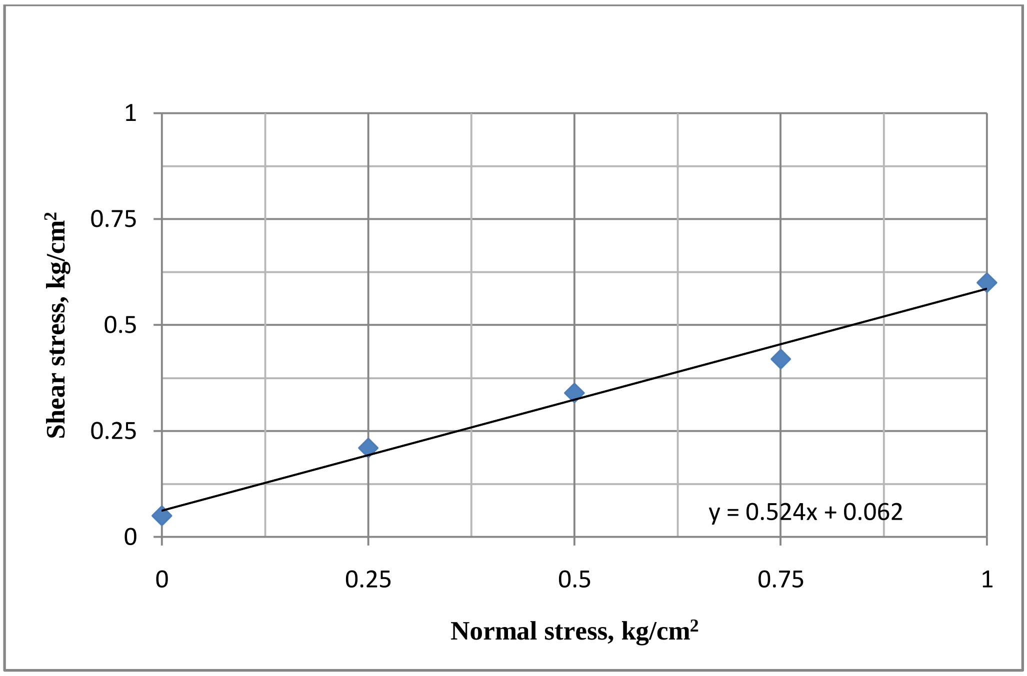

Fig. 4.28 Normal Stress v/s Shear Stress plot for soil + 50% Stone Dust

From the above Table and Fig it is concluded that there is a decrese in value of cohesion from 0.27 to 0.05 kg/cm^2 while angle of internal friction increases from 24.23° to 32.6° when the stone dust percentage increases from 0 to 50%. This increment in angle of internal friction and decrement in cohesion is due to the replacement of cohesive soil with stone dust which has very low cohesion value and high angle of internal friction in comparison with soil.

4.4.5 UNCONFINED COMPRESSIVE STRENGTH

Strain-controlled UCS tests were conducted on soil sammples. Compessive strength was obtained in undrained state.

The UCS test was done on soil samples with percentages of stone dust varying from 0-50% (*i.e.*, 0%, 10%, 20%, 30%, 40% and 50%). Stress values at failure for these soil samples are presented in Table 4.7. The variation of stress at failure with increasing percentage of stone dust is presented in Fig. 4.29.

Table 4.7 Stress at Failure of soil with stone dust mixes

S.No.	Material	Stress at failure, kN/m^2
1	Soil + 0% Stone Dust mixes	175.16
2	Soil + 10% Stone Dust mixes	207.68
3	Soil + 20% Stone Dust mixes	220.93
4	Soil + 30% Stone Dust mixes	229.49
5	Soil + 40% Stone Dust mixes	194.8
6	Soil + 50% Stone Dust mixes	181.93

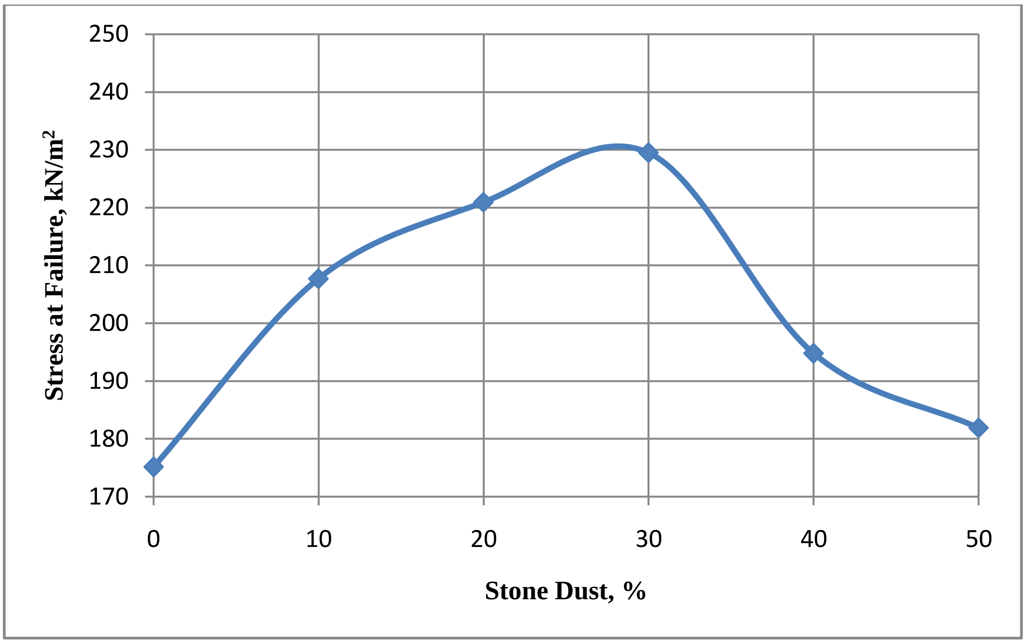

Fig. 4.29 Variation of stress at failure with soil and stone dust mixes

4.4.6 CALIFORNIA BEARING RATIO

The CBR test was conducted on soil mixed with different percentages of stone dust (*i.e.*, 0%, 10%, 20%, 30%, 40% and 50%). Table 4.8 shows the value of CBR of the soil mixes. The load v/s penetration fraphs for these soil samples are shown in Fig. 4.30 to Fig. 4.35.

Table 4.8 CBR Values of soil with stone dust mixes

S.No.	Material	CBR, %
1	Soil + 0% Stone Dust mixes	1.86
2	Soil + 10% Stone Dust mixes	2.39
3	Soil + 20% Stone Dust mixes	2.55
4	Soil + 30% Stone Dust mixes	2.81
5	Soil + 40% Stone Dust mixes	2.40
6	Soil + 50% Stone Dust mixes	1.82

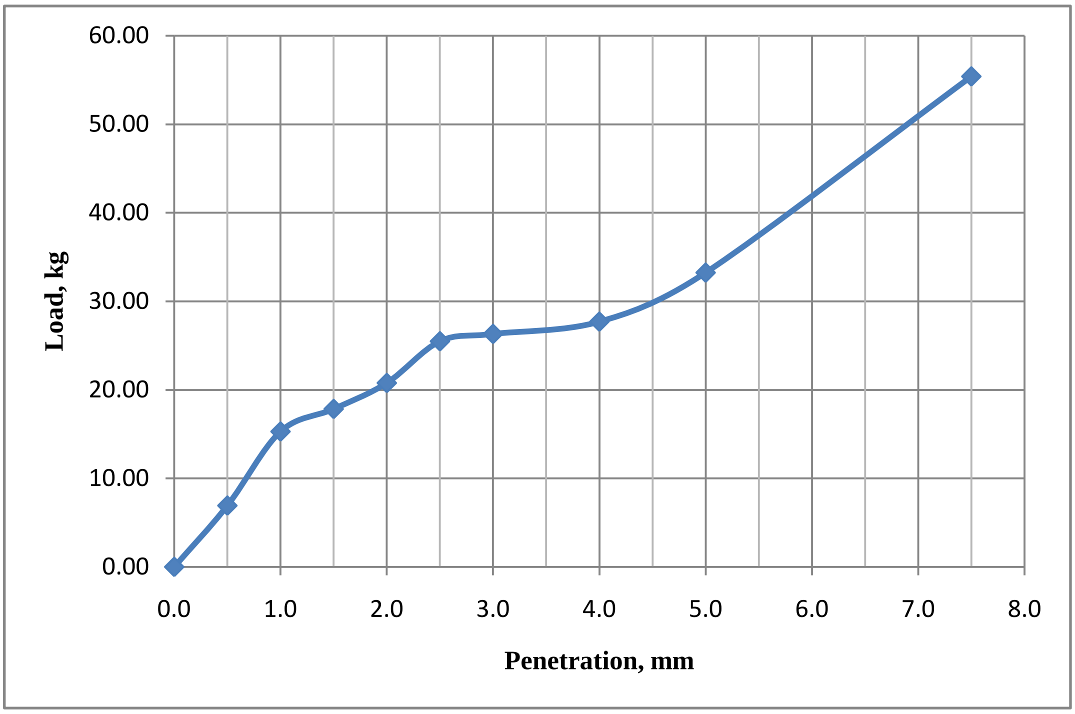

Fig 4.30 Relation between load and penetration of virgin soil

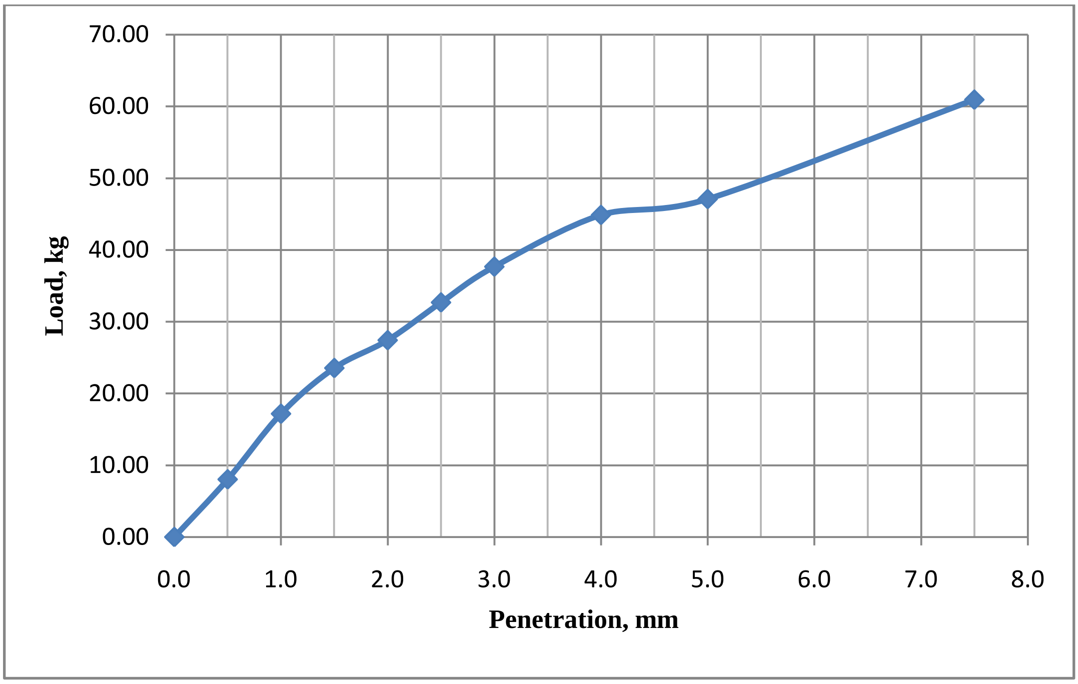

Fig. 4.31 Relation between load and penetration of soil + 10% Stone Dust

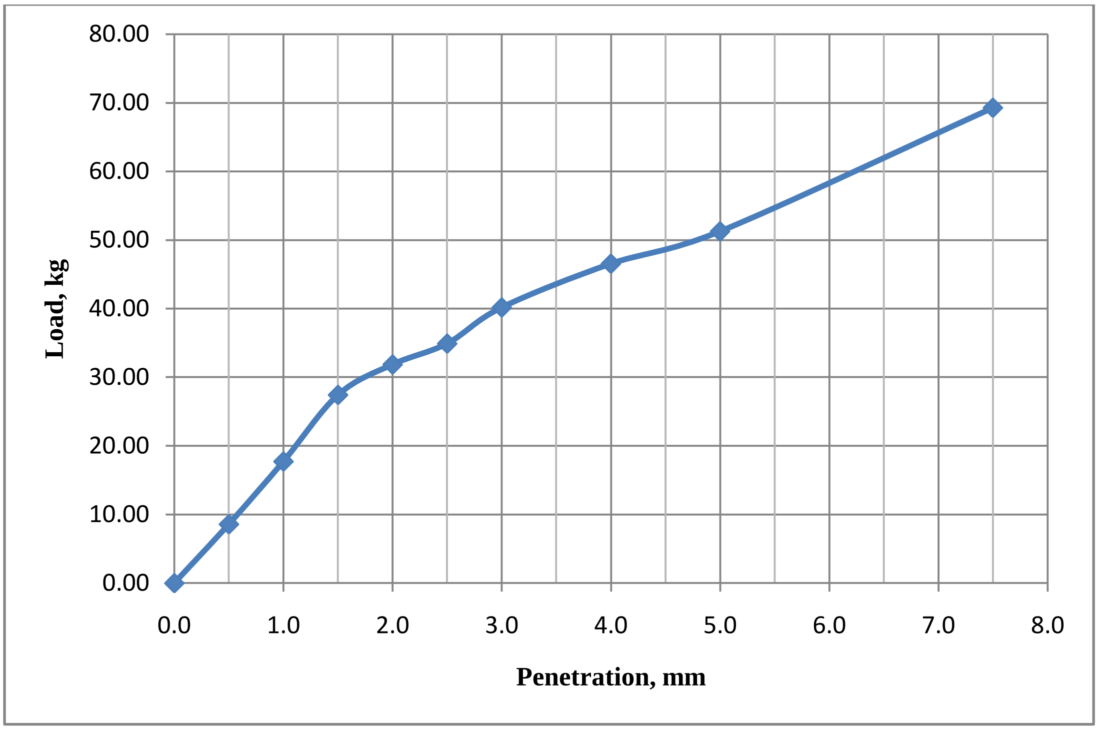

Fig. 4.32 Relation between load and penetration of soil + 20% Stone Dust

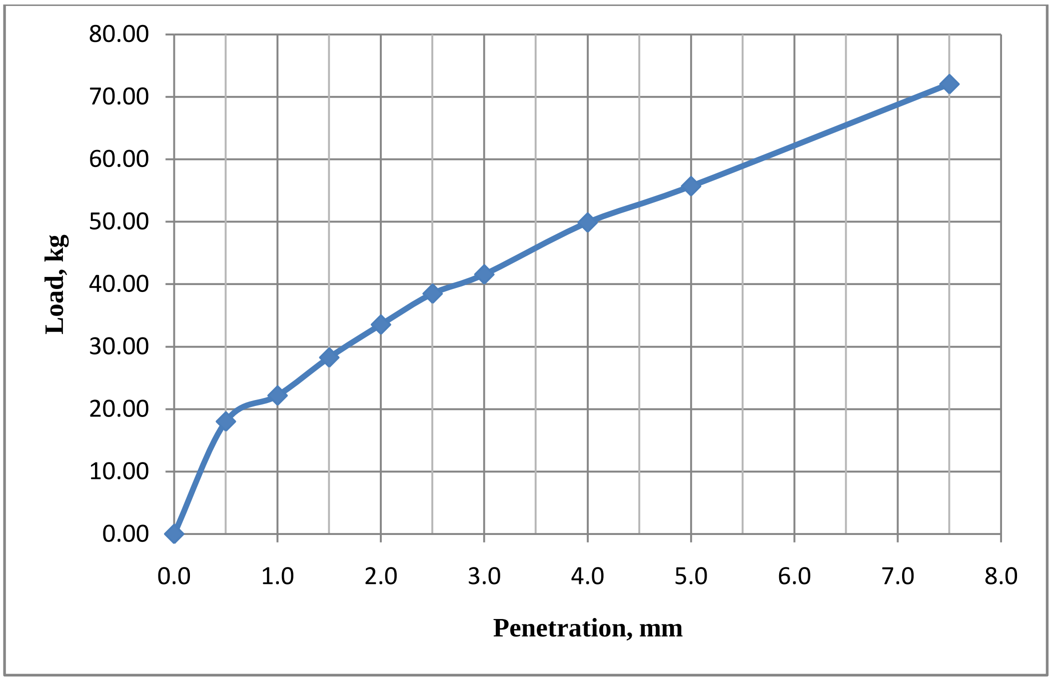

Fig. 4.33 Relation between load and penetration of soil + 30% Stone Dust

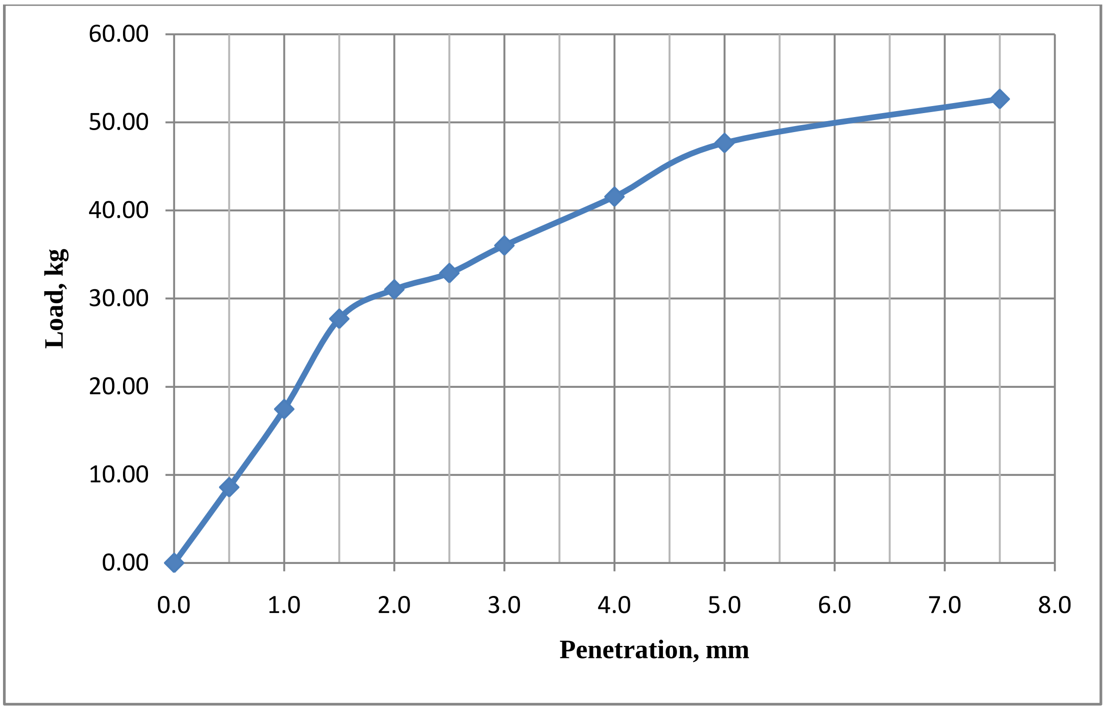

Fig. 4.34 Relation between load and penetration of soil + 40% Stone Dust

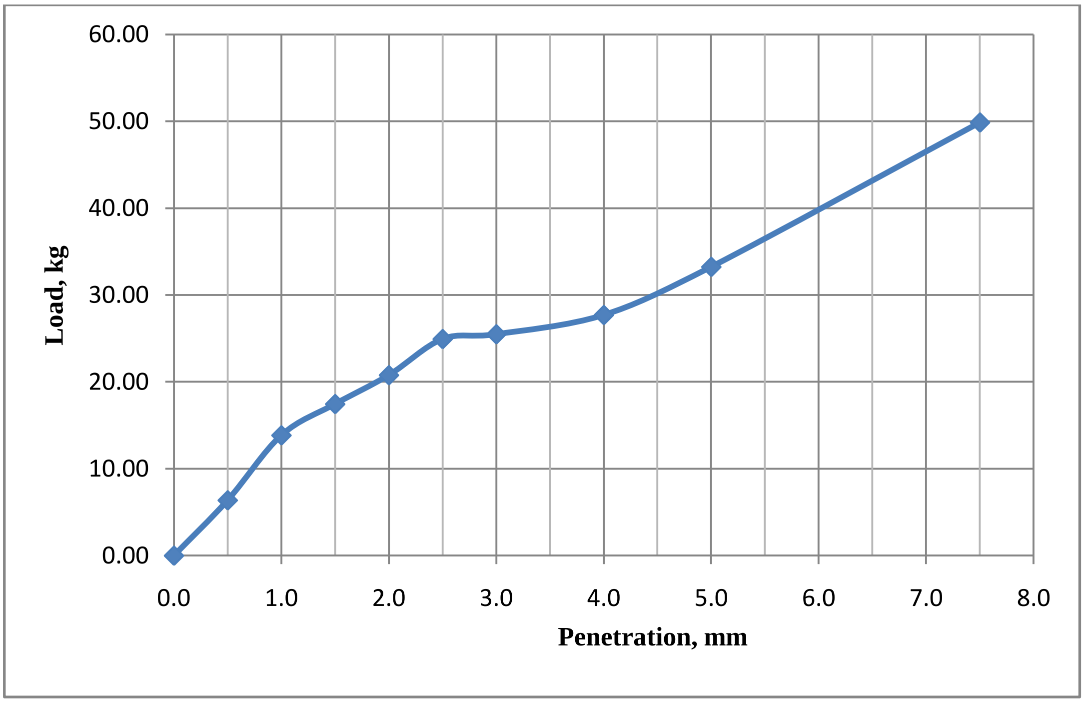

Fig. 4.35 Relation between load and penetration of soil + 50% Stone Dust

From the above figures and table of CBR reslts it is seen that the soaked values of CBR and stone dust mixes firstly increases from 1.86% to 2.81% when stone dust percentage increases upto 30% and then it decreases to 1.95% when the stone dust percentage was increase upto 50%. This increment upto 30% stone dust percentage may be due to the fact that soil containng clay particles are replaced by stone dust which has good compaction as compared to clay offering more shear resistance against compression.

4.4.7 COEFFICIENT OF PERMEABILTY

Permeability is the property of porous material which allow the water to flow through its interconnected voids. Its calculation is important for various problems like seepage through body of dam and stability of slopes, settlement of saturated compressible layer of soil, uplift pressure and safety against piping. The falling head permeability test was carried out on different soil samples cointaing stone dust in vaious percentages (*i.e.*,0%, 10%, 20%, 30%, 40% and 50%). The Table 4.9 shows the permeablity of such soil samples and Fig. 4.36 shows the variation of permebility of soil with change in the percentage of stone dust.

Table 4.9 Permeability Values of Soil with Stone Dust Mixes

S.No.	Material	Permeability, cm/sec
1	Soil + 0% Stone Dust mixes	1.78E-06
2	Soil + 10% Stone Dust mixes	2.03E-06
3	Soil + 20% Stone Dust mixes	2.45E-06
4	Soil + 30% Stone Dust mixes	3.9E-06
5	Soil + 40% Stone Dust mixes	5.6E-06
6	Soil + 50% Stone Dust mixes	10E-06

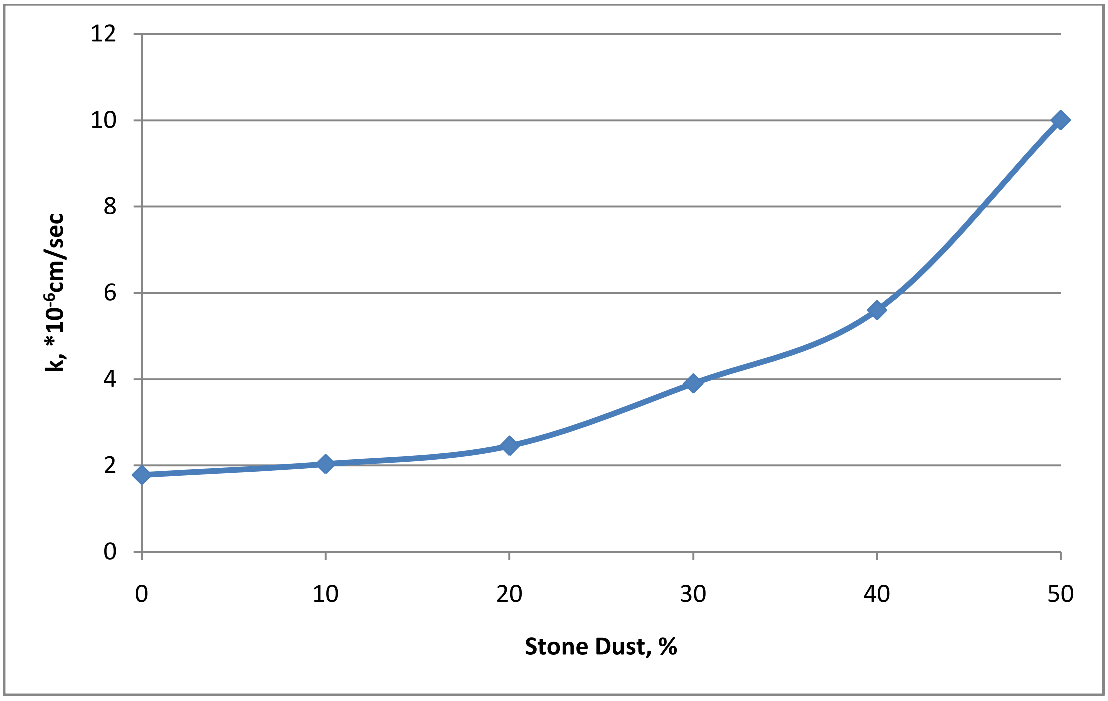

Fig. 4.36 Variation of coefficient of permeability with stone dust

From the above results of permeability it is observed that with increasing in percentage of stone dust, permeability of mixtures also increases. Coefficient of permeablity increases from 1.78E-06 to 10E-06 with increase in the percentage of stone dust from 0 to 50%. This increase in value is due to the fact that with adding increased percentage of stone dust, coarser aprticles of stone dust replaces fine frained particles of orignal soil sample.

4.5 CONCLUDING REMARKS

The results obtained by peforming various geotechnical tests on soil and on soil when mixed with different percentages of stone dust are discussed in this chapter. Graphs for studying variation of some geotechnical properties of soil with increasing percentage of stone dust is presented in Appendix. The conclusions drawn from these results are discussed in the next chapter.

Summary & Conclusions

Chapter 5 SUMMARY AND CONCLUSIONS

5.1 General

Stone dust is the waste which is produced in bulk from stone crushers in India and finally is dumped on barren lands. Hence its utilization becomes necessary. Utilization of this waste can be done in many geotechnical engineering applications like as a construction material in embankments, as backfill material, as sub-base material etc. The present study provides an outline of the effects of different percentages of stone dust when mixed with local soil on some geotechnical properties of soil like its consistency limits, specific gravity, shear strength parameters, unconfined compression strength, California bearing ratio and permeability of the soil. This study clarifies the suitability of stone dust as a substitute for conventional fill material in civil engineering construction.

Table 5.1 summarizes the results of the experimental investigation. The following conclusions are drawn in the view of the results as discussed in chapter 4:

5.2 Conclusions

- Stone dust used in the study is poorly graded sand (SP) with 7.5 and 0.867 values of coefficient of uniformity and coefficient of curvature respectively.
- The mass specific gravity (G) of the samples increases from 2.33 to 2.96 as the percentage of stone dust increases from 0 to 30% and thereafter it decrease to 2.83 when stone dust increases to 50%.
- On observing consistency limit results, it is concluded that the liquid limit of the soil samples decreases from 20.32% to 5.31% on adding stone dust to 40% and beyond 40% a non-plastic behaviour of soil was observed.
- Plastic limit results show a decrement in its value from 7.51% to 3.2% when stone dust was increased up to 40% and again beyond 40% a non-plastic behaviour of soil was observed.
- The plasticity index of soil was observed to decrease from 12.81% to 2.11% with the increment in percentage of stone dust from 0 to 40% and beyond this limit a non plastic behaviour of soil was observed.

Table5.1 Summary of Results

S. No.	**Material**	**Specific Gravity**	**LL (%)**	**PL (%)**	**PI (%)**	**OMC (%)**	**MDD (g/cm^3)**	**Cohesion (kg/cm^2)**	**Angle of Internal Friction (°)**	**Stress at Failure (kN/m^2)**	**CBR Soaked (%)**	**Permeability (E-06 cm/sec)**
1	Soil + 0% SD	2.33	20.32	7.51	12.81	16.75	1.74	0.27	24.23	175.16	1.86	1.78
2	Soil + 10% SD	2.5	16.28	6.82	9.46	16	1.83	0.18	25.6	207.68	2.39	2.03
3	Soil + 20% SD	2.81	12.31	5.1	7.21	15	1.85	0.13	26.75	220.93	2.55	2.45
4	Soil + 30% SD	2.96	8.64	3.62	5.02	15	1.86	0.11	27.47	229.49	2.81	3.9
5	Soil + 40% SD	2.93	5.31	3.2	2.11	14	1.88	0.1	28	194.8	2.4	5.6
6	Soil + 50% SD	2.83	NP	NP	NP	13.25	1.89	0.05	32.6	181.93	1.82	10

- On studying compaction tests results, it is seen that the optimum moisture content of soil samples show a decrement from 16.75% to 13.25% with increase in the stone dust percentage from 0 to 50%.
- The maximum dry density of soil samples increases from 1.74g/cm^3 to 1.89g/cm^3 on increasing the stone dust percentage from 0 to 50% which shows that it is advantageous to use stone dust up to 50% in civil engineering projects.
- By analyzing direct shear tests results, a decrement in the value of cohesion was observed which were 0.27kg/cm^2 when no stone dust was added while it decreases to 0.05kg/cm^2 when the percentage of stone dust was increases up to 50%.
- The angle of internal friction increases from 24.23° to 32.6° when the stone dust percent was increased from 0 to 50%.
- The stress at failure from UCS tests of the soil samples increases from 175.16kN/m^2 to 229.49kN/m^2 when stone dust increases from 0 to 30% and then decreases to 181.93kN/m^2 when the percentage of stone dust added was increased to 50%.
- The soaked CBR values of soil samples increases from 1.86% to 2.81% when stone dust percentage added was 0 to 30% and thereafter it decreases to 1.82% on increasing the stone dust to 50%.
- The permeability of soil samples increases from 1.78E-06cm/sec to 10.00E-06cm/sec on increasing the stone dust percentage from 0 to 50%.

After analyzing the results, the optimum percentage of stone dust to be added is found to be 30% which gives better results in order to improve the geotechnical properties of soil. Hence, soil stabilization using stone dust is the most appropriate option which not only helps in improving the engineering properties of soil but will also help to protect our environment. Using such additive can also help to reduce the cost while construction. Hence, this study can be beneficial for many civil engineering projects design.

5.3 Scope of Future Study

- A study can be done by adding stone dust in different percentages and in different curing periods to get more reliable results and design.

- Other synthetic additives in different percentages can be added with stone dust to study their combined effect and their suitability in civil engineering projects.
- More geotechnical tests like consolidation test, triaxial strength test can be performed to study more properties.

Field studies can also be performed to find the suitability of stone dust in foundations.

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Appendix

Variation of geotechnical parameters with increased percentage of stoner dust

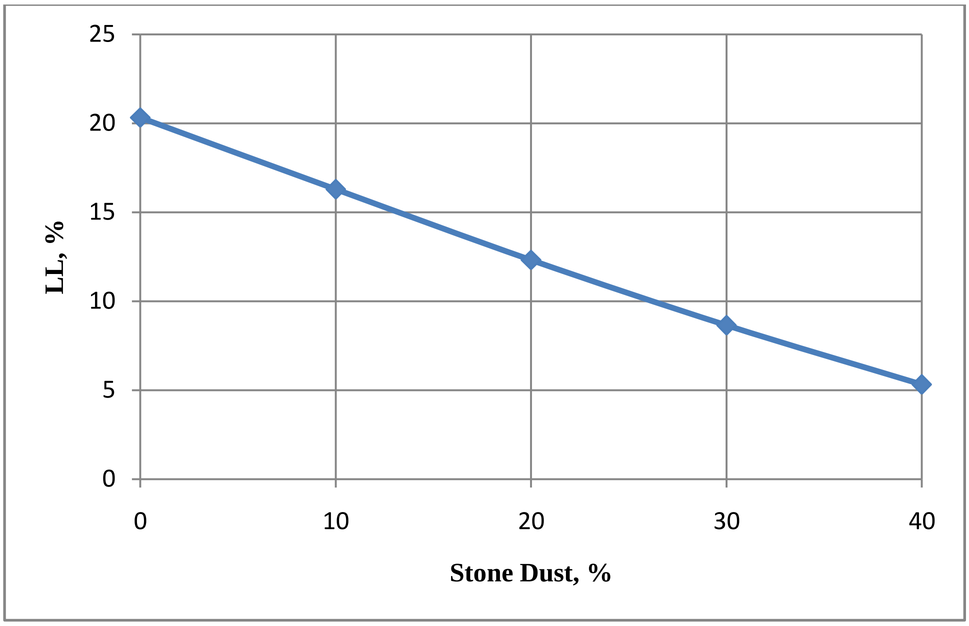
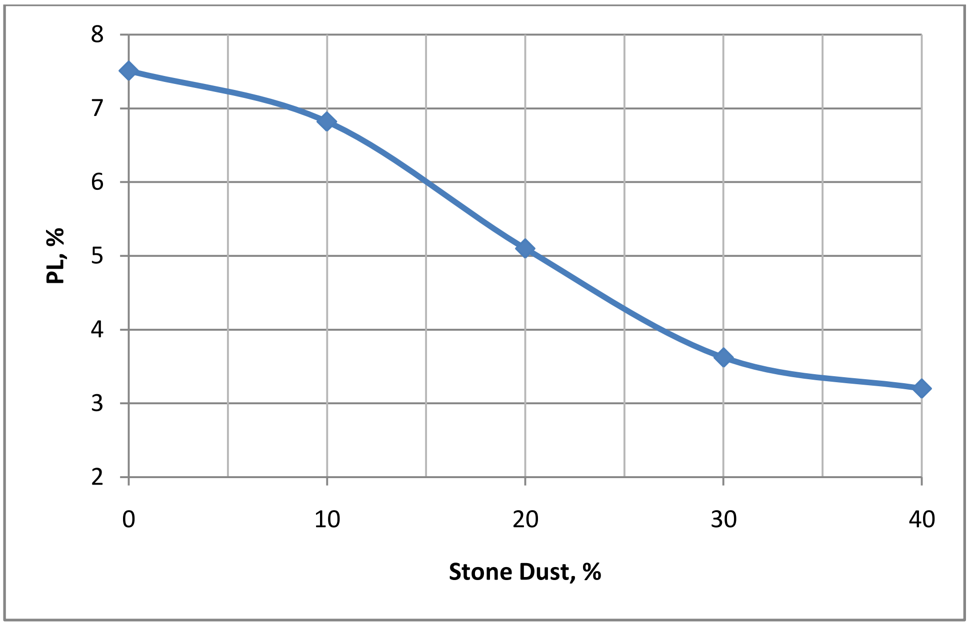
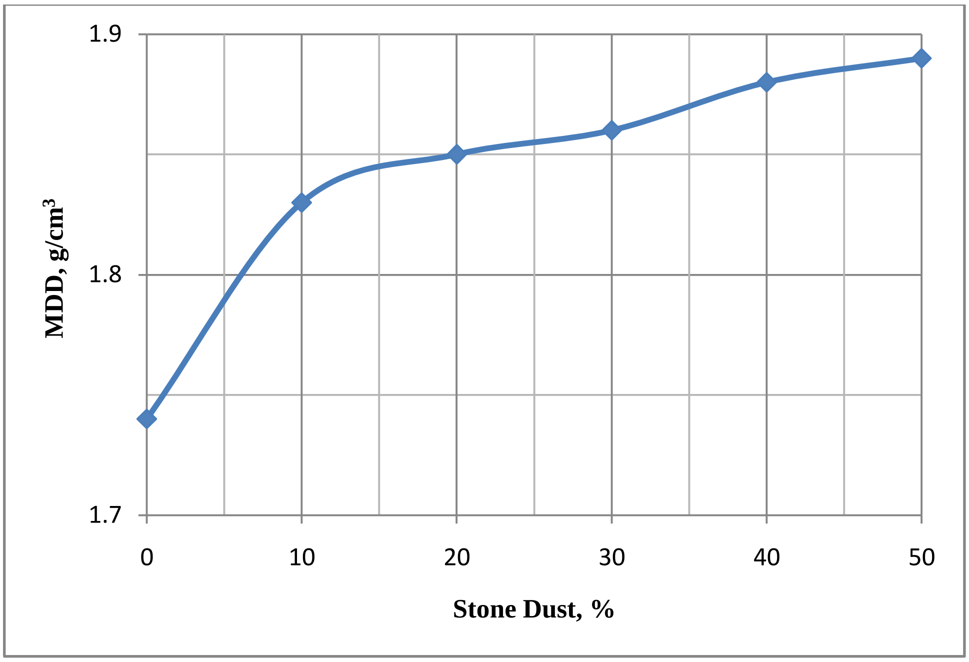


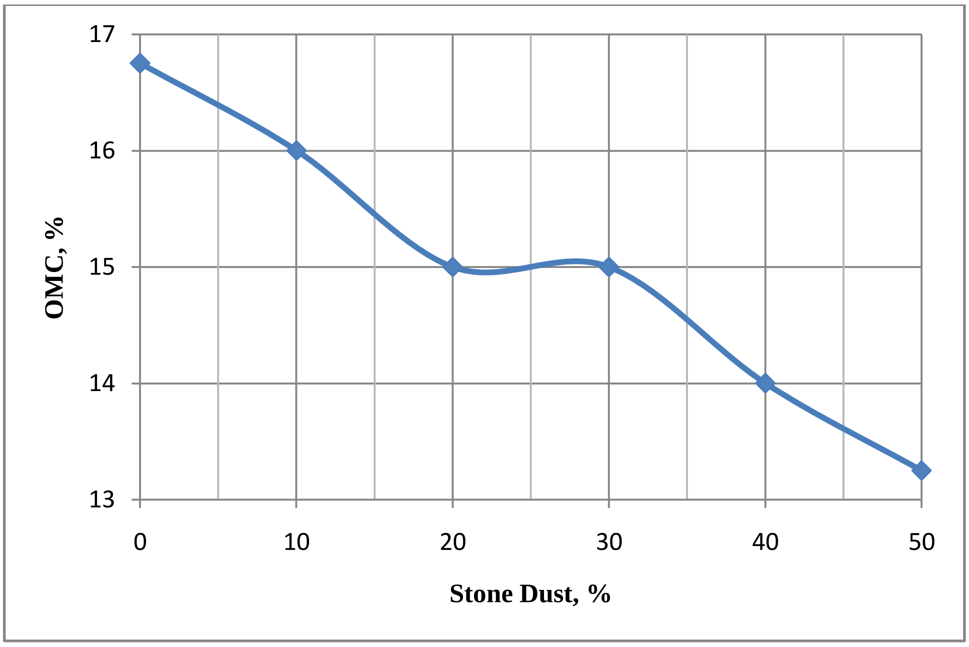
Fig.A1 Variation of Liquid Limit with Increased Percentage of Stone Dust



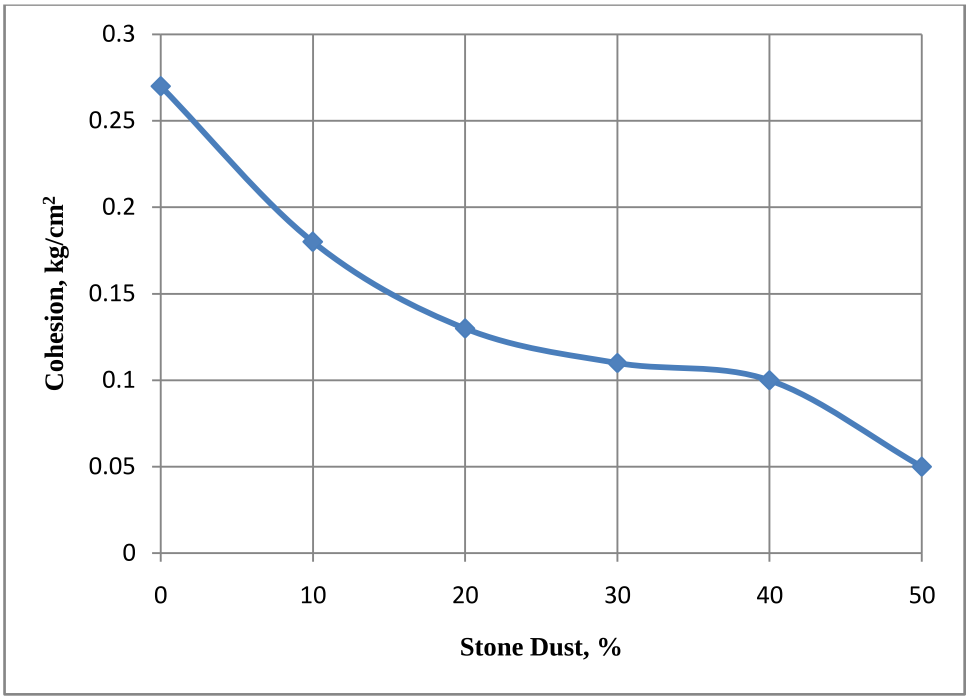
FigA2 Variation of Plastic Limit with Increased Percentage of Stone Dust



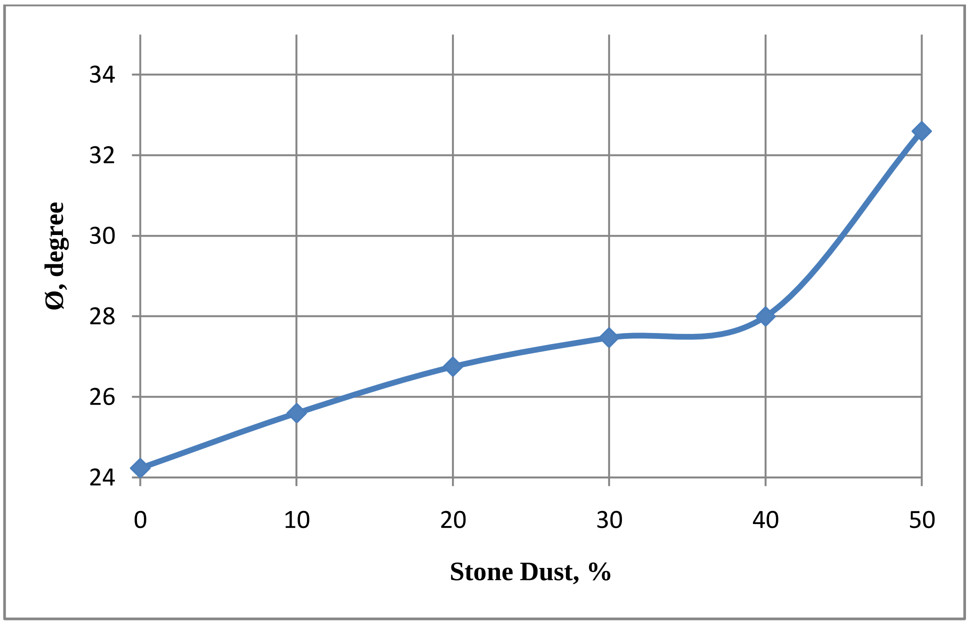
FigA3 Variation of MDD with Increased Percentage of Stone Dust



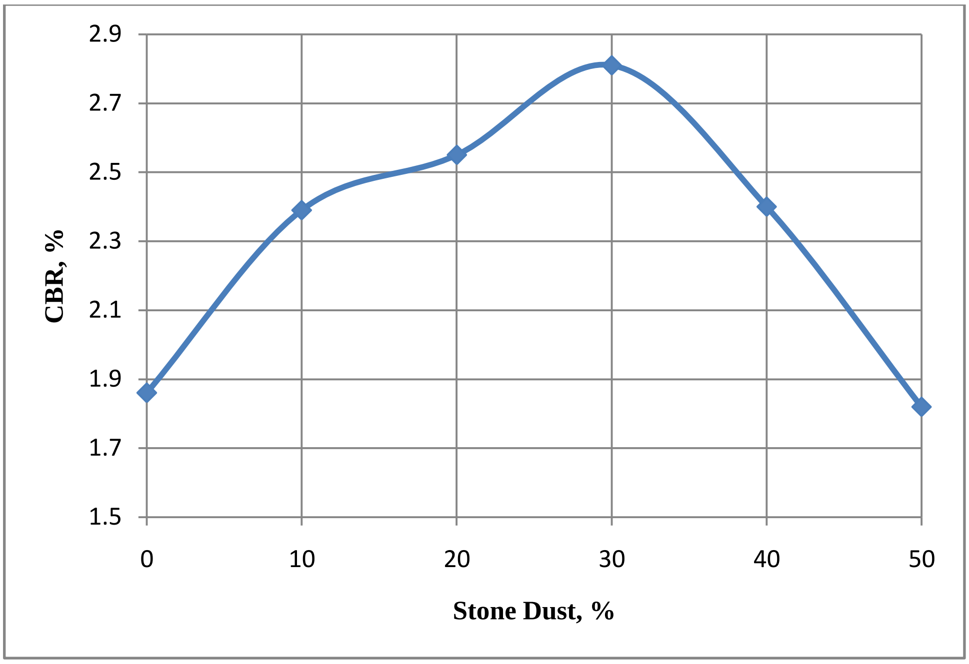
FigA4 Variation of OMC with Increased Percentage of Stone Dust



FigA5 Variation of Cohesion with Increased Percentage of Stone Dust



FigA6 Variation of Angle of Internal Friction with Increased Percentage of Stone Dust



FigA7 Variation of CBR with Increased Percentage of Stone Dust

VITA

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ABSTRACT

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Thesis Title	:	A Sustainable Methodology to Ameliorate Stabilization of Fine Grained Soils			
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In the past few decades there have been a tremendous increase in Civil Engineering activities like construction of buildings, roads, highway and airfields. But, there is inadequacy of appropriate land for such activities because, frequently the soil available at these sites may be clayey, loamy or black cotton soil which have posed complexities to the Geotechnical engineers. As, such soils reflect different types of volumetric changes when acted as supported material below foundations. So, many researchers try to find solution to overcome this issue. And one such economic solution is soil stabilization i.e. to enhance the soil properties by introducing different additives in it.

Initially, introduction of sand to soils followed by compaction was one of the common practices adopted by engineers for stabilization of such weak soil deposits. But, competent substitute for sand becomes necessary with time, as the application of sand is not only limited to geotechnical applications but also in construction industry (Chetia.M. et al. 2018). So, many researchers after doing their critical evaluation and comparison suggest stone dust to be a competent substitute for sand. Stone dust possesses both pozolanic as well as coarser particles in it. These characteristics of stone dust make it an appropriate admixture in fine grained soils in order to achieve desirable strength.

Stone dust is a solid waste that is produced every year to a generous extent of 200 million tonnes per annum. It is a waste that cannot be treated before it is dumped which therefore creates environmental problems. Therefore, utilization of these wastes becomes obligatory. With the advancement in technology, researchers utilize these wastes to ameliorate the geotechnical properties of soil to make it suitable for construction activities.

Over the past several decades, many investigators have devoted considerable efforts to study the suitability of stone dust as additive to modify the geotechnical properties of weak soil deposits. Extensive investigations and testing programmes have been conducted by them to study the strength and compaction characteristics, index properties, consistency when stone dust is used as an additive in weak soils. However, the review of literature has revealed that only a limited study has been done on shear strength properties and permeability.

In view of the above, the main objective of this paper is to investigate the effect of stone dust (introducing in varying percentages) on shear strength and permeability characteristics of clayey soil. In addition, laboratory experiments like consistency limit tests, compaction test, unconfined compression test and CBR tests are also performed with an objective to reach for better results and conclusions.

In order to fulfil our objective, a comprehensive investigation of using stone dust as additive has been planned. Firstly, stone dust was procured from Pal stone industries Limited located at Lalkuan, Uttarakhand. Soil was acquired from the campus of G.B. Pant University of Agriculture & Technology, Pantnagar (Uttarakhand). The overall testing programme was conducted in three phases. In the first phase experimental investigation of soil was carried out. In the second phase laboratory examinations was done on stone dust. In the third and last phase soil was mixed with different percentages of stone dust (i.e. 0%, 10%, 20%, 30%, 40% and 50%).

By analyzing the results obtained from various laboratory experiments, it was concluded that utilization of stone dust as additive is satisfactory when mixed with clayey soil. Plasticity index was observed to decrease from 12.81% to 2.11% with increment in percentage of stone dust from 0 to 40% and beyond this a non plastic behaviour of soil was observed. Proctor results reflect that optimum dry density was decreased from 16.75% to 13.25% while maximum dry density increases from 1.74g/cm^3 to 1.89g/cm^3 on increasing the stone dust percentage from 0 to 50%. The stress at failure and CBR increased up to 30% addition of stone dust and afterwards they show a decreasing trend. The value of cohesion decreases while angle of internal friction increases. The permeability of samples was increased when stone dust was mixed up to 50%. These results were compared with the previous study's results done by many researchers which reveal a fairly good agreement between the observations.

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मुख्य विषय	: मृदा यांत्रिकी और फाउंडेशन इंजीनियरिंग	विभाग	: सिविल इंजीनियरिंग
शोध का षीर्शक	: मृदा के स्थायित्व सुधार की एक टिकाऊ विधि		
सलाहकार	: डा0 अजीत कुमार		

पिछले कुछ दशकों में सिविल इंजीनियरिंग गतिविधियों में इमारतों, सड़कों, राजमार्ग और हवाई क्षेत्रों के निर्माण में जबरदस्त वृद्धि हुई है। लेकिन, इस तरह की गतिविधियों के लिए उपयुक्त भूमि की अपर्याप्तता है क्योंकि, अक्सर इन साइटों पर उपलब्ध मिट्टी में मिट्टी, दोमट या काली कपास मिट्टी हो सकती है, जिन्होंने भू–तकनीकी इंजीनियरों के लिए जटिलताएं उत्पन्न की हैं। जब ऐसी मिट्टी विभिन्न प्रकार के वॉल्यूमेट्रिक परिवर्तनों को दर्शाती है, जब नींव के नीचे समर्थित सामग्री के रूप में कार्य किया जाता है। इसलिए, कई शोधकर्ता इस मुद्दे को दूर करने के लिए समाधान खोजने की कोशिश करते हैं। और ऐसा ही एक आर्थिक समाधान है मृदा स्थिरीकरण अर्थात इसमें विभिन्न योजकों को शामिल करके मृदा गुणों को बढ़ाना।

प्रारंभ में, संघनन के बाद रेत का परिचय इस तरह की कमजोर मिट्टी के निस्तारण के लिए इंजीनियरों द्वारा अपनाई गई सामान्य प्रथाओं में से एक था । लेकिन, रेत के लिए सक्षम विकल्प समय के साथ आवश्यक हो जाता है, क्योंकि रेत का आवेदन केवल भू–तकनीकी अनुप्रयोगों तक ही सीमित नहीं है, बल्कि निर्माण उद्योग में भी है । इसलिए, अपने महत्वपूर्ण मूल्यांकन और तुलना करने के बाद कई शोधकर्ता पत्थर की धूल को रेत के लिए एक सक्षम विकल्प बताते हैं । पत्थर की धूल में दोनों पोजोलैनिक और साथ ही इसमें मोटे कण होते हैं। पत्थर की धूल की ये विशेषताएं इसे वांछनीय शक्ति प्राप्त करने के लिए ठीक दाने वाली मिट्टी में एक उपयुक्त मिश्रण बनाती हैं।

पत्थर की धूल एक ठोस अपशिष्ट है जो हर साल 200 मिलियन टन प्रति वर्ष की उदार सीमा तक उत्पादित होता है। यह एक ऐसा कचरा है जिसे डंप करने से पहले इसका इलाज नहीं किया जा सकता है, जिससे पर्यावरणीय समस्याएं पैदा होती हैं। इसलिए, इन कचरे का उपयोग अनिवार्य हो जाता है। प्रौद्योगिकी में प्रगति के साथ, शोधकर्ता इन कचरे का उपयोग मिट्टी के भू–तकनीकी गुणों को सुधारने के लिए करते हैं ताकि यह निर्माण गतिविधियों के लिए उपयुक्त हो।

पिछले कई दशकों में, कई जांचकर्ताओं ने पत्थर की धूल की उपयुक्तता का अध्ययन करने के लिए काफी प्रयासों को समर्पित किया है ताकि कमजोर मिट्टी के भंडार के भू–तकनीकी गुणों को संशोधित किया जा सके। कमजोर मिट्टी में एक योज्य के रूप में पत्थर की धूल का उपयोग करने पर ताकत और संघनन विशेषताओं, सूचकांक गुणों, स्थिरता का अध्ययन करने के लिए उनके द्वारा व्यापक जांच और परीक्षण कार्यक्रम आयोजित किए गए हैं। हालांकि, साहित्य की समीक्षा से पता चला है कि कतरनी शक्ति गुणों और पारगम्यता पर केवल एक सीमित अध्ययन किया गया है।

उपरोक्त के मद्देनजर, इस पत्र का मुख्य उद्देश्य मिट्टी की मिट्टी की कतरनी शक्ति और पारगम्यता विशेषताओं पर पत्थर की धूल (अलग–अलग प्रतिशत में परिचय) के प्रभाव की जांच करना है। इसके अलावा, निरंतरता परीक्षण परीक्षणों, संघनन परीक्षण, अपुष्ट संपीड़न परीक्षण और सीबीआर परीक्षण जैसे प्रयोगशाला प्रयोगों को भी बेहतर परिणाम और निष्कर्ष पर पहुंचने के उद्देश्य से किया जाता है।

हमारे उद्देश्य को पूरा करने के लिए, पत्थर की धूल को योज्य के रूप में उपयोग करने की एक व्यापक जांच की योजना बनाई गई है। सबसे पहले, उत्तराखंड के लालकुआं स्थित पाल पत्थर उद्योग लिमिटेड से पत्थर की धूल की खरीद की गई थी। जीबी पंत कृषि एवं प्रौद्योगिकी विश्वविद्यालय, पंतनगर (उत्तराखंड) के परिसर से प्राप्त किया गया था। समग्र परीक्षण कार्यक्रम तीन चरणों में आयोजित किया गया था। पहले चरण में मिट्टी की प्रायोगिक जाँच की गई। दूसरे चरण में पत्थर की धूल पर प्रयोगशाला परीक्षण किए गए। तीसरे और अंतिम चरण में मिट्टी को पत्थर की धूल के विभिन्न प्रतिशत (यानी 0%, 10%, 20%, 30%, 40% और 50%) के साथ मिलाया गया था।

विभिन्न प्रयोगशाला प्रयोगों से प्राप्त परिणामों का विश्लेषण करके , यह निष्कर्ष निकाला गया कि मिट्टी के मिट्टी के साथ मिश्रित होने पर पत्थर की धूल का उपयोग योज्य के रूप मंन संतोषजनक है। 0 से 40 % तक पत्थर की धूल के प्रतिशत में वृद्धि के साथ 12. 81% से 2.11% तक की कमी के साथ प्लास्टिसिटी इंडेक्स देखा गया था और इसके अलावा मिट्टी का एक गैर प्लास्टिक व्यवहार देखा गया था। प्रॉक्टर परिणाम दर्शाते हैं कि इष्टतम शुष्क घनत्व 16.75% से घटकर 13.25% हो गया, जबकि पत्थर का प्रतिशत प्रतिशत 0 से बढ़ाकर 50% करने पर अधिकतम शुष्क घनत्व 1.74g / सीसी से 1.89g / सीसी तक बढ़ जाता है। असफलता पर तनाव और सीबीआर पत्थर की धूल के 30ः तक बढ़ गया और बाद में वे एक घटती प्रवृत्ति दिखाते हैं। आंतरिक घर्षण के कोण में वृद्धि होने पर सामंजस्य का मूल्य घट जाता है। पत्थरों की धूल 50% तक मिश्रित होने पर नमूनों की पारगम्यता बढ़ गई थी । इन परिणामों की तुलना कई शोध र्कर्ताओं द्वारा किए गए पिछले अध्ययन के परिणामों के साथ की गई थी, जो टिप्पणियों के बीच एक निष्पक्ष गीत अच्छा समझौता प्रकट करते हैं।

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