IMPROVEMENT OF SOIL USING FLY ASH & STONE DUST

Under guidance of
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Submitted By:-

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CERTIFICATE

This is to certify that the work titled "**IMPROVEMENT OF SOIL USING FLY ASH & STONE DUST**" submitted by Sugam Sharma in partial fulfilment for the award of degree of B. Tech. Civil Engineering of Jaypee University of Information Technology, Waknaghat has been carried out under my supervision. This work has not been submitted partially or wholly to any other University or Institute for the award of this or any other degree or diploma.

Sign of Supervisor	……………………..
Name of Supervisor	Mr.Ashish Kumar
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Date	……………………..

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Abstract

Soil improvement is of major concern in the construction activities due to rapid growth of urbanization and industrialization. The term soil improvement is used for the techniques which improve the index properties and other engineering characteristic of weak soils. In India expansive soil cover about 0.8x106 km2 area which is approximately one-fifth of its surface area.

These soils contain montmorillonite mineral; due to this they swell and shrink 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. Expansive soils can be stabilised by the addition of a small percentage of lime and other admixtures.

These techniques have been used for many construction purposes, notably in highway, railroad and airport construction to improve subgrades and sub-bases. The Granite dust is a by-product produced in granite factories while cutting huge granite rocks to the desired shapes. About 3000 metric ton of granite dust slurry is produced per day as a by-product during manufacturing of granite tiles and slabs from the raw blocks. The marble and granite cutting industries are dumping these wastes in nearby pits or open lands. This leads to serious environmental pollution and occupation of vast area of land especially after the slurry dries up.This study envisages the effect of granite dust on the consistency limits and differential free swell (DFS) of Black Cotton Soil mixed with 5% lime and 0 to 30% granite dust by weight of soil.

Contents

INTRODUCTION

In many road construction projects, if weak soils exist, stabilization and improvement of their properties is necessary. The stabilization process aims at increasing the soil strength and reducing its permeability and compressibility. The stabilization processes may include mechanical, chemical, electrical or thermal processes (Ingles and Metcalf, 1972). The process used depends on the type of soil at the site, the time available to execute the project and the stabilization cost compared to the overall cost of the project and to the cost of full replacement of the soil at the site. The engineer may consider one method or several methods together.

The main objective of this research is to utilize stone powder and lime in the improvement of fine soil properties at the routes of medium traffic roads. The two additives are cheap and available in commercial quantities in Palestine. This research will help dispose large quantities of stone slurry and lime by utilizing them in the construction of roads. Therefore, the reduction in the exploitation of raw materials and the mitigation of threats to the environment by stone slurry wastes are of great importance. The variables of this research will include the two additives and three percentages 10%, 20% and 30%.

LITERATURE REVIEW

This research is an attempt to investigate the effect of stone powder and lime on the strength, compaction and CBR properties of fine grained soil. The basic properties: direct shear, compaction and CBR were determined first. The stone powder and lime were added at specific percentages (10%, 20% and 30%) by weight of soil and mixed with the optimum moisture content obtained from the compaction test. The direct shear, compaction and CBR tests were conducted directly without curing or soaking of the specimens. The results revealed that the addition of 30% stone powder has increased the angle of internal friction (φ) by about 50% and reduced cohesion by about 64%. The addition of 30% of lime has decreased the friction angle and cohesion by 57% and 28%, respectively. The maximum dry density and optimum moisture content decreased slightly by addition of 30% stone powder, however, the addition of 30% lime decreased the maximum dry density and optimum moisture content by 19% and 13.5%, respectively. The CBR values have increased from 5.2 to 16 and 18 by the addition of 30% stone powder and lime, respectively. The thicknesses of flexible pavement were determined based on the CBR values and assumed daily traffic volume and found to be reduced from 38 cm for soil without additives to 20 cm and 17cm by the addition of 30% of stone powder and lime respectively.

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. This project presents the results of an experimental program undertaken to investigate the effect of stone dust &flyash 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/flyash alone to the soil in controlling the swelling nature.

OBSERVATIONS FROM THE LITERATURE REVIEW

- The stone powder and lime were added at specific percentages (10%, 20% and 30%) by weight of soil and mixed with the optimum moisture content obtained from the compaction test.

- The results revealed that the addition of 30% stone powder has increased the angle of internal friction (φ) by about 50% and reduced cohesion by about 64%.

- The maximum dry density and optimum moisture content decreased slightly by addition of 30% stone powder, however, the addition of 30% lime decreased the maximum dry density and optimum moisture content by 19% and 13.5%, respectively.

- The CBR values have increased from 5.2 to 16 and 18 by the addition of 30% stone powder and lime, respectively.

- 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.

- 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.

OBJECTIVES OF THE PROJECT

- To analyze the effect of various percentages of fly ash on CBR value of soil.

- To analyze the effect of various percentages of stone dust on CBR value of soil.

- To analyze the effect of various percentages of fly ash on shear strength value of soil.

- To analyze the effect of various percentages of stone dust on shear strength value of soil.

<u>Materials to Be Used:</u>

Fly Ash:

Fly ash, also known as flue-ash, is one of the residues generated in combustion, and comprises the fine particles that rise with the flue gases. Ash which does not rise is termed bottom ash. In an industrial context, fly ash usually refers to ash produced during combustion of coal. Fly ash is generally captured by electrostatic precipitators or other particle filtration equipment before the flue gases reach the chimneys of coal-fired power plants, and together with bottom ash removed from the bottom of the furnace is in this case jointly known as **coal ash**. Depending upon the source and makeup of the coal being burned, the components of fly ash vary considerably, but all fly ash includes substantial amounts of silicon dioxide (SiO_2) (both amorphous and crystalline) and calcium oxide(CaO), both being endemic ingredients in many coal-bearing rock strata.

Fly ash material solidifies while suspended in the exhaust gases and is collected by electrostatic precipitators or filter bags. Since the particles solidify rapidly while suspended in the exhaust gases, fly ash particles are generally spherical in shape and range in size from 0.5 µm to 300 µm. The major consequence of the rapid cooling is that only few minerals will have time to crystallize and that mainly amorphous, quenched glass remains. Nevertheless, some refractory phases in the pulverized coal will not melt (entirely) and remain crystalline. In consequence, fly ash is a heterogeneous material. SiO_2, Al_2O_3, Fe_2O_3 and occasionally CaO are the main chemical components present in fly ashes. The mineralogy of fly ashes is very diverse. The main phases encountered are a glass phase, together with quartz, mullite and the iron oxides hematite, magnetite and/or maghemite. Other phases often identified arecristobalite, anhydrite, freelime, periclase, calcite, sylvite, halite, portlandite, rutile and anatase . The Ca-bearing minerals anorthite, gehlenite, akermanite and various calcium silicates and calcium aluminates identical to those found in Portland cement can be identified in Ca-rich fly ashes. The mercury content can reach1 ppm , but is generally included in the range 0.01 - 1 ppm for bituminous coal. The concentrations of other trace elements vary as well according to the kind of coal combusted to form it. In fact, in the case of bituminous coal, with the notable exception of boron, trace element concentrations are generally similar to trace element concentrations in unpolluted soils.

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TYPES OF FLY ASH

Class F fly ash

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

Class C fly ash:

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

CHEMICAL COMPOSITION OF FLY ASH

	Lime	Cement	Ash F	Ash C
SiO_2	2.1	18.6	32.5	39.9
Al_2O_3	1	5	5.59	16.7
Fe_2O_3	0.5	2.8	10.8	5.8
Sum	3.6	26.4	48.89	62.4
CaO reactive	90	1	6.0	3.3
TOC			0.2	0.5

STONE DUST

Stone dust also known as rock powders, rock minerals, rock flour, soil remineralization, and mineral fines, consists of finely crushed rock, processed by natural or mechanical means, containing minerals and trace elements widely used in organic farming practices.

The igneous rocks basalt and granite often contain the highest mineral content, whereas limestone, considered inferior in this consideration, is often deficient in the majority of essential macro-compounds, trace elements, and micronutrients.

Rock dust is not a fertilizer, for it lacks the qualifying levels of nitrogen, potassium, and phosphorus.

TABLE OF COMPOSITION OF FLY ASH

Element	unit	
calcium	%w/w	6.44
iron	%w/w	10.5
magnesium	%w/w	6.54
sulfur	%w/w	0.21
potassium	%w/w	1.25
phosphorus	mg/kg	30.30
cobalt	mg/kg	35
copper	mg/kg	43
manganese	mg/kg	790
molybdenum	mg/kg	<5
zinc	mg/kg	92
silicon	%w/w	21.6

TESTS TO BE PERFORMED:

CBR

The California bearing ratio (CBR) is a penetration test for evaluation of the mechanical strength of road subgrades and base courses. It was developed by the California Department of Transportation before World War II.

The test is performed by measuring the pressure required to penetrate a soil sample with a plunger of standard area. The measured pressure is then divided by the pressure required to achieve an equal penetration on a standard crushed rock material. The CBR test is described in ASTM Standards D1883-05 (for laboratory-prepared samples) and D4429 (for soils in place in field), and AASHTO T193. The CBR test is fully described in BS 1377 : Soils for civil engineering purposes : Part 4, Compaction related tests.

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

$CBR = \frac{\mathrm{p}}{p_s}$

CBR = CBR [%]

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

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

California Bearing Ratio (CBR) – The ratio expressed in percentage of force per unit area required to penetrate a soil mass with a circular plunger of 50 mm diameter at the rate of 1.25 mm/min to that required for corresponding penetration in a standard material. The ratio is usually determined for penetration of 2.5 and 5.0 mm. Where the ratio at 5 mm is consistently higher than that at 2.5 mm, the ratio at 5 mm is used.

Moulds with base plate stay rod and wing nut:

Collar

Spacer disc

Metal Rammer

Expansion Measuring Apparatus

Weights

Loading Machine

Penetration plunger

Dial gauges –two dial gauges reading to 0.01 mm.

Sieves- 4.75 mm IS Sieve and 19 mm IS Sieve.

Miscellaneous Apparatus

PROCEDURE

The mould containing the specimen with the base plate in position but the top face exposed shall be placed on the lower plate of the testing machine. Surcharge weights, sufficient to produce an intensity of loading equal to the weight of the base material and pavement shall be placed on the specimen. If the specimen has been soaked previously, the surcharge shall be equal to that used during the soaking period. To prevent upheaval of soil into the hole of the surcharge weights, 2.5 kg annular weight shall beplaced on the soil surface prior to seating the penetration plunger after which the remainder of the surcharge weight shall be placed. The plunger shall be seated under a load of 4 kg so that full contact is established between the surcharge of the specimen and the plunger. The load and deformation gauges shall then be set to zero (In other words, the initial load applied to the plunger shall be considered as zero when determining the load penetration relation). Load shall be applied to the plunger into the soil at the rate of 1.25 mm/min. Reading of the load shall be taken at penetrations of 0.5, 1.0, 1.5, 2.0, 2.5, 4.0, 5.0, 7.5, 10.0 and 12.5 mm(The maximum load and penetration shall be recorded if it occurs for a penetration of less than 12.5 mm). The plunger shall be raised and the mould detached from the loading equipment. About 20 to 50 g of soil shall be collected from the top 30 mm layer of the specimen and the water content determined according to IS: 2720 (Part 2) – 1973. If the average water content of the whole specimen is desired, water content sample shall be taken from the entire depth of the specimen. The undisturbed specimen for the test should be carefully examined after the test is completed for the presence of any oversize soil particles which are likely to affect the results if they happen to be located directly below the penetration plunger.

Liquid limit test

Liquid Limit (W_L) : LA flow curve' shall be plotted on a semi logarithmic graph representing water content on the arithmetical scale and the number drops on the logarithmic scale. The flow curve is a straight line drawn as nearly as possible through the four or more plotted points. The moisture content corresponding to 25 drops as read from the curve shall be rounded off to the nearest whole number and reported as the liquid limit of the soil.

Flow Index (I_f) :The flow curve (straight line) plotted on semi-logarithmic graph as in shall be extended at either end so as to intersect the ordinates corresponding to 10 and 100 drops. The slope of this line expressed as the difference in water contents as 10 drops and at 100 drops shall be reported as the flow index.

Plastic limit test

It is the moisture content at which soil begins to behave as a plastic material

Apparatus:
Porcelain evaporating dish
Flat glass plate
Spatula
Palette knives
Surface for rolling
Containers
Balance
Oven
Rod

Soil sample

A sample weighing gm from the thoroughly mixed portion of the material passing 425-micron IS sieve obtained in accordance with shallbe taken.

Test procedure

1. The soil sample shall be mixed thoroughly with distilled water in an evaporating dish or on the flat glass plate till the soil mass becomes plastic enough to be easily moulded with fingers.
2. In the case of clayey soils the plastic soil mass shall be left to stand for a sufficient time (24 hours) to ensure uniform distribution of moisture throughout the soil.
3. A ball shall be formed with about 8 gm of this plastic soil mass and rolled between the fingers and the glass plate with just sufficient pressure to roll the mass into a thread of uniform diameter throughout its length.
4. The rate of rolling shall be between 80 and 90 strokes/min counting a stroke as one complete motion of the hand forward and back to the starting position again.
5. The rolling shall be done till the threads are of 3mm diameter.
6. The soil shall then be kneaded together to a uniform mass and rolled again.
7. This process of alternate rolling and kneading shall be continued until the thread crumbles under the pressure required for rolling and the soil can no longer be rolled into a thread.
8. The crumbling may occur when the thread has a diameter greater then 3 mm.

9. This shall be considered a satisfactory end point, provided the soil has been rolled into a thread 3mm in diameter immediately before. At no time shall an attempt be made to produce failure at exactly 3 mm diameter by allowing the thread to reach 3mm, then reducing the rate of rolling or pressure or both, and continuing the rolling without further deformation until the thread falls apart.

DETERMINATION

The moisture content (%) at which the soil when rolled into threads of 3 cm in diameter, will crumble gives the plastic limit.

PLASTICITY INDEX

The plasticity index is calculated as the difference between the liquid limit and plastic limit .
Plasticity index (Ip)=liquid limit (W_l)-plastic limit(W_p)

SPECIFIC GRAVITY BY DENSITY BOTTLE METHOD

Specific gravity G is defined as the ratio of the weight of an equal Volume of distilled water at that temperature both weights taken in air.

APPARATUS:

The following apparatus is required:

1.Two density bottles of approximately 50 ml capacity with Stoppers.
2. A water-bath maintained at a constant temperature 27C
3. A vacuum desiccator (size 200mm to 250mm in diameter)

TEST PROCEDURE

1. The complete density bottle with stopper shall be dried at 105 - 110 C cooled in the desiccator and weighed to the nearest 0.001 gm.

2. A 5 to 10gm Sub sample shall be obtained by riffling, and oven dried at 105 to 110C.
3. The bottle and contents together with the stopper shall be weighed to the nearest 0.001 gm.
4. Sufficient air- free distilled water shall be added so that the soil in the bottle is just covered.
5. AIR BUBBLES from the bottle are removed and further air-filled liquid added until the bottle is full.
6. The bottle is wipped dry and the whole weighed to the nearest .001 gm.
7. Remove the soil from the bottle
8. Bottle is cleaned ,wiped and dried again
9. Now air free liquid is added until the bottle is full
10. The bottle is wiped dry and the whole weighed to the nearst .001 gm
11. Two determinations of the specific gravity of the soil samples shall be made.

DETERMINATION

1. The specific gravity of the soil particles G shall be measured at room temperature If water has been used as the air free liquid, then the following equation shall be used.

$$G=(M_2- M_1)/(M_4 - M_1)- (M_3-M_2)$$

Where

M_1 = mass of density bottle in gm

M_2 =massof bottle and dry soil in gm

M_3 =mass of bottle, soil and water in gm

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

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GRAIN SIZE ANALYSIS BY SIEVE ANALYSIS

APPARATUS

Balance

I.S sieve

Mechanical sieve shaker

PROCEDURE

1. The proportion of soil sample retained on 75 micron I.S sieve is weighed and recorded weight of soil sample is as per I.S 2720.
2. I.S sieves are selected and arranged in the order as shown in the table.
3. The soil sample is separated into various fractions by sieving through above sieves placed in the above mentioned order.
4. The weight of soil retained on each sieve is recorded.
5. The moisture content of soil if above 5% it is to be measured and recorded

DETERMINATION

Draw graph between log sieve size vs % finer. The graph is known as grading curve. corresponding to 10%, 30% and 60% finer, obtain diameters from graph are designated as D_{10}, D_{30}, D_{60}.

MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENTUSING LIGHT COMPACTION

APPRATUS

1) Cylindrical metal mould:It shall be either of 100 mm diameter and 1000 cm^3

3) Balances

4) Oven

5) Container

6) Hammer of 2.6 kg

8) Mixing Tools

PROCEDURE

1. A 5 kg sample of air dried soil passing the 19 mm IS test sieve shall be taken (see Note 2). The sample shall be mixed thoroughly with a suitable amount of water depending on the soil type.

2. The mould of 1000 cm^3 capacity with base plate attached shall be weighed to the nearest 1gm (m1). The mould shall be placed on a solid base, such as a concrete floor or plinth and the moist soil shall be compacted into the mould, with the extension attached, in three layers of approximately equal mass, each layer being given 25 blows from the 2.6 Kg rammer dropped from a height of 310 mm above the soil.

3. The blows shall be distributed uniformly over the surface of each layer. The operator shall ensure that the tube of the rammer is kept clear of soil so that the rammer always falls freely.

4. The amount of soil used shall be sufficient to fill the mould, leaving not more then about 6 mm to be struck off when the extension is removed.

5. The extension shall be removed and the compacted soil shall be leveled off carefully to the top of the mould by means of the straightedge. The mould and soil shall then be weighed to 1gm (m^2)

6. The compacted soil specimen shall be removed from the mould and placed on the mixing tray. The water content of a representative sample of the specimen shall be determined.

7. The remainder of the soil specimen shall be broken up, rubbed through the 19 mm IS test sieve, and then mixed with the remainder of the original sample. Suitable increments of water (see Note 6) shall be added.

8. Observation table is shown in Appendix A.

DETERMINATION

GRAPH IS PLOTTED water content on abscissa and dry density on ordinate. The maximum value of DRY density gives maximum dry densityand the corresponding water content gives optimum moisture content.

Wikipedia images of compaction test

UNCONFINED COMPRESSION TEST

Unconfined compression test is the load per unit area at which an unconfined cylindrical specimen of soil will fail in the axial compression test.

APPRATUS

1. Compression Device
2. proving Ring
3. Deformation Dial Gauge
4. Vernier Calipers
5. Timer
6. Oven
7. Weighing Balances
8. Mixing tools

PREPERATION OF TEST

SPECIMEN SIZE

The specimen for the test shall have a minimum diameter of 38 mm and the Largest particle contained within the test specimen shall be smaller than 1/8 of the specimen diameter.The height to diameter ratio shall be within 2 to 2.5

UNDISTRIBUTED SAMPLE

1. Undisturbed specimens shall be prepared from large undisturbed samples or samples secured in accordance with IS: 2132:1986.

2. When samples are pushed from the drive sampling tube the ejecting device shall be capable of ejecting the soil core from the sampling tube in the same direction of travel in which the sample entered the tube and with negligible disturbance of the sample. Conditions at the time of removal of the sample may dictate the direction of removal but the principal concern should be to keep the degree of disturbance negligible.

3. The specimen shall be handled carefully to prevent disturbance, change in cross section, or loss of water. If any type of disturbance is likely to be caused by the ejection device the sample tube shall be split lengthwise or be cut off in small sections to facilitate removal of the specimen without disturbance. If possible carved specimen should be prepared in a humid room to prevent, as far as possible, change in water content of the soil.

4. The specimen shall be of uniform circular cross section with ends perpendicular to the axis of the specimen.

5. Specimen of required size may be carved from large undisturbed specimen. When sample condition permits use of a vertical lathe which will accommodate the total sample the same may be used as an aid in carving the specimen to the required diameter Tube specimens may be tested without trimming except for squaring of ends.

6. Where the prevention of the possible development of appreciable capillary forces is required the specimen shall be sealed with rubber membranes, thin plastic coatings, or with a coating of grease or sprayed plastic immediately after preparation and during the entire testing cycle.

7. Representative sample cutting taken from the tested specimen shall be used for the determination of water content.

PROCEDURE

1. The initial length diameter and weight of the specimen shall be measured and the specimen placed on the bottom plate of the loading device .The upper plate shall be adjusted to make contact with the specimen.

2. The deformation dial gauge shall be adjusted to a suitable reading preferably in multiples of 100 Force shall be applied so as to produce axial strain at a rate of 0.5 to 2 percent per minute causing failure with 5 to 10. The force reading shall be taken at suitable intervals of the deformations dial reading.

3. The specimen shall be compressed until failure surfaces have definitely developed or the stress strain of 20 percent is reached

4. The failure pattern shall be sketched carefully and shown on the date sheet or on the sheet presenting the stress strain plot. The angle between the failure surface and the horizontal may be measured if possible and reported.

5. The water content of the specimen shall be determined in accordance with using samples taken from the failure zone of the specimen.

DETERMINATION

GRAPH is plotted between axial strain on abscissa and axial strain on ordinate.The maximum value of axial strain gives unconfined compressive strength of soil.

http://www.impact-test.co.uk/products/4801-Direct-Digital-Shear-Apparatus/

DIRECT SHEAR TES*T*

APPRATUS

1.The shear box grid plates, porous stones, base plates and loading pad and water jacket shall confirm to IS: 11229-1985.
2.Loading frame
3.Weights

4.Proving Ring
5.Micrometer dial-gauges – accurate to 0.01 mm; one suitably mounted to measure horizontal movement and the other suitably mounted to measure the vertical compression of the specimen.
6.Sample trimmer or core cutter
7.Stop cock
8.Balance of 1 kg capacity
9.spatula

PROCEDURE

The shear box with the specimen, plane grid plate over the base plate at the bottom of the specimen, and plane grid plate at the top of the specimen should be fitted into position in the load frame. The serration of the grid plates should be at right angles to the direction of shear. The loading pad should be placed on the top grid plate. The water jacket should be provided so that the sample does not get rate of longitudinal displacement/shear stress application so adjusted that no drainage can occur in the sample during the test. The upper part of the two parts of the box. The test may now conduct by applying horizontal shear load to failure or to 20 percent longitudinal displacement, which ever occur first. The shear load readings indicated by the proving ring assembly and the corresponding longitudinal displacement should be noted at regular intervals. If necessary, the vertical compression, if any of the soil specimens may be measured to serve as a check to ensure that drainage has not taken place from the soil specimen. At the end of the test, the specimen should be removed from the box and the final moisture content measured

DETERMINATION

The maximum shear stress & the corresponding longitudinal displacement and applied normal stress should be recorded for each test and the results should be presented in the form of a graph in which the applied normal stress in plotted as abscissa and the maximum shearing stress is plotted as ordinate to the same scale. The angle which the resulting straight line makes with the horizontal axis and the intercept which the straight line makes with the vertical axis shall be reported as the angle of shearing resistance and cohesion intercept respectively.

TRIAXIAL TEST

This test is for the determination of the compressive strength of a specimen of saturated cohesive soil in the triaxial compression apparatus under conditions in which the cell pressure is maintained constant and there is no change in the total water content of the specimen.

This test is limited to specimens in the form of right cylinders of normal diameter 38, 50, 70 and 100 mm and of height approximately equal to twice the nominal diameter. In case of remoulded samples ratio of diameter of specimen to maximum size of particle in the soil should not be less than 5.

APPARATUS

1. Split Mould
2. Trimming knife –
3. Piano wire saw
4. Metal straightedge
5. Metal scale
6. Non-corrodible metal or plastic end-caps-
7. Seamless Rubber Membrane –.
8. Membrane Stretcher- to suit the size of the specimen.
9. Rubber rings – of circular cross-section to suit the diameter of the end caps.
10. Apparatus for moisture content determination

a) **Triaxial Test Cell-** A triaxial test cell of dimensions appropriate to the size of the specimen, capable of being opened for the insertion of the specimen, suitable for use with the fluid selected for use at internal pressure up to 1 MPa and provided with a means of applying additional axial compressive load to the specimen by means of a loading ram. A transparent chamber is recommended. The base of the cell shall be provided with a suitable central pedestal with drainage outlets with valves.

b) **An Apparatus For Applying And Maintaining The Desired Pressure On The FluidWithin The Cell** – To an accuracy of 10 KPa (preferably 5 KPa) with a gauge for measuring the pressure. The gauge shall be regularly calibrated.

1 kPa = 100 kgf/m^2	1 kN = 100 kgf

c) **Machine Capable of Applying Axial Compression to the Specimen**– At convenient speeds to cover the range 0.05 to 5 mm per minute. The machine should have a capacity of 50 kN. A means of measuring the axial compression of the specimen to an accuracy of 0.01 mm shall be provided and the machine shall be capable of applying an axial compression of about one third the height of the specimen tested.

4. TESTING PROCEDURE

1. The specimen placed centrally on the pedestal of the tri-axial cell. The cell shall be assembled with the loading ram initially clear of the top cap of the specimen and the cell containing the specimen shall be placed in the loading machine. The operating fluid shall be admitted to the cell and the pressure raised to the desired value.

2. The loading machine shall be adjusted to bring the loading ram a short distance away from the seat on the top cap of the specimen and the initial reading of the load measuring gauge shall be recorded. The loading machine shall then be further adjusted to bring the loading ram just in contact with the seat on the top cap of the specimen and the initial reading of the gauge measuring the axial compression of the specimen shall be recorded. A rate of axial compression shall be selected such that failure is produced within a period of approximately 5 to 15 minutes. The test shall be commenced a sufficient number of simultaneous readings of the load and compression measuring gauges being taken to define the stress strain curve. The test shall be continued until the maximum value of the stress has been reached. The specimen shall then be unloaded and the final reading of the load measuring gauge shall be recorded as a check on the initial reading.

3. The cell shall be determined of fluid and dismantled and the specimen taken out. The rubber membrane shall be removed from the specimen and the mode of failure shall be noted. The specimen shall be weighed and samples for the determination of the moisture content of the specimen shall be taken. If there is a moisture change in the specimen it should be recorded and discretion used with regard to acceptability of the test.

OBSERVATIONS

1. Initial diameter of specimen
2. Initial weight of specimen
3. Bulk density
4. Moisture content.
5. Load gauge no.
6. Load gauge Constant
7. Cell pressure ($\sigma 3$)
8. Rate of strain
9. Description of sample

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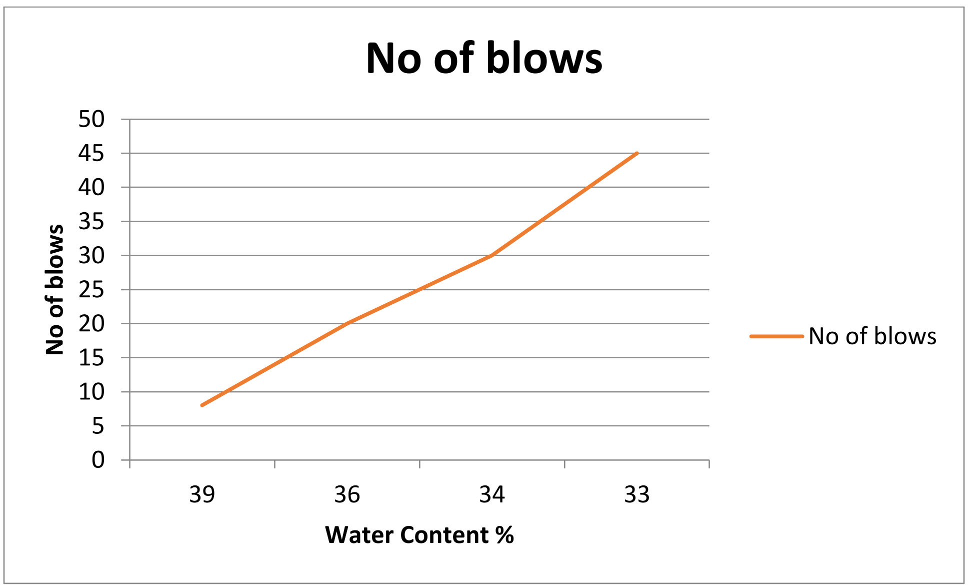
RESULTS AND DISCUSSION

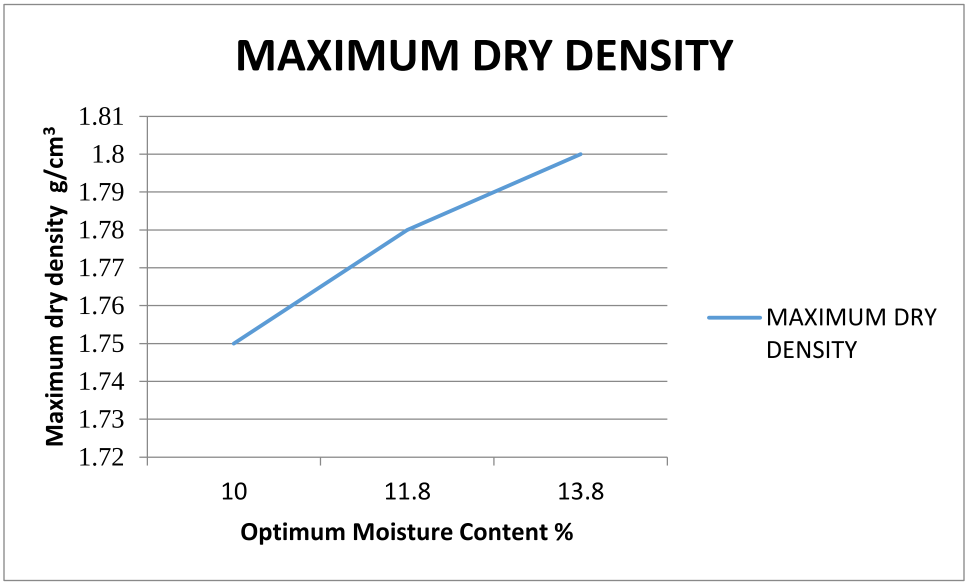
Liquid limit

Water content(%)	No of blows
39	8
36	20
34	30
33	45



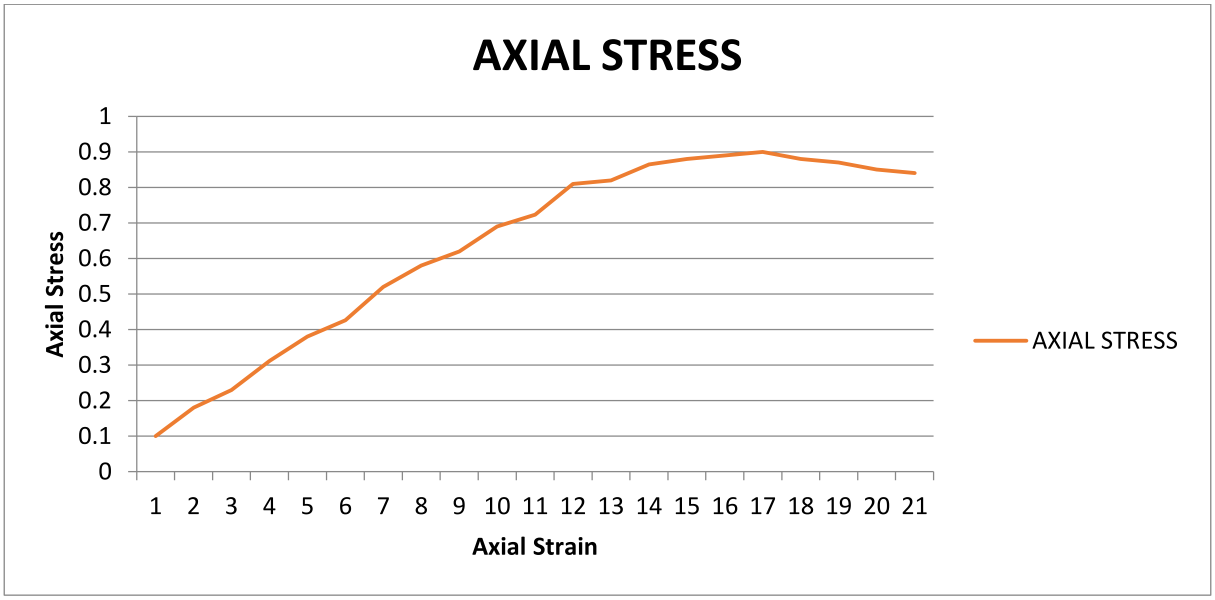
MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT

MAXIMUM DRY DENSITY	OPTIMUM MOISTURE CONTENT
1.75	10
1.78	11.8
1.8	13.8



UNCONFINED COMPRESSIVE STRENGTH

AXIAL STRESS	AXIAL STRAIN
0	.1
.012	.18
.02	.23
.024	.312
.031	.38
.04	.426
.045	.52
.051	.58
.058	.62
.065	.69
.07	.723
.0.079	.81
.085	.82
.095	.865
.102	.88
.12	.89
.128	.9
.123	.88
.125	.87
.138	.85
.141	.84



s. no	PROPERTIES	REPETITION 1	REPETITION 2	REPETITION 3	AVERAGE
1	LIQUID LIMIT	37.54	36.99	37.9	37.46
2	PLASTIC LIMIT	14.29	14.50	13.99	14.26
3	SPECIFIC GRAVITY	2.55	2.62	2.50	2.56
4	MAXIMUM DRY DENSITY	1.80	1.82	1.78	1.80
5	Optimum moisture content	15.1	15.0	15.5	15.2
6	UNCONFINED COMPRESSION STRENGTH	.98	1.02	.93	.97
7	DIRECT SHEAR TEST	S 10.68 .066	S 11.4 .076	S 10.87 .072	S 10.98 .071
8	TRIAXIAL TEST				
9	CBR TEST				

DIRECT SHEAR TEST PERFORMED

TEST ON SOIL USING FLY ASH WITH 15% FLY ASH

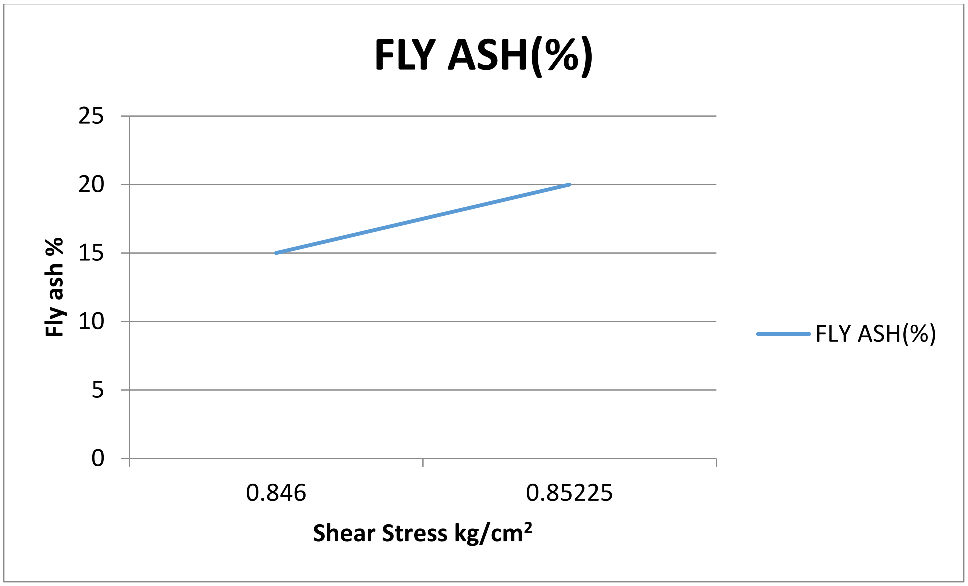
NORMAL STRESS	SHEAR STRESS
.5	.336
1	.668
1.5	1.001
2	1.379
AVG	.846

DIRECT SHEAR TEST PERFORMED WITH 20% FLY ASH

NORMAL STRESS	SHEAR STRESS
.5	.337
1	.677
1.5	1.015
2	1.38
AVG	.85225

FLY ASH CC SHEAR STRESS

FLY ASH(%)	SHEAR STRESS(kg/cm2)
15	.846
20	.85225



PLASTIC LIMIT WITH 20% FLY ASH

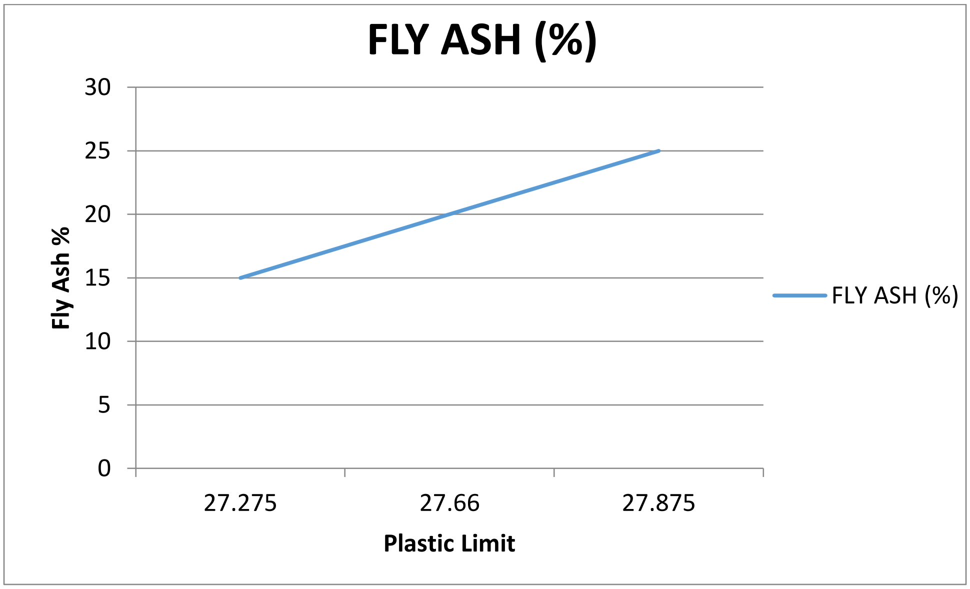
WEIGHT OF CAN (GM)	27	29
WEIGHT OF CAN +DRY SOIL(gm)	32.80	31.6
WEIGHT OF CAN +WET SOIL(gm)	41060	41.7
PLASTIC LIMIT(%)	27.76	27.76
AVERAGE	27.660	

PLASTIC LIMIT WITH 25% FLY ASH

WEIGHT OF CAN +DRY SOIL(gm)	27	29
WEIGHT OF CAN +DRY SOIL(gm)	31.01	29.956
WEIGHT OF CAN +WET SOIL(gm)	40.26	38.76
PLASTIC LIMIT(%)	27.80	27.95
AVERAGE	27.875	

FLY ASH CC PLASTIC LIMIT

FLY ASH (%)	PLASTIC LIMIT(%)
15	27.275
20	27.660
25	27.875



LIQUID LIMIT WITH 15% FLY ASH

WEIGHT OF CAN(gm)	26	27
WEIGHT OF CAN+dry soil(gm)	560	55
WEIGHT OF CAN +wet soil(gm)	65.63	64.02
No of blows	32	34
Moisture content	32.09	32.15
Liquid limit	32.91	33.17
Avg	33.04	

LIQUID LIMIT WITH 20% FLY ASH

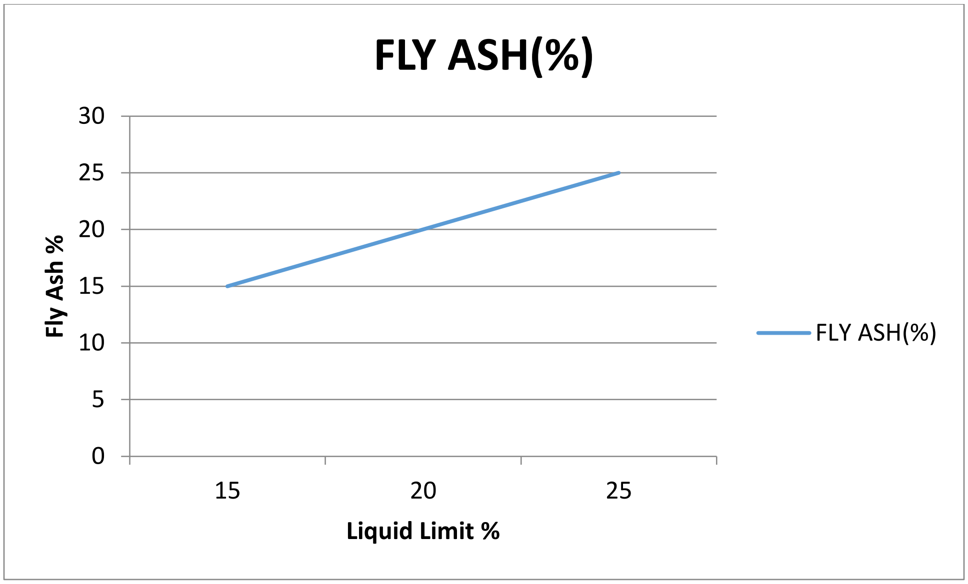
WEIGHT OF CAN(gm)	26	29
WEIGHT OF CAN+dry soil(gm)	57	61
WEIGHT OF CAN +wet soil(gm)	67.31	81.36
No of blows	35	34
Moisture content	33.26	33.38
Liquid limit	34.42	34.44
Avg	34.415	

LIQUID LIMIT WITH 25% FLY ASH

WEIGHT OF CAN(gm)	26	29
WEIGHT OF CAN+dry soil(gm)	58	67
WEIGHT OF CAN +wet soil(gm)	68 .5	75.32
No of blows	36	35
Moisture content	34.11	34.1
Liquid limit	34.80	34.78
Avg	34.79	

FLY ASH CC LIQUID LIMIT

FLY ASH(%)	LIQUID LIMIT(%)
15	33.04
20	34.415
25	34.798

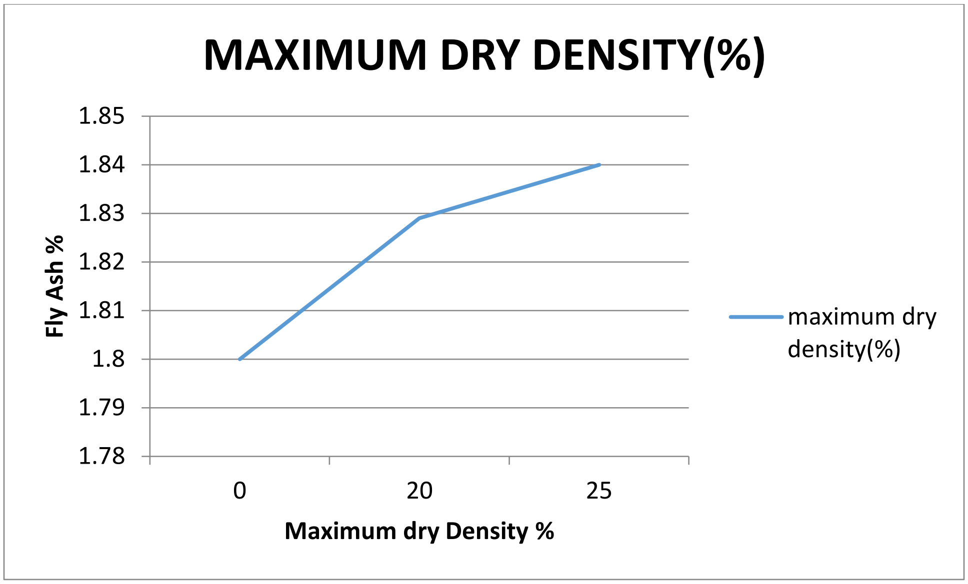


COMPACTION TEST WITH FLY ASH

FLY ASH(%)	OMC(%)	MAX. DRY DENSITY (gm/cm3)
15	15.12	1.822
20	15.1	1.829
25	14.4	1.84

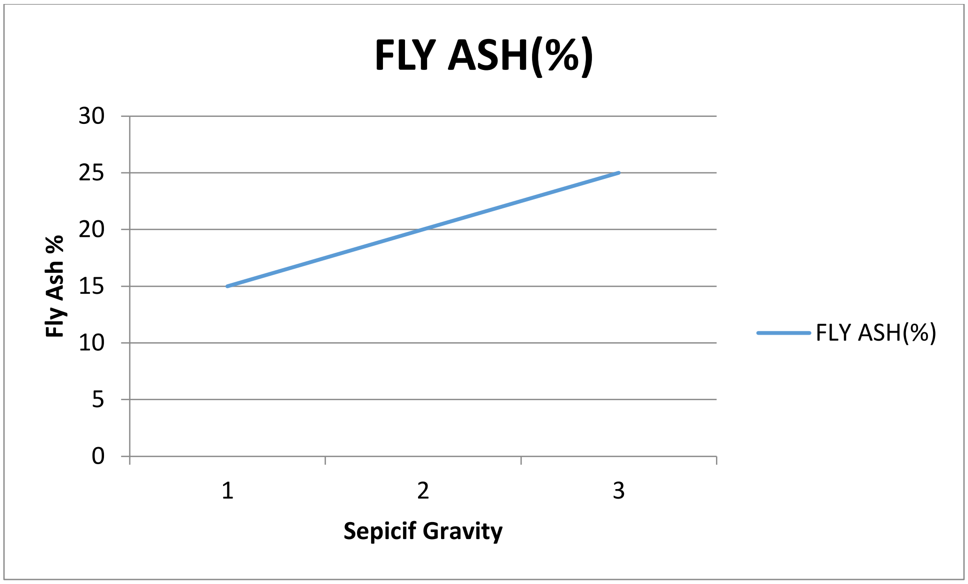
MAXIMUM DRY DENSITY

MAXIMUM DRY DENSITY(%)	PERCENTAGE FLY ASH ADDED(%)
1.8	0
1.829	20
1.84	25



SPECIFIC GRAVITY TEST WITH VARYING PERCENTAGE OF FLY ASH

FLY ASH(%)	W1(gm)	W2(gm)	W3(gm)	W4(gm)	S.G
15	479	678	1380	1269	2.16
20	480	679	1380	1270	2.2
25	482	680	1380	1265	2.2



CBR Test for normal soil

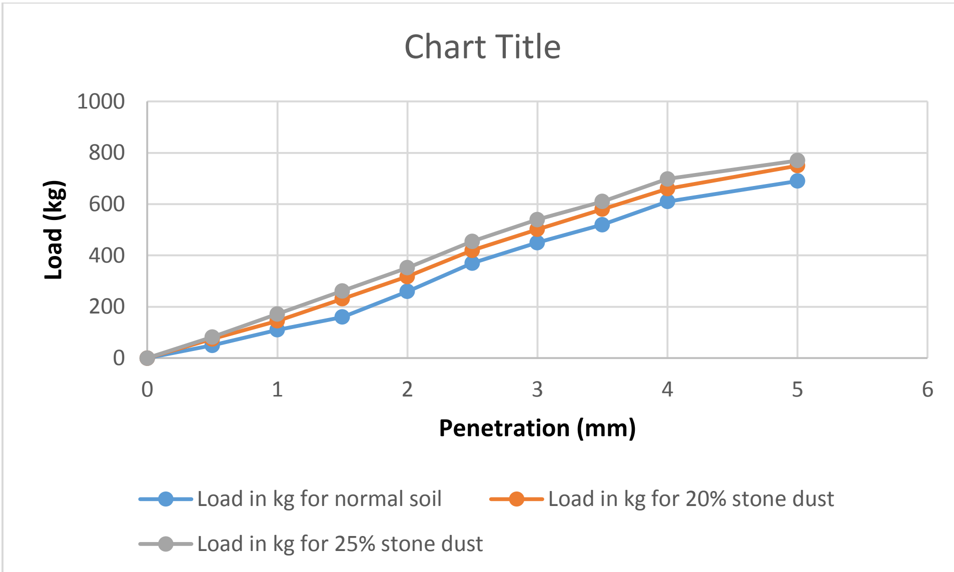
Penetration (mm)	Load in kg for normal soil	Load in kg for 20% stone dust	Load in kg for 25% stone dust
0	0	0	0
0.5	50	75	82
1	110	145	172
1.5	160	231.5	262
2	260	318	352
2.5	370	420	455
3	450	502	540
3.5	520	580	610
4	610	660	698
5	690	750	770

Value of cbr as calculated comes out to be

For normal soil 14.97

For 20% stone dust 1874

For 25% stone dust 20.66



TRIXIAL TEST ON SOIL WITH STONE DUST

TRIAXIAL TEST FOR NORMAL SOIL

AXIAL STRAIN	σ1	σ3	σ1-σ3	EXCESS PORE PRESSURE	(σ1+σ3)/2
0	0	0	0	0	0
0.5	79.22	56.33	22.89	16.34	67.775
0.9	81.91	49.16	32.75	23.57	65.535
1.7	82.81	38.61	44.2	34.06	60.71
2.6	81.56	34.33	47.23	38.34	57.945
2.9	81.64	33.37	48.27	39.3	56.67
4	81.05	31.71	49.34	40.96	56.38
5.5	79.43	29.58	49.85	43.09	54.515
6.4	79.36	29.58	49.78	43.09	54.57
7.2	78.05	28.54	49.51	44.13	53.295
8.2	75.49	26.4	49.09	46.27	50.945

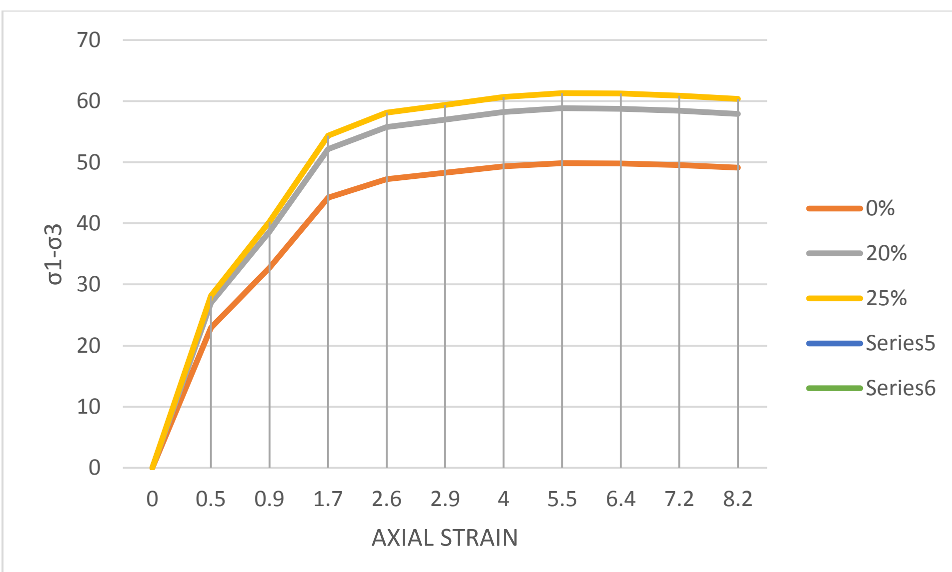
TRIAXIAL TEST FOR 20% STONE DUST

AXIAL STRAIN	σ1	σ3	20%	EXCESS PORE PRESSURE	(σ1+σ3)/2	(σ1-σ3)/2	A
0	0	0	0	0	0	0	0
0.5	93.4796	66.4694	27.01	16.34	79.9745	13.5051	0.60496
0.9	96.6538	58.0088	38.645	23.57	77.3313	19.3225	0.60991
1.7	97.7158	45.5598	52.156	34.06	71.6378	26.078	0.65304
2.6	96.2408	40.5094	55.731	38.34	68.3751	27.8657	0.68794
2.9	96.3352	39.3766	56.959	39.3	67.8559	28.4793	0.68997
4	95.639	37.4178	58.221	40.96	66.5284	29.1106	0.70352
5.5	93.7274	34.9044	58.823	43.09	64.3159	29.4115	0.73254
6.4	93.6448	34.9044	58.74	43.09	64.2746	29.3702	0.73357
7.2	92.099	33.6772	58.422	44.13	62.8881	29.2109	0.75537
8.2	89.0782	31.152	57.926	46.27	60.1151	28.9631	0.79877

TRIAXIAL TEST FOR 25% STONE DUST

AXIAL STRAIN	σ1	σ3	25%	EXCESS PORE PRESSURE	(σ1+σ3)/2	(σ1-σ3)/2	A
0	0	0	0	0	0	0	0
0.5	97.4406	69.2859	28.1547	16.34	83.36325	14.07735	0.580364912
0.9	100.7493	60.4668	40.2825	23.57	80.60805	20.14125	0.585117607
1.7	101.8563	47.4903	54.366	34.06	74.6733	27.183	0.6264945
2.6	100.3188	42.2259	58.0929	38.34	71.27235	29.04645	0.659977381
2.9	100.4172	41.0451	59.3721	39.3	70.73115	29.68605	0.661927067
4	99.6915	39.0033	60.6882	40.96	69.3474	30.3441	0.674925274
5.5	97.6989	36.3834	61.3155	43.09	67.04115	30.65775	0.702758683
6.4	97.6128	36.3834	61.2294	43.09	66.9981	30.6147	0.703746893
7.2	96.0015	35.1042	60.8973	44.13	65.55285	30.44865	0.72466267
8.2	92.8527	32.472	60.3807	46.27	62.66235	30.19035	0.766304465

CHART OF AXIAL STRAIN VS DEVIATOR STRESS



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