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The Major Project- II On Study on Modelled Granular Column of various Diameters in Soils Submitted

in Partial Fulfillment for the Award of the Degree Of MASTERS of TECHNOLOGY 10

IN CIVIL

ENGINERING With Specialization In GEOTECHNICAL ENGINEERING By Rahul Siddarth (Roll No. 2K13/GTE/13) Under The Guidance Of Prof. A. K. Sahu

Department of Civil Engineering Delhi Technological University, Delhi Department Of Civil Engineering Delhi 7

Technological University, Delhi-110042 2015 DELHI TECHNOLOGICAL UNIVERSITY, DELHI

CERTIFICATE This is to certify that the major project report entitled 4

â€œStudy on Modelled Granular Column of various Diameters in Soils.â€ is a bonafide record of work carried out by Rahul Siddarth (Roll No. 2K13/GTE/13) under my guidance and supervision, during the session 2015

in partial fulfillment of the requirement for the degree of Master of 18

Technology (Geotechnical Engineering) from Delhi Technological University, Delhi. The work embodied in this major

project has not been submitted for the award of any other degree to the 5

best of our knowledge. (Prof.

A. K. Sahu) Department of Civil Engineering Delhi Technological University, Delhi 13

Delhi-110042 2015 DELHI TECHNOLOGICAL UNIVERSITY, DELHI ACKNOWLEDGEMENT As I write this acknowledgement, I must clarify that this is not just a formal acknowledgement but also a sincere note of thanks and regard from my side. I feel a deep sense of

gratitude and affection for those who were associated with the project and without whose co-operation and guidance this project could not have been conducted properly. 5

I would like express my deep gratitude to 4

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Department of Civil Engineering, Delhi Technological University, Delhi 13

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Professor, Department of Civil Engineering, Delhi Technological University, Delhi 7

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Department of Civil Engineering Delhi Technological University, Delhi DELHI 7

TECHNOLOGICAL UNIVERSITY, DELHI DECLARATION I hereby declare that the work being presented in this Project Report entitled â€œStudy on Modelled Granular Column of various Diameters in Soilsâ€ is a bona fide record of work carried out by me as a part of major project

in partial fulfillment of the requirement for the degree of Master of Technology in Geotechnical Engineering. 10

I have not submitted the matter presented in this report for the award of any degree. (Rahul Siddarth) M. Tech (Geotechnical Engineering) Roll No. 2K13/GTE/13

Department of Civil Engineering Delhi Technological University, Delhi 24

ABSTRACT

Continuous usage with time soil has been degrading. The actions of environment and urbanisation were devastating at places where soil lost its strength and resistance to lateral deformation, so it becomes very

important to restore the properties stated above in order to carry on construction. The project presented here aims to study the variation of load carrying capacity and shear parameters of soil after introducing granular columns of varying diameter. Studies show that the granular columns derive the strength from the soil confining them. The granular columns also help in easy drainage, reduction in pore pressure. The initial portion of the study deals with theoretical analysis and detailing regarding the brief description of granular columns and studies conducted to learn the effect of granular column on soils when reinforced with them. A deep analysis was conducted on the materials adopted for the study. A series of CBR and Direct Shear Test were performed after the installation of granular columns by varying the diameter in the soil where it was found that there was improvement in load carrying capacity and

shear strength parameters of the soil. The 12

study also presents swelling behaviour of soil against time. The improvement has been presented graphically in later section of the thesis. Also it was established that granular columns can also be used to rectify the swelling behaviour of expansive soil and they are suitable means of ground improvement too. Keywords: Soil, Granular columns, Stone dust, direct shear test, BCS, CBR test CONTENTS S.NO. CHAPTER

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- Abstract v 6. - List of Figures viii-x 7. - List of Tables xi-xii 8. 1 Introduction 13-17 9. 2 Literature review 18-22 10. 3 Material used 23-45 11. 4 Experimental Programme 46-76 12. 5 Result and Discussion 77-81 13. 6 Conclusions and Future scope 82 14. - References 83-84 LIST OF SYMBOLS 1. BCS Black Cotton Soil 2. CBR California Bearing Ratio

3. MDD Maximum Dry Density 4. OMC Optimum Moisture Content 20

5. % Percentage 6. Fig Figure 7. DST Direct Shear Test 8. c Cohesion value 9. Ï• Angle of internal friction 10. SS Silty sand 11. G Specific gravity LIST OF FIGURES Fig. 1.1 Stages Occurring in the Vibro Compaction Technique 14 Fig.1.2 Stages Occurring in the Vibro CompozerTechnique 15 Fig. 1.3 Stages occurring in the Cased borehole Technique 16 Fig. 1.4 Stages occurring in the Vibro replacement Technique 16 Fig. 3.1 Sieve Analysis for sand 24 Fig. 3.2 Maximum dry density and optimum moisture content determination 25 Fig. 3.3 load Vs displacement curve for sand 26 Fig. 3.4 Shear Stress versus Normal Stress Curve for sand 27 Fig. 3.5 Sieve Analysis for stone dust. 29 Fig. 3.6 maximum dry density and optimum moisture content determination 30 Fig. 3.7 load Vs displacement for stone dust 31 Fig. 3.8 Shear Stress versus Normal Stress Curve for stone dust 31 Fig. 3.9 hydrometer analysis for BCS 33 Fig. 3.10 maximum dry density and optimum moisture content determination 34 Fig. 3.11 load Vs displacement for BCS 35 Fig. 3.12 Shear Stress versus Normal Stress Curve for BCS 36 Fig. 3.13 liquid limit determination curve 36 Fig 3.14 stress vs. strain curve 39 Fig 3.15 Graph for Sieve Analysis for ss soil 40 Fig 3.16 Graph for mdd and omc of ss soil 41 Fig. 3.17 load Vs displacement for SS soil 42 Fig. 3.18 Shear Stress versus Normal Stress Curve for SS soil 43 Fig. 3.19 liquid limit determination 43 Fig. 3.20 Stress Vs Strain curve for Ss soil 45 Fig. 4.1 CBR load Vs deformation curve for BCS 47 Fig. 4.2 CBR load Vs

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an immense progression in construction projects. But soon it was realized that properties of soil werenâ€Ÿt same everywhere but they varied from place to place, hence it was obvious of major problems coming up with time and the biggest of them was the strength characteristics offered by the soil. Other then the strength criteria one of the other problems encountered by the soil engineer is the expansive nature of soil found in major parts of the world. The central part of India is covered with such soil. These soils are highly expansive when brought in contact of water and thus offer high uplift capacity to the structures built upon it leading to destruction of integrity of the structure, hence it becomes very important to study such soils and methods to stabilize them. Reinforcing soil for modifying the bearing capacity, shear strength parameters, minimizing settlement as well as swelling and so on has been one of the important goals of studies and researches conducted on soil. These studies have proven to be very useful in every small and large construction projects related with soil where soil serves as foundation material or load bearing strata. The previous studies reveal that the granular column offers resistance to shear stress due to its orientation as well as in axial loading condition

due to the confining pressure offered by soil surrounding the column 6

when the column bulges under the vertical loading. The present study aims at studying the effect of granular columns on shear strength parameters, bearing capacity and swelling characteristics offered by the soil with and without granular columns of varying diameter. The scope of the project conducted can be applied in following fields: â€¢ Ground improvement â€¢ Analysis of shear parameters of soil â€¢ Stability analysis of a structure â€¢ Soil stabilization But before heading further it becomes very important to know about granular columns in detail. Starting with granular material, it can defined as concentration of discrete particles that are characterized with a unique property of loosing energy whenever brought in contact with each other and losses are generally to be the frictional losses. Referring to the size of the granular material used, it shouldnâ€Ÿt be very fine that it could be affected by thermal motions. However the suitable lower limit as per studies conducted was kept to be 1Î¼m. Columns composed of such material were termed as granular columns. Granular columns construction was a cost effective method developed in order to encounter soft soils. The soft soils refer to clay and fine silts which possesâ€Ÿ high water content and are subjected to low undrained strength, compression strength and poor settlement properties. In such kind of soils granular columns are driven space closely to form a grid of granular columns that imparts higher strength and stiffness to existing soil. 1.2 Methods of installation of granular columns 1.2.1 Vibro-Compaction Method The method was developed to densify the cohesion less soils. It uses a devices called as Vibrofloat that sinks into the soil strata to be reinforced under action of its own weight , pressure and jetting action of water provided through external means. After the device has reached a pre determined depth the hole is filled with granular material and the device is allowed to vibrate at a designated frequency. This is continued at different depths by pulling the device in upward direction till the borehole is completely filled with the granular material and is well compacted. For better understanding following diagram can be analyzed: Figure 1.1: Stages Occurring in the Vibro Compaction Technique (Source: www.google.com) 1.2.2 Vibro-Compozer method The method was started and gained importance in Japan and was found extremely helpful in clay deposits with high ground water table. In this method a casing is driven into the soil strata with the help of impacts of hammer at top portion. Now the casing is filled with requisite amount of granular material and casing is pulled out upto certain depth and is redriven back into soil with help of a vibrating unit fitted at top of the casing. The above steps are repeated until we have got a full compacted granular column. Figure 1.2: Stages Occurring in the Vibro CompozerTechnique (Source: www.google.com) 1.2.3 Cased borehole method The method has proven itself to be very cost effective against the vibratory compaction techniques. A casing of requisite length is driven into the soil using suitable methodology and then is filled with the required granular material. Now the casing is withdrawn granular material in borehole is compacted with a hammer of considerable weight (15 to 20 kN) falling around height of around 2 meter. The technique although is cost effective but it produces high disturbances through noises as well as it may affect the sensitive soils. 1.2.4 Vibro replacement method The method is very much similar to Vibro compaction method applicable to cohesive soft soils. The method can be marked as dry or wet which actually refers to usage of water for formation of a borehole. The Vibrofloat used here leaves the borehole of greater size than the size of float. The uncased borehole is cleaned out using a suction pump. After the borehole is cleaned it is filled with imported granular material in stages applying required degree of compaction. Figure1.3: Stages occurring in the Cased borehole Technique (Source: www.google.com) Figure 1.4: Stages occurring in the Vibro replacement Technique (Source: www.google.com) 1.3 Advantages of granular columns Following are the advantages of using granular columns: â€¢ Granular materials are cost effective and are widely available; hence construction of granular columns is quite cheap. â€¢ Construction of a granular column can be done very easily in a very short span of time. â€¢ The columns also help in preventing the erosion of nearby soil. â€¢ The columns provide a better option for reinforcement of soil that are highly susceptible to liquefaction. â€¢ The columns help to reduce pore pressure and reduce settlements. 1.4 Disadvantages of granular columns Following are the disadvantages of using granular columns: â€¢ They offer low permeability as a function to vertical drain during times of earthquake. â€¢ Whenever casing is pushed into the ground a smear zone is developed around column that reduces its lateral permeability and its effectiveness as sand drain. â€¢ They offer low stiffness and angle of internal friction compared to stone columns. 1.5 Objectives of the study The project makes an attempt to analyze the detailed study done on soils adopted for the study of the effect of reinforcement. However in particular following are the objectives of the thesis presented: â€¢ To study and obtain the geotechnical properties of various types of soils and granular materials considered for the project. â€¢ To perform the direct shear test on the soil in plain case as well as reinforced with granular columns of varying diameter. â€¢ To perform the CBR test on the soils in plain case and reinforcing it with granular columns of varying diameter. â€¢ To analyze and study the effect of swelling property of expansive soil in plain as well as reinforced state. â€¢ Presentation of above results in graphical and tabular manner for better analysis. CHAPTER 2: LITERATURE REVIEW There has been a severe shortage of land in past few years. Rise in population, development in commercial, industrial projects are few of the factors that contributed well in above. The demand of land was continuous which allowed people to use land with weaker strata section. The continuous necessitated land usage grabbed the attention of researchers and soil engineers for developing ground improvement techniques .over last few years several ground improvement methodologies have been developed , however the adopting criteria depends upon the type of soil , load , structure to be built. Although some of the common methods of ground improvements are dynamic and static compaction, explosive methods, stabilization through use of admixtures, use of fibres reinforcements, but it was for granular columns which are being used extensively for enhancement of the shear and bearing strength of soil. Granular columns was a technique of soil reinforcement that was developed to deal with the

soft cohesive soils in order to augment the bearing capacity of soil supporting structure, to lower down the settlement and to accelerate the consolidation of the surrounding saturated soil. [1] studied the 6

behavior of

ground reinforced with sand columns. The research paper presented is 12

a study and investigation after the conventional triaxial test were performed on 200 mm long and 100 mm diameters were installed with sand compacted piles to stimulate the strength behavior of ground . The soft clay with high water content offers very low bearing capacity .the clay specimens were taken for preparation of test specimens each encased with sand column. The investigation consist of 20 triaxial test which on completion revealed that after the installation of sand piles smear zone was created. The tests revealed that the stress strain behavior of clay was influenced by the presence of smear zone. The shear induced pore pressure was found to be low in soil with smear effect since the water wasnâ€Ÿt allowed to flow towards column. The columns induced also caused variation in the horizontal load bore by the soil. It was [2] whose analysis raised the question on the feasibility of usage of sand columns. The concept of testing sand columns efficiency actually came after conducting the tests on granular material used as columns. Although the granular columns provided good strength but when they were tested on clays, clays being of expansive nature exerted such pressure they started failing. Alternatively this led to low swelling behavior of soil reinforced with granular columns. Considering the crushing factor for granular columns they were replaced with granular material mixed with sand. a series of triaxial tests were conducted . The tests were conducted on two groups of soil, first was the soil specimen with isolated column and the other was with increased number of columns. The tests conducted were triaxial test; the columns were compacted to 95 percent and were cured for 21 days. After preparing the sample triaxial tests were conducted keeping confining pressure to be 25 kpa. on completion of the tests it was found that even though the global behavior of soil improved but there was increase in the stiffness of soil and also there was minimization of strain taking behavior of soil. There was modification in load settlement curve. The confining stress resulted in low mobilization of intergranular bonding strength increasing the ductility and shear strength of soil. [3] made efforts to study and analyze the secondary efforts that can be served by sand columns apart from being used as sand drains. To analyze the effect of sand columns 32 samples containing sand column were prepared and consolidated undrained shear strength tests were performed on it. After the tests were completed it was found that there was significant rise in undrained shear strength of soil with youngâ€Ÿs modulus too. Although the pore pressure decreased but drained shear parameters remained

unaffected by sand columns reinforcement except for fully penetrated columns with higher replacement 21

ratio. As per his research he discovered that sand drains not only accelerate the construction work but also helps in dissipation of excess pore pressure. Till date every study had been ignoring the effect of improving bearing capacity but the recent studies revealed the case of one dimensional loading with controlled drainage. After the tests were completed it was analyzed that there was increase in youngâ€Ÿs modulus of the soil specimen. The presence of columns reduced the generation of excess pore water pressure. There was increase in undrained shear strength however the effect of confining pressure on strength wasnâ€Ÿt revealed by him. [4] made effort to analyze the difference between

clay reinforced with sand piles and geotextile encased 11

piles . However making this effort he was able to analyze the ultimate yield strength and settlement behavior of soil in plain as well as reinforced state. Soil was reinforced with sand piles and series of loading test were performed with help of universal testing machine. After tests concluded it was founded that plain clay sample failed at a value much less than 2.5 mm and sample reinforced with sand piles failed at a higher value. The bearing capacity of clay was found to be 11.8 kPa and the corresponding settlement was found to be 2.4 mm while the bearing capacity of soil reinforced with sand piles was 38.7 kPa which was 3.3 times greater than the reinforced state. The settlement needed for development of bearing capacity is 1.58 times the unreinforced state. [5] studying the research work of [6] and [7] carried his work in determining the effect of size of granular material used as column for reinforcing the soft soil. As per their research it was found that sand at greater confining pressure offered far better

bearing capacity than the gravel material. However at lower 26

confining pressure they offered nearly same bearing capacity. The samples prepared were either hollow or augmented with granular columns of varying particle size.

Series of consolidated undrained triaxial tests were performed. 11

After the tests concluded it was found that specimen reinforced with sand showed greater undrained shear strength. There was also increase in youngâ€Ÿs modulus too with the reinforcement. [8] studied the research and analyses made by previous authors and researchers each with a single motive of studying the soil soil reinforced with sand or stone columns that modified the soil strength and settlement properties

.studies conducted by [9] revealed that rate of improvement of the soft soil reinforced with granular column depended upon the lateral support provided by the soft soil to columns, columns diameter and the degree of compaction done on the soil. He stated that when load is applied over the soil mass the load is directly transferred to the columns under the effect of which they try to bulge. Their bulge failure is prevented through the lateral support offered by surrounding soil. In this manner the

load carrying capacity of the soil is amplified upto a **14**

limiting value at which column fails in any of the failure mode. He also observed that at the site under study the settlements have reduced to half for reinforced soil as compared to that of unreinforced soil.[10] conducted test on soft kaolin clay reinforced by a granular column. Several loading tests were conducted on the soil and the loading was done in stages ensuring that pore pressure is completely dissipated. After the tests completed it was founded that the rate of settlement has increased upto 4 times however the vertical displacement was lowered by a factor by 6.[11] conducted footing load tests on clay soil reinforced with different granular material. The series of tests revealed that with increase in the density of the granular material there was increase in the bearing capacity of the soils too. [12] took fine silty soil in both unreinforced and reinforced state . A series of triaxial test was conducted in drained and undrained state. After the tests completed it was found that shear strength offered in drained case was more than the undrained case, however the strength in both the cases was greater than the plain unreinforced state. [13] carried out the consolidated undrained tests on kaolin clay samples in reinforced and unreinforced state both. The study aimed to sort out the difference between the properties of plain clay sample and clay sample augmented by sand column in plain state as well as geogrid encased case. The samples were first prepared slurry state and were allowed to consolidate under pressure and later on they were augmented by sand columns in plain and encase state. After preparation of sample

series of undrained triaxial tests were performed. **11**

The results concluded that in samples having column penetration ratio more than .6, for them there was increase in bearing capacity. For fully penetrated column there was thirty percent increase in the value of ultimate deviator stress. [14] conducted loading tests on reinforced clay in order to study the load strain behavior for given group of granular columns under pad , strip and circular footing. The clay was prepared in formed in form of slurry and was allowed to consolidate under the effect of given pressure. After the soil was consolidated it was augmented with sand columns. The whole reinforced soil was fixed to plate. The length to diameter ratio was kept as 6 and area replacement ratio was kept as 24 percent. The samples were subjected to strain controlled loading tests. After the completion of tests it was found that for l/d ratio upto 6 the load carrying capacity had increased by 130 percent and for value of l/d beyond 6 there was an additional 5 percent increase. The tests also concluded that shorter columns reinforced soil had 4 times greater stiffness value while the longer columns reinforced soil had 5.7 times greater stiffness value compared to plain soil case. Hence the author concluded that longer column can be used to control the settlement problems.[15] took clay samples for triaxial tests that were augmented with crushed basalt aggregate. After the completion of test he was able to conclude that most suitable l/d ratio lies between 6 and 10. [16] conducted triaxial compression tests on clay sample reinforced with granular columns and concluded that the strength criteria was also depending on the drainage conditions. However he also concluded that keeping the drainage factor aside there was increase in the bearing strength of soil when it was reinforced. Also to mention there was increase in angle of internal friction after the granular columns were installed in the plain soil. [17] performed consolidated undrained triaxial tests on clay samples reinforced with quartz sand. After completion of tests he concluded

that there was increase in undrained shear strength of soil. The 2

pore pressure had dropped down while there was rise in the stiffness of the soil. [18] used large triaxial chambers for testing the soil . Preconsolidated samples were obtained from site and were trimmed to required criteria. The samples now were reinforced with granular columns and tests were performed and reported about the improved settlement behavior and as per best suited l/d ratio lies between 8 and 10. [19] performed similar test on clay augmented with granular columns. He concluded that there was fall in the compressibility value of soil after the installation of the granular columns. [20] conducted tests on the clay reinforced with simple sand material .the load was applied through a small footing resting over the sand column , loading was done small increments with sufficient allowance of consolidation. The test gave stress strain curve in concave upward direction. The test also revealed the increase stress taking behavior of soil and the axial strain was reduced upto 25 percent correspondingly. The soil that fails in required strength for a supporting a given project or a soil that has lost its strength due to environmental changes generally attracts the attention for soil improvement techniques. General practices adopted for improving the properties of an adopted for improving the properties of an inferior soil is to mix it with superior soil this modifies the properties of inferior soil up to the requirements. One such property is CBR value. CBR value is one such property that is frequently checked during initial as well as in construction stages of transportation projects.CBR value is measure of soil strength and its relation to soil parameters like gradation and mineral type. In these types of projects large number of samples have to be tested which is always laborious and time consuming task. Hence to ease out this issue researcher developed empirical formulaeâ€Ÿs that helped to find CBR value by relating it to index properties of soil. [21], Black (1962) made efforts to develop a chart for estimating CBR value of cohesive soil from plasticity and liquidity index. The other researchers who worked in similar field of development were Johnson and Bhatia (1969), Agarwal and Ghanekar (1970), Ojha and Nath (2005), Purkar and Naik (2007) and Patel and Desai. The national cooperative highway research programme (2001) of United States of America developed relationship between soil index properties and CBR value of soil. CHAPTER 3: MATERIALS USED 3.1 General The materials used for the study are the various types of soils such as silty sand, black cotton soil, sand, and stone dust. The geotechnical properties of various materials are obtained as per various parts of code IS-2720. 3.2 Geotech properties of Sand 3.2.1 Geological history of sand Sand is naturally occurring granular material comprising of finely divided rock and mineral particles. Considering the size factor it is finer than gravel and coarser than the silt. Sand varies from place to place depending upon local mineral and rock availability in the area. Soils containing sand are well suited for agriculture purposes. Sand is widely used in construction projects and soil stabilization as they possesâ€Ÿ good drainage properties. 3.2.2 2 Location of procurement of sand Geotechnical lab, Delhi Technological University, Bawana, Delhi, 28.7499â °N, 77.1170â °E 3.2.3 3 Specific gravity Table 3.1.: Results of specific gravity of sand S. No Sample 1 Sample 2 Sample 3 Mass of empty pycnometer (gm) 697.50 698.13 699.10 Mass of empty pycnometer and soil sample (gm) 896.40 897.10 901.31 Mass of empty pycnometer, water and soil sample (gm) 1672.10 1677.72 1684.12 Mass of empty pycnometer and water (gm) 1551.32 1552.11 1556.36 G 2.546 2.718 2.715 The Specific gravity â€žGâ€Ÿ for adopted sand is 2.659. 3.2.4 4 Sieve analysis Table 3.2.: Results of sieve analysis for sand Sieve size (mm) Mass retained (gm) % mass retained Cumulative % retained % finer 4.75 31.46 3.14 3.14 96.85 2.36 9.80 0.98 4.12 95.87 1.18 9.40 0.94 5.06 94.93 0.600 13.80 1.38 6.44

93.53 0.300 928.40 62.84 68.30 31.70 0.150 26.48 26.48 95.76 4.23 0.075 3.05 3.05 98.82 1.18 Pan 2.11 0.21 99.03 0.97 Figure 3.1: Sieve Analysis for sand From the semi-log graph plotted between percentage finer and sieve size (on log scale), the value of D10=.18,D30= .29, D60= 0.41; CU, the coefficient of uniformity and CC, the coefficient of curvature are given as : Cu= D60/D10 = 0.41/0.18 = 2.27 Cc= (D30)2/ (D60 x D10) = (0.29)2/ (0.41 x 0.18) = 1.13 The result shows that the sand is poorly graded, (SP). 3.2.5

Maximum dry density and optimum moisture content TABLE 3. 2

3.: Results for

maximum dry density and optimum moisture content Water content (w) (%) (1) 9

Mass of mould + soil (2) (gm) Mass of mould + soil ((2)-4282) (3) (gm) Density of soil ((3)/1000) (4) (gm/cc) Dry density (4/(1+wa)) (5) (gm/cc) Dry unit weight (5)*9.81 (6) (kN/m3) 0 6046 1786 1.786 1.786 17.52 3 6048 1788 1.788 1.750 17.16 6 6125 1865 1.865 1.791 17.55 9 6230 1970 1.970 1.857 18.22 12 6325 2065 2.065 1.907 18.71 15 6210 1950 1.950 1.735 17.02 Figure 3.2: Variation of dry density with water content The curve gives the maximum dry unit weight equal to 18.83KN

/m3 at an optimum moisture content of 1

11.56%. 3.2.6 6

Direct shear test The direct shear test was conducted 1

on the soil sample to obtain the

friction angle of the soil (É¸) and the cohesion (c) using the 3

direct shear apparatus with mould of size 60mm x 60mm. The test was conducted for three different normal loadings of 50kN/m2, 100kN/m2 and 150kN/m2. The horizontal displacement corresponding to different shear force is noted and curves are plotted for different normal loadings. The shear stress corresponding to normal load is obtained by dividing the maximum shear force with the initial area of the mould i.e. Shear stress (kN/m2) = (Shear Force x 1000)/3600 The shear force versus horizontal displacement curves are obtained below for different normal loadings are shown below: Figure 3.3: load Vs displacement curve for sand TABLE 3.4.: Results of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 27.42 100 59.45 150 84.45 Figure 3.4: Shear Stress versus Normal Stress Curve for sand The value of apparent

cohesion, c and internal frictionÉ¸ are obtained from 8

the plot of maximum shear stress corresponding to respective normal loadings, where c is the Intercept and

É¸ is the slope of the failure line. Thus c =0.076 KN/m2 and É¸= tan- 1(0.570) =29.6o. 3.2. Geotech properties of Stone Dust 3.2.1 Geological history of stone dust Stone dust is a multipurpose material obtained during the crushing and cutting action of stone and rock during sculpting or construction work. In earlier times it was discarded considering it to be useless but soon it became of great importance when given a chance to serve as a reinforcing material after the research studies were carried on by compacting a stone dust layer as a passageway which offered a great resistance to deformation. 3.2.2 Location of procurement of stone dust Jhansi, Uttar Pradesh, 25.4486Â° N, 78.5696Â° E,284001 3.2.3 Moisture content Weight of empty pan = 6.72 gm Weight of pan + weight of soil =14.12 gm Weight of pan + dried sample = 13.2 gm The moisture content of the adopted sample is 14.20 percent. 3.2.4 Specific gravity Table 3.5.: Results of specific gravity S. No Sample 1 Sample 2 Sample 3 Mass of empty pycnometer (gm) 696.28 696.28 696.28 Mass of empty pycnometer and soil sample (gm) 896.24 946.32 996.28 Mass of empty pycnometer, water and soil sample (gm) 1688.66 1720.22 1752.80 Mass of empty pycnometer and water (gm) 1561.12 1560.20 1561.00 G 2.76 2.78 2.77 The Specific gravity â€žGâ€Ÿ for adopted stone dust is 2.77 3.2.5 5 Sieve analysis Table 3.6.: sieve analysis for stone dust Sieve size (mm) Mass retained (gm) % mass retained Cumulative % retained % finer 4.75 8.42 0.84 0.84 99.16 2.36 186.30 18.63 19.47 80.53 1.18 227.10 22.71 42.18 57.82 0.600 182.30 18.23 60.41 39.59 0.300 169.60 16.96 77.37 22.63 0.150 116.20 11.62 88.99 11.01 0.075 63.50 6.35 95.34 4.66 Pan 44.90 4.49 99.83 0.17 Figure 3.5: Graph for Sieve Analysis for stone dust From the semi-log graph plotted between percentage finer and sieve size (on log scale), the value of D10=.15, D30=0.45, D60 = 1.1; CU, the coefficient of uniformity and CC, the coefficient of curvature are given as : Cu= D60/D10 = 1.10/0.15 = 7.33 Cc= (D30)2/ (D60 x D10) = (0.45)2/ (0.15 x1.1) = 1.22 The result shows that the stone dust is well graded. 3.2.6

Maximum dry density and optimum moisture content TABLE 3. 2

7.:

Maximum dry density and optimum moisture content Water content (w) (%) (1) 9

Mass of mould + soil (2) (gm) Mass of mould + soil ((2)-4282) (3) (gm) Density of soil ((3)/1000) (4) (gm/cc) Dry density (4/(1+wa)) (5) (gm/cc) Dry unit weight (5)*9.81 (6) (kN/m3) 0 6052 1770 1.770 1.770 17.360 3 6060 1778 1.778 1.726 16.932 6 6092 1810 1.810 1.737 17.039 9 6240 1958 1.958 1.870 18.340 12 6310 2020 2.020 1.925 18.880 15 6240 1988 1.988 1.882 18.462 Figure 3.6: variarion of dry density with moisture content The curve gives the maximum dry unit weight equal to 18.88 KN

/m3 at an optimum moisture content of 1

12.22%. 3.2.7

Direct shear test The direct shear test was conducted 1

on the soil sample to obtain the

friction angle of the soil (É¸) and the cohesion (c) using the 3

direct shear apparatus with mould of size 60mm x 60mm. The test was conducted for three different normal loadings of 50kN/m2, 100kN/m2 and 150kN/m2. The horizontal displacement corresponding to different shear force is noted and curves are plotted for different normal loadings. The shear stress corresponding to normal load is obtained by dividing the maximum shear force with the initial area of the mould i.e. Shear stress (kN/m2) = (Shear Force x 1000)/3600 The shear force versus horizontal displacement curves are obtained below for different normal loadings are shown below: Figure 3.7: load Vs displacement for stone dust TABLE 3.8.: Result of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 49.21 100 96.60 150 146.66 Figure 3.8: Shear Stress versus Normal Stress Curve The value of apparent

cohesion, c and internal frictionÉ¸ are obtained from 8

the plot of maximum shear stress corresponding to respective normal loadings, where c is the Intercept and É¸ is the slope of the failure line. Thus c=0.04 KN/m2 and É¸= tan-1(0.974) =77.2o. 3.3 Geotech properties of Black Cotton soil 3.3.1 General history of the soil In the field of geotechnical engineering, one of the major problems encountered is expansive soil and India suffers from it too. These soils on addition of water expand highly due to presence of clay minerals and finer particles, also when the soil is dried it shrinks leaving deep cracks. The central portion of India is covered with expansive soil where the rainfall is medium and underlying strata is basalt rock. The soil is having

iron rich granular structure makes it resistant to wind and water erosion, 17

the soil is best suited for irrigation because it has high water retaining capacity and rich humus content. Expansive soil in India is also called as black cotton soil, since cotton is highly grown in this area. The soil particle share a highly cohesive bond due to which during rainy period, the soils are highly sticky and difficult to transverse. Last but not least for such soils it is beneficiary to have construction to be done during summer and all of it should be brought to end before the rainy season. 3.3.2 Location of procurement of soil Jhansi, Uttar Pradesh, 25.4486Â° N, 78.5696Â° E,284001 3.3.3 Specific gravity Table 3.9.: Results of specific gravity S. No Sample 1 Sample 2 Sample 3 Mass of empty pycnometer (gm) 696.28 696.28 696.28 Mass of empty pycnometer and soil sample (gm) 896.29 947.31 995.98 Mass of empty pycnometer, water and soil sample (gm) 1684.22 1719.94 1752.21 Mass of empty pycnometer and water (gm) 1557.47 1561.20 1562.12 G 2.73 2.72 2.73 The Specific gravity â€žGâ€Ÿ for given BCS is 2.728 3.3.4 Sieve analysis When the soil was first sieved it was found that it contained good amount of finer particles. Hence wet sieve analysis was carried out. After the wet sieve analysis it was found that almost 92 percent particles were finer than 75 micron hence hydrometer analysis was conducted. Table 3.10.: hydrometer analysis Particle size (mm) Percentage finer (%) 0.044 81.690 0.0314 78.451 0.0228 75.244 0.0163 62.890 0.0121 56.320 0.0092 48.090 0.0067 44.180 0.0046 39.330 0.0032 33.450 0.0025 27.260 0.0014 22.610 0 0 . Figure 3.9: Hydrometer analysis for BCS As per the plot plotted above percentage of particle finer than or equal to 2 Î¼ is 27.30 percent. 3.3.5

Maximum dry density and optimum moisture content TABLE 3. 2

11.:

Maximum dry density and optimum moisture content Water content (w) (%) (1) 9

Mass of mould + soil (2) (gm) Mass of mould + soil ((2)-4282) (3) (gm) Density of soil ((3)/1000) (4) (gm/cc) Dry density (4/(1+wa)) (5) (gm/cc) Dry unit weight (5)*9.81 (6) (kN/m3) 6 5700 1480 1.480 1.427 13.99 12 5940 1720 1.720 1.658 16.26 18 6100 1880 1.880 1.812 17.77 24 6120 1900 1.900 1.697 16.64 30 5720 1500 1.500 1.122 11.006 Figure 310: Variation of dry density with water content The curve gives the

maximum dry unit weight equal to 17. 756 KN/m3 at an optimum moisture content of 19

21.23 %. 3.3.6 rect

shear test The direct shear test was conducted 1

on the soil sample to obtain the

friction angle of the soil (É¸) and the cohesion (c) using the 3

direct shear apparatus with mould of size 60mm x 60mm. The test was conducted for three different normal loadings of 50kN/m2, 100kN/m2 and 150kN/m2. The horizontal displacement corresponding to different shear force is noted and curves are plotted for different normal loadings. The shear stress corresponding to normal load is obtained by dividing the maximum shear force with the initial area of the mould i.e. Shear stress (kN/m2) = (Shear Force x 1000)/3600 The shear force versus horizontal displacement curves are obtained below for different normal loadings are shown below: Figure 3.11: load Vs displacement for BCS TABLE 3.12.: Result of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 59.72 100 61.38 150 65.27 Figure 3.12: Shear Stress versus Normal Stress Curve The value of apparent

cohesion, c and internal frictionÉ¸ are obtained from 8

the plot of maximum shear stress corresponding to respective normal loadings, where c is the Intercept and É¸ is the slope of the failure line. Thus c = 56.57 KN/m2 and É¸= tan- 1(0.055) =3.148 o. 3.3.7 Liquid limit TABLE 3.13.: Liquid limit determination No. of blows Water content 23 61.0 26 52.6 34 49.2 Figure 3.13: liquid limit determination curve The Liquid limit for given BCS is 56.60 %. 3.3.8 Plastic limit Weight of empty pan = 9.60 gm Weight of pan + weight of soil =15.83 gm Weight of pan + dried sample = 14.57 gm The plastic limit of the adopted sample is 25.32 percent. 3.3.9 Plasticity index Plasticity index = Liquid limit â€“ Plastic limit = 56.60-25.32 = 31.28 % 3.3.10 Activity Activity = plasticity index/ % by weight finer than 2Î¼ = 31.28/27.3 = 1.145 The soil is normally active contains illite in moderate amount. When activity increases, Montomorillionite mineral increases too. 3.3.11 Relative Consistency Relative consistency = (liquid limit â€“ natural water content)/ plasticity index = (56.60-9.85)/31.28 = 1.49 Since relative consistency is greater than soil is in semi solid state. 15

3.3.12 Liquidity Index Liquidity index =

(natural water content â€“ plastic limit)/plasticity index 15

= (9.85-25.32)/31.28 = -0.494 Negative value indicates that the soil is in hard state. 3.3.13 Flow Index Flow index = (w1-w2)/log (N2/N1) = (52.60-49.20)/log (34/26) = 29.18 3.3.14 ess Index Toughness index = plasticity index/flow index = 31.28/29.18 = 1.071 3.3.15 UCS Length of sample = 7.6 cm Diameter of sample = 3.8 cm Area of sample = 11.341 cm2 TABLE 3.14.: UCS value determination âˆ†L, mm Dial gauge reading Load, P, kg Æ = âˆ†L/L A = A0/1-Æ Ïƒ = P/A (kg/sq cm) Ïƒ = P/A (kN/sq m) 0 0 0 0 0 0 0 0.5 0.1 0.329256 0.006579 11.41626 0.028841 2.828318 1 0.2 0.658512 0.013158 11.49236 0.0573 5.619175 1.5 0.3 0.987768 0.019737 11.56949 0.085377 8.372571 2 0.6 1.975535 0.026316 11.64767 0.169608 16.63276 2.5 0.9 2.963303 0.032895 11.7269 0.252693 24.78056 3 1.3 4.280326 0.039474 11.80722 0.362518 35.55065 3.5 1.7 5.59735 0.046053 11.88865 0.470814 46.17089 4 2.2 7.243629 0.052632 11.97121 0.605087 59.33849 4.5 2.5 8.231397 0.059211 12.05493 0.682824 66.96184 5 2.6 8.560652 0.065789 12.13982 0.705171 69.15332 5.5 2.9 9.54842 0.072368 12.22592 0.780998 76.58936 6 3.1 10.20693 0.078947 12.31325 0.828939 81.29073 6.5 3.5 11.52396 0.085526 12.40183 0.929214 91.12429 7 3.9 12.84098 0.092105 12.4917 1.027961 100.808 7.5 4.3 14.158 0.098684 12.58288 1.12518 110.3419 8 4.3 14.158 0.105263 12.6754 1.116967 109.5365 8.5 4.3 14.158 0.111842 12.76929 1.108754 108.731 Figure 3.14: stress vs. strain curve The UCS value for BCS soil is 110.3496 kN/m2. 3.4.1 SILTY SANDY SOIL 3.4.2 General data about soil Ss soil is classified as composition of silty and sand type of soil. They are also termed as cohesive frictional type of soil. Such kinds of soils are best suited for construction work considering the research work conducted earlier, however it will be interesting to learn the effect of granular columns reinforcement on them. 3.4.3 Location of procurement of soil Geotechnical lab, Delhi Technological University, Bawana, Delhi, 28.7499â °N, 77.1170â °E 3.4.4 Specific gravity Table 3.15.: Result of specific gravity S. No Sample 1 Sample 2 Sample 3 Mass of empty pycnometer (gm) 696.50 697.30 698.13 Mass of empty pycnometer and soil sample (gm) 894.41 896.70 898.30 Mass of empty pycnometer, water and soil sample (gm) 1671.90 1670.71 1672.61 Mass of empty pycnometer and water (gm) 1563.22 1564.80 1565.45 G 2.217 2.213 2.152 The Specific gravity â€žGâ€Ÿ for given SS soil is 2.164 3.4.5 Sieve Analysis Table 3.16.: Results of sieve analysis of SS soil Sieve size (mm) Mass retained (gm) % mass retained Cumulative % retained % finer 4.75 32.48 3.24 3.24 96.76 2.36 8.28 0.82 4.07 95.93 1.18 8.37 0.83 4.91 95.09 0.6 13.30 1.33 6.24 93.46 0.300 543.01 54.30 60.54 39.46 0.150 208.81 20.88 81.42 18.58 0.075 11.92 1.19 82.62 17.38 pan 169.80 16.98 99.60 0.40 Figure 3.15: Sieve Analysis for SS soil From the semi-log graph plotted between percentage finer and sieve size (on log scale), the value of D10=.016, D30=0.25, D60 = 0.4; CU, the coefficient of uniformity and CC, the coefficient of curvature are given as : Cu= D60/D10 = 0.40/0.016= 25 Cc= (D30)2/ (D60 x D10) = (0.25)2/ (0.016 x 0.40) = 9.76 The soil contains fines greater than 15 percent, itâ€Ÿs well graded silty sand. 3.4.6

Maximum dry density and optimum moisture content Table 3. 2

17.: Results of actual water content determination Water content added (%) Mass of empty can(M1) Mass of can+wet soil(M2) Mass of can+dry soil(M3) Actual water content (w) (%) 0 4.74 8.20 8.05 4.53 4 5.53 13.06 12.60 6.51 8 13.65 27.22 25.95 10.32 12 14.76 27.48 25.86 14.50 16 8.77 48.32 42.87 15.98 Table 3.18.: Calculation of maximum

dry density and omc Water content (w) (%) (1) Mass of mould + soil 28

(2) (gm) Mass of mould + soil ((2)-4282) (3) (gm) Density of soil ((3)/1000) (4) (gm/cc) Dry density (4/(1+wa)) (5) (gm/cc) Dry unit weight (5)*9.81 (6) (kN/m3) 4.53 12300 6000 1.940 1.856 18.207 6.51 12990 6590 2.130 1.999 19.610 10.32 13426 6998 2.262 2.054 20.150 14.50 13095 6685 2.161 1.887 18.051 15.98 12903 6500 2.101 1.811 17.766 Figure 3.16: Variation of dry density with water content The curve gives the maximum dry unit weight equal to 20.15KN

/m3 at an optimum moisture content of 1

9.67%. 3.4.7

Direct shear test The direct shear test was conducted 1

on the soil sample to obtain the

friction angle of the soil (É¸) and the cohesion (c) using the 3

direct shear apparatus with mould of size 60mm x 60mm. The test was conducted for three different normal loadings of 50kN/m2, 100kN/m2 and 150kN/m2. The horizontal displacement corresponding to different shear force is noted and curves are plotted for different normal loadings. The shear stress corresponding to normal load is obtained by dividing the maximum shear force with the initial area of the mould i.e. The shear force versus horizontal displacement curves are obtained below for different normal loadings are shown below: Shear stress (kN/m2) = (Shear Force *1000)/3600 Figure 3.17: Load Vs Displacement for SS soil TABLE 3.19.: Result of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 36.11 100 64.72 150 92.50 Figure 3.18: Shear Stress versus Normal Stress Curve The value of apparent

cohesion, c and angle of internal frictionÉ¸ are obtained from 8

the plot of maximum shear stress corresponding to respective normal loadings, where c is the intercept and É¸ is the slope of the failure line. Thus, c=8.05 KN/m2 and É¸= tan-1(0.53) =29.4o 3.4.8 Liquid Limit TABLE 3.20.: Liquid limit determination No. of blows Water content 13 61.26 18 41.35 23 28.30 31 17.1 Figure 3.19: Liquid limit determination The liquid limit for given soil sample is 24.20 percent. 3.4.9 Plastic limit Weight of empty pan = 10.23 gm Weight of pan + weight of soil =23.83 gm Weight of pan + dried sample = 21.44 gm The plastic limit of the adopted sample is 21.32 percent 3.4.10 UCS Length of sample = 7.6 cm Diameter of sample = 3.8 cm Area of sample = 11.341 cm2 TABLE 3.21.: UCS value determination âˆ†L, mm Dial gauge reading Load, P, kg Æ = â^†L/L A = A0/1-Æ Ïƒ = P/A (kg/sq cm) Ïƒ = P/A (kN/sq m) 0 0 0 0 0 0 0 0.5 0.2 0.658512 0.006579 11.41626 0.057682 5.656636 1 0.28 0.921916 0.013158 11.49236 0.08022 7.866845 1.5 0.34 1.11947 0.019737 11.56949 0.09676 9.488914 2 0.37 1.218247 0.026316 11.64767 0.104591 10.25687 2.5 0.41 1.349949 0.032895 11.7269 0.115116 11.28892 3 0.47 1.547503 0.039474 11.80722 0.131064 12.85293 3.5 0.5 1.646279 0.046053 11.88865 0.138475 13.57967 4 0.54 1.777982 0.052632 11.97121 0.148521 14.5649 4.5 0.58 1.909684 0.059211 12.05493 0.158415 15.53515 5 0.63 2.074312 0.065789 12.13982 0.170868 16.75638 5.5 0.68 2.23894 0.072368 12.22592 0.183131 17.95888 6 0.72 2.370642 0.078947 12.31325 0.192528 18.88043 6.5 0.76 2.502345 0.085526 12.40183 0.201772 19.78699 7 0.8 2.634047 0.092105 12.4917 0.210864 20.67856 7.5 0.8 2.634047 0.098684 12.58288 0.209336 20.52872 8 0.75 2.469419 0.105263 12.6754 0.19482 19.1052 8.5 0.7 2.304791 0.111842 12.76929 0.180495 17.7004 9 0 0.118421 0 0 0 Figure 3.20: Stress Vs Strain curve The UCS value for SS soil is 20.6786 kN/m2 . CHAPTER 4: EXPERIMENTAL INVESTIGATION 4.1 Introduction This portion of thesis covers the study and experiment conducted to observe the effect of granular columns of varying diameter on engineering properties of soil taken for consideration. The motive of the project was to analyze the variation in CBR value, cohesion value, angle of internal friction and swelling behaviour of soil after the installation of granular columns. As mentioned earlier soils considered are BCS and SS soil while the material for column was kept as stone dust and sand. The tests conducted were: â€¢

California bearing ratioâ€¢ Direct shear test California Bearing Ratio Test â€¢ CBR 27

test are performed according to IS2720 PART 16. â€¢ According to IRC 37: 2012, the CBR results depend on a various factor and wide variation in values can be expected. â€¢ In current investigation the improvement in the given soil has been achieved through installation of granular columns of varying diameter. â€¢ The soil was compacted to mdd at omc. â€¢ All the CBR tests were performed in soaked condition. Direct Shear Test â€¢ Test performed as per IS 2720-part 13-1972 â€¢ The soil was compacted to mdd at omc â€¢ The tests were performed with a single objective to see the variation in cohesion and

angle of internal friction in the soil after the installation of columns. The 25

experimental investigation shall be conducted in following stages: â€¢ Study the â€žVariation in CBR value for plain Black Cotton Soil and Black Cotton Soil installed with granular columnâ€Ÿ. â€¢ Study the â€žVariation in CBR value for plain Silty sand soil and Silty sand soil installed with granular columnâ€Ÿ â€¢ Study the â€žVariation in c-Ï• value for plain Black Cotton Soil and Black Cotton Soil installed with granular column using DSTâ€Ÿ. â€¢ Study the â€žVariation in c-Ï• value for plain Silty sand soil and Silty sand soil installed with granular column using DSTâ€Ÿ. â€¢ Study the â€žVariation in the swelling behavior of Black Cotton Soil in plain as well as reinforced stateâ€Ÿ. 4.1.1 .0 California Bearing Ratio Test 4.1.2 Plain BCS TABLE 4.1.: CBR value determination for BCS Penetration Sample 1(kg) Sample 2(kg) Sample 3(kg) 0.0 0 0 0 0.5 3.4 3.2 3.3 1.0 6.4 6.2 6.3 1.5 11.2 11.3 11.5 2.0 18.5 19.6 20.4 2.5 30.2 28.2 29.20 3.0 30.4 30.6 30.8 3.5 31.2 31.1 30.9 4.0 32.6 32.8 33.1 4.5 33.8 34.2 34.2 5.0 36.2 36.4 36.9 5.5 36.8 37.2 37.1 6.0 37.3 37.6 37.8 Figure 4.1: Load Vs Deformation curve The CBR value for BCS is 2.13 4.1.3 BCS + 10 mm stone dust column TABLE 4.2. : CBR value determination for BCS+10 mm stone dust column Penetration Sample 1(kg) Sample 2(kg) Sample 3(kg) 0 0 0 0 0.5 5.9 6.1 5.8 1 9.16 16.4 11.6 1.5 16.6 22.4 18.2 2 23.2 26.2 23.4 3 41.2 40.4 42.1 3.5 43.2 44.3 42.1 4 49.6 49.9 48.6 4.5 52.6 53.1 56.1 5 54.6 54.7 57.2 5.5 56.2 55.2

58.9 6 56.4 56.2 59.1 Figure 4.2: Load Vs Deformation curve The CBR value for given is 2.746 4.1.4 BCS + 15 mm stone dust column TABLE 4.3. : CBR value determination for BCS + 15 mm stone dust column Penetration Sample 1(kg) Sample2(kg) Sample 3(kg) 0 0 0 0 0.5 6.8 5.4 6.4 1 12.2 9.8 14.2 1.5 20.6 22.3 26.3 2 28.4 32.6 39.9 2.5 43.2 44.6 45.2 3 44.6 46.1 47.2 3.5 46.4 47.8 49.6 4 51.8 55.3 57.3 4.5 53.7 57.1 58.1 5 58.6 58.2 59.1 5.5 59.4 60.1 60.1 6 60.2 62.1 60.5 Figure 4.3: Load Vs Deformation curve The CBR value for given sample is 3.235 4.1.5 BCS + 20 mm stone dust column TABLE 4.4. : CBR value determination for BCS + 20 mm stone dust column Penetration Sample 1(kg) Sample 2(kg) Sample 3(kg) 0 0 0 0 0.5 7.4 9.2 8.6 1 13.1 14.6 16.1 1.5 24.6 26.6 25.1 2 32.6 30.2 39.6 2.5 47.4 48.2 49.1 3 51.8 50.1 52.3 3.5 52.6 55.2 54.2 4 56.4 58.2 55.1 5 5.5 63.4 62.5 62.1 6 63.6 62.9 63.2 4.1.6 BCS + 10 mm sand column TABLE 4.5. : CBR value determination for BCS + 10 mm sand column Penetration Sample 1(kg) Sample 2(kg) Sample 3(kg) 0 0 0 0 0.5 4.8 6.6 5.4 1 10.6 12.2 11.9 1.5 16.8 17.1 15.6 2 22.6 26.6 24.1 2.5 34.8 35.6 35.2 4 47.8 46.2 48.1 4.5 51.6 54.6 55.2 5 53.8 55.1 58.2 5.5 56.1 58.2 58.6 6 56.3 58.8 61.2 4.1.7 BCS + 15 mm sand column TABLE 4.6. : CBR value determination for BCS + 15 mm sand column Penetration Sample 1(kg) Sample 2(kg) Sample 3(kg) 0 0 0 0 0.5 7.6 9.4 8.2 1 14.2 19.5 21.1 2 2.5 38.7 39.9 41.4 3 47.2 44.4 43.3 3.5 51.6 49.3 48.2 4 53.6 56.1 57.8 4.5 57.8 59.2 58.8 5 61.4 61.2 60.2 5.5 62.6 63.3 63.7 6 62.8 63.6 64.1 Figure 4.6: Load Vs Deformation curve The CBR value for given sample is 2.919. 4.1.8 BCS + 20 mm sand column TABLE 4.7. : CBR value determination for BCS + 20 mm sand column Penetration Sample 1(kg) Sample 2(kg) Sample 3(kg) 1 1.5 26.8 24.2 25.6 2 34.2 33.2 34.2 2.5 41.2 42.1 42.6 3 48.2 50.3 47.6 3.5 51.8 52.4 55.2 4 54.2 56.4 57.8 4.5 58.6 58.2 59.7 5 61.6 60.3 59.9 5.5 62.8 62.3 60.1 6 63.6 63.8 62.2 Figure 4.7: Load Vs Deformation curve The CBR value for given sample is 3.063. 4.1.9 SS Soil TABLE 4.8. : CBR value determination for SS soil Penetration Sample 1(kg) Sample 2(kg) Sample 3(kg) 0 0 0 0 0.5 19.6 20.1 21.1 1 27.5 24.4 27.4 1.5 29.6 30.8 32.1 2 42.4 46.6 47.1 2.5 47.1 49.6 48.7 3 52.3 57.8 56.4 3.5 54.3 58.4 57.6 4 59.6 60.2 59.8 4.5 61.1 63.1 61.2 6 70.6 71.2 70.4 Figure 4.8: Load Vs Deformation curve The CBR value for given sample is 3.53 4.1.10 SS Soil + 10 mm sand column TABLE 4.9. : CBR value determination for SS soil +10 mm sand column Penetration Sample 1(kg) Sample 2(kg) Sample 3(kg) 0 0 0 0 0.5 20.4 22.3 21.8 1 26.3 27.9 28.4 1.5 30.3 31.2 36.5 2 37.8 46.6 41.2 2.5 49.6 52.3 50.3 4 59.6 61.6 62.1 4.5 62.3 64.8 63.7 5 66.8 66 67.4 5.5 67.5 70.2 71.2 6 71.9 73.5 72.8 Figure 4.9: Load Vs Deformation curve The CBR value for given sample is 3.70. 4.1.11 SS Soil + 15 mm sand column TABLE 4.10. : CBR value determination for SS soil +15 mm sand column Penetration Sample 1(kg) Sample 2(kg) Sample 3(kg) 0 0 0 0 0.5 21.6 24.1 22.6 2 38.2 46.6 41.2 2.5 52.2 54.1 51.2 3 54.1 57.1 56.2 3.5 56.7 59.6 59.4 4 60.2 62.5 63.3 4.5 62.3 64.8 63.7 5 67.1 67.8 66.5 5.5 67.8 70.9 71.7 6 70.5 72.9 72.2 Figure 4.10: Load Vs Deformation curve The CBR value for given sample is 3.832. 4.1.12 SS Soil + 20 mm sand column TABLE 4.11. : CBR value determination for SS soil +20 mm sand column Penetration Sample 1(kg) Sample 2(kg) Sample 3(kg) 0 0 0 0 0.5 22.1 29.6 26.8 1 29.3 33.6 32.7 1.5 32.6 39.5 37.6 2 39.6 49.7 55.3 2.5 54.8 56.7 57.6 3 57.5 58.6 56.2 3.5 59.7 62.1 63.2 4 61.2 63.9 64.7 4.5 63.4 64.5 65.8 5 69.6 68.3 69.1 Figure 4.11: Load Vs Deformation curve The CBR value for given sample is 4.11. 4.1.13 SS Soil + 10 mm stone dust column TABLE 4.12. : CBR value determination for SS soil +10 mm stone dust column Penetration Sample 1(kg) Sample 2(kg) Sample 3(kg) 0 0 0 0 0.5 19.6 23.1 24.2 1 26.1 28.2 31.4 1.5 30.2 39.6 35.4 2 36.3 49.8 45.6 2.5 51.1 53.4 49.9 3 54.3 58.7 56.6 4.5 60.1 63.1 69.1 5 64.3 65.4 69.9 5.5 66.4 69.1 72.4 6 71.2 72.6 72.6 Figure 4.12: Load Vs Deformation curve The CBR value for given sample is 3.756 4.1.14 SS Soil + 15 mm stone dust column TABLE 4.13. : CBR value determination for SS soil +15 mm stone dust column Penetration Sample 1(kg) Sample 2(kg) Sample 3(kg) 0 0 0 0 0.5 25.5 26.5 29.8 1 31.1 33.6 36.2 2.5 54.2 56.6 53.6 3 56.8 59.7 58.4 3.5 58.9 62.4 60.9 4 60.2 65.4 67.4 4.5 61.3 66.3 70.1 5 65.4 67.9 73.9 5.5 70.1 71.4 74.5 6 74.5 73.6 75.6 Figure 4.13: Load Vs Deformation curve The CBR value for given sample is 4.0. 4.1.15 SS Soil + 20 mm stone dust column TABLE 4.14. : CBR value determination for SS soil +20 mm stone dust column Penetration Sample 1(kg) Sample 2(kg) Sample 3(kg)

0 0 0 0.5 27.8 29.3 25.6 1 32.2 36.5 37.8 1.5 42.3 49.1 46.5 2 49.1 56.1 54.1 2.5 54.2 60.2 58.8 3 59.9 64.8 62.8 3.5 60.5 69.3 65.5 4 62.8 72.4 69.7 4.5 66.7 75.9 73.2 5 70.3 76.9 74.3 5.5 74.6 78.2 78.4 6 Figure 4.14: Load Vs Deformation curve The CBR value for given sample is 4.21. 4.2.1 Direct Shear Test 4.2.2 Plain BCS Figure 4.15: Load Vs Deformation curve for various normal loads TABLE 4.15. : Result Of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 59.72 100 61.38 150 65.27 Figure 4.16: shear stress Vs normal load for plain BCS c=56.57 KN/m2 and É¸= tan- 1(0.055) =3.14o 4.2.3 BCS + 10 mm stone dust column Figure 4.17: Load Vs Deformation curve for various normal loads TABLE 4.16. : Result Obtained from Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 53.88 100 56.11 150 63.33 Figure 4.18: shear stress Vs normal load for BCS reinforced with 10 mm stone dust column c=48.34 KN/m2 and É¸= tan-1(0.094) =5.37 o 4.2.4 BCS + 15 mm stone dust column Figure 4.19: load Vs deformation curve for various normal loads TABLE 4.17. : Result Of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 50.27 100 60 150 66.38 Figure 4.20: shear stress Vs normal load for BCS reinforced with 15 mm stone dust column c=42.77 KN/m2 and É¸= tan-1(0.161) =9.141o 4.2.5 BCS + 20 mm stone dust column Figure 4.21: load Vs deformation curve for various normal loads TABLE 4.18. : Result Of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 47.77 100 65.55 150 71.38 Figure 4.22: shear stress Vs normal load for BCS reinforced with 20 mm stone dust column c=37.95 KN/m2 and É¸= tan-1(0.236) =13.278o 4.2.6 BCS + 10 mm sand column Figure 4.23: load Vs deformation curve for various normal loads TABLE 4.19. : Result Of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 55.0 100 57.77 150 63.61 Figure 4.24: shear stress Vs normal load for BCS reinforced 10 mm sand column c=50.18 KN/m2 and É¸= tan-1(0.086) =4.91o. 4.2.7 BCS + 15 mm sand column Figure 4.25: load Vs deformation curve for various normal loads TABLE 4.20. : Result Of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 54.722 100 59.72 150 69.16 Figure 4.26: shear stress Vs normal load for BCS reinforced 15 mm sand column c=46.76 KN/m2 and É¸= tan-1(0.144) =8.19o. 4.2.8 BCS + 20 mm sand column Figure 4.27: load Vs deformation curve for various normal loads TABLE 4.21. : Result Of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 50.83 100 61.38 150 73.16 Figure 4.28: shear stress Vs normal load for BCS reinforced 15 mm sand column 4.2.9 SS soil +10 mm sand column Figure 4.29: load Vs deformation curve for various normal loads TABLE 4.22. : Result Of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 22.22 100 26.66 150 54.72 4.2.10 SS soil +15 mm sand column Figure 4.31: load Vs deformation curve for various normal loads TABLE 4.23. : Result Of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 23.24 100 28.05 150 58 4.2.11 SS soil +20 mm sand column Figure 4.33: load Vs deformation curve for various normal loads TABLE 4.24. : Result Of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 34.16 100 68.33 150 100.83 Figure 4.34: shear stress Vs normal load for SS soil with 20 mm sand column c=1.103KN/m2 and É¸= tan-1(0.666) =33.663o 4.2.12 SS soil +10 mm stone dust column Figure 4.35: load Vs deformation curve for various normal loads TABLE 4.25. : Result Of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 24.9 100 28.33 150 54.44 Figure 4.36: shear stress Vs normal load for SS soil with 10 mm stone dust column c=6.35 KN/m2 and É¸= tan-1(0.295) =16.43o. 4.2.13 SS soil +15 mm stone dust column Figure 4.37: load Vs deformation curve for various normal loads TABLE 4.26. : Result Of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 30.27 100 36.94 150 72.5 Figure 4.38: shear stress Vs normal load for SS soil with 15 mm stone dust column c=4.34 KN/m2 and É¸= tan-1(0.422) =22.8o 4.2.14 SS soil + 20 mm stone dust column Figure 4.39: load Vs deformation curve for various normal loads TABLE 4.27. : Result Of Direct Shear Test Normal load (kN/m2) Shear stress(kN/m2) 50 41.94 100 48.88 150 102.22 Figure 4.40: shear stress Vs normal load for SS soil with 20 mm stone dust column c=4.066KN/m2 and É¸= tan-1(0.602) =31.04o Figure 4.41.: DST box with silt soil with granular material columns Figure 4.42.: failed samples 4.3.1 Swelling behavior of BCS

In order to evaluate the effect of granular column on swelling behavior of BCS the soil was kept in soaking condition compacted at omc in CBR mould. The swelling was recorded every 24 hrs for a duration of 4 days. This was adopted for both soil in plain and in reinforced condition. Following results were obtained: TABLE 4.28. : Swelling for plain BCS and stone dust reinforced BCS Time (Hours) Swelling(mm) Plain BCS 10 mm column 15 column 20 column 24 6.45 6.55 6.65 6.35 48 11.10 10.75 10.30 9.70 72 12.65 12.05 11.50 10.90 96 13.00 12.50 12.10 11.10 Figure 4.43: swelling behavior of plain and reinforced soil TABLE 4.29. : Swelling for plain BCS and sand reinforced BCS Time (Hours) Swelling(mm) Plain BCS 10 mm column 15 column 20 column 24 6.45 6.25 6.35 6.25 48 11.10 10.85 10.65 9.50 72 12.65 12.50 11.30 10.70 96 13.00 12.75 11.90 11.0 Figure 4.44: swelling behavior of plain and reinforced soil CHAPTER 5: RESULTS AND DISCUSSION The properties of the materials adopted for the study have been analyzed earlier however speaking of BCS in particular that soil can be classified as CH. The relative consistency was found to be greater than 1 hence the expansive soil was in semi solid state, similar deductions can be drawn from liquidity index too. Variation of CBR TABLE 5.1.: Variation of CBR with diameter of granular columns Type of cases BCS SS soil Stone Dust Sand Stone Dust Sand CBR (%) % increase CBR (%) % increase CBR (%) %increase CBR (%) %increase plain 2.130 0 2.130 0 3.53 0 3.530 0 10 mm 2.746 28.92 2.560 20.18 3.76 6.51 3.700 4.81 15 mm 3.235 51.8 2.919 37.04 4.0 13.31 3.832 8.55 20 mm 3.520 65.2 3.063 43.8 4.21 19.26 4.110 16.43 The results above show after the installation of granular columns there has been increase in the load taking capacity of soils considered, the possible reason for such behavior of soil can be deduced from previous research studies. When the load is applied on the soil in the CBR mould the load is concentrated on the granular column, under the effect of load the column tries to fail but its failure is resisted

by the lateral support and resistance provided by the surrounding soil. 3

However when we compare the BCS to SS soil which is basically comprising of the granular material itself, the granular columns prove more fruitful in BCS, the granular columns in SS soil give low value due presence of sand in soil which lowers the cohesion value restricting the lateral support to bulging action of granular column when loaded. Figure 5.1: variation of cbr for given soil with varying diameter TABLE 5.2.: Variation of c-Ï• with diameter of granular columns Variation of c-Ï•Type of cases BCS SS soil Stone Dust Sand Stone Dust Sand c Ï• c Ï• c Ï• c Ï• plain 56.57 3.14 56.57 3.14 8.05 29.4 8.05 29.4 10 mm 48.34 5.37 50.18 4.91 6.35 16.43 2.033 18.001 15 mm 42.77 9.14 48.76 8.19 4.34 22.8 1.67 19.13 20 mm 37.95 13.278 39.16 12.78 4.066 31.04 1.103 33.663 The tabular form of data presented above after conduction of dsts show that after the soil was installed with granular columns there was decrease in cohesion value of the soil. The decrement of cohesion value increases with the diameter of the column, the reason for happening so is reduction in the area of plane of failure

as the diameter of column is increased. With the increase in the diameter of column 23

there is an increase in Ï• value of soil again for the similar reason as stated above, however the effect of granular column on Ï• value of SS soil unpredictable which was due to presence of fine sand particles in the soil. A better understanding can be developed from the graph plotted below: Figure 5.2: variation of cohesion value for given soil with varying diameter Figure 5.3: variation of Ï• value for given soil with varying diameter After studying the diameter variation of reinforcements on soilâ€Ÿs shear parameters and the load carrying capacity, it also becomes very important to go through the result which we got from the swelling tests that were performed during the investigation program. It was seen that soil like BCS that could swell upto large extent was cut down short after installation of granular columns. The possible reason for this could be easy the drainage facility allowed by the granular columns and minimization of pore pressure caused by them. They also allow expansion in lateral direction. Hence in this manner swelling was reduced. However when it comes to choose between the two granular material, considering the alternations it caused in load bearing capacity, shear strength criteria and the swelling criteria itâ€Ÿs rational to choose stone dust over sand. Let us also learn the variation of CBR and c-Ï• value variation with area replacement ratio where: Area replacement ratio = (area of granular column / area of the mould in which sample is placed). Ar= Ag/Am Diameter of CBR mould=150 mm TABLE 5.3.: Variation of CBR with respect to area replacement ratio Type of cases BCS SS soil - Ar Stone Dust Sand Stone Dust Sand CBR (%) % increase CBR (%) % increase CBR (%) %increase CBR (%) %increase 0 0 2.130 0 2.130 0 3.53 0 3.530 0 10 mm .0044 2.746 28.92 2.560 20.18 3.76 6.51 3.700 4.81 15 mm .0010 3.235 51.8 2.919 37.04 4.0 13.31 3.832 8.55 20 mm .0177 3.520 65.2 3.063 43.8 4.21 19.26 4.110 16.43 Figure 5.4: Variation of CBR with respect to area replacement ratio TABLE 5.4.: Variation of c-Ï• with respect to area replacement ratio Variation of c-Ï•Type of cases BCS SS soil - Ar Stone Dust Sand Stone Dust Sand c Ï• c Ï• c Ï• c Ï• plain 0 56.57 3.14 56.57 3.14 8.05 29.4 8.05 29.4 10 mm .0021 48.34 5.37 50.18 4.91 6.35 16.43 2.033 18.001 15 mm .0490 42.77 9.14 48.76 8.19 4.34 22.8 1.67 19.13 20 mm .0872 37.95 13.278 39.16 12.78 4.066 31.04 1.103 33.663 Figure 5.5: Variation of c with respect to area replacement ratio Figure 5.6: Variation of Ï• with respect to area replacement ratio CHAPTER 6: CONCLUSIONS

On the basis of the experimental results following conclusion can be drawn: â€¢ The **22**

results of both CBR and DST reveal that the strength of the silty soil and black cotton soil increases due to installation of granular columns. â€¢ The granular column made up of stone dust gives better results as compared to sand columns in both the soils adopted for the study. Therefore stone dust obtained from the rock crushing industries often discarded as a waste material should be used wisely. â€¢ With increase in % area of fraction of granular material the strength of overall composite reinforced soil increases. â€¢ The swelling behavior of black cotton soil is also reduced after the installation of granular column. This is an extra advantage that could be induced in expansive soil after the installation of granular columns. Future Scope â€¢ In place of stone dust other kinds of waste material can also be used as granular column after conducting the require experimentation. â€¢ These experiments can be repeated

to study the group effect of columns. â€¢ The interference of **12**

group of columns can also be observed. â€¢ A real prototype model may be constructed for correlation. REFERENCES 1. Mir,B.A.and Juneja,a:2012,strength behavior of composite ground reinforced with sand columns, asce journal of geotechnical engineering. 2. Ilan Juran and Oraccio Riccobono: 1988, reinforcing soft soils with artificially cemented sand columns, asce journal of geotechnical engineering. 3. Shadi S. Najjar,Salah Sadek and Tarek Maakaroun: 2010, effect of sand columns on the undrained load response of soft clays , asce journal of geotechnical engineering. 4. Wankyu Yoo, Byoung-Il Kim, and Wanjei Cho:

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