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Study of Stress- Strain Behaviour of Cohesive Soil Mixed With Granite Dust under Triaxal Test

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ABSTRACT

One of the most important characteristics in construction which is widely used in connection with road pavements and foundation is soil stabilization. In other to enhance the engineering properties of the soil, stabilization techniques is required. The addition of waste materials like, fly ash, rice husk ash etc. increases the physical as well as chemical properties of the soil. Some of the expected properties to be improved are shear strength, liquidity index, plasticity index, unconfined compressive strength and bearing capacity. One of the primary objective of this research is to analyse the effect of granite dust on cohesive soil. This research studied the application of granite dust material on the geotechnical properties of soil. For the purpose of study, three soil samples at different location were collected and considered. All test in raw soil and treated/stabilized soil were performed at The Polytechnic, Ibadan soil laboratory. The soil were collected from location at Sango-Ojoo Road, Ijokodo-Apete Road, and UI-Ajibode Road Ibadan, Oyo State. Nigeria, with the coordinates of Latitude 7.45708, (N7o27’25.48”), Longitude 3.90869 (E 3o54’31.29”), Latitude 7.44603 (N 7o26’45.70”), Longitude 3.87602, (E 3o52’33.67”) and Latitude 7.43509, (N7o24’42.47”), Longitude 3.76253, (E 7o23’45.35”) respectively. The granite dust was collected from the RCC Quarry site, within Ibadan for the stabilization studies. The experimental program were carried out on the samples treated with different percentages of granite dust at 0%, 2%, 4%, 6% and 8%. The research depicts the study carried out to check the improvements in the strength properties of soil stabilization by granite dust in varying percentages. The dry density of the soil increases with addition of granite dust at varying percentages and a corresponding decrease in the moisture content. The test carried out shows that there is an improvement in the strength properties of the soil as a result of an increase in the cell pressure with an increase in the granite dust proportion. It is therefore concluded that granite dust is an effective stabilizer which can be used as additive to improve geotechnical properties of cohesive soil when necessary. However, no assumption should be made that granite dust should be considered as suitable stabilizer for all type of soil until they are tested with the sample.

Keywords: *Granite Dust, Soil Stabilization, Cell Pressure*

INTRODUCTION

It is very important to establish the mechanical properties of soils when designing foundations, embankments and other soil structures. Building constructions, excavations, tunneling and similar applications have several effects on the subsoil structures. These effects are successfully simulated with Triaxial Tests where the stress-strain relation of undisturbed soil specimen is investigated by subjecting the soil sample to different stress levels and drainage conditions. The process for constructing a building or road involves analyzing the soil on which construction will take place. Soil environments provide the physical foundation for a building or road to stand on for years. The engineering aspects of soil composition examine the differences in texture, strength, and consistency that distinguish cohesive soils from non-cohesive soil environments. (Abdul Samad, et. al., 2017)

Geotechnical engineering involves the use of soils as engineering materials. This enables engineers to identify suitable soil environments for building and construction purposes. A soil’s ability to compact and maintain its consistency under pressure determines whether it will provide a suitable foundation for building. (Akolade and Olaniyan, 2014).In effect, the physical characteristics of a soil environment are examined as part of the pre-planning process involved with construction projects. As a result, difference between cohesive and non-cohesive soil play a significant role in determining whether a particular area will work with a construction’s plan. Texture differences in soils result from the types of rock that make up a particular area. Over time, the effects of weather and water erosion break down preexisting rocks into soil particles. Texture differences appear in the shapes, sizes and arrangement of particles that make up the soil. The presence or absence of clay or fine particles determines the cohesive qualities found within a soil environment. In effect, clay and fine particle materials act as binding agents that hold soil together. So non-cohesive soil environments contain little to no clay or fine particles while cohesive soils contain high amounts of clay and fine particles. (Marta, et. al., 2014)

A soil’s ability to compact has to do with the size of its particles and the amount of clay present in the sample. As a material, clay tends to readily absorb water when compared to a sand-type material. This absorption factor increases a soil’s capacity to compact into a mold. A soil sample is analyzed to gauge its plasticity, or how well it molds together. Hence, difference between cohesive and non-cohesive soils appear as high versus low plasticity properties. In effect, the higher a soil’s plasticity properties, the more likely it will hold its shape when subjected to additional weight or pressure. Non-cohesive soils consist of large or irregular-sized soil particles with little to no clay content. These soils tend to shift or change in consistency under different environmental conditions. Rain and wind conditions cause water and air materials to move in and out of soils. These conditions create spaces in between soil particles. (Odewumi, et. al., 2018)

The strength and deformation behavior of soil materials are often evaluated by using conventional triaxial compression tests, where a cylindrical soil specimen under axisymmetric loading condition is tested to simplify the field conditions. However, most of the field problems, such as strip footings, embankments, and slopes are three dimensional or close to a plane strain condition. The stress-strain and strength behavior under plane strain condition may differ from those measured under triaxial condition. To evaluate the soil behavior in such cases, a variety of plane strain or true triaxial devices have been developed. (Akai, et. al., 1981).

Soil masses are often subjected to repeated or transient loads. With the increases usage of offshore structures and an increased concern for adequate seismic design, the required accuracy of dynamic soil analysis has arisen dramatically. A number of studies have been concerned with the stress and deformation responses of soil subjected to repeated loadings. It has been recognized that the behavior of soils subjected to repeated cycles of loading may differ considerably from their behavior during a single loading cycle. There are many natural situations in which the duration of the series of loading cycle is such that little or no drainage of the pore water can take place during the period of the repeated loading. It is therefore useful to study the effects of repeated loading under undrained conditions in the laboratory. (Lekha, 2012)

In a triaxial shear test, stress is applied to a sample of the material being tested in a way which results in stresses along one axis being different from the stresses in perpendicular directions. This is typically achieved by placing the sample between two parallel platens which apply stress in one (usually vertical) direction, and applying fluid pressure to the specimen to apply stress in the perpendicular directions. From the triaxial test data, it is possible to extract fundamental material parameters about the sample, including its angle of shearing resistance, apparent dilatancy angle. These parameters are then used in computer models to predict how the material will behave in a large-scale engineering application. An example would be predict the stability of the soil on a slope, whether the slope will collapse or whether the soil will support the shear stresses of the slope and remain in place. Triaxial tests are used along with other tests to make such engineering predictions.

MATERIAL AND METHODS

Soil samples collected was obtained from different site location at Sango-Ojoo Road, Ijokodo-Apete Road, and UI-Ajibode Road Ibadan, Oyo State, Nigeria, with the coordinates of Latitude 7.45708, (N7^{o}27'25.48"), Longitude 3.90869 (E 3^{o}54'31.29"), Latitude 7.44603 (N 7^{o}26'45.70"), Longitude 3.87602, (E 3^{o}52'33.67") and Latitude 7.43509, (N7^{o}24'42.47"), Longitude 3.76253, (E 7^{o}23'45.35") respectively was used in this study.

The soil samples obtained from different locations in Oyo State, Nigeria, were tagged with unique identification numbers A, B and C. They were dug out from the ground at a depth not less than 150mm (natural ground formation with at least 300 mm top soil removed). The soil samples were kept safe and dry in the Soil laboratory at the Department of Civil Engineering, The Polytechnic, Ibadan. Oyo State. Nigeria. The soil samples were kept in clean bags and properly sealed with adhesive tapes. Laboratory tests was conducted on the soil samples and granite dust. The various test method employed to determine some geotechnical properties of the soil in accordance with BS 1377 (1990) includes: Moisture Content, Sieve analysis, Atterberg Limit (Plastic limit test, Liquid limit test), Specific Gravity, Compaction Test, Triaxial test as shown in figure 1 and 2 below. The granite dust was used as partial replacement on each sample with varying proportion of 2%, 4%, 6% and 8% respectively

Fig 1: Experimental operation (Compaction and Soil Sample preparation for Triaxial Test)

Fig 2: Experimental operation (Tri-axial Test)

Result and Discussion

The summary of the result of the laboratory test (grain size analysis, compaction and atterbergs limit) shown below. The engineering property test (triaxial) is presented in the tables and figures below.

3.1 Natural Moisture Content

This is the amounts of moisture present at the time of collection and storing in the laboratory. The natural moisture content also called the natural water content and this is the ratio of the weight of water to the weight of the solids in a given mass of soil. This ratio is usually expressed as percentages. The result of the natural moisture content of soil sample A, B & C were 17.59%, 18.63% and 24.70% respectively while the Granite Dust sample was 13.18%.

3.2 Sieve Analysis

In order to classify a soil for engineering purposes, it is important to know the grain size distribution in a given soil mass. Sieve analysis is a method used to determine the grain size distribution of soils. This method is applicable to most granular soils with some fines. Sieve analysis on the other hand does not provide information as to the shape of particles present in the soil mass. Table 1 and Fig 3 below shows the particle size distribution of soil samples A, B and C.

Table 1: Particle Size Distribution of Soil Samples A, B and C.

Plot data	**Sample A**	**Sample B**	**Sample C**
Size (mm)	**% Passing**	**% Passing**	**% passing**
10	78.7	90.58	72.95
4.75	55.5	76.77	49.35
2	29.5	53.43	25.75
1.18	18.5	31.05	16.23
0.6	8.9	11.56	7.01
0.3	3.2	3.42	1.51
0.15	0.8	0.64	0.00
0.075	0.0	0.00	0.00

Table 2: AASHTO and USC Classification of the Soil Samples

Soil Samples	Liquid Limit (LL)%	Plastic Limit (PL)%	Plasticity Index (PI)%	Uniformity Coefficient (Cu)	Coefficient of Curvature (Cc)	Typical Name and General Rating AASHTO and USC
A	11.84	32.22	20.38	7.14 (well graded)	1.16 (well graded)	AASHTO: A-6 (Clayey Soil), Fair to Good; A-7/A-7-5 (Clayey Soils); Fair to Good; A-2/A-2-6 (Silty or Clayey Soil, Excellent to Good. USC:GM(Silty Gravel), GC(Clayey Gravel): Gravel with fines
B	11.80	36.90	25.10	3.63 (poorly graded)	0.514(poorlygraded)	
C	14.00	38.89	24.89	8.89 (well graded)	1.36 (well graded)	

3.5 Compaction Test

The compaction test was carried out in order to determine the Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) of the samples A-C. The AASHTO compaction test type was used with proctor mould and 4.5kg weight rammer with 62 blows per each of three layer dropping with free fall from an height 300mm having a capacity of 944 cc with an internal diameter of 102mm and height of 116mm in accordance with B.S Codes 1377 part 4. The MDD and OMC for the soil samples (A -C) without inclusion of Granite Dust are (1.19kg/m^3, 10.90%; 1.25 kg/m^3, 16.4%; 1.34 kg/m^3, 10.7%) respectively. The MDD and OMC for the soil samples (A -C) with the inclusion of Granite Dust at various percentages of (2, 4, 6 and 8)% are: Soil Sample A (1.28 kg/m^3, 15.3%; 1.29 kg/m^3, 9.05%; 1.31 kg/m^3, 8.77% and 1.36 kg/m^3, 5.06%), Soil Sample B (1.36 kg/m^3, 8.82%; 1.36 kg/m^3, 8.48%; 1.37 kg/m^3, 8.12% and 1.38, 6.64%) and Soil Sample C (1.30 kg/m^3, 12.6%; 1.31 kg/m^3, 11.8%; 1.39 kg/m^3, 5.22% and 1.40 kg/m^3, 4.56%) respectively.

Hence, there is an increase in the Maximum Dry Density (MDD) with a corresponding decrease in the Optimum Moisture Content (OMC) of the three soil samples.

3.6 Triaxial Test

Triaxial test was carried out in order to the determine shear strength and to examine the generalized states of stress of the soil by cylindrical compression test. The cylindrical specimen is sealed in a water-tight rubber membrane and enclosed in a cell which subjected to fluid pressure. The water pressure, (cell pressure) is measured directly from a manometer or gauge. After the performance of stress analysis on the soil sample assumed as a continuum, the components of the Cauchy stress tensor at a particular material point are known with respect to a coordinate system. Therefore, the Mohr circle is used to determine graphically the stress components acting on a rotated coordinate system, i.e., acting on a differently oriented plane passing. The result analysis of the Mohr circle is presented in Table 3 and Fig 4, 5 & 6 below.

Table 3: Result of Mohr Circle Plot of Soil Sample A, B and C with Granite Mix.

Soil Samples	Granite Dust Percentage	Cell Pressure (∂_3) (Kg/m^2)	Principal Stress (∂_1) (Kg/m^2)
A	2%	50.00	140.00
	4%	100.00	249.00
	6%	150.00	450.00
	8%	200.00	450.00
B	2%	0.00	28.00
	4%	20.00	59.00
	6%	40.00	95.00
	8%	60.00	128.00
C	2%	25.00	124.00
	4%	50.00	187.00
	6%	75.00	246.00
	8%	100.00	310.00

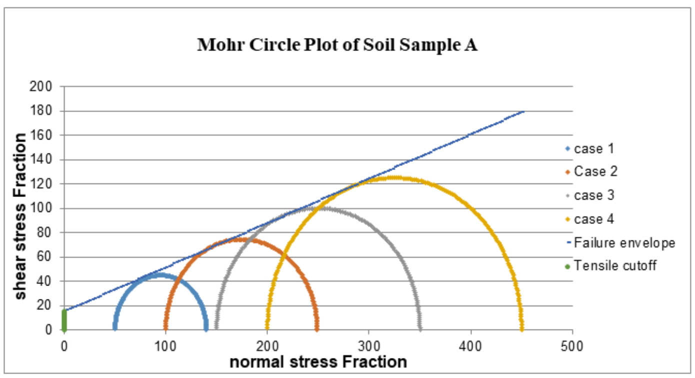

Figure 4: Mohr Circle Plot of Soil Sample A

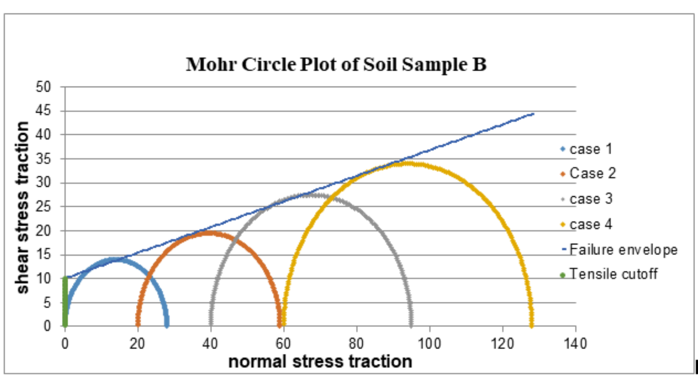

Figure 5: Mohr Circle Plot of Soil Sample B

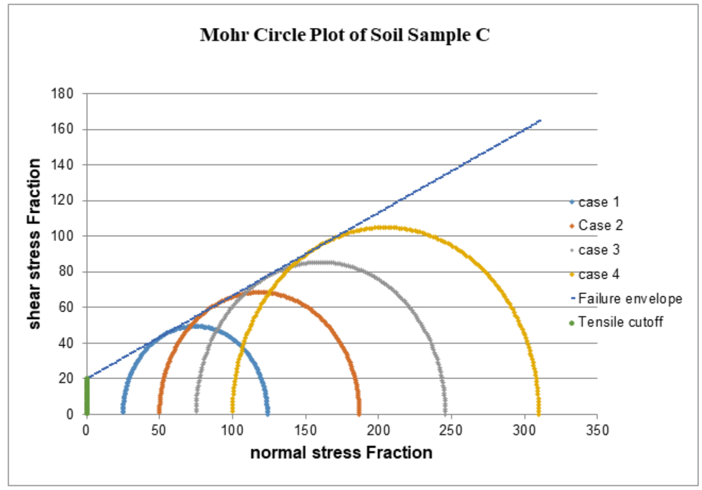

Figure 6: Mohr Circle Plot of Soil Sample C

CONCLUSION AND RECOMMENDATION

A comprehensive investigation into some geotechnical properties of cohesive soil before and after addition of different percent of Granite Dust material has been carried out. Based on experiment, test and findings, the cohesive soils contains medium sized gravel to fine sand following the results of sieve analysis. The cohesive soil A, B and C has a liquid limit, plastic limit and plasticity index of (11.84%, 11.8%, 14%); (32.33%, 36.90%, 38.89%) and (20.38%, 25.10%, 24.90%) respectively. The soil with addition of different percent of Granite Dust has LL and PI which lies within the range of 11.80% - 14.00% and 20.38%-25.10% respectively. This result describes the soils has a clayey soil which is a fair material because liquid limit is less than 30%, plastic limit is greater than 30% and plasticity index is less than 30% (LL<30%, PL>30%, and PI<30%). Therefore the soil is classified under A-7-5 type of soil using AASHTO classification system. From the liquid limit, plasticity index, percent of soil passing sieve 0.425 mm (No. 40), the soils A,B and C are classified as A-2-6 and A-6 using AASHTO classification system and they are rated as well graded materials. The general description of soil is silty or clayey gravel sand.

Using the USC classification system, the soils are rated as Silty Gravel (GM), Clayey Gravel (GC), i.e., Gravel with fines. Hence, the soils are rated as well graded subgrades. The uniformity coefficient (Cu) and the coefficient of curvature (Cc) of soil A and C are classified as well graded material while soil B is classified as a poorly graded material. The cohesive soil with the addition of Granite Dust at increasing percentages has an increment in the Maximum Dry Density with a successive reduction of Optimum Moisture Content. Thus making it more suitable as a stabilizing agent. That is, the higher the granite dust added to the soil the higher the value of the dry density and the lower the moisture content. As more percent of Granite Dust was added to the cohesive soil A, B and C, the shear strength of the soils increased. Thus, Granite Dust increases the shear strength of soil. From the Mohr Circle analysis, it is observed that there is an increase in the principal stress as a result of an increase in the cell pressure with increase in the granite dust proportion.

On this note, it can be concluded that Granite Dust material can be used as additive to improve geotechnical properties of cohesive soil when necessary. Hence, Granite Dust is an effective soil stabilizing agent. This study also concludes that the shear strength and stress deformation characteristics changes with the increase of confining pressure. When soils fails to meet up with requirement, additive such as Granite Dust can be used with cement to improve its geotechnical properties. Further research should also be carried out with varying proportion of Granite Dust different from those used in this study to determine the possible effect of higher percent of Granite Dust on the geotechnical properties of the soil.

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