

# Triaxial Behaviour of Stabilized Soil by Quarry Dust

S. K. Vigneshwar<sup>1</sup>, G.Vigneswaran<sup>2</sup>, S.Gobinath<sup>3</sup>, R. Thirumalai<sup>4</sup>, Dr. S. Suresh Babu<sup>5</sup>

UG Scholar, Dept. of Civil Engineering, Adhiyamaan College of Engineering, Hosur, Tamil Nadu, India<sup>1,2</sup>

Research Scholar, VTU Research Resource Centre, Visvesvaraya Technological University, Belgaum, Karnataka<sup>3</sup>,

Assistant Professor, Dept. of Civil Engineering, Adhiyamaan College of Engineering, Hosur, Tamil Nadu, India<sup>4</sup>

Professor & Head, Dept. of Civil Engineering, Adhiyamaan College of Engineering, Hosur, Tamil Nadu, India<sup>5</sup>

**ABSTRACT:** Soil stabilization is one of the most important characteristics for the construction which is widely used in connection with road pavements and foundation construction. To enhance the engineering properties of the soil, it requires various soil stabilization techniques, some waste materials such as fly ash, rice husk ash, quarry dust, pond ash may use to make the soil to be stable. Addition of such materials will increase 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 study is to determine the effect of quarry dust, Quarry dust is a waste obtained during quarrying process. It has very recently gained good attention to be used as an effective filler material instead of fine aggregate. The present paper depicts the study carried out to check the improvements in the strength properties of soil stabilization by quarry dust in varying percentages [10%, 20%, 30%, 40% & 50%].

**KEYWORDS:** Soil stabilization, Quarry dust, Tri-axial tests, bearing capacity

## I. INTRODUCTION

### A. GENERAL

Triaxial test methods are used in the laboratory to measure the shear strength parameters of a soil, where in the stress and strain conditions and drainage patterns are different. However, these methods are affected by many factors, one such factor is the rate of loading at which the external load is applied to the specimen. The test may however, be performed in various ways; in order to distinguish between the different types of test and relate them to the more common practical problems, it is necessary to make a brief survey of the basic factors controlling shear strength and deformation.

### B. THE THEORY OF TRIAXIAL TEST:

Ideally, the triaxial test should permit independent control of the three principle stresses so that generalized states of stress can be examined, including the important special case corresponding to plane strain. The type of triaxial test most commonly used in research work and in routine testing is the cylindrical compression test. The cylindrical specimen is sealed in a water-tight rubber membrane and enclosed in a cell in which it can be subjected to fluid pressure. The water pressure, usually called cell pressure, can be measured directly from a manometer or gauge, and an adaptation also enables the pore water pressure inside the sample to be recorded. During the application of this load the sample experience shortening in the vertical direction with a corresponding expansion in the horizontal direction. Under these conditions the axial stress is the major principle stress  $\sigma_1$ ; the intermediate and minor principle stresses ( $\sigma_2$  and  $\sigma_3$ , respectively) are both equal to the cell pressure. Connections to the ends of the sample permit either the drainage of water and air from the voids in the soil or, alternatively, the measurement of the pore pressure under conditions of no drainage.

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## C. MERITS AND DEMERITS OF TRIAXIAL TEST

### i. Merits of triaxial test

- Triaxial test has complete control over the drainage conditions. Tests can be easily conducted for all three types of drainage conditions.
- Pore pressure and volumetric changes are possible to be measured directly.
- Stress distribution is uniform on the failure plane.
- Another advantages is that the specimen will fail on its weakest plane.
- The state of stress at all intermediate stages upto failure is known. It is possible to draw the Mohr circle at any shear stage.
- The test is suitable for accurate research work. The apparatus is adaptable to special requirements such as extension test and tests for different stress paths.

### ii. Demerits of triaxial test

- Triaxial apparatus is expensive.
- It is impossible to calculate cross sectional area of the specimen accurately at large strains, as the assumption that the specimen remains cylindrical does not hold well.
- The test simulates only axis symmetrical problems. In the field, the problem is generally 3 dimensional. A general test in which all the three stresses are varied would be more useful.
- Specimen consolidation in the triaxial test is isotropic; whereas in the field, the consolidation is generally anisotropic.

## D. OBJECTIVES OF THE PROJECT:

- ✓ To quantify the triaxial behavior of plain soil and soil mixed with different volumetric proportions of quarry dust.
- ✓ To identify the shift in failure envelope in the above two cases.

## II. LITERATURE SURVEY

Jagmohan Mishra et.al (ISSN 0976-3945, volume 3, November 2014) presented a paper on the topic of "Effect of Granite dust on Index properties of lime stabilized black cotton soil". In this research, they presented the effect of granite dust on the characteristics of soil in International Journal of Engineering Research and Science and Technology. He conducted compaction, triaxial and durability test on lime stabilized soil quarry dust. They concluded that there is a marked improvement in the properties of soil when granite dust is mixed in different proportion. It is reported that there is a significant control in the swelling behavior of the expansive soil when mixed with granite dust. Jagmohan Mishra found that the plasticity index of the soil decreases with increasing the percentage of granite dust.

O.Arun Kumar et.al (ISSN 2319-5347, volume 03. no.01, Jan 2014) presented a paper in the topic of "Soft Subgrade Stabilization with Quarry dust on Industrial waste". In this study, they conducted various experiments on the sample blended with waste material (i.e.) quarry dust at different percentage to determine the index and engineering properties of the soil. It is interested to note that the quarry dust has various effect on the properties of stabilized soil. They concluded that the addition of quarry dust to soil decreases the liquid limit, plasticity index, optimum moisture content and increases the maximum dry density. He identified that addition of quarry dust yield high CBR value. It is interested to investigate the brittle behaviour of cemented soils was more obvious in drained condition than in undrained state. They concluded the failure envelopes were not curved for the studied soil. Moreover, the failure envelopes of drained tests were lower than those of undrained ones.

S.S.Mofiz & M.M.rahman (ISSN 2249-2256, volume 5, 5-10 Sep 2010) presented a paper in the topic of "Shear Strength behaviour of barind soil on Triaxial extension stress path tests". In this study, it is observed that the test was carried out at consolidation pressure ranging from 100 – 500 K Pa and at a confining pressure of 200 K Pa. In this study, triaxial testing programs are divided into 3 stress paths (i).Conventional Triaxial Extension (CTE) (ii). Reduced Triaxial Extension (RTE) (iii). Triaxial Extension (TE). This study produces a non-linear stress strain relationships upon compression loading and the curves do not produce any distinct peak points. The experimental results exhibit

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contraction volume change behaviour at lower stress levels and the rate of contraction decreases at higher stress levels. This study concludes the shear strength & stress deformation characteristics changes with the increase of confining pressure. The elastic parameters such as modulus number, bulk modulus number, & bulk modulus exponent of barind soil are determined which also dependent on stress path. After analyzing, they found that there is a higher initial tangent modulus in CTE stress path and the lower value for RTE stress path. This is due to the fact that lower value of failure strain was found for CTE path in comparison to the RTE path.

Md.Kumruzzaman & Jain – Hua Yin. (ISSN 0983-1106, volume: 6, no.1, 2012) presented a paper in the topic of “Stress – Strain behavior of a completely decomposed granite in both Triaxial and Plane strain conditions”. In this paper, they studied the behaviour of a residual soil (a completely decomposed granite soil) under plane strain & triaxial condition. This study reveals that the excess pore water pressure is lower & the volumetric strain is higher for triaxial loading than those for plane strain loading. The experimental finding shows that the frictional angle for the plane strain is about 22% higher than for triaxial loading.

Shi Wei Cheng, Zhu Jun-Gao et al (ISSN 1095-1102, volume 2, 2010) presented a paper in the topic of “Strength and Deformation behaviour of coarse grain soil by true Triaxial test”. In this study, the true triaxial tests were conducted with constant minor & intermediate principle stress ratio. It is interested to note that the intermediate principal strain increases from negative to positive value & the minor principle strain is always negative. From the test results, the author presented an empirical equation of  $g(b)$  that is the shape function of the failure surface on the  $\pi$ -length. From the study it was found that the triaxial stress conditions decreases with increasing the magnitude of minor principle stress.

### III. MATERIALS AND METHODS

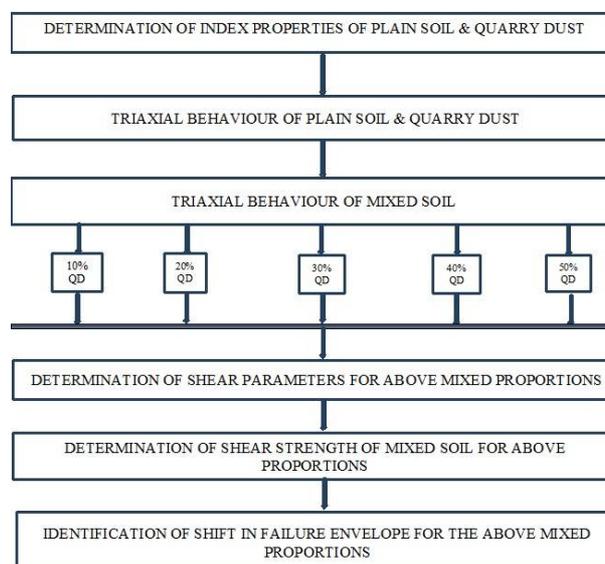
#### A. SOIL

Soil collected from AERI campus was used in this study, the specific gravity of soil is 2.65 having uniformity coefficient ( $C_u$ ) of 4.16 and coefficient of curvature ( $C_c$ ) of 1.161. The soil is classified as WG group of soil of permeability, compressibility, shear strength and workability is impervious, very high, good and bad respectively. The maximum dry density and optimum moisture content of soil as determined from the relative tests were found 19.14 KN/m<sup>3</sup> and 12% respectively.

#### B. QUARRY DUST:

Quarry dust collected from Hill Top granites, Shoolagiri, Hosur was used in this study, the specific gravity of soil is 2.86 having uniformity coefficient ( $C_u$ ) of 4.67 and coefficient of curvature ( $C_c$ ) of 1. The soil is classified as WG group of soil of permeability, compressibility, shear strength and workability is impervious, very high, good and bad respectively. The maximum dry density and optimum moisture content of quarry dust as determined from the relative tests were found as 17.02 kN/m<sup>3</sup> and 12.16% respectively.

#### C. METHODOLOGY



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### D. LABORATORY INVESTIGATIONS

Tests conducted for both soil and quarry dust are,

- Sieve analysis
- Specific gravity
- Liquid limit test
- Plastic limit test
- Triaxial test

#### i. SIEVE ANALYSIS:

The determination of the proportions of particles within certain size ranges in granular material by separation on sieves of different openings.

**Table -3.1 Sieve Analysis for Quarry dust and Plain soil**

sieve size	weight retained (gm)		% weight retained		Cumulative % weight retained		% Finer	
	QS	PS	QS	PS	QS	PS	QS	PS
4.75	66.5	59.1	6.65	5.91	6.65	5.91	93.35	94.09
2	143.7	193	14.37	19.3	21.02	25.21	78.98	74.79
1	114.1	209.2	11.41	20.92	32.43	46.13	67.57	53.87
0.6	261.3	285.2	26.13	28.52	58.56	74.65	41.44	23.35
0.3	183.3	146.7	18.33	14.67	76.89	89.32	23.11	10.68
0.212	104.9	50.5	10.49	5.05	87.38	94.37	12.62	5.63
0.15	56.2	24.9	5.62	2.49	93	96.36	7	3.14
0.075	35.2	21.2	3.52	2.12	96.52	98.98	3.48	1.02
PAN	34.8	10.2	3.48	1.02	100	100	0	0

#### Sieve Analysis for plain soil

$D_{10}=0.18$ ;  $D_{30}=0.39$ ;  $D_{60}=0.84$ ;  $CU= 4.666$ ;  $CC= 1$

#### Sieve Analysis for plain soil

$D_{10}=0.3$ ;  $D_{30}=0.66$ ;  $D_{60}=1.25$ ;  $CU = 4.16$ ;  $CC = 1.161$

\**CU- Coefficient of Uniformity; CC- Coefficient of Curvature*

#### ii. SPECIFIC GRAVITY:

The Specific Gravity of solid particles (G) is the ratio of the mass density of solids to that of water. It is determined in the laboratory using the relation,

$$\text{Specific Gravity, } G = \frac{(w_2 - w_1)}{(w_2 - w_1) - (w_3 - w_4)}$$

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**Table - 3.3 Specific Gravity for Quarry dust & plain soil:**

Trial no	SPECIFIC GRAVITY	
	QUARRY DUST	PLAIN SOIL
1	2.85	2.65
2	2.88	2.64
3	2.85	2.66

From the three trial which we have carried out in the laboratory, the average Specific Gravity of Quarry Dust is 2.86, From the three trial which we have done in the laboratory, the average Specific Gravity of plain soil = 2.65

### iii. ATTERBERG'S LIMIT:

#### 3.4.3.1 LIQUID LIMIT & PLASTIC LIMIT:

Liquid limit is defined as "the moisture content at which soil changes from liquid state to plastic state".

Plastic limit defined as "The moisture content at which the soil changes from plastic state to semi solid state".

**Table – 3.5 Liquid limit & Plastic limit values for plain soil and soil mixed with different percentage of quarry dust:**

Sl. No.	Percentage of soil + Quarry dust	Liquid limit (%)	Sl. No.	Percentage of soil + Quarry dust	Plastic limit (%)
1	100% + 0%	47.70	1	100% + 0%	25.65
2	90% + 10%	46.20	2	90% + 10%	24.72
3	80% + 20%	44.40	3	80% + 20%	24.09
4	70% + 30%	40.60	4	70% + 30%	23.44
5	60% + 40%	39	5	60% + 40%	22.57
6	50% + 50%	37.6	6	50% + 50%	21.83
7	0% + 100%	26.20	7	0% + 100%	17.39

## IV. RESULTS AND DISCUSSION

### A. TRIAXIAL TEST

The triaxial test is carried out on a cylindrical soil sample having a length to diameter ratio of 2. The usual sizes are 76mm x 38mm and 100mmx50mm. Three principle stresses are applied to the soil sample, out of which two are applied water pressure inside the confining cell and are equal. The third principle stress is applied by a loading ram through the top of the cell and is different to the other two principle stresses.

The soil sample is placed inside a rubber sheet which is sealed to a top cap and bottom pedestal by rubber o-rings. For test with pore pressure measurement, porous discs are placed at the bottom, and sometimes at the top of the specimen. Filter paper drains may be provided around the outside of the specimen in order to speed up the consolidation process. Pore pressure generated inside the specimen during testing can be measured by means of pressure transducers.

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**i. FOR PLAIN SOIL:**  
Pressure: 0.10 bars

**Table 4.1 Load & displacement table for plain soil**

LOAD IN KN	DISPLACEMENT IN MM (+ve)	DEVIATOR STRESS (kN/M <sup>2</sup> )	AXIAL STRAIN
0.01	1.67	0.0081	0.9810
0.05	1.63	0.0405	0.9814
0.10	1.58	0.0809	0.982
0.15	1.55	0.1214	0.9823
0.20	1.53	0.1618	0.9826
0.25	1.48	0.2022	0.9831
0.30	1.44	0.2425	0.9836
0.35	1.41	0.2829	0.9839
0.40	1.38	0.3233	0.9843
0.46	1.34	0.3716	0.9847
0.50	1.32	0.4038	0.985
0.55	1.30	0.4441	0.9852
0.60	1.23	0.4842	0.9860
0.65	1.22	0.5245	0.9861
0.70	1.19	0.5646	0.9864
0.76	1.17	0.6129	0.9867
0.80	1.10	0.6448	0.9875
0.85	1.05	0.6847	0.988
0.90	1	0.7247	0.9886

**ii. FOR SOIL + 10 % QUARRY DUST**  
Pressure - 0.15 bars

**Table 4.2 Load and Displacement table for soil + 10% QD**

LOAD IN KN	DISPLACEMENT IN MM (+ve)	DEVIATOR STRESS (KN/M <sup>2</sup> )	AXIAL STRAIN
0	7.99	0	0.9092
0.01	7.94	0.00862	0.9097
0.02	7.89	0.0172	0.9103
0.03	7.83	0.0258	0.9110
0.04	7.76	0.0344	0.9118
0.05	7.72	0.0429	0.91227
0.06	7.65	0.0515	0.9130

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iii. FOR SOIL + 20% QUARRY DUST:  
Pressure - 0.15 bars

Table 4.3 Load and Displacement table for soil + 20% QD

LOAD IN KN	DISPLACEMENT IN MM (-ve)	DEVIATOR STRESS (kN/M <sup>2</sup> )	AXIAL STRAIN
0.07	11.85	0.0627	0.8653
0.10	11.85	0.0897	0.8653
0.20	11.81	0.1793	0.8657
0.25	11.75	0.2240	0.8664
0.30	11.70	0.2687	0.8670
0.35	11.65	0.3133	0.8676
0.40	11.59	0.3578	0.8682
0.45	11.54	0.4023	0.8688
0.50	11.49	0.4468	0.8694
0.55	11.44	0.4912	0.87
0.60	11.39	0.5356	0.8705
0.65	11.32	0.5798	0.8713
0.72	11.24	0.6417	0.8722
0.75	11.21	0.6683	0.8726
0.80	11.14	0.7123	0.8734
1	10.89	0.8881	0.8762
1.05	10.83	0.9319	0.8769
1.10	10.78	0.9758	0.8775
1.15	10.74	1.0197	0.8779

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iv. FOR SOIL + 30% QUARRY DUST:  
Pressure: 0.10 bars

Table 4.4 Load and Displacement table for soil + 30% QD

LOAD IN KN	DISPLACEMENT IN MM (-ve)	DEVIATOR STRESS (KN/M <sup>2</sup> )	AXIAL STRAIN
0.00	11.35	0	0.8710
0.05	11.31	0.04460	0.8714
0.10	11.25	0.08914	0.8721
0.15	11.17	0.1336	0.8730
0.20	11.10	0.1780	0.8738
0.25	11.03	0.2223	0.8746
0.30	10.96	0.26663	0.8754
0.35	10.91	0.3109	0.8760
0.40	10.86	0.3551	0.8765
0.45	10.79	0.3992	0.8773
0.50	10.71	0.4432	0.8782
0.55	10.65	0.48725	0.8789
0.60	10.59	0.5312	0.8796
0.65	10.49	0.5748	0.88079
0.70	10.42	0.6186	0.8815
0.75	10.36	0.6624	0.8822
0.80	10.31	0.70623	0.8828
0.85	10.25	0.7499	0.8835
0.90	10.17	0.7933	0.8844

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v. FOR SOIL + 40% QUARRY DUST:  
Pressure: 0.12 bars

Table 4.5 Load and Displacement table for soil + 40% QD

LOAD IN KN	DISPLACEMENT IN MM (-ve)	DEVIATOR STRESS (KN/M <sup>2</sup> )	AXIAL STRAIN
0.02	5.77	0.0168	0.9344
0.10	5.76	0.0843	0.9345
0.15	5.71	0.1264	0.9351
0.20	5.63	0.1684	0.9360
0.25	5.55	0.2103	0.9369
0.30	5.48	0.2522	0.9377
0.35	5.42	0.2941	0.9384
0.40	5.35	0.3358	0.9392
0.45	5.28	0.3776	0.94
0.50	5.22	0.4193	0.9406
0.55	5.15	0.4609	0.9414
0.60	5.09	0.5025	0.9421
0.65	5.01	0.5439	0.9430
0.70	4.94	0.5854	0.9438
0.75	4.86	0.6267	0.9447
0.80	4.78	0.6680	0.9456
0.85	4.69	0.7091	0.9467
0.90	4.60	0.7502	0.9477
0.95	4.50	0.7911	0.94886

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vi. FOR SOIL + 50% OF QUARRY DUST  
Pressure: 0.11 bars

Table 4.6 Load and Displacement table for soil + 50% QD

LOAD IN KN	DISPLACEMENT IN MM (-ve)	DEVIATOR STRESS (KN/M <sup>2</sup> )	AXIAL STRAIN
0.07	6.28	0.05932	0.9286
0.08	6.27	0.0677	0.92875
0.09	6.26	0.07625	0.92886
0.10	6.24	0.0847	0.9290
0.11	6.22	0.09316	0.9293
0.12	6.21	0.10162	0.9294
0.13	6.19	0.1100	0.9296
0.14	6.16	0.1185	0.93
0.15	6.14	0.1269	0.9302
0.16	6.12	0.1353	0.93045
0.17	6.10	0.1438	0.93068
0.18	6.08	0.1522	0.9309
0.19	6.06	0.1606	0.9311
0.20	6.03	0.1690	0.9314
0.21	6.00	0.1774	0.9318
0.22	5.98	0.1858	0.9320
0.23	5.95	0.1942	0.9323
0.24	5.90	0.2026	0.9329
0.25	5.85	0.21096	0.9335

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vii. PLAIN QUARRY DUST:  
Pressure: 0.14 bars

**Table 4.7 Load and Displacement table for plain QD**

LOAD IN KN	DISPLACEMENT IN MM (-ve)	DEVIATOR STRESS (KN/M <sup>2</sup> )	AXIAL STRAIN
0.00	7.36	0	0.9163
0.01	7.34	0.00856	0.9165
0.02	7.30	0.01712	0.9170
0.03	7.27	0.02567	0.9173
0.04	7.24	0.0342	0.9177
0.05	7.20	0.0427	0.9181
0.06	7.17	0.0513	0.9185
0.07	7.14	0.0598	0.9188
0.08	7.10	0.0683	0.9193
0.09	7.07	0.0768	0.9196
0.10	7.05	0.0854	0.9198
0.11	7.02	0.0939	0.9202
0.12	6.99	0.1024	0.9205
0.13	6.96	0.11092	0.9209
0.14	6.93	0.1194	0.9212
0.15	6.90	0.1279	0.9215
0.16	6.87	0.1363	0.9219
0.17	6.82	0.1448	0.9225
0.18	6.80	0.1533	0.9227

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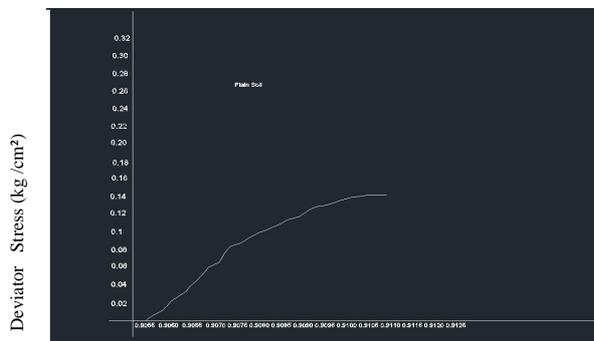
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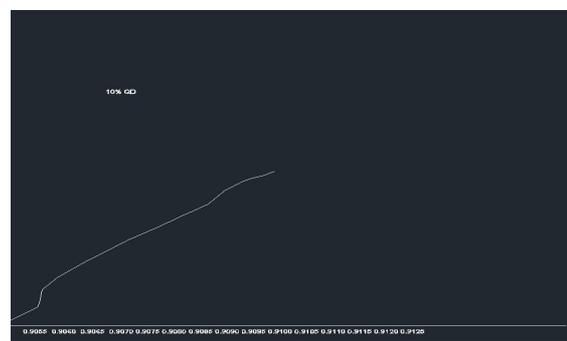
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## B. DEVIATOR STRESS VS AXIAL STRAIN CURVES:

The most common way of depicting the relation between stress and strain is through a **STRESS-STRAIN CURVE**. Stress-strain curves are obtained experimentally and provide useful material properties such as **Young's Modulus, yield strength, ultimate tensile strength**, etc.

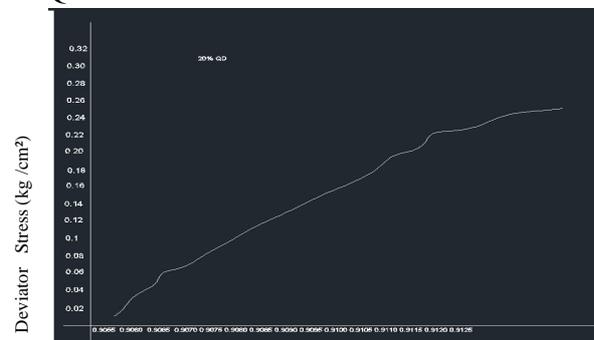


Axial strain Graph 4.1: For plain soil

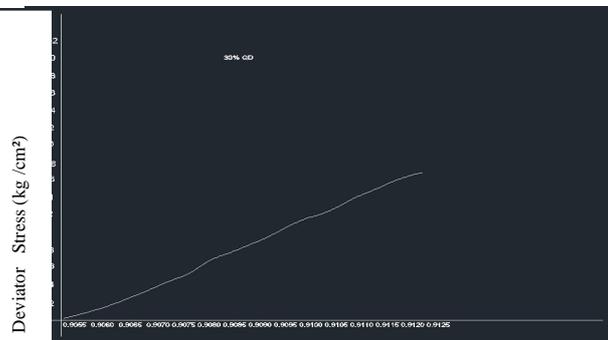


Axial strain Graph 4.2: For soil + 10% QD

10% QD

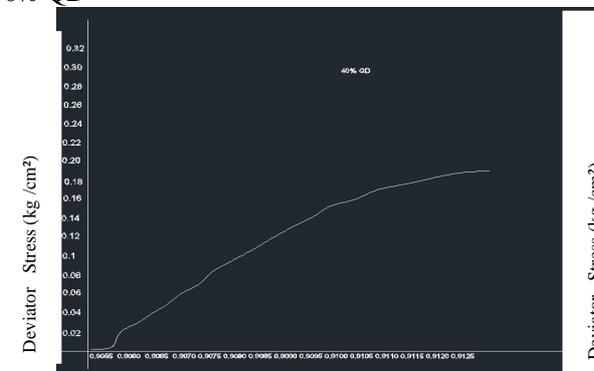


Axial strain Graph 4.3: For Soil + 20% QD

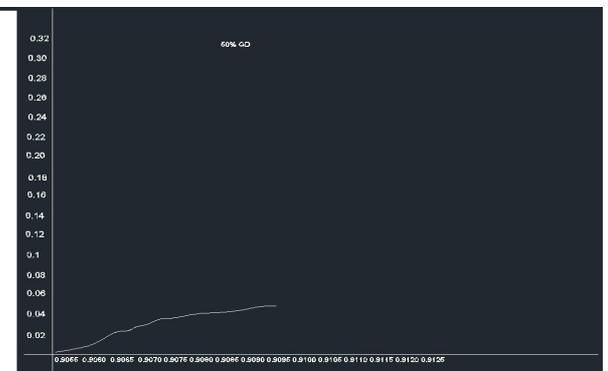


Axial strain Graph 4.4: soil + 30% QD

30% QD



Axial strain Graph 4.5: soil + 40% QD



Axial strain Graph 4.6: soil + 50% of QD

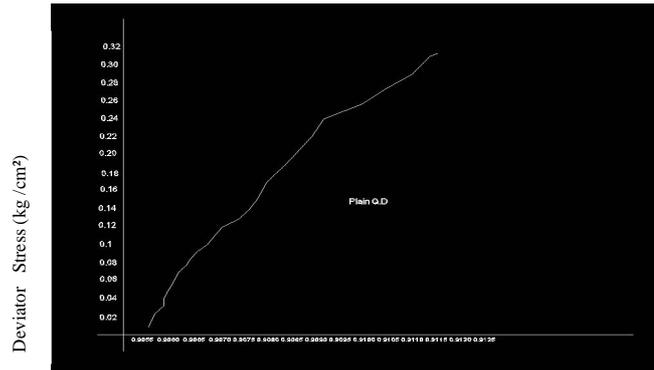
50% of QD

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Axial strain Graph 4.7: plain quarry dust

From the deviator stress versus axial strain curves (from the graph 4.1 to graph 4.7), the following points are observed:

- For same intensity of deviator stress (say 0.15 kg/cm<sup>2</sup>), the increase in the value of axial strain is observed with 50% mix of quarry dust.
- Quarry dust alone is insignificant in accepting the deviator stress to that extent.
- The behaviour of plain soil is such that there is a marginal increase in the value of poisson's ratio ( $\mu$ ).
- For 10% mix of quarry dust, elastic properties of the mixed soil is almost linear, within which there is a paradigm shift in the elastic properties of plain soil.
- Quarry dust has insignificant elastic properties and the range of variation is again a marginal for 10% mix.

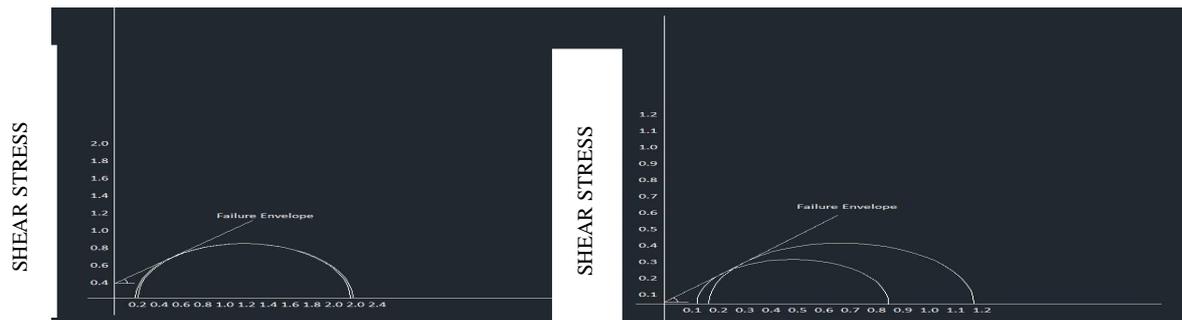
### D. FAILURE ENVELOPE GRAPHS:

After performing a stress analysis on a material body assumed as a continuum, the components of the Cauchy stress tensor at a particular material point are known with respect to a coordinate system. The Mohr circle is then used to determine graphically the stress components acting on a rotated coordinate system, i.e., acting on a differently oriented plane passing through that point. The Normal stress  $\sigma_n$  and shear stress  $\tau_n$  of each point on the circle, are the magnitudes of the normal stress and shear stress components, respectively, acting on the rotated coordinate system. In other words, the circle is the locus of points that represent the state of stress on individual planes at all their orientations, where the axes represent the principle axes of the stress element. Alternative graphical methods for the representation of the stress state at a point include the Lamé's stress ellipsoid and Cauchy's stress quadric. The Mohr circle can be applied to any symmetric 2x2 tensor matrix, including the strain and moment of inertia tensors. The Failure envelope graphs for plain soil and soil mixed with different volumetric proportions of quarry dust are drawn as below.



Normal stress Graph 4.8: For plain soil

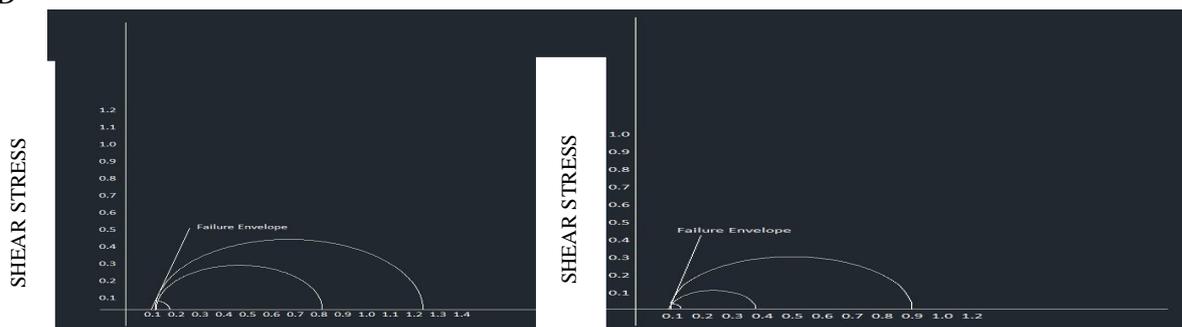
Normal stress Graph 4.9: For soil + 10% QD



Normal stress Graph 4.10: For soil + 20% QD

Normal stress Graph 4.11: For soil + 30%

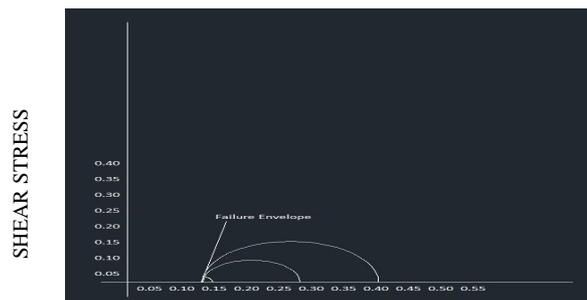
QD



Normal stress Graph 4.12: for soil + 40% QD

Normal stress graph 4.13: for soil +

50% QD



Normal stress graph 4.14: For Plain Quarry Dust

From the failure envelope graphs (from the Graph 4.8 to Graph 4.14), the following points are observed:

- There is an increase in the value of frictional angle ( $\phi$ ), as the dosage of quarry dust is increased.
- Though the raise in cohesion (C) is in significant, because of flocculent behaviour of quarry dust (plastic limit and liquid limit decreases with an increase in percentage of quarry dust), angle of shearing resistance is irresponsible for the steepness of stability envelope.
- Shear strength of the mixed soil is profound with the quarry dust mix of 40% and above. Whereas, with 10% of quarry dust mix there is a substantial reduction in the value of shear strength. This would be due to improper interlocking of the structural grain of quarry dust with soil solids.

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## V. CONCLUSION

### CONCLUSION

Quarry dust upon mixing with the soil, imparts plasticity to the mix. Initially (upto 20%), the quarry dust particles are flocculent in nature and becomes dispersive at 50% mix. Thereby, the quarry dust tends to impart a pseudo cohesion to the soil.

At higher dosages (beyond 40%)

- The quarry dust itself acts as a self-super plasticizer.
- Improper interlocking of particles of quarry dust with soil solids is evidenced.

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