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The Study about Stabilization of Soil with Waste Material

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Abstract— For satisfactory performance of a structure, its foundation must satisfy the following three basic criteria:

a) Location and depth criterion
b) Shear failure or bearing capacity criterion
c) Settlement criterion

The properties influencing the above cited criterions i.e. shear strength as well as compressibility can be improved by stabilizing the weaker soil deposits. Generally, soil stabilization has been adopted in various civil engineering works. Some important applications are in foundations, retaining structures, stability of slopes, underground structure, earth dam etc. Hence, in broad sense, we can say:

1. Soil stabilization is the process of the improving the engineering properties of soil and thus making it more stable.
2. It required when soil available for construction is not suitable for intended purpose.
3. 3. In broad, the soil stabilization includes compaction, pre-consolidation, drainage and many other processes.
4. A cementing material or a chemical is added to a natural soil for the purpose of stabilization.
5. Soil stabilization is used to reduce the permeability and compressibility of soil mass in earth structures and to increase its shear strength.

Key Words : Soil Stabilization, Liquid Limit, Plastic Limit, Shrinkage
Sub Area : Construction Technology & Management
Broad Area :**Civil Engineering**

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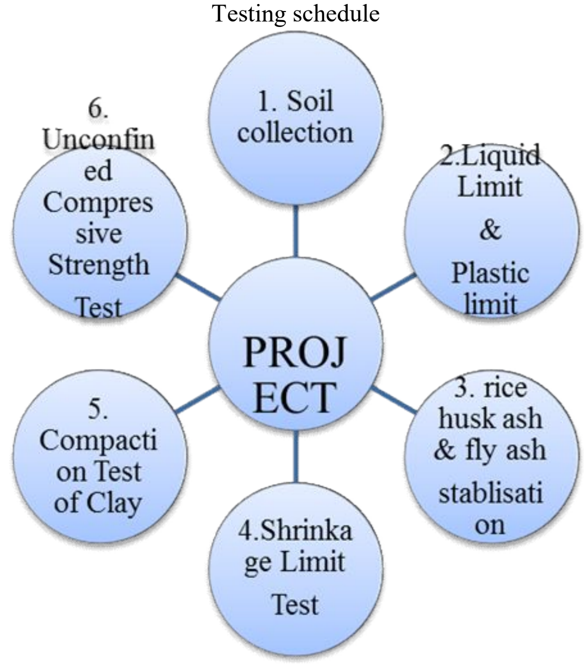
EXPERIMENTAL INVESTIGATIONS:

Testing schedule



Liquid limit and plastic limit test

The Liquid limit of fine grained soil is the water content at which soil behaves practically like a liquid, bit has small shear strength. If flow close the groove in just 25 blows in cassagrandes liquid limit device. It is one of the Atterbergs limit. The Atterbergs limits consist of liquid limit and shrinkage limit. As it difficult to get exactly 25 blows in the test. 3 to 4 tests are conducted, and the number of blows (N) required in each test determined. A semi-log plot is drawn between log N and the water content (w).
The Liquid limit is the water content corresponding to N = 25. This index property helps in classification.
The plastic limit of a fine-grained soil is the water content of the soil below which it ceases to be plastic. It begins to crumble when rolled in to threads of 3 mm diameter.

APPARATUS

1. Cassagrande's limit device
2. Grooving tools of both standard and ASTM types
3. Oven
4. Evaporating dish
5. Spatula
6. 425 micron IS sieve
7. Weighing balance with 0.01 g accuracy
8. Wash bottle
9. Air-tight and non-corrodible container for determination of content.

Figure1: Cassagrande Apparatus

Figure2: Spatula and Grooving tool

PREPARATION OF SAMPLE

2.3.1 Airs dry the soil sample and break the clods. Remove the organic matter like tree roots, pieces of bark etc.

2.3.2 About 100 g of the specimen passing through 150 micron IS sieve is mixed thoroughly with water in the evaporating dish and left for 24 hours for soaking.

PROCEDURE

1. A portion of cup placed in the cup of the Liquid limit device.
2. Leveled the mix so as to have a maximum depth of 1 cm.
3. Grooving tool drawn through the sample along the symmetrical axis of the cup, holding the tool perpendicular to the cup.
4. After soil pat has been cut by proper grooving tool, the handle is rotated at the rate of about 2 resolution per second and the nos. of blows counted till the two parts of the soil sample come into contact for about 10 mm length.
5. 10 g of soil taken near the closed groove & water contend determined.
6. The soil of the cup is transferred to the dish containing the soil paste and mixed thoroughly after adding a little more water. The test is repeated for 3 times.
7. The range of blows comes out only between 15 to 35 blows.
8. Liquid limit is determined by plotting a 'flow curve' on the semi-log graph between nos. of blows on logarithm scale and water content on arithmetical scale.
9. Water content corresponding to 25 blows is the value of liquid limit.
10. After this procedure for finding plastic limit 8 g of soil taken and roll it with fingers on a glass plate.
11. The rate of rolling shall between 80 to 90 strokes per minutes to form a 3 mm diameter.
12. The diameter of the soil not became less than 3 mm.
13. The plastic limit of the soil sample is nearly about zero.

OBSERVATION TABLE

Determination No.	1	2	3
(1).No of blows	30	22	13
(2).Container No.	3	2	16
(3).Mass of container + wet soil, (gm.)	32	35	58
(4).Mass of container + dry soil, (gm)	27	27	45
(5).Mass of water (3) –(4), (gm.)	5	8	13
(6).Mass of container, (gm.)	10	12	19
(7).Mass of dry soil (4)-(6), (gm.)	22	23	39
(8).Moisture content (5)/(7)*100, (%)	29.41	53.33	50

Table1: Liquid Limit Determination

SHRINKAGE LIMIT TEST

To Shrinkage limit is the water content of the soil when the water is just sufficient to fill the pores of the soil and the soil is just saturated. The volume of the soil does not decrease when the water content is reduced below the Shrinkage limit. It can be determined from the following relation –

$W = ((W_1-W_s) - (V_1-V_2)\ Y_w)/W_s)*100$

Where W_1 = Initial wet mass, W_s = Dry mass

V_1 = Initial volume, V_2 = Volume after drying

APPARATUS

1. Shrinkage dish, having a flat bottom, 45 mm diameter and 15 mm height.
2. Two large evaporating dishes about 120 mm diameters, with a pour out and flat bottom.
3. One small mercury dish, 60 mm diameter.
4. Two glass plates, one with prongs, 75*75*3 mm size.
5. Glass cup, 50 mm diameter and 25 mm height.
6. IS sieve 150 micron.
7. Oven.
8. Desiccators.
9. Weighing balance, accuracy 0.01 g.
10. Spatula
11. Straight edge mercury.

Figure3 : Shrinkage test apparatus

Figure4 : Shrinkage dish having wet soil pat

PROCEDURE

1. A shrinkage dish is taken.
2. The Wt. of empty dish is 40 gm is recorded.
3. A soil sample is taken and water mixed into the soil.
4. Wt. of wet soil with dish is 80 gm is recorded. It placed into oven for drying for a period of 24 hours.
5. After drying the Wt. of dry pat with dish is 63 is recorded.
6. After this procedure a weighing dish Wt. is recorded i.e. 40 gm.
7. Mercury is placed in the weighing dish. Wt. of mercury with weighing dish is 375 gm.
8. After this the dry soil pat is placed into the weighing dish, mercury is displaced through weighing dish.
9. The wt of weighing dish with mercury displaced is 335 gm.

OBSERVATION TABLE

Determination No.	I
(1). Shrinkage dish No.	1
(2). Mass of dish + wet soil pat, (gm.)	80
(3). Mass of dish + dry soil pat, (gm.)	63
(4). Mass of water, (2) - (3), (gm.)	17
(5). Mass of shrinkage dish empty, (gm.)	40
(6). Mass of dry soil pat (Ws) = (3) – (5), (gm.)	23
(7). Initial water content (w1) = (4) / (5)*100, (%)	42.5
(8). Mass of weighing dish + mercury (filling shrinkage dish), (gm.)	375
(9). Mass of weighing dish empty, (gm)	40
(10) .Mass of mercury (8) –(9), (gm.)	335
(11). Vol. wet soil pat (V1) = (10)/13.6, (cc.)	24.63
(12). Mass of weighing dish + displaced mercury (by dry pat), (gm.)	330
(13). Mass of mercury displaced (12) – (9), (gm.)	218
(14). Vol. dry pat (V2) = (13)/13.6, (cc.)	16.03

Table 2: Shrinkage factor determination

Similarly this work is done by two stabilizer Rice Husk Ash & Fly-Ash and then find the Shrinkage limit.

UNCONFINED COMPRESSIVE STRENGTH TEST

The purpose of this laboratory is to determine the unconfined compressive strength of a cohesive soil sample. We will measure this with the unconfined compression test, which is an unconsolidated undrained (UU or Q-type) test where the lateral confining pressure is equal to zero (atmospheric pressure).

To perform an unconfined compression test, the sample is extruded from the sampling tube. A cylindrical sample of soil is trimmed such that the ends are reasonably smooth and the length-to-diameter ratio is on the order of two. The soil sample is placed in a loading frame on a metal plate; by turning a crank, the operator raises the level of the bottom plate. The top of the soil sample is restrained by the top plate, which is attached to a calibrated proving ring. As the bottom plate is raised, an axial load is applied to the sample. The operator turns the crank at a specified rate so that there is constant strain rate. The load is gradually increased to shear the sample, and readings are taken periodically of the force applied to the sample and the resulting deformation. The loading is continued until the soil develops an obvious shearing plane or the deformations become excessive. The measured data are used to determine the strength of the soil specimen and the stress-strain characteristics. Finally, the sample is oven dried to determine its water content. The maximum load per unit area is defined as the unconfined compressive strength, qu.

In the unconfined compression test, we assume that no pore water is lost from the sample during set-up or during the shearing process. A saturated sample will thus remain saturated during the test with no change in the sample volume, water content, or void ratio. More significantly, the sample is held together by an effective confining stress that results from negative pore water pressures (generated by menisci forming between particles on the sample surface). Pore pressures are not measured in an unconfined compression test; consequently, the effective stress is unknown. Hence, the undrained shear strength measured in an unconfined test is expressed in terms of the total stress.

APPARATUS

The loading frame consists of two metal plates. The top plate is stationary and is attached to the load-measuring device. The bottom plate is raised and lowered by means of a

Crank on the front of the loading frame. After the soil sample has been placed between the plates, the bottom plate is gradually raised; the resistance provided by the stationary top plate applies an axial force to the sample. Although the loading frames in our laboratory are hand operated, electric motor-driven and hydraulic load frames are common. Loads are measured with a calibrated proving ring or an electronic load cell. Vertical deformations are measured with a dial gauge; the dial gauge is attached to the top plate and measures the relative movement between the top and bottom plates. We will be performing a strain-controlled test, in which the load is applied at a constant rate of strain or deformation

PROCEDURE

1. The first step in the procedure is to examine the loading frame. Turn the crank and learn how to read the load and deformation dial gages. Determine the calibration constant for the proving ring and the units of the deformation dial gauge.
2. We will be shearing the samples at a strain rate of 1% per minute. From the length of your soil sample, determine the deformation at 1% strain. Depending on the units of the vertical deformation dial gauge (usually 0.001 inches or 0.0001 inches), determine the number of dial divisions per 1 strain- Practice turning the crank at his number of dial divisions/minute. It is important that the soil sample not be sheared faster than this specified rate
3. Measure the initial height and diameter of the soil sample with calipers. It is unlikely that the sample will be a perfect right cylinder. Therefore, it will be necessary to find the average height and diameter by taking several measurements in different places along the soil sample. The measurements should be taken by more than one member of a lab team to be sure that the calipers are read correctly. If you have any questions about how to take measurements with calipers, ask the laboratory instructor for instruction.
4. Record the weight of the soil sample and determine the total (moist) unit weight.
5. Place the soil sample in the loading frame, seat the proving ring and zero the dials.
6. During this lab you will record the load applied at specified strain values. It is recommended that readings be taken at strains of 0,0.1,0.2, 0.5, 1,2,3,4,5,6,8, 10, 12 14, 16, 18 and 20 percent. You should prerecord the vertical deformation dial readings at these strain values. With the measured initial height of sample (H0), the desired percent strain (ε) and the initial dial reading (S0), calculate the dial readings (S)
7. Readings of force (F) are taken from the proving ring dial gauge and the stress applied to the ends of the sample (01, or major principal stress) is computed as follows: where A is the cross-sectional area of the sample. Because the soil sample height decreases during shear and the volume of the sample remains constant, the cross sectional area must increase. For a saturated soil that undergoes no volume change during shear (no flow of water into or out of the sample), the equivalent or average area (A) at any strain (e) is computed from the initial area (Ac) and the assumption that volume is conserved:
8. Shear the sample at a strain rate of 1% per minute. Typically, the sample fails in one of two ways. In stiffer clays, a distinct failure plane forms. For this type of failure, it is likely that the point of failure will be indicated by the measurement of a peak and then a decrease in load. If this is the case, continue to take four or five readings past the point of failure. (Caution: before you stop shearing the sample, be sure that the sample has failed.) A "barreling" failure is more typical for softer clays. In this type of failure, a distinct failure plane doesn't form, rather the sample bulges in the middle the unconfined compressive strength (qu) is the maximum value σ1, which may or may not coincide with the maximum force measurement (depending on the area correction). It is also equal to the diameter of Mohr's circle as indicated in Fig. 1. The untrained shear strength (su) is typically taken as the maximum shear stress, or: If σ1 continues to increase up until 20 vertical strain, i.e. does not reach a maximum and then decrease, the sample has failed by "barreling". In this case, qu is defined as the value of σ1 measured at 20% strain.
9. When your lab team has completed the experiment, dismantle the loading frame and measure the water content of the soil sample. It is recommended that you reduce the data for this test during the lab period.

UNCONFINED COMPRESSION TEST

OBSERSERVATION AND CALCULATIONS

ONLY WITH CLAY

Type of specimen – Undisturbed/Remoulde = Remoulde

Least count of deformation dial gauge (mm/divn.) = 0.01

Proving ring constant (N/divn.) = 20

Table24: unconfined processed data

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S.No.	**Elapsed Time (sec.)**	**Deformation (delta L)**		**strain**	**Corrected area**	**Load**		**Compress-ive stress ,q (N/cm*cm)**
		(divn.)	(m.m)			(divn)	(N)	
(1)	(2)	(3)	(4)=(3) *L.C.	(5)	(6)	(7)	(8)= (7)*(iii)	(9)=(8)/(6)
1.	30	21	0.21	0.00276	11.373	0.4	08	0.703
2	30	32	0.32	0.00421	11.389	0.68	13.6	1.194
3	30	45	0.45	0.00592	11.409	0.92	18.4	1.613
4	30	56	0.56	0.00737	11.425	1.16	23.2	2.031
5	30	80	0.80	0.01052	11.462	1.40	28	2.443
6	30	109	1.09	0.01434	11.506	1.62	32.4	2.816
7	30	141	1.41	0.01855	11.556	1.78	35.6	3.080
8	30	176	1.76	0.02316	11.610	1.90	38	3.273
9	30	218	2.18	0.02868	11.676	1.96	39.2	3.357

UNCONFINED COMPRESSION TEST
OBSERSERVATION AND CALCULATIONS
CLAY + 5% RICE HUSK ASH
Type of specimen – Undisturbed/Remoulded = Remoulded
Least count of deformation dial gauge (mm/divn.) = 0.01
Proving ring constant (N/divn.) = 20Table25:

UNCONFINED COMPRESSION DATA

UNCONFINED COMPRESSION TEST
OBSERSERVATION AND CALCULATIONS
CLAY + 10% RICE HUSK ASH
Ttype of specimen – Undisturbed/Remoulde = Remoulded
Least count of deformation dial gauge (mm/divn.) = 0.01
Proving ring constant (N/divn.) = 20

Table26: UNCONFINED COMPRESSION DATA

S.No.	Elapsed Time (sec.)	Deformation (delta L)		strain	Corrected area	Load		Compress-ive stress ,q (N/cm*cm)
		(divn.)	(m.m)			(divn)	(N)	
(1)	(2)	(3)	(4)=(3) *L.C.	(5)	(6)	(7)	(8)= (7)*(iii)	(9)=(8)/(6)
1.	30	0	0	0.00000	11.34	0.1	2	0.17637
2.	30	0	0	0.00000	11.34	0.2	4	0.35273
3.	30	10	0.1	0.00131	11.35	0.2	4	0.35398
4.	30	12	0.12	0.00157	11.36	0.3	6	0.52863
5.	30	15	0.15	0.00197	11.36	0.4	8	0.70422
6.	30	15	0.15	0.00197	11.36	0.5	10	0.88028
7.	30	18	0.18	0.00237	11.37	0.6	12	1.05541
8.	30	21	0.21	0.00276	11.37	0.7	14	1.23131
9.	30	36	0.36	0.00473	11.39	0.8	16	1.40474
10.	30	52	0.52	0.00684	11.42	0.9	18	1.57618
11.	30	70	0.70	0.00921	11.45	1.0	20	1.74672
12.	30	90	0.90	0.01184	11.48	1.1	22	1.91637
13.	30	111	1.11	0.01461	11.51	1.2	24	2.08514
14.	30	138	1.38	0.01815	11.55	1.4	26	2.25108
15.	30	160	1.60	0.02105	11.59	1.4	28	2.41588
16.	30	188	1.88	0.02473	11.63	1.4	28	2.40757
17.	30	208	2.08	0.02736	11.66	1.4	28	2.40137

CONCLUSION

The study demonstrates the influence of rice husk ash and fly ash on the shrinkage and strength characteristics of highly compressible locally available clay. The following conclusions have been drawn based on the laboratory investigations carried out in this study:

1. Values of optimum moisture content (OMC) and maximum dry density (MDD) for parent clay were found to be 14.71% and 1.93 g/cc. It was observed that with increase in percentage of rice husk ash as stabilizer the value of OMC increases from 14.71% to 32.5% and value of MDD decreases from 1.93 g/cc to 1.54 g/cc. On the other hand, when fly ash was mechanically mixed with parent clay, no significant changes were observed in the values of OMC and MDD. Generally OMC increases and MDD decreases in case of fly ash stabilized clay samples. This significant increase in optimum moisture content can be attributed to water absorbing tendency of rice husk ash that too present in fly ash in small quantity. Decrease in MDD of stabilized samples can be attributed to the addition of a material of low specific gravity to parent clay.

2. A series of unconfined compressive strength tests were conducted to determine the strength characteristics of parent clay treated with various percentages of pozzolanic wastes as per specifications of IS: 2720 (Part 10) (1973).It was observed that shear strength of clayey soil stabilized with rice husk ash varies from 3.36 N/Cm^2 to 3.85 N/Cm^2. While shear

strength of clayey soil stabilized with fly ash varies from 3.36 N/Cm^2 to 3.65 N/Cm^2. Stress – strain relationships for clay and clay stabilized with various percentages of pozzolanic wastes are shown in Graph 14 to Graph 24 below.

3. A considerable decrease in values of shrinkage limit was observed when soil was stabilized with fly ash and rice husk ash.

1. Settlement criteria analysis by consolidation test
2. Strength criteria by Triaxial Shear test.
3. If bearing capacity is very low and settlement increases, the locally available soil can be stabilized and their Geotechnical property can be improved.
4. The above Stated Stabilized Soil can be tested for settlement as well as bearing capacity. Also the optimum % of stabilization can be calculated.

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