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EXPERIMENTAL STUDY ON BEHAVIOUR OF STONE COLUMN ON SOFT SOIL WITH AND WITHOUT GEOSYNTHETICS

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ABSTRACT
The fine grained soils such as soft clays, loose silts and most organic soils are characterized by low undrained shear strength, low bearing capacity, high compressibility and low permeability. These lead to soil stability problems when construction work is taken on such type of soil. On account of poor permeability of the soil, the dissipation of pore water is very slow. It takes a long time for the soil to consolidate and stabilize on its own, thereby delaying the construction activities on such soils. In such situations, soil needs stabilization before construction. Hence suitable ground improvement method has to be adopted to improve the engineering properties of soil. This paper presents the utilization of stone column to improve the load capacity of soft soil with clay in naturally consolidated state. Load tests through the compression testing machine are performed on single un-encased stone column in sandy silt soil with. (Red soil 50%+ Kaolin clay 50%) Un-encased and encased (with geotextile) stone column behavior on layered soil also discussed in this investigation. In case of unencased stone column load carrying capacity increases with the increasing diameter of the stone column but in un-encased and encased layered soil load carrying capacity decreases with the increasing diameter of the stone column. The load bearing capacity of stone column in un-encased and encased layered soil decreases with the increasing diameter of stone column.

INTRODUCTION
Stone columns have been increasingly used for improvement of soft soils to increase the load bearing capacity and to reduce the settlements. Improvement of soft clay deposits by the Installation of stone columns is one of the most popular techniques followed worldwide. The stone columns not only act as reinforcing material increasing the overall strength and stiffness of the compressible soft soil, but also they promote consolidation through effective drainage. Potential applications include stabilizing foundation soils to support embankments and approach fills, supporting retaining structures (including reinforced earth), bridge bent and abutment structures on slightly marginal soft to stiff clays and loose silty sands, landslide stabilization and reducing liquefaction potential of clean sands. Also, stone columns under proper conditions can greatly decrease the time required for primary consolidation. This ground improvement technique has been successfully applied for the foundations of structures like oil storage tanks, earthen embankments, raft foundations etc where large settlement is possible.

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METHODOLOGY

DESIGN OF STONE COLUMNS

The design of stone columns was carried out by the following two methods.

A) AS PER IS 15284 (PART 1) 2003

1 Estimation of Load Capacity of Stone Column

Capacity Based on Bulging of Column

The capacity based on bulging of column is calculated as given in equation.

Q_1 = Yield Load / Factor of safety

$Q_1 = \sigma\pi D/4)/2$

Surcharge effect Due to surcharge effect, increase in safe load of the column is calculated as per equation.

$Q_2 = k_{pcol}\Delta 6_{ro} A_s/2$

Bearing Support Provided by the Intervening Soil Due to the effect of intervening soil's the safe load is calculated as per the equation.

$Q_3 = q_{safe}\, A_g$

B) DESIGN OF STONE COLUMNS BY PRIEBE'S METHOD

The design of stone columns by Priebe's method, mainly involves the following steps:

(a) Determination of basic improvement factor , (b) determination of the improvement factors by considering, column compressibility and overburden pressure (c) finding the compatibility controls and (d) determination of the shear values of improved ground.

EXPERIMENTAL INVESTIGATIONS

3.1 MATERIALS

The materials used for this study are locally available soil, kaolin clay, aggregates, stone dust, river sand.

- **LOCALLY AVAILABLE SOIL**

The soil used is of CH classification excavated from the nearby area in Airoli sector-3. Surface soil was excavated after removal of vegetation, air-dried and pulverized. The soil was sieved through 4.75 mm sieve to remove the coarser fraction.

- **KAOLIN CLAY**

The clay used in the testing programme is a Kaolin clay with a liquid limit of 67% and a plasticity index 34%. It is obtained from the Gujarat clay mills Pvt. Ltd., Bhiwandi.

- **AGGREGATES**

Stone aggregates of size 6mm-10mm have been used to form stone column. It is obtained from the construction site in Airoli sector-3.

- **STONE DUST**

Crushed stone of size less than 4.75mm have been used for stone columns. It is obtained from the nearby supplier in Airoli.

- **RIVER SAND**

The sand used is clean river sand of a size less than 4.75mm. The properties of this are to be determined. It is obtained from the supplier in Airoli.

INDEX PROPERTIES OF SOIL

Properties	Soil (Red soil 50%+Kaolin clay 50%)
Liquid limit (%)	62
Plastic limit (%)	33
Plasticity index (%)	29
Max. dry density (g/cc)	1.6
Optimum moisture content(%)	19
Soil classification as per IS	CH

MAIN EXPERIMENTAL PROGRAMME

The following are the tests which are to be conducted on soil with 50% soil and 50% kaolin clay(with SC and ESC) to study the effect of stone column individually and with group of three columns with geosynthetics:-

1. Unconfined compressive strength test
2. Consolidation test
3. California bearing ratio test

DETERMINATION OF UNCONFINED COMPRESSIVE STRENGTH

1 SCOPE

This describes the method for determining the unconfined compres- sive strength of clayey soil, undisturbed, remolded or compacted, using controlled rate of strain.

2 TERMINOLOGY

For the purpose of this standard, the following definitions shall apply. Unconfined Compressive Strength, qu:- It is the load per unit area at which an unconfined cylindrical specimen of soil will fail in the .axial compression test.

3 PROCEDURE

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

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

The specimen shall be compressed until failure surfaces have definitely developed, or the stress-strain curve is well past its peak, or until an axial strain of 20 percent is reached.

The failure pattern shall be sketched carefully and shown on the data sheet or on the sheet presenting the stress-strain plot. The angle be- tween the failure surface and the horizontal may be measured, if possible, and reported.

DETERMINATION OF CONSOLIDATION PROPERTIES OF SOIL

1 SCOPE

1.1 This standard covers the method for conducting one dimensional consolidation test using either fixed or the floating ring for determining the consolidation characteristics of soil.

2 PROCEDURE

PREPARATION OF TEST SPECIMEN

Weigh the empty consolidation ring, designated W1.

If the specimen is to be prepared from a tube sample, a representative sample for testing shall be extruded and cut off, care being taken to ensure that the two plane faces of the resulting soil disc are parallel to each other. The thickness of the disc of soil shall be somewhat greater than the height of the consolidation ring.

If the specimen is to be prepared from a block sample, a disc similar in size to that specified above shall be cut from the block, with two parallel faces. The diameter of the disc shall beat least 10 mm greater than the inside diameter of the consolidation ring. Care shall be taken to ensure that the soil stratum is oriented such that the laboratory test will load the soil in the same direction relative to the stratum as the applied force in the field.

Using the weighed consolidation ring as a template, the edges of the disc obtained in shall be trimmed carefully until the ring just slides over the soil. The last fraction of soil is pared away by the cutting edge of the ring as it is pushed down slowly and evenly over the sample, with no unnatural voids against the inner face of the ring; this process is best done using a mechanical guide to prevent tilting or horizontal move- ment of the ring. The top and bottom surfaces shall project above and below the edges of the ring to enable final trimming. An alternate procedure is described in

Should an occasional small inclusion interfere with the trimming operation, it shall be removed, and the cavity filled completely with material from the parings. Alternatively, if sufficient sample is available, it would be preferable to eventually extrude and discard the portion of the specimen containing the inclusion from the ring, leaving a specimen free of such disturbed zones. If inclusions are known to exist in a soil sample, a large diameter consolidation ring should be used, in order to minimize the relative effect of the disturbed zones. If excessive inclusions are encountered during trimming, the sample should be discarded. If no alter- native exists, the tube sample shall be extruded directly into a consolidation ring of equal diameter.

An alternative procedure for obtaining a specimen from a soil disc as obtained in 3.1.2 is to use the consolidation ring as sampling device. The ring should be gradually inserted into the sample by pressing with hands and carefully removing the material and the ring. This can also be accomplished using a mechanically operated jig.

The soil sample thus obtained according to Above Statement shall be trimmed flush with the top and bottom edges of the ring. For soft to medium soils, excess soil should be removed using a wire saw, and final trimming may be done with a straight edge if necessary. For stiff soils, a straight edge alone may be used for trimming. Excessive remoulding of the soil surface by the straight edge should be avoided. In the case of very soft soils, special care should be taken so that the specimen may not fall out of, or slide inside the ring during trimming.

A sample of soil similar to that in the ring, taken from the trimmings, shall be used for determining moisture content.

The thickness of the specimen (H0) shall be measured and it shall be weighed immediately (W,). Should the nature of the soil make satisfactory thickness determination difficult, the ring height may be assumed as specimen height.

Assembly of Apparatus

The bottom porous stone shall be centered on the base of the consolidation cell. If over consolidated clay, or soils sensitive to moisture increase (swelling or collapsing soils) are being tested, the stone should be placed dry. When testing softer, normally consolidated clays, the stone should be wet, and it may be covered by a wet filter paper. No filter paper shall be used for the stiffer and moisture sensitive soils.

The ring and specimen shall be placed centrally on the bottom porous stone, and the upper porous stone and then the loading cap shall be placed on top. The top stone shall be placed dry or wet, and with or without filter paper, in accordance with 3.2.1.

The consolidometer shall be placed in position in the loading device and suitably adjusted. The dial gauge is then clamped into position for recording the relative movement between the base of the consolidation cell and the loading cap. A seating pressure of 0.05 kgf/cm^2 shall be applied to the specimen.

The consolidation cell shall be filled with water, preferably of the same ionic content as the specimen pore water. If this is no& possible, distilled water shall be used. The type of water used shall be noted in the data sheet.

The specimen shall then be allowed to reach equilibrium for 24 h.

LOADING

For consolidation testing, it is generally desirable that the applied pressure at any loading stage be double than that at the preceding stage. The test may, therefore, be continued using a loading sequence which would successively apply stress of 0.1, 0.2, 0.4, 0.8, 1.6, 3.2, etc, kgf/cm^2 on the soil specimen.

For each loading increment, after application of load, readings of the dial gauge shall be taken using a time sequence such as 0,0.25, 1, 2.25, 4, 6.25, 9, 12.25, 16, 20, 25, 36, 49, 64, 81, 100, 121, 144, 169, 196, 225, etc, min, up to 24 h or 0,1, 2, 4, 8, 15, 30, 60 min, and 2, 4, 8, 24 h. These time sequences facilitate plotting of thickness or change of thickness of specimen against square root of time or against logarithm of time.

The loading increment shall be left at least until the slope of the characteristic linear secondary compression portion of the thickness versus log time plot is apparent, or until the end of primary consolidation is indicated on a square root' of time plot. A period of 24 h will usually be sufficient, but longer times may be required. If a period of 24 h is seen to be sufficient, it is recommended that this commonly used load period be used for all load increments. In every case, the same load increment duration shall bc used for all load increments during a consolidation test.

It is desirable that the final pressure be of the order of at least four times the pre-consolidation pressure, and be greater than the maxi- mum effective vertical pressure which will occur in situ due to the overburden and the proposed construction.

On completion of the final loading stage, the specimen shall be unloaded by pressure decrements which decrease the load to one-fourth of the last load. Dial gauge readings may be taken as necessary during each stage of unloading. If desired, the time intervals used during the consolidation increments may be adopted; usually it is possible to proceed much more rapidly.

In order to minimize swell during disassembly, the last unloading stage should be to 0.05 kgf/cm2 which should remain on the specimen for 24 h. On completion of this decrement, the water shall be siphoned out of the cell and the consolidometer shall be rapidly dismantled after the release of the final load. The specimen, preferably within the ring, shall be wiped free of water, weighed (W3), and thereafter placed in the oven for drying. If the ring is required for further testing, the specimen may carefully be removed from the ring in order to prevent loss of soil, and then weighed and dried.

Following drying, the specimen (plus ring) shall be reweighed (w,).

The porous stones shall be boiled clean after the test, in order to prevent clay from drying on them and reducing their permeability.

DETERMINATION OF CALIFORNIA BEARING RATIO

1 SCOPE

1.1 It covers the laboratory method for determination of California Bearing Ratio (CBR).

2 PROCEDURE

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

RESULTS AND DISCUSSIONS

UNCONFINED COMPRESSIVE STRENGTH TEST

The testing is carried out on soil sample with single stone column (20 mm dia.) and group of three stone columns with and without encasement of woven geotextiles. The unconfined compression test results for samples of size 100 mm diameter and 155 mm height are discussed below.
The following table shows the undrained cohesion strength qu (kg/cm^2) of samples:

Table:-6.1

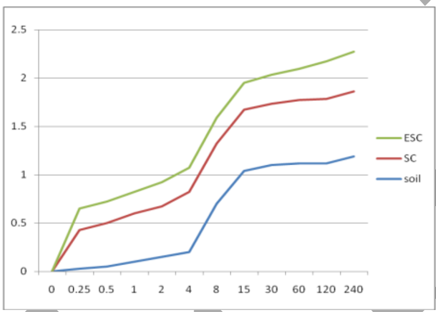
Samples	**Soil only**	**SC**	**GSC**	**ESC**	**EGSC**
!00 mm loading area	4.45	5.46	6.75	7.5	7.25
20 mm loading area	_	0.32	_	0.5	_
75 mm loading area	_	_	5.19	_	5.4

It is observed from the table 6.1 that construction of stone column in the soil has increased the strength i.e load carrying capacity. The load carrying capacity of soil with single stone column is more as compared to soil sample only. Also load carrying capacity of group of stone columns is more than that of soil with single stone column. The test is also conducted on above mentioned samples with loading area 20mm dia. on soil with single stone column and 75 mm dia. on soil with group of stone columns with and without encasement of geotextile. It is also concluded that load carrying capacity of single stone column is less as compared to group effect. Encasement of geotextile on stone column provides confinement to stone column thus helps in increasing load carrying capacity to some extent in addition to drainage, separation and filtration.

CONSOLIDATION TEST

The results for tests carried out on samples of soil, soil with single stone column with and without encasement of geotextiles as shown in graphs attached.

From the mentioned graphs plotted between time versus dial gauge reading, it has been clearly observed that the soil with single stone column gets consolidated at early stage. Thus the rate of settlement of soil with stone column with and without encasement of geotextile is high as compared to that of soil sample only. Thus, stone column will act like a sand drains which will help in vertical as well as radial drainage an geotextile will help in drainage, filtration, separation and reinforcement for soil.



PLOT DGR vs TIME

CALIFORNIA BEARING RATIO TEST

The results foe tests carried out on sample of dia. 150 mm in standard CBR mould are indicated in graphs shown below.

The following results are obtained from the test carried out on soil sample with and without encasement.

Table:- 6.3

Sample (Unsoaked)	Normal loading		Loading area 20 mm		Loading area 75 mm	
Penetration	2.5mm	5 mm	2.5mm	5 mm	2.5mm	5 mm
Soil	205.4 (15 %)	290.72 (14 %)	—		—	
SC	403.0 (29.4%)	526.5 (25.6%)	84.5 (6.1%)	117.00 (5.6%)	—	
GSC	568.78 (41.5%)	715 (34.8%)	—		123.5 (9%)	156.00 (7.6%)
ESC	409.00 (29.8%)	534 (25.9%)	92.0 (6.6%)	128.5 (6.2%)	—	
EGSC	570.50 (42%)	715 (34.8%)	—		130.0 (9.5%)	162.0 (7.9%)

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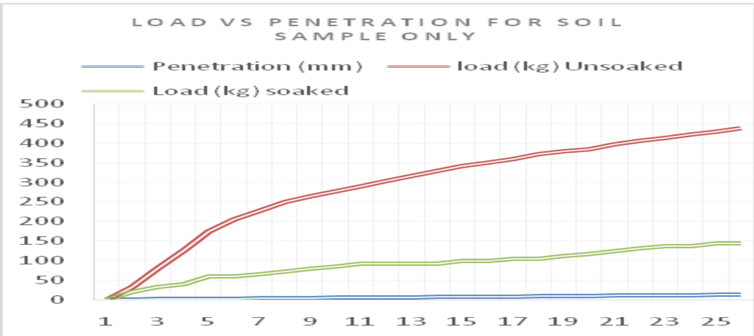
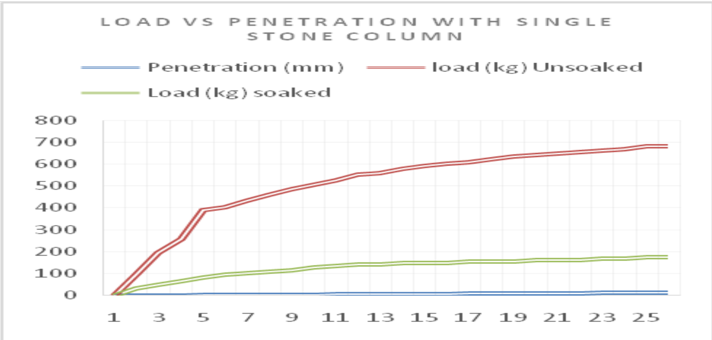
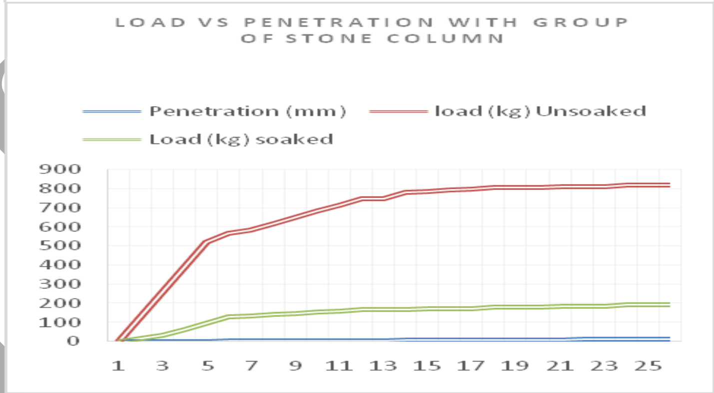
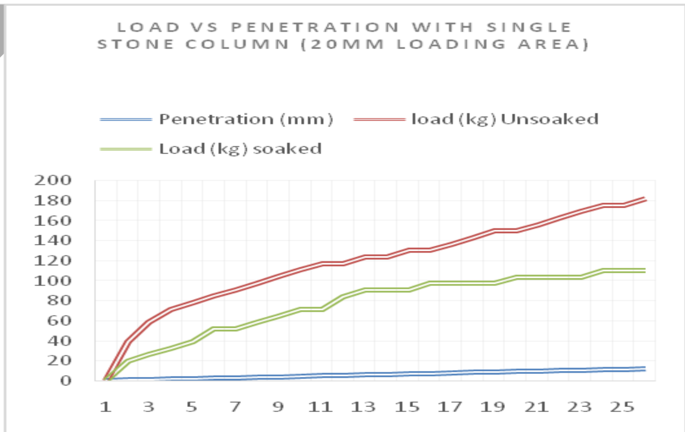
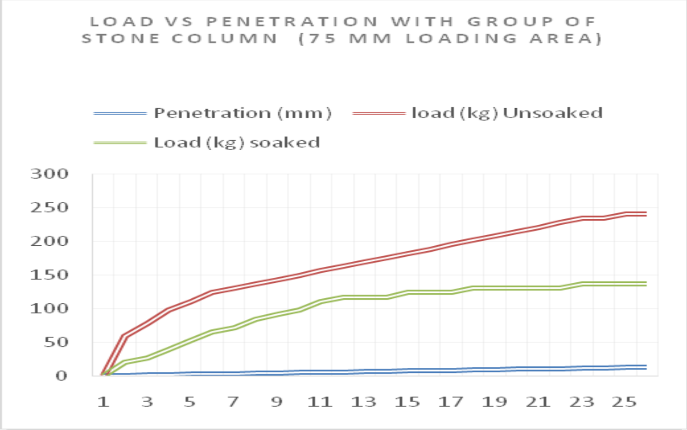
The test is carried out on soil, soil with single stone column and group of three stone columns along with 20 mm dia. loading area and 75 mm dia. loading area on group of stone columns for both soaked and unsoaked conditions. Soaking is done for 4 days. The CBR values has been increased due to construction of stone column with and without encasement of geotextile. This also indicates that the improved soil can carry greater load with lesser penetration as compared to soil sample only. The soil with CBR less than 10 will require improvement of this sort which helps in increasing the CBR value os soil, thus making it suitable for construction purpose.

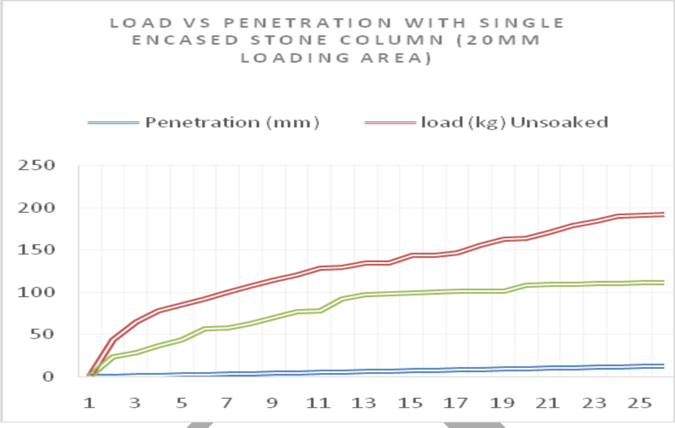


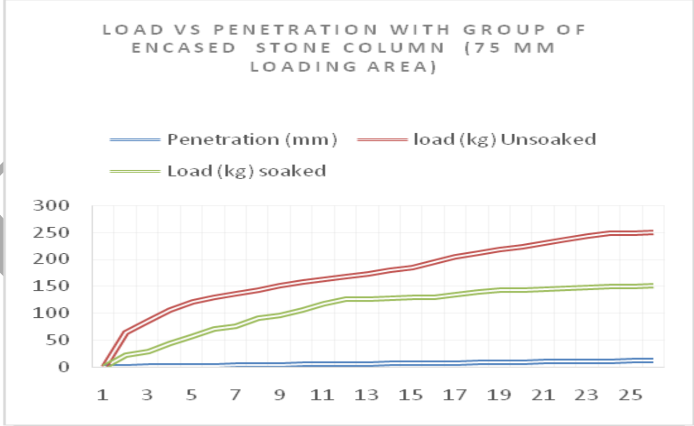


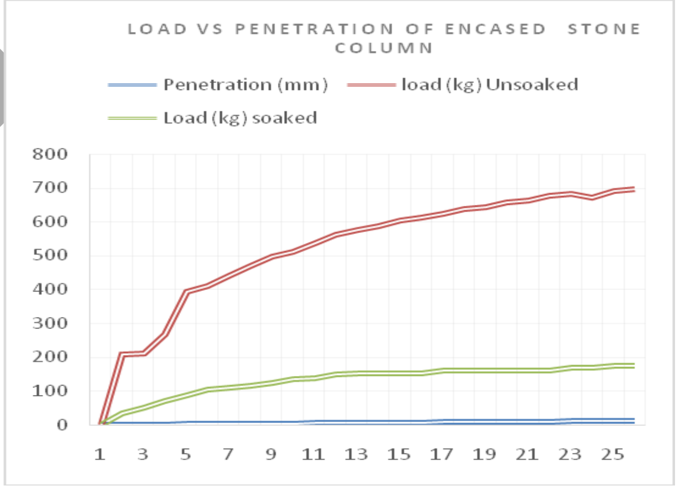


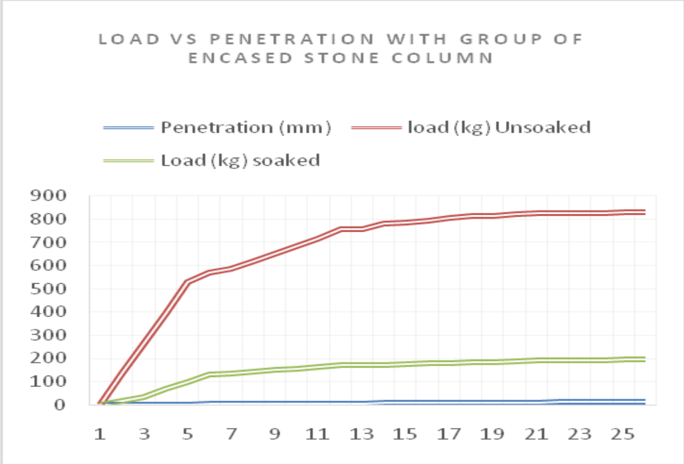



LOAD VS PENETRATION WITH GROUP OF STONE COLUMN (75 MM LOADING AREA)
Penetration (mm)
load (kg) Unsoaked
Load (kg) soaked
300
250
200
150
100
50
0
1 3 5 7 9 11 13 15 17 19 21 23 25


LOAD VS PENETRATION WITH SINGLE ENCASED STONE COLUMN (20MM LOADING AREA)
Penetration (mm)
load (kg) Unsoaked
250
200
150
100
50
0
1 3 5 7 9 11 13 15 17 19 21 23 25


LOAD VS PENETRATION WITH GROUP OF ENCASED STONE COLUMN (75 MM LOADING AREA)
Penetration (mm)
load (kg) Unsoaked
Load (kg) soaked
300
250
200
150
100
50
0
1 3 5 7 9 11 13 15 17 19 21 23 25


LOAD VS PENETRATION OF ENCASED STONE COLUMN
Penetration (mm)
load (kg) Unsoaked
Load (kg) soaked
800
700
600
500
400
300
200
100
0
1 3 5 7 9 11 13 15 17 19 21 23 25



CONCLUSION

The results of the testing programme give some important insight into the performance of the ESCs. The tends obtained in the laboratory tests are in good agreement with the results reported in literature.
The major conclusions that can be drawn from this research work are as follows:-

- The load carrying capacity and stiffness of the column can be increased by all-round encasement by geosynthetic. The encasement also exhibits better and stronger response.
- The benefit of encasement decreased with increase in diameter of stone column due to reduction of hoop tension in encasement.
- The performance of ESCs of smaller diameter is superior to that of larger diameter stone columns because of mobilization of higher confining stress in smallest diameter stone columns.
- The load versus response gets improved due to introduction of stone column. The rate of settlement of soil with stone column increases and encasement will help in drainage and filtration.
- When column area is loaded alone failure is indicated by bulging of stone column at a mid-depth height approximately.
- As the spacing increases, effectiveness of group of stone column decreases.

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