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Failure of Stone Column due to material yielding

By

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Abstract:

Stone column or granular piles is one of the most effective ground improvement techniques for soft cohesive soil to increase in bearing capacity and reduction in settlement. This is partly due to replacement of soil by a relatively stiffer materials as well as drainage of action of stone column. The failure of stone column may be taken place due to the yielding of the column if the column has to support a rigid raft due to difference in stiffness and also may take place due to yielding of the stone column material upon loading beyond the critical state. The behaviour of such yielding stone column can be considered to be in the triaxial state of stress due to confinement provided by the surrounding soil or the reinforcement (encased stone column). The behaviour of the column material can be taken as an elasto-plastic and strain hardening and hence the yield function for the stone column material can be determined using different model parameters which can be determined from a series drained of triaxial tests data. This paper will present an approximate yield function for the yielded encased stone column.

Keywords:

Stone column, Triaxial Test, Encased Column, Critical state and Yield Function

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1. Introduction:

Continuous rarefaction of good construction sites throughout the globe results in increases the land prices which in turn building up pressure on the Geotechnical Engineers to formulate some economic and simple ground improvement technique other than deep foundation to develop soft and marginal for infrastructural activities. Due to the availability of effective ground improvement methods, many projects are now being constructed on sites that, due to poor ground conditions were not previously have been considered economical. Stone columns were first employed in Europe in the 1830's and have been used extensively after its discovery as a by product of Vibrofloatation technique since 1950. It provides a reinforcing effect, increases the horizontal effective stress and acts as a vertical drain.

The pioneering work on stone column is credited to Greenwood (1970). Since then the use of sand columns in geotechnical engineering to improve the bearing capacity of weak or soft soil, and reduce the settlement of foundations resting on weak soil are found in many literature (Bergado et al., 1991, 1992). A granular column is constructed by filling a cylindrical cavity with granular material. The soft soil improvements with stone columns are achieved due to faster rate of consolidation and load carrying capacity increase and/or settlement reduction due to inclusion of stiffer granular material. When vertical and corresponding lateral deformations occur in a stone column under vertical load, the surrounding soil exerts passive earth pressure to the column materials and increases with the increase in column depth. The mechanical behavior of the granular material is usually controlled by the lateral confining pressure; most granular columns fail from bulging near the top due to insufficient lateral support (Hughes and Withers, 1974; Madhav and Miura, 1994). However, the difference in the confining condition for a sand column subjected to triaxial compression and that embedded in soil is that in case of triaxial conditions This approach arises from an assumption used in the analysis by Hughes and Withers (1974) that stone columns in the ground can be modeled by a column in a triaxial cell confined by an approximately constant radial stress but, it must be noted that this approach has its own limitations as an increase in strength of the soil with an increase in depth cannot be modeled.

2 DETAILS OF EXPERIMENTAL INVESTIGATIONS:

2.1 Material properties

Standard Ennore sand was used for preparing the sand samples for the drained triaxial tests. The index properties of the sand were determined by using standard test procedures as outlined in the code practice of American Standard for Testing of Materials. The specific gravity of this sand is found to be 2.58 (ASTM D 0854–06). The particle size distribution was determined by dry sieve analysis as per ASTM D 6913-04 is shown in Fig.1. The different particle sizes D_{10}, D_{30}, D_{50} and D_{60} are found to be 0.6mm, 0.68mm, 0.69mm and 0.7mm respectively. The Coefficient of uniformity (C_u) and Coefficient of curvature (C_c) are obtained as 1.1 and 1.05 respectively. As per United Soil Classification System (USCS) [ASTM D 2487-06], the soil is classified as poorly graded sand (SP). The maximum dry density (γ_{dmax}) as determined using a vibratory table (ASTM-D 4253-00) is 16.8kN/m^3. The minimum dry density (γ_{dmin}) is found to be 14.7 kN/m^3 (ASTM D 4254-00). The maximum void ratio (e_{max}) and minimum void ratio (e_{min}) are 0.89 and 0.58 respectively. The angle of shearing resistance (Dry) as determined from direct shear test [ASTM D 6528-07] and Coefficient of permeability of the soil were found out to be 42º and 0.067cm/sec respectively. The relative densities of the sand samples were kept as 71% for all tests.

2.2 Details of test series

All total 50 tests were conducted for both the series and the out of which 40 tests shows consistent results and the best set of results are presented in the results and discussion section. The test conducted on saturated sand samples can broadly be catagorised as follows and their brief descriptions are presented here.

2.2.1 Isotropic consolidation with loading and unloading

In case of isotropic consolidation test the samples are subjected to equal compression from the all the three dimensions and hence the sample get compressed equally in all directions. In this investigation the samples were first loaded isotropically to a specified stress level in the triaxial cell and then unloaded gradually to zero confining stress. In this case the sample was first isotropically consolidated in two stages with firstly loaded to 500kPa and then unloaded to zero confining pressure. In the second loading cycle the sample was loaded to higher stress level i.e. upto 1000kPa and then unloaded to zero. The behaviour of the sample in this test is depicted in Fig.5 and Fig. 6. This test was carried out to evaluate the change in volume at specific stress level, the plastic volumetric strain due to hydrostatic stress by unloading it to atmospheric pressure.

2.2.2 Consolidated drained triaxial test with loading and unloading

Vertical loading and unloading tests were conducted on the isotropically consolidated samples by applying deviatoric stress keeping the confining pressure constant. The tests were conducted with different confining pressure ranging from 400kPa to 1000kPa. The tests are done with loading upto certain deviatoric stress level and unloading the samples to zero. In every stages of loading the samples were loaded to higher deviatoric stress level and then unloaded to zero. The response of loading–unloading is shown in Fig.7, 8, and 9

2.3 Test equipments

Standard triaxial testing apparatus (HEICO, Mechanical type) as well as computerised GDS Triaxial machine (GDS Instruments, London) were used for testing saturated cylindrical samples of sand. The use of both the instruments was done to determine the consistency of the test results. Only few triaxial tests were conducted in the GDS triaxial

testing machine as the machine was procured as the latter stage of my thesis work. The mechanical type triaxial testing equipment comprise of the following:
(i) Triaxial cell (ii) Loading frame (iii) Load cell (iv) Pore pressure measuring cell (v) Strain dial gauge and (vi) Sensitive volume change gauge. The details of which are shown in Fig. 2, 3 and 4.

2.3.1 Application of cell pressure

All valves except the pinch cock on the plastic tube, valve 23, and the WIV are opened to allow the water to find the level in the system. Ensuring the pump is full of water; pressure is developed to push the mercury into the AUL to make the level equal in both the cylinders.

Sensitive volume change gauge

Any change in pressure or in axial load in triaxial test is generally results in volume changes of the sample. In case of consolidated undrained test on fully saturated sample, this volume change is of negligible order owing to the low compressibility of water in the pore space. For over consolidated or compacted soils in unconsolidated undrained without pore pressure measurement or drained test, volume change may be of appreciable order. The measurement of this volume change is of great importance in determination of compressibility of soil, calculation of the actual cross sectional area of the sample at failure and the rates of dilatancy during the shearing deformation. The volume change gauge attached to the triaxial cell is based on the principle of the measurement of the volume of the fluid expelled from the pores of the sample. The sensitivity of the volume change apparatus is $0.003cm^3/min$ with $6cm^3$ volume of fluid per cycles.

2.4 Test procedure

The standard procedure of testing as outlined by Bishop and Henkel (1969) was used for the drained triaxial test on Ennore sand samples. The preparations of the sand samples were done by dry vibratory method as per ASTM 5311-92(04). Weight of dry sand required to obtain a specimen of desired density was determined based on the volume of the spilt mould. The soil was divided into six equal parts and each part was spooned into the membrane lined split mould attached to the bottom plates of the triaxial cell. The soil in each lift was compacted to the desired dry unit weight by vibration. For all the tests, a strain rate of 0.35% per minute was used. Most of the tests were continued up to a strain level of 20%. Corrections such as membrane penetration, membrane force, cell expansion, and cross-sectional area were considered and applied.

2.4.1 Mounting of the specimen

The test specimen was placed on the base of the triaxial test apparatus after the drainage line and the base was lubricated with water. A good quality of rubber membrane was used with no weak spots. It was slipped into the sheath stretcher and keeping equal length of sheath projecting on both sides, the ends were turned on the outside of the stretcher ensuring the rubber membrane was sticking to the inside surface of the stretcher. Sine the tests were drained tests; porous stone with cut to size filter paper was always placed below the bottom of the sample. A filter paper was placed over the top of the specimen and bottom of the top platen. The stretcher along with the stretcher rubber membrane was placed to encase the sample and the sheath was transferred from the stretcher to the platens carefully. 'O' rings were slipped onto the platens using in pairs to ensure that the rubber was held tightly. The sheath stretcher was then removed with care. The cell was then reassembled without disturbing the set-up. The cell was filled air free cleaned water upto 0.5cm gap at the top. The gap is then filled with caster oil through air vent and let air go out through another vent. The layer of oil is required check the leakage around plunger.

2.4.2 Saturation of the sample

All the tests were carried out on saturated and consolidated specimens. Firstly the specimen is saturated using the concept of Skempton's pore pressure coefficient. The value of B in Skempton pore pressure parameters indicates the degree of saturation. For a saturated soil the value of 'B' is equal to unity. The sample was put under 50kPa and a back pressure of 50kPa to accelerate the saturation. Both cell and back pressure were applied using self compensating mercury control system. The pressure was later increased to 100kPa and 150kPa. The samples were found to get saturated at this level.

3. ANALYSIS OF TEST RESULTS

3.1 Evaluation of the model parameters: The different model parameters that required to determined from the triaxial test results are M, λ, k, H, A, G and K. These parameters were obtained based on the experimental results as functions of the main parameters which include the confining pressure, the angle of friction and the relative density using the best-fitting curve technique.

i) The parameter M: The q-p curve shown in Fig.10 is used to determine the value of the model parameter 'M'. The slope of the q-p diagram gives the value of this parameter. The slope of the yield point is taken as the value of 'M'. The slope of p-q curve in this present investigation is found out to be 1.32 for this type of sand and this line indicates that no state of stress can go beyond this line.

ii) The parameter λ: The response of hydrostatic stress [Log (p)] with volumetric strain is depicted in Fig. 6. The value of λ can be directly determined from the slope of the compression curve. In this experimental investigation it is found as 0.01724.

iii) The parameter k: The hydrostatic stress (p) - volumetric strain (ε_v) response for the isotropic consolidation of the specimen is shown in Fig. 6. From the response curve we can directly determine the value of k which is the value of the slope of the swelling curve. The value of k in this experimental investigation is found out to be equal to 0.052.

iv) The elastic shear modulus: The slope of the loading–reloading branch of σ_3=constant curves (Fig. 7-9) can be used to evaluate the value of this parameter and is given by

$G = k \times p_a[(p-0.33q)/p_a]^n$

Where p_a is the atmospheric pressure and p, q are the hydrostatic stress and deviatoric stress respectively. K and n are constants to be determined from logG Vs logP curve. These values for this present investigation are determined as below:

$$k = 280 \text{ and } n=0.68$$

So, the elastic shear modulus G can be written as,

$$G = 280p_a\ [(p-0.33q)/p_a]^{0.68}$$

iv) The parameter K: The bulk modulus of the sand (i.e. K) can directly be computed by using the relation $K=dp/d\varepsilon_v^p = 126910 kN/m^2$.

4. DETERMINATION OF YIELD FUNCTION

The approximate yield function for the material can be written as the function of the following parameters, The yield Loci for this present investigation on Ennore Sand for different volumetric strain level is shown in Fig. 10

$G = f(p, q, H) = 0$; where $H = \varepsilon^p v$ (Eq. 1)

Mathematically the elliptic cap function can be expressed as

$f = (I_1 - L + H)^2 + R^2 J_2 - (X-L+H)^2$ (Eq. 2)

Where, $I_1 = 3p$ (p = Hydrostatic stress in kPa); $J_2 = q^2/3$ (q = Deviatoric stress in kPa)

H=Strain hardening parameter which is determined through trial and error method and found to be

$H = [(\varepsilon_p{}^v)^2 + 0.05(\varepsilon^{-p})^2] \times 10^4 + 2\sqrt{(\varepsilon^{-p})}$ (Eq. 3)

X = Maximum 'X' co-ordinate of yield surface or plastic potential surface from origin in the plastic potential or yield surface.

L = Minimum X coordinate of yield or plastic potential surface from origin in the plastic potential or yield surface.

R = shape function which derives from the maximum 'Y' ordinate of the yield surface in the plastic potential and related as R = (X-L)/R i.e., $R = \sqrt{(X-L)}$

By substituting the above known values in Eq. 2 and after simplification the yield function can be determined as:

$f = q^2 + 10.4\ (p-6.92H)^2 + 120H - 299 = 0$ (Eq. 4)

So the different elasto - plastic parameters pertaining to this present investigation can be summerised as follows:

M = 1.39 (Failure Envelope), $\lambda = 0.01724$, $k = 0.0521$, $K = 126910 kN/m^2$ and $G = 280p_a[(p-0.33q)/p_a]^{0.68}$

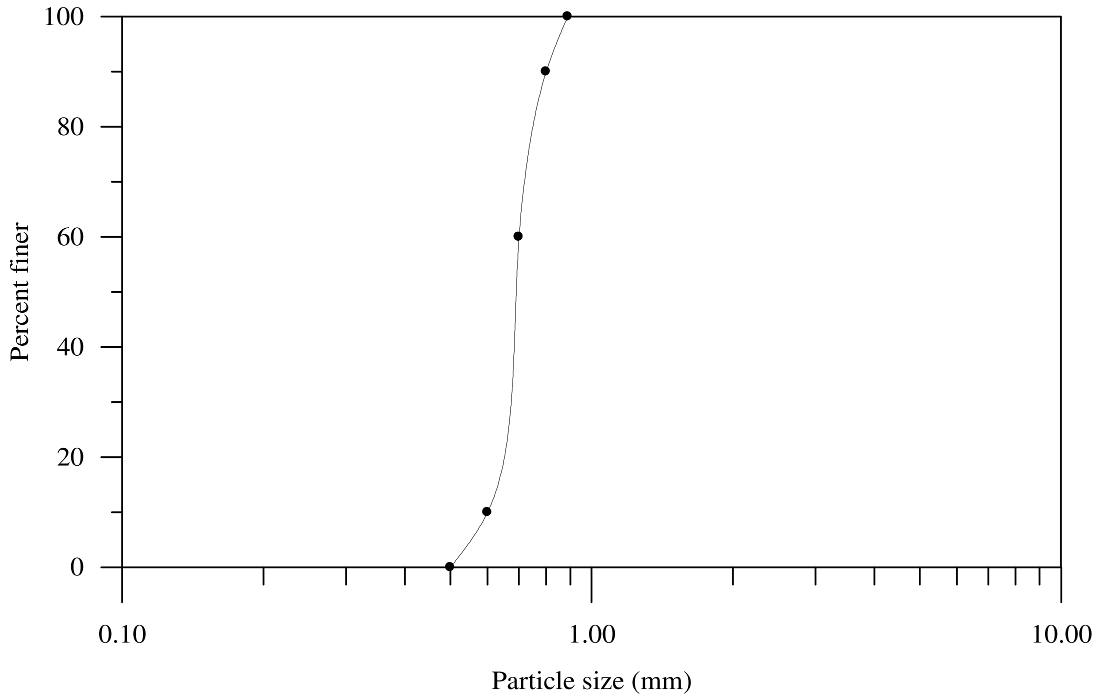

Fig.1 Particle size distribution of sand used in experimental work

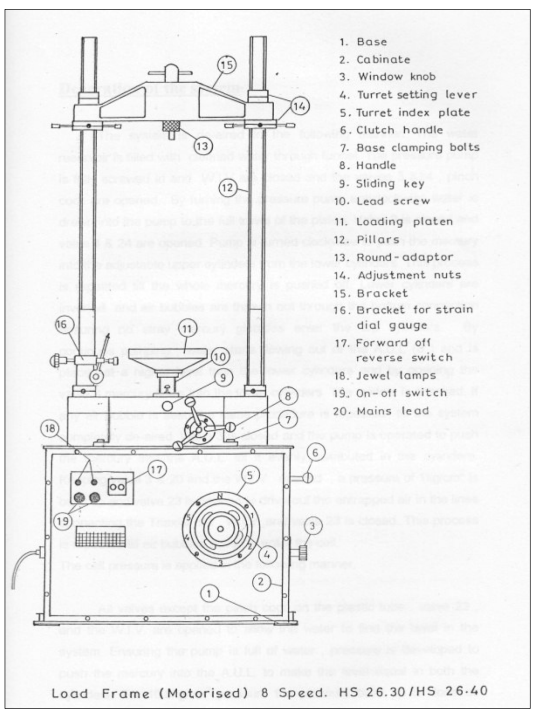

Fig.2: Schematic diagram of the triaxial loading frame

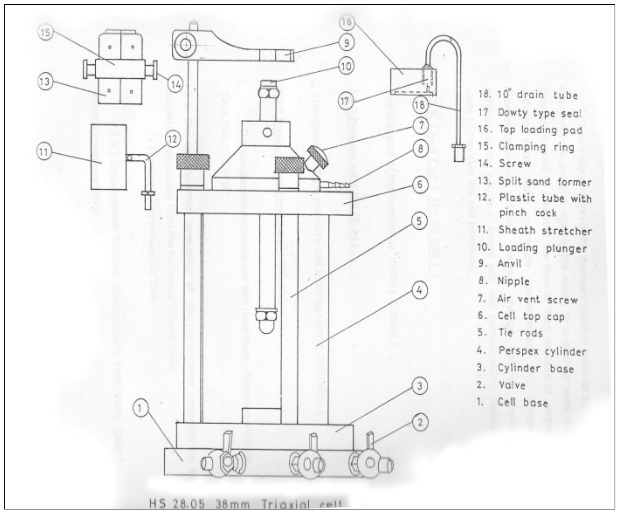

Fig. 3: Schematic diagram of the triaxial cell assembly

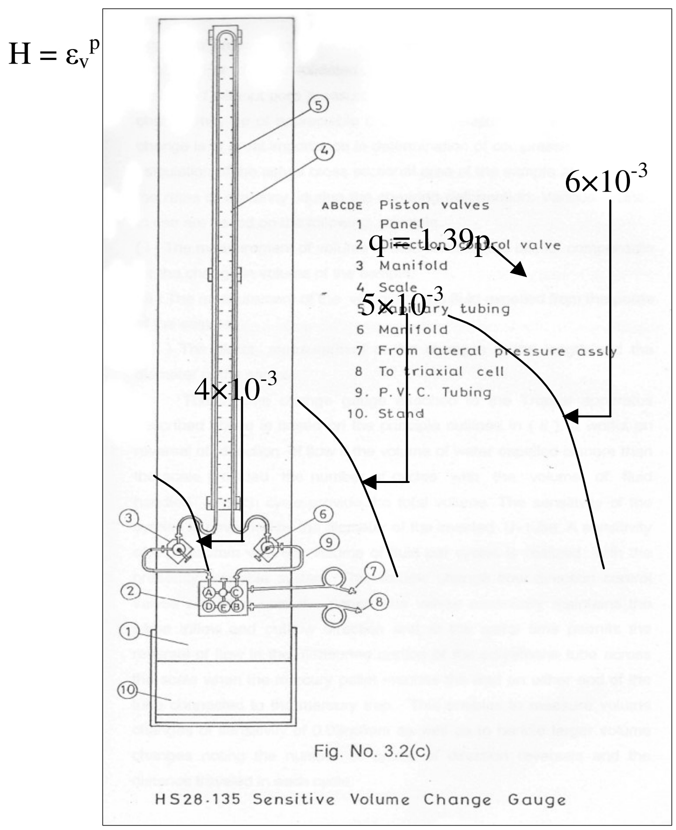

Fig.4: Schematic diagram of Volume change apparatus

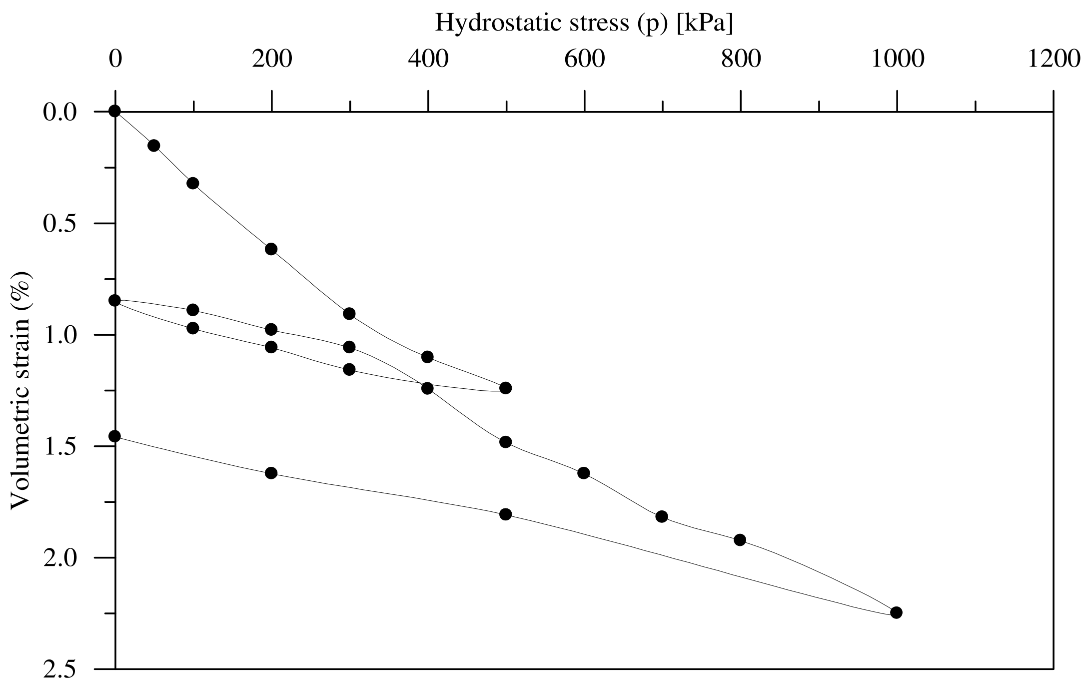

Fig.5: Hydrostatic stress (p) - volumetric strain (ε_v) response under of sand in isotropic consolidation

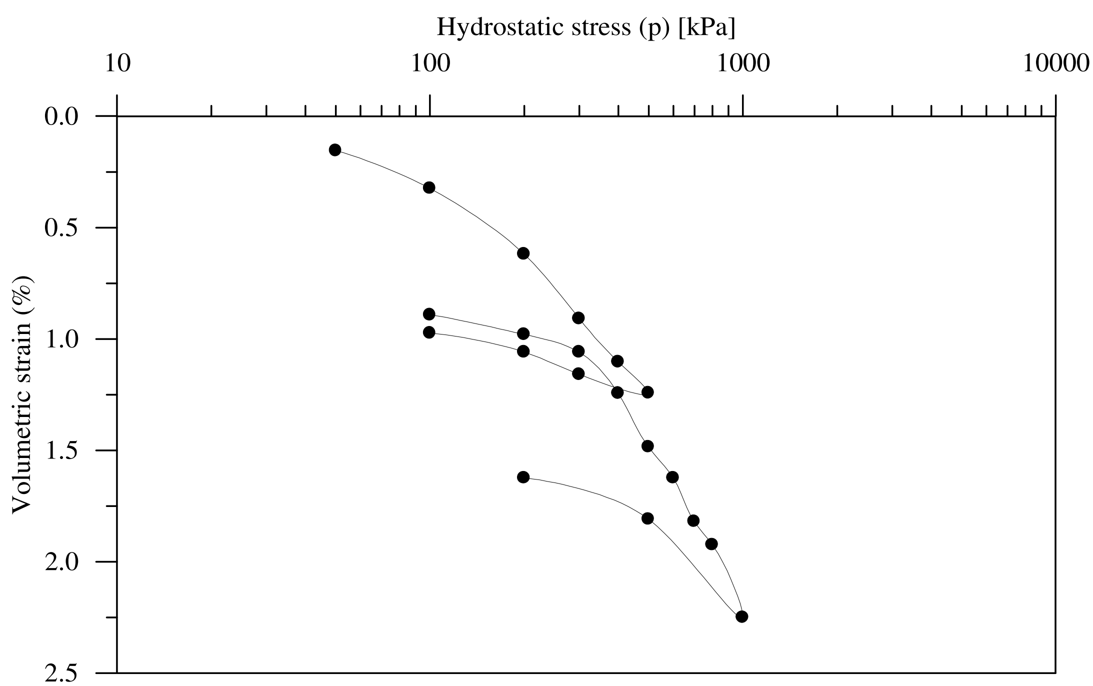

Fig.6: Log (p) - volumetric strain (ε_v) response under isotropic consolidation of sand

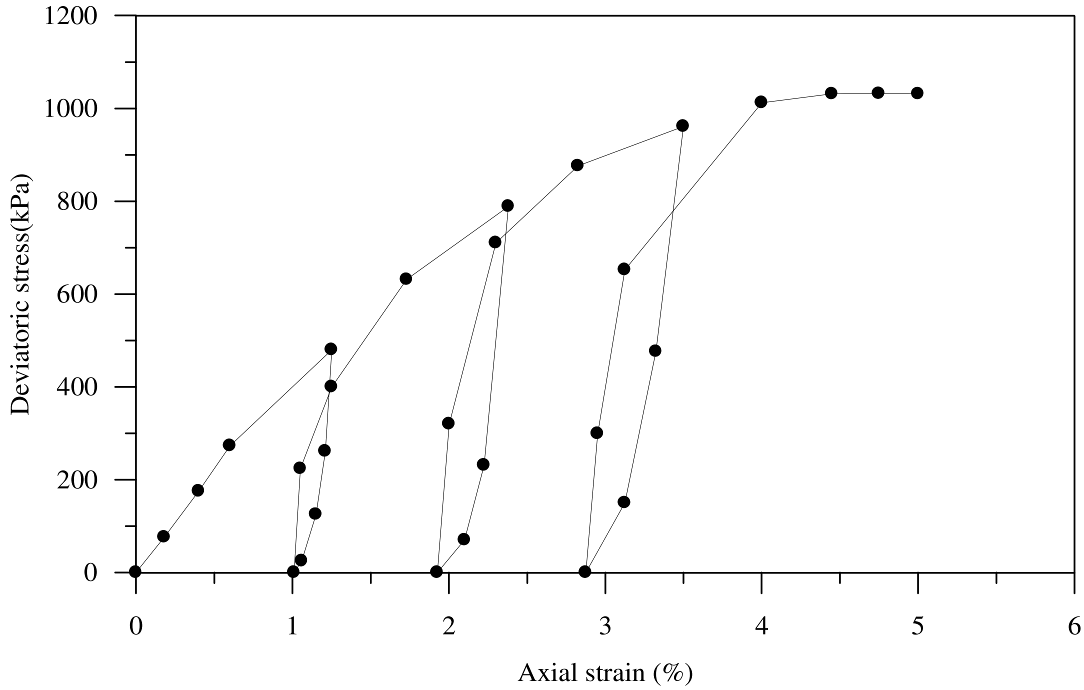

Fig.7: Loading - Reloading response of sand in triaxial test ($\sigma_3 = 400$kPa)

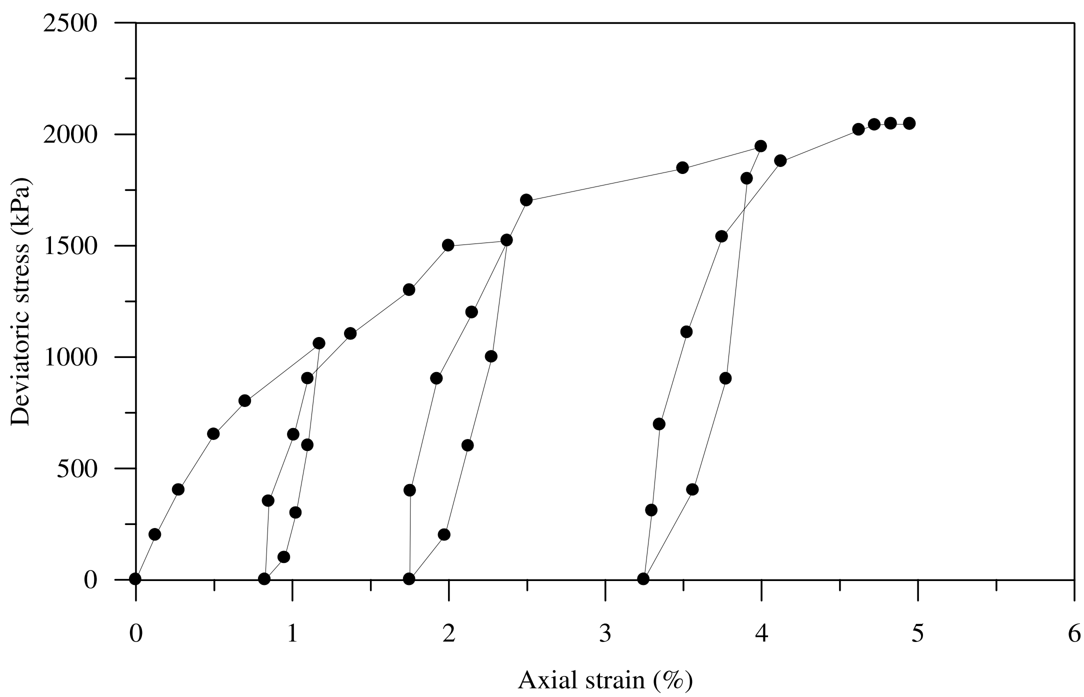

Fig.8: Loading-reloading response of sand in triaxial test ($\sigma_3 = 700$kPa)

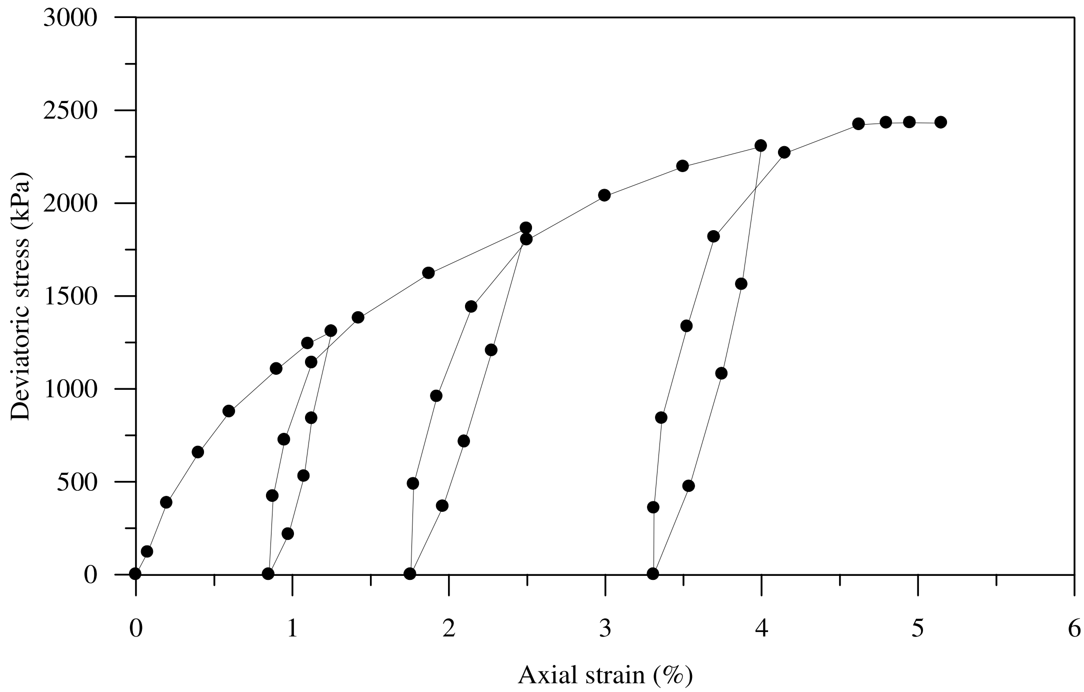

Fig.9: Loading-Reloading response of sand in triaxial test ($\sigma_3 = 1000$kPa)

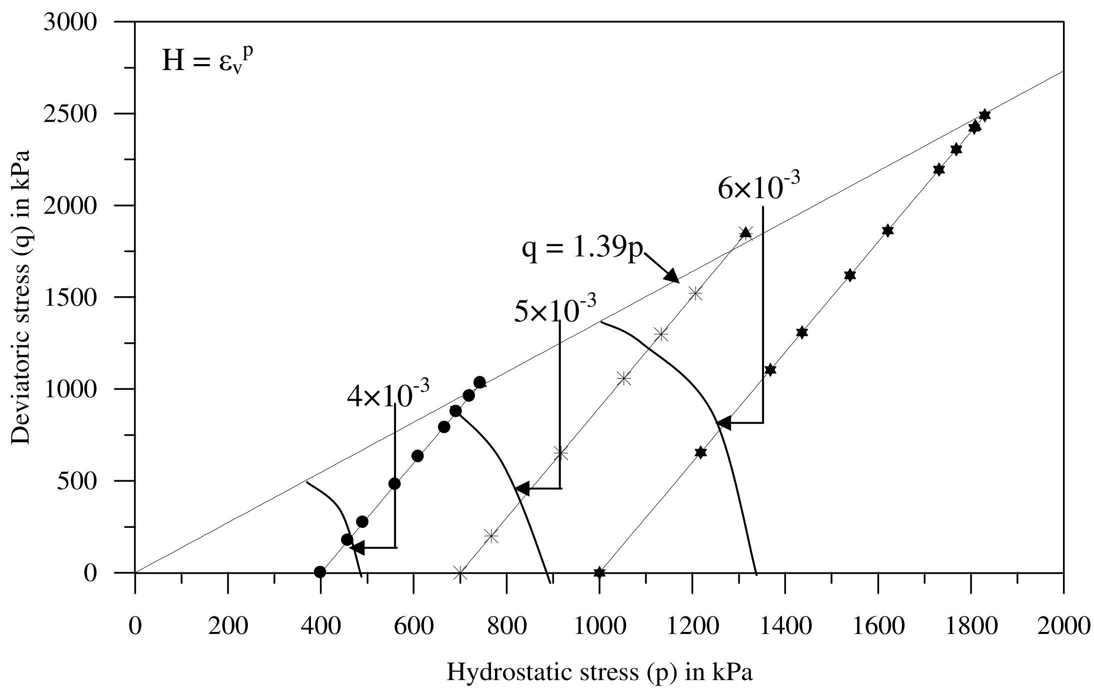

Fig. 10: Hydrostatic stress vs deviatoric stress response for sand

6. Conclusions:

From the above discussion it can be concluded that the behaviour of the stone column and its material yielding can be studied in the laboratory with the help of drained triaxial tests and will provide vital information regarding the loading conditions which leads to the critical state of the column materials. If the field condition of the stone column is properly simulated then it will be the very effective methods for the study of the critical state parameter of the column material. Although this method is useful for all types of stone column but can be more useful for encased or reinforced stone column where bulging is prevented by extra confinement. So this can be considered as a beginning towards the study on the critical state of stone column material under loading conditions.

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Nomenclatures:

σ_3 … Confining stress in kPa
p … Hydrostatic stress in kPa
q…. Deviatoric stress in kPa
ε_v … Volumetric strain
G…. Elastic Shear Modulus in kPa
M…. Slope of the ‘p-q’ Curve
λ….. Slope of the hydrostatic compression curve
k…... Slope of the swelling curve
K…. Bulk modulus of Sand
p_a…. Atmospheric pressure in kPa
H…. Hardening Parameter
f….. Yield function