Abstract

Stone columns or granular piles can have widespread application in the field of ground improvement. It is a cost effective measure used to improve bearing capacity of weak soils for supporting a wide variety of structures such as residential, commercial and industrial buildings, raft foundations, oil storage tanks, highways and other applications. It also helps in reducing settlements. The stone columns derive its load carrying capacity mainly from confinement of the surrounding soil. In the present investigation, load versus settlement response of silty clay bed is compared with the load versus settlement response of silty clay bed reinforced with stone column for different aggregate mixes and different depth of stone column with and without encasement. The plate load tests were carried out on a single column in a large rectangular tank (1.5m×0.6m×0.9m). The test bed was prepared using locally available silty clay soil. Investigations were carried out by varying the size of aggregate in the stone column as well as mixing different size aggregates. The results from the tests indicated a clear improvement in load carrying capacity of silty clay bed due to stone column as well as a reduction in settlement. The bearing capacity improvement factors have been found out to be 1.11, 1.23, 1.32 for stone columns with 20mm aggregates; 1.08, 1.22, 1.31 for stone columns with 10mm aggregate; 1.15, 1.24,1.41 for stone columns with a mixture of 10mm and 20mm aggregates in the ratio of 1:1 by weight for depths of 250mm, 500mm and 750mm respectively at 25mm settlement. It has been observed that for encased stone column the bearing capacity improvement factors at 25mm settlement for full depth of silty clay bed i.e. 750mm has been observed as 1.87,1.83 and 1.91 for stone columns with 20mm, 10mm and a mixture of 10mm and 20mm aggregate respectively.

Chapter 1
Introduction

1.0 Introduction

With the ever increasing population of the country and the limited land resources, it has but become imperative that we develop sites which were previously considered marginal and not viable for construction activities. Although previously it was prohibitively costly but now with the advent of new ground improvement techniques it has become economically feasible. The high cost of conventional foundations coupled with environmental concerns has made development of weak soil deposits a necessity. For structures such as low rise buildings, oil storage tanks and other structures like embankments fill supports for highways, bridge approaches, factories, abutments etc. which can tolerate settlement to a certain extent, stone columns or granular piles provide a cost effective method for ground improvement. Stone columns are either constructed as fully penetrating the soil layer which is supported on a hard stratum or as floating with their tips embedded on the soil layer itself.

Stone columns are primarily composed of granular material compacted in long cylindrical holes and are used for improving strength and consolidation characteristics of soil. Load carrying capacity of a stone column is primarily governed by frictional properties of stone mass, frictional properties of soil surrounding the stone column, rigidity or flexibility of the foundation transmitting stresses to the ground, lateral pressure developed on the surrounding soil mass which acts on the sides of the stone column due to interaction between various elements of the system etc. The axial capacity of the stone column is mainly on account of passive earth pressure developed due to bulging effect of the column and increased resistance to lateral deformation under superimposed surcharge load. Stone columns on several occasions have been used instead of piles to support manycritical structures such as oil storage tanks, pipe racks, raft foundations leading to high degree of economy and speedy construction.

Stone columns also have certain secondary roles. It helps in mitigating liquefaction and its consequences in saturated sandy soils. However, its effectiveness is limited in low permeable silty soils that are prone to liquefaction. Stone columns provide drainage to hinder excess pore pressure development during an earthquake. They also act as vertical drains and speed up the process of consolidation. Also in case of sensitive clays stone columns have certain limitations. As there is limited lateral restraint, the settlement of the bed increases. The clay particles are clogged around the surface of the stone column thereby decreasing radial drainage.

The presence of stone columns creates a composite material. This material has lower compressibility and more shear strength compared to virgin soil. On application of vertical stresses the soil and stone column move downward together resulting in stress concentration in the stone column. This stress concentration primarily occurs on account of column material being stiffer than soil. An axial load applied at the top of a stone column produces a bulge at a depth of around 2 to 3 times column diameter. This bulge in turn increases the lateral stresses which provides additional confining pressure to the stone column. An equilibrium state is ultimately reached which reduces settlement in comparison to untreated soil.

The concept of stone columns was first formulated in France in 1830. Since 1950s, it has been extensively used in Europe to improve soil conditions. But its application has somewhat been limited in India. Infrastructure development in India at present is on full swing. As the availability of good land for construction is depleting fast, hence it is becoming necessary that we develop soils which have low shear strength and bearing capacity as well as high compressibility. Of all the ground improvement techniques, the stone column method provides us with a cheap and fast method through which we can improve ground conditions through increased bearing capacity and reduced settlements.

In this study we are going to compare load deformational characteristics of untreated silty clay bed with that of silty clay bed reinforced with stone column. The study is being carried out for different column depths having different aggregate gradation. 120mm diameter stone column of varying depths is constructed at the centre of the rectangular tank. The columns are constructed using 10mm, 20mm and a mixture of 50% 10mm and 50% 20mm aggregate by weight. The ultimate bearing capacity is also computed using relations given by IS code and reputed journals.

1.1 Objectives

The main objectives of this present study are:

- To identify the characteristics of soil and stone aggregate
- To study the load settlement behaviour of silty clay bed reinforced with and without stone column for different depths and gradation
- To study the load settlement behaviour of silty clay bed with stone columns of a particular depth with and without casing with geotextiles.
- To compare the settlement characteristics of silty clay bed with and without stone column reinforcement
- To compare theexperimental and theoretical values of bearing capacity and settlement.
- Conclusions

Chapter 2
Review of Literature

2.0 Literature Review

Stone Columns have wide spread application in the field of soil stabilization. They are frequently used for stabilization of soft clay or silts and lose silty sands. For low rise buildings, highway facilities, embankments, storage tanks, bridge abutments and other structures that can tolerate some settlement, stone columns are one of the most frequently used methods for ground improvement. Its advantages include low cost, effectiveness and ease of installation. The beneficial effects of stone columns are reduced settlement, increased stiffness, increased time rate of settlement and the reduction of liquefaction potential.Several researchers have worked on stone columns and many field tests, numerical analysis have been carried out to study the effects of stone columns on poor ground. However, the design of stone columns till date is based on the empirical approach as the load settlement behavior of stone columns is influenced by a number of factors. The available literature on stone column is discussed in this chapter.

2.1 Methods of stone column installation

Various methods for installation of stone columns are in vogue all over the world depending on the availability of equipment and their proven applicability. A few of them are presented in the following paragraphs.

2.1.1 Vibro-compaction method

In Vibro-compaction method the density of cohesionless, granular soils is improved using a vibroflot which sinks in the ground under its own weight and with the assistance of water and vibration [4]. After reaching the predetermined depth, the vibroflot is withdrawn gradually from the ground with the subsequent addition of granular backfill.

2.1.2 Vibro-replacement method

The vibro-replacement method is used to improve cohesive soils with more than 18% passing no. 200 US standard sieve. The equipment used is similar to vibro-compaction. The vibroflot is pushed into the ground under its own weight assisted by water or air jets as a flushing medium until it reaches the required depth[4]. This method can be either carried out with wet or dry process. In wet process, a hole is formed in ground by jetting a vibroflot down the desired depth with water. When vibroflot is withdrawn, it leaves a borehole of greater diameter than vibrator. The uncased hole is flushed out and filled in stages with 12-75 mm size coarse aggregates. The densification is provided by an electrically or hydraulically actuated vibrator near the bottom of the vibroflot. The wet process is generally suitable for unstable boreholes and a high ground water table.

Penetrate to full depth	Compact base	Compact in steps to surface

Fig 1: Wet top feed system of stone column construction
(Ref: http://www.zetas.com.tr/index.php?dil=EN&id=222000)

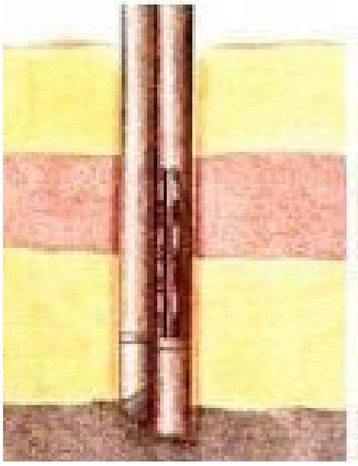
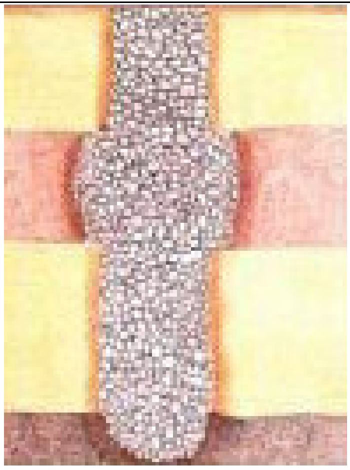
 Penetrate to full depth	 Compact base	 Compact in steps to surface

Fig 2: Dry bottom feed system
(Ref: http://www.zetas.com.tr/index.php?dil=EN&id=222000)

2.1.3 Cased-borehole method

In this method, the piles are constructed by ramming granular materials into the prebored holes in stages using a heavy falling weight(15kN to 20kN) from a height of 1.0m to 1.5m [7]. The method is good substitute for vibrator compaction considering its low cost. The method is useful in developing countries like India utilizing only indigenous equipment in contrast to the methods described previously.

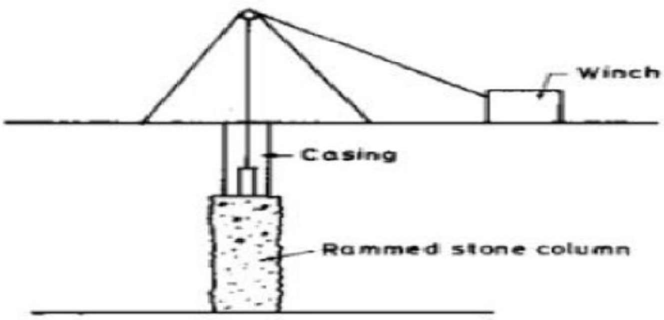


Fig 3: Cased rammed stone column (Ref: http://ethesis.nitrkl.ac.in/5229/1/211CE1229.pdf)

2.2 Design Concept

The design of stone columns has not been fully understood by researchers. It is as empirical as the design of pile foundation. A stone column's load carrying capacity is mainly dependent on the lateral resistance provided by the surrounding soil to the expansion caused by bulging of the aggregates on application of load.

Theparameters which are used in measuring the load carrying capacity of stone columns are

a) Angle of internal friction of stone aggregates in column
b) Diameter of the stone column
c) Undrained shear strength of soil surrounding the stone column
d) Lateral stress in the soil
e) Radial pressure or deformation characteristics of the soil

2.3 Failure mechanism of stone column

Possible modes of failure of stone columns are

- Bulging failure
- Shear failure
- Punching failure

For single isolated stone column, the most probable mode of failure is by bulging. This mechanism develops whether the column is floating in soft soil or fully penetrating and bearing on a firm layer.

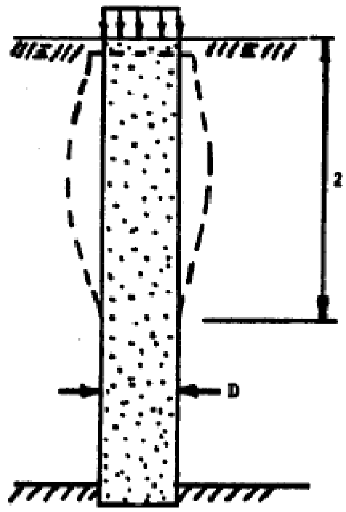
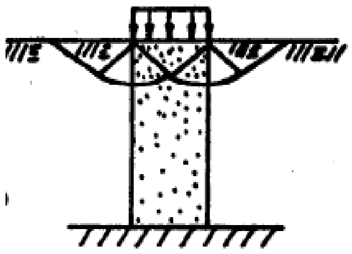
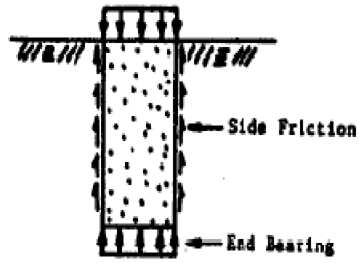
		
Long column with firm or floating support (Bulging failure)	Short column with rigid base (Shear failure)	Short floating column (Punching failure)

Fig 4: Different modes of failure of stone column (Ref: http://www.ejge.com/2012/Ppr12.118e.pdf)

2.4 Ultimate bearing capacity of single, isolated granular piles

A realistic assessment of the ultimate bearing capacity of the supporting soil is of paramount importance for safe and economic design of the foundation.

Since most stone columns have length to diameter ratio equal to or greater than 4 to 6, bulging failure usually develops whether the tip of the column is floating in soft soil or is supported by a firm bearing layer. [12], [13] observed that the bulged developed at a depth of 2 to 3 diameters beneath the surface. These small scale model tests were performed using columns of 12.5mm to 38mm in diameter

As early as 1835, Moreau (referred by [12], [13]) observed that very little of the applied load reaches the bottom of a single column if the column is more than twice its depth. The fact that load applied to a single stone column is transferred to its surroundings was further verified by [12], [13].

A number of theories have been presented for predicting the bearing capacity of an isolated single stone column surrounded by soft soil [1], [4], [8], [10], [12], [14], [16], [30], and [32]. Most of the early analytical solution assumed a triaxial state of stress exists in the stone column and both the column and the surrounding soil is at failure [1], [4], [10], [30] and [32].

The lateral confining stress σ_3 which supports the stone column is usually taken in these methods as the ultimate passive resistance which the surrounding soil can mobilize as the stone column bulges outward against soil. Since the column is assumed to be in a state of failure, the ultimate vertical stress σ_1, whichthe column can take is equal to the co-efficient of passive pressure of the stone column, K_p times the lateral confining stress, σ_3 , which from classical plasticity theory can be expressed as

$$\frac{\sigma_1}{\sigma_3} = \frac{1 + \sin \emptyset_{column}}{1 - \sin \emptyset_{column}}$$

Where $\emptyset_{column}$ = angle of internal friction of stone column material

$\frac{\sigma_1}{\sigma_3}$= K_p= co-efficient of earth pressure for stone column

2.5 Settlement calculation of single, isolated granular piles

The methods for calculating settlement of isolated stone columns can be broadly divided into two categories

a) Simple approximate methods which make important simplifying assumptions
b) Sophisticated methods which are based on fundamental elasticity or plasticity theory which model material and boundary conditions

The equilibrium method described by [1] offers a very simple yet realistic approach for the reduction in settlement of ground improved with stone columns. In applying this simple method the stress concentration factor must be estimated using past experience. If a conservatively low stress concentration factor is used, a safe estimate of the reduction in settlement due to ground improvement will be obtained

The change in vertical stress in clay, σ_c, due to applied external stress is given by

$$\sigma_c = \mu_c \sigma$$

Where σ is the average externally applied stress,

and μ_c is given by $\frac{1}{[1+(n-1)a_s]}$

where, a_s is the area replacement ratio defined as $\frac{A_s}{A_s+A_g}$

A_s, A_g are area of stone column and area of ground surrounding the column

And $n = \frac{\sigma_s}{\sigma_g}$ as per IS 15284 (Part 1): 2003 page 12

where, σ_s is vertical stress in compacted column

σ_g is vertical stress in surrounding ground

From conventional one dimensional consolidation theory

$$S_t = \left(\frac{C_c}{1+e_0}\right) \log_{10}\left(\frac{\sigma_0 + \sigma_c)}{\sigma_0}\right. H$$

Where, S_t is primary consolidation settlement occurring over a depth H of stone column

σ_0 is initial effective stress in clay layer

C_c is compression index

e_0 is initial void ratio

2.6 Experimental and numerical studies

From the latter part of the 20^{th} century researchers have started working on theoretical, experimental and field study on behavior of stone columns. However, there is little information available regarding the design procedure to be adopted for a given situation.Semi empirical design approach based on the allowable stress on stone columns and the undrained shear strength of clay have been proposed by [11], [12], [13], [22], [23]. Semi empirical design approach based on pressuremeter theory was proposed by [12], [13]. Cavity expansion approach was proposed by [31] have been used by [21] and [7]. The theory of load transfer, estimation of ultimate bearing capacity and prediction of settlements of stone columns was first proposed by [11] and later by [12], [13],[20],[1], [3], [15].

The tests carried out indicated that ultimate capacity of stone column was governed primarily by the maximum radial reaction of the soil against the bulging and the extent of vertical movement in the stone column was limited to about 4 times the diameter [12], [13]. Experimental studies to study the effect of pattern of installation of stone columns showed that triangular pattern seems to be optimum and rational [25]. [18]studied the effect of different factors influencing the capacity of stone column improved ground from the available literature and showed that in the case of columns failing by bulging the critical length is about 3 to 5 times the stone column diameter. [17]compared the field performance of stone columns with the predictions by finite element analysis and reported that the agreement was generally good. [24]studied the load response behavior of stone columns in soft soil environment by using a finite element software package (ANSYS).An overview of recent contributions for the analysis and design of stone columns and different equations available in the literature for finding bearing capacity and settlement of stone column improved ground have also being given [15].

The experimental studies are carried out to study the behavior of stone column by varying spacing, shear strength of soft clay, moisture content etc.[2]. They found that when column area alone is loaded, the failure is by bulging of the column with maximum bulging at 0.5 to 1 times the column diameter below the top and when the entire area is loaded bulging failure does not take place. The load settlement behaviour and the ultimate axial capacities obtained from model test compares well with that of finite element analysis.

The experiments were conducted to determine load versus settlement of clay bed stabilized with stone and reinforced stone columns [28]. They found that on encasing stone columns with geogrids there was an appreciable increase in load carrying capacity of both end bearing and floating columns. It was also observed that length/diameter ratio had less influence on bearing capacity of columns for the tests carried out in the investigation. The increase in ultimate bearing capacity for encased and non-encased stone columns was observed to be in the range of three and two times that of the untreated bed.

The study on behavior of single and group of geosynthetic encased stone columns found that all round encasement using geosynthetic resulted in increased stiffness and loading capacity of a stone column [29]. The encased stone columns offered stiffer and stronger response. The elastic modulus of geosynthetic also played an important role in enhancing capacity and stiffness of encased stone column. For higher modulus of encasement confinement pressure generated was more. They also found that encased stone columns have higher stress concentration compared to ordinary stone columns. The stress concentration increased with increase in modulus of encasement which showed that encased stone columns also act as semi rigid piles.

The experiments were conducted to study the improvement in load carrying capacity, stiffness, resistance to bulging of stone column installed in soft soil due to a series of laboratory plate load test[26]. Vertical nails are inserted along the circumference of stone column and it is found that stone column reinforced with nails has higher load carrying capacity, lesser compression and lesser lateral bulging.

A series of model tests on unreinforced and geogrid reinforced sand bed resting on stone column were carried out [9]. The load carrying capacity of soft soil, depth of bulge of stone column increases and bulge diameter decreases due to the placement of sand bed and it is more beneficial in sand bed reinforced with geogrids.

The behaviour of stone column in layered soil comprising of weak soil in the top layer under a number of plate load tests was studied [27]. Load was applied over the entire area in the unit cell tank and stiffness of improved ground estimated. Secondly the stone column was loaded and axial capacity determined. It was found that the depth of top weak soil layer has a great influence on load bearing capacity and bulging of stone column.

The study on consolidation and deformation around end bearing columns under distributed loads and compared the laboratory results with analytical solution and numerical simulation was carried out [6]. Stress concentration factors, equivalent coefficient of consolidation and settlement reduction were analyzed. Soil improvement was found to be directly dependent on the stress distribution between the soil and column.

One the basis of the literature survey, the objectives are being fulfilled in the succeeding chapters.

Chapter 3

Materials and Methods

3.0 Materials and equipment

Soil was excavated from Delhi Technological University campus after removing the surface vegetation. A total quantity of 1.5 cu.m of soil was excavated, air dried and pulverized. Investigations were subsequently conducted to determine the index properties of soil for its classification. Coarse aggregate of 20mm and 10mm size were used for construction of stone column. A rectangular steel tank of 1.5m×0.6m×0.9m is used for preparation of soil bed. The diameter of stone column is chosen such that the loaded area does not affect the walls of the rectangular tank.

Picture 1:Rectangular tank used for soil bed preparation (1.5m×0.6m×0.9m)

3.1 Loading system

Loading frame consists of a rigid truss which is fixed and anchored to sustain very heavy loading. A hydraulic jack consisting of a loading mechanism fitted with load measuring gauge is used to apply load to the silty sand bed. The jack is supported against the truss and loading is applied.

Picture 2: Truss used for loading

Picture 3: Hydraulic jack used for loading

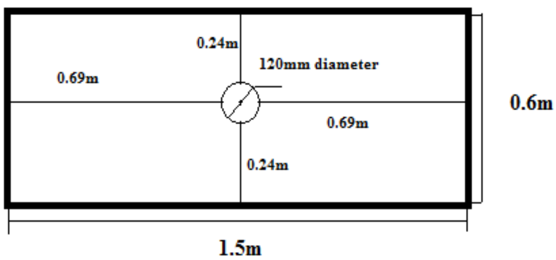


Fig 5: Set up of stone column in rectangular tank

3.2 Geotextile used for encasement

Woven Geotextiles have been used to provide encasement for stone columns. Woven geotextiles are developed from synthetic or natural fibers using weaving techniques. The weaving process gives these geotextiles an appearance of two sets of parallel threads interlaced at right angles. "warp" runs along the length of the loom and "weft" runs in the transverse direction across the loom. The yarn used to produce a woven geotextile may be monofilament or multifilament or a combination of each type. However, slit film tapes have recently become the most common form of yarn used in the manufacture of woven geotextiles. The yarn in the warp direction has to withstand the action of the loom's reeds continually pulling and pushing it apart to make way for the shuttle which pulls the weft yarn through. As a consequence it is usual for a slightly stronger yarn to be selected for the warp direction

Properties of geotextile used in experimentation are as follows:

- Type Woven
- Material Polypropylene
- Thickness in mm 0.58mm
- Colour white
- Permeability 0.47×10-3m/s
- Ultimate tensile strength 2.86 N/mm^2

3.3 Stone Column installation

The stone column is installed in the centre of the rectangular tank. A PVC pipe of 120mm diameter is placed at the centre of the tank and the silty clay bed is prepared around it. The aggregates are filled in three different layers and each layer is tamped using a 10mm rod to achieve a density close to15kN/m^3. For the construction of different types of stone columns, an area greater than the effective area of the stone column is excavated upto full depth and the procedure is repeated, and stone columns of different gradation are formed.

For encased stone columns, the geotextile is wrapped around the PVC pipe after stitching. Aggregates are charged inside the stone column and the PVC pipe is raised in layers with tamping also being carried out in parallel. The procedure is repeated till the desired depth of stone column is obtained.

3.4 Method used

The air dried pulverized silty clay soil is passed through 4.75mm sieve and then filled in three different layers. Each layer is compacted with more than 100 blows with 2.5kg rammer spread over the whole surface. Ramming is also done by wooden rammer to avoid any entrapped air. The soil is filled upto a depth of 750mm. Load is applied gradually and the deformation of the bed at each 5kN increment loading was calculated with the help of dial gauge connected to the base plate. A base plate of size 300mm diameter was used for application of load. Stone column of diameter 120mm was constructed at the centre of the rectangular tank with depths varying from 250mm, 500mm and 750mm; and settlement behaviour of the silty clay bed were observed for each 5kN increment in load. The aggregates used in construction of stone column were of 10mm and 20mm size as well as a mixture of both in the ratio of 1:1 by weight. The stone columns were also encased using woven geotextile and its effect on the load deformation behaviour is observed.

Chapter 4
Experimental Investigations on Soil and Aggregates

4.0Moisture content determination:

Sample is being taken in three different containers of about 30 gms. It was kept in the oven for about 24 hours at 110°C.After 24 Hours weight is again taken through weighing machine and moisture content is determined.

Table 1: Moisture content of soil

Sample Name	**Initial Weight (g)**	**Dried Weight (g)**	**Water Content (%)**
A	30	28.8	4.16
B	30	27.9	7.5
C	30	28.6	4.9

Average moisture content = 5.52%

4.1Specific Gravity by Pycnometer Method(I.S. – 2720 (Part II) 1964)

Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature, generally taken at 4 degree centigrade. The test is being carried out according to the procedure mentioned in IS 2720 Part II, 1964

Table 2: Calculation of specific gravity

Sl. No.	Observations and Calculations	Determination Number	
		1	2
1	Mass of empty pycnometer (M_1)	700.45g	700.46g
2	Mass of pycnometer and dry soil (M_2)	900.49g	900.52g
3	Mass of pycnometer, soil and water (M_3)	1700.9g	1701.2g
4	Mass of pycnometer and water (M_4)	1574.42g	1576.15g
5	$M_2 - M_1$	200.04g	200.06
6	$M_3 - M_4$	126.48g	125.05g
7	$G = \frac{(5)}{(5)-(6)}$	2.72	2.67

Average Specific gravity of soil sample is 2.69

4.2Particle size distribution (IS: 2720 (Part IV) 1985)

Soil consists of an assembly of discrete particles of various shapes and sizes. The object of a particle size analysis is to group these particles into separate ranges of sizes and to determine the relative proportion by weight of each size range. Tests are carried out according to procedure mentioned in IS 2720 Part IV, 1985

Table 3: Grain size distribution calculation

Sl. No.	Sieve size	Mass of soil retained in each sieve (g)	Percentage retained (%)	Cumulative percentage retained (%)	Percentage finer
1	4.75mm	3.3	0.33	0.33	99.67
2	2.36mm	8.2	0.82	1.15	98.85
3	1.18mm	15	1.5	2.65	97.35
4	600µ	59.6	5.96	8.61	91.39
5	300µ	45.1	4.51	13.12	86.88
6	180µ	100.2	10.02	23.14	76.86
7	75µ	175.1	17.51	40.65	59.35
8	Pan	593.5	59.36	100	0

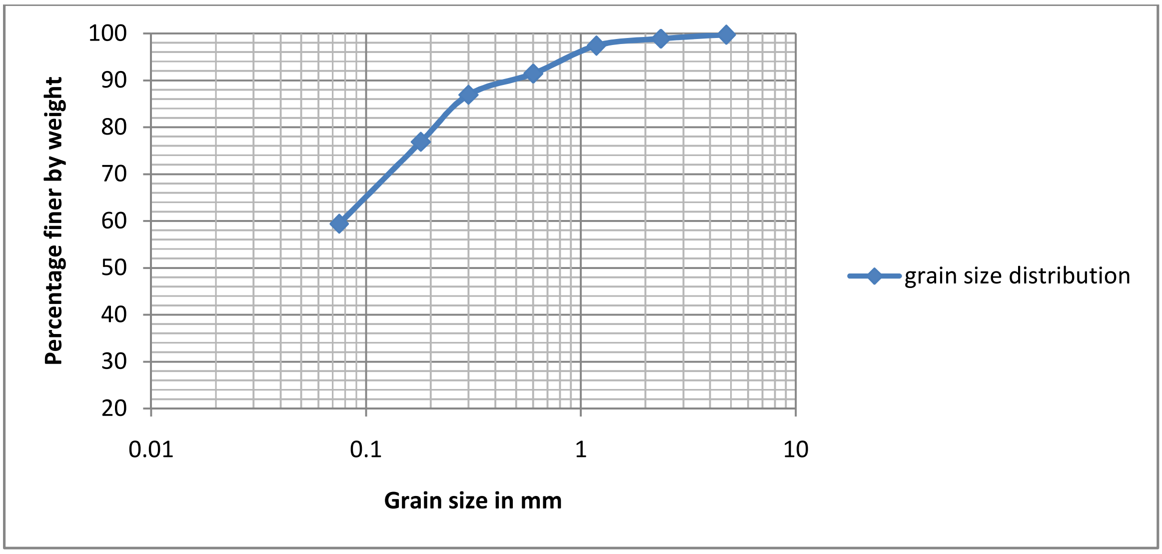


Figure 6: Grain size distribution of soil particle

As more than 50% is passing 75 micron sieve, it is a fine grained soil and hydrometer analysis is carried out.

4.3 Particle size distribution using Hydrometer Analysis (IS 2720 Part 4 1985)

The hydrometer method is based on the measurement of velocity of soil particles in a sedimentation solution and the dry mass of soil in the solution in different intervals of time. The velocity of falling particles and dry mass of soil at a specific depth are measured by a hydrometer. The results are combined with Stokes' law, which gives the relation between velocity of a spherical particle and its diameter while settling within its solution. The tests are carried out according to procedure mentioned in IS 2720 Part 4 1985.

Picture 4: Hydrometer analysis

Calibration of hydrometer:

Initial reading = 760mL

Final reading = 850mL

Volume of hydrometer, VH = 90 mL

Area of cross section of the cylinder, A = ((700-600))/(2.8) = 35.71 cm2

Height of bulb, h = 15.5 cm

Table 4: Calibration of hydrometer

Actual Hydrometer reading (R_H)	**Distance between neck to each mark on hydrometer (H) in cm**	**Effective depth $H_e = H + \frac{1}{2}(h - \frac{V_h}{A})$ in cm**
25(1025)	1.7	8.2
20(1020)	3.4	9.9
15(1015)	5.1	11.6
10(1010)	6.8	13.3
5(1005)	8.5	15
0(1000)	10.2	16.7
-5(995)	11.9	18.4

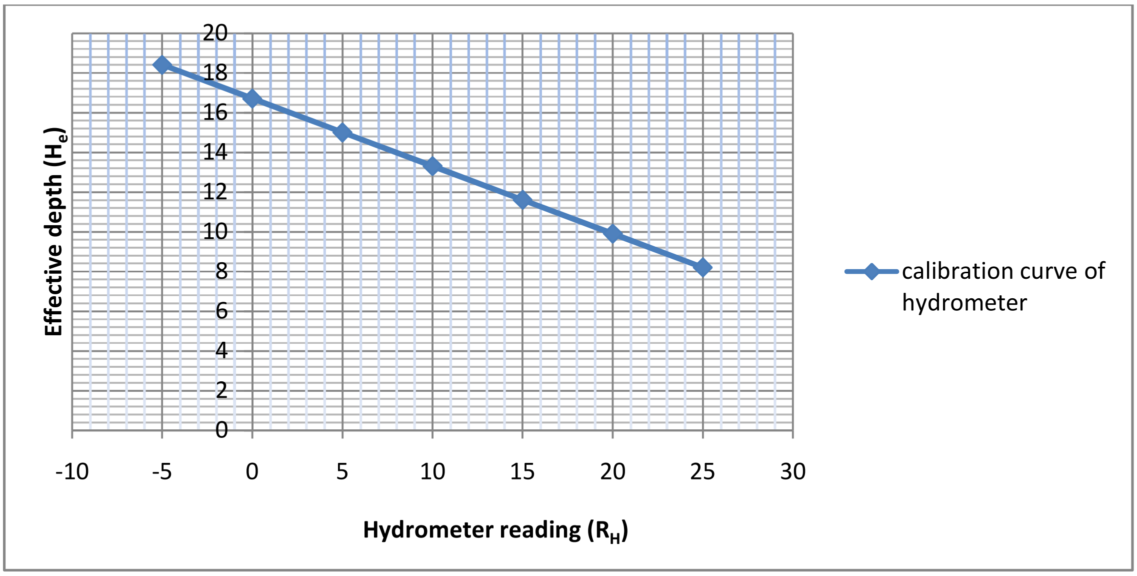


Fig7: Calibration curve of hydrometer

Observations: Mass of dry soil (M_s) = 50g; Meniscus correction (C_m) = +0.5

Specific gravity of solids (G) = 2.69; Composite correction C = -0.5

Table 5: Data sheet for hydrometer test

Sl. No.	Observations				Calculations					
	Elapsed time(t)	**Hydrometer reading (R_H)**	**Temperature**	**Composite correction (C)**	**Corrected hydrometer reading $R_H' = R_H + C_m$**	**Height H_e (cm)**	**Reading R= R_H + C**	**Factor M**	**Particle size (mm) $D = M\sqrt{\frac{H_e}{t}}$**	**Percentage finer (N)**
1	0.5 min	10	27^0C	-0.5	10.5	13.3	9.5	0.01258	0.0639	30.24
2	1 min	9.75	27^0C	-0.5	10.25	13.39	9.25	0.01258	0.0453	29.44
3	2 min	9.5	27^0C	-0.5	10	13.47	9	0.01258	0.0322	28.65
4	4 min	8.75	27^0C	-0.5	9.25	13.73	8.25	0.01258	0.0229	26.26
5	8 min	7.75	27^0C	-0.5	8.25	14.07	7.25	0.01258	0.0164	23.07
6	15 min	7.25	27^0C	-0.5	6.75	14.24	6.75	0.01258	0.0120	21.49
7	30 min	6.75	27^0C	-0.5	7.25	14.41	6.25	0.01258	0.00859	19.89
8	1hr	6	27^0C	-0.5	6.5	14.67	5.5	0.01258	0.00436	17.5
9	2 hr	5.5	25^0C	-0.5	6	14.83	5	0.01258	0.0031	15.9
10	4 hr	5	25^0C	-0.5	5.5	15	4.5	0.01258	0.00281	14.32
11	8 hr	4	25^0C	-0.5	4.5	15.33	3.5	0.01258	0.00224	11.14
12	12 hr	3	25^0C	-0.5	3.5	15.56	2.5	0.01258	0.00185	7.96
13	24 hr	1.5	25^0C	-0.5	3	15.73	1	0.01258	0.00131	3.18

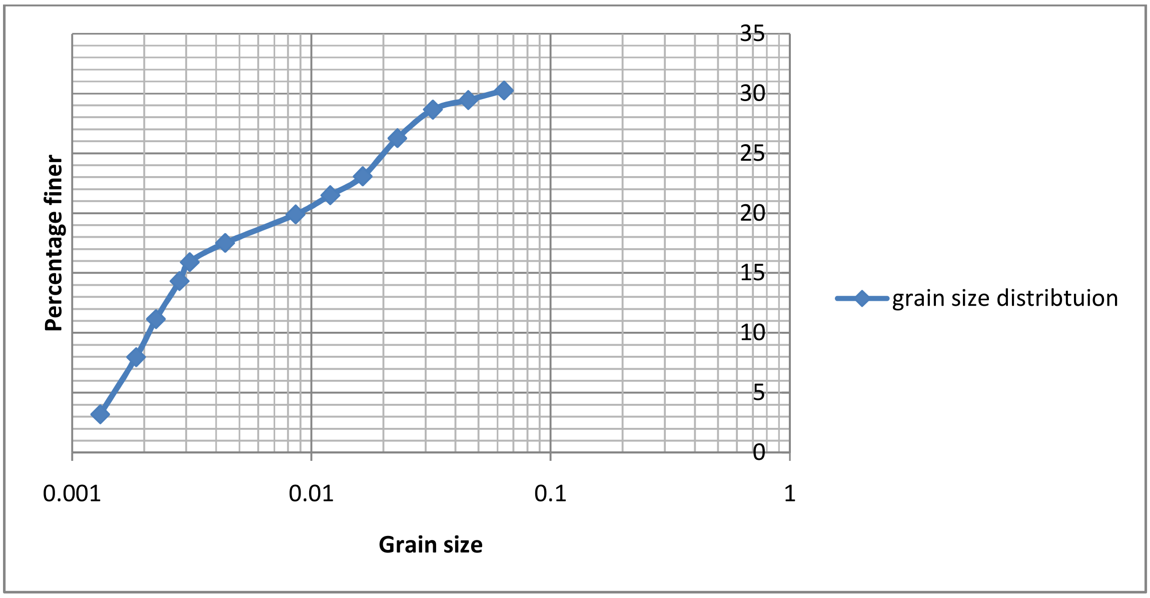


Fig8: Grain size distribution based on hydrometer analysis

From hydrometer analysis the soil has been identified as silty claywith approximately 9% clay content.

4.4 Atterberg's limits (IS 2720 Part 5 1985)

4.4.1 Liquid Limit Test

For a fine grained soil consistency means the physical state in which it exists. Consistency is used to denote the degree of firmness of a soil. It is indicated by such terms as soft, firm or hard. In 1911, a Swedish agriculture engineer Atterberg mentioned that a fine grained soil can exist in four states, namely, liquid, Plastic, semi-solid and solid state.

The water content at which the soil changes form one state to another are known as consistency limits or Atterbergs limits. The water content alone is not an adequate index property of a soil. At the same water content, one soil may be relatively soft; whereas other soil may be hard. However, the soils with same consistency limit behave somewhat in similar manner. Thus, consistency limits are very important index properties of fine grained soils. A soil containing high water content is in a liquid state. It has no resistance to shear deformation and therefore, the shear strength is equal to zero. As the water content is reduced, the soil becomes stiffer and resistance to shear deformation is gradually developed. At particular water content, the soil becomes plastic.

The water content at which the soil changes from the liquid state to plastic state is known as liquid limit. In other words the liquid limit is the water content at which the soil ceases to be liquid.

Table 6: Liquid limit determination

Sl. No.	**Determination No.**	**1**	**2**	**3**	**4**
1	Number of blows	35	31	23	10
2	Mass of container (M_1)g	6.54	6.01	5.37	5.63
3	Mass of container + wet soil (M_2)g	20.96	21.98	22.38	16.91
4	Mass of container + Dry soil (M_3)g	18.34	18.68	18.26	13.32
5	Mass of water (M_2- M_3)g	2.62	3.3	4.12	3.59
6	Mass of oven dry soil (M_3- M_1)g	11.8	12.67	12.89	7.69
7	Water content (%) = $\frac{M_2-M_3}{M_3-M_1} \times 100$	22.22	26.04	31.96	46.68

The flow curve is plotted on semi log graph with water content as the ordinate and no. of blows as abscissa. The water content corresponding to 25 blows is taken as the liquid limit of the soil.

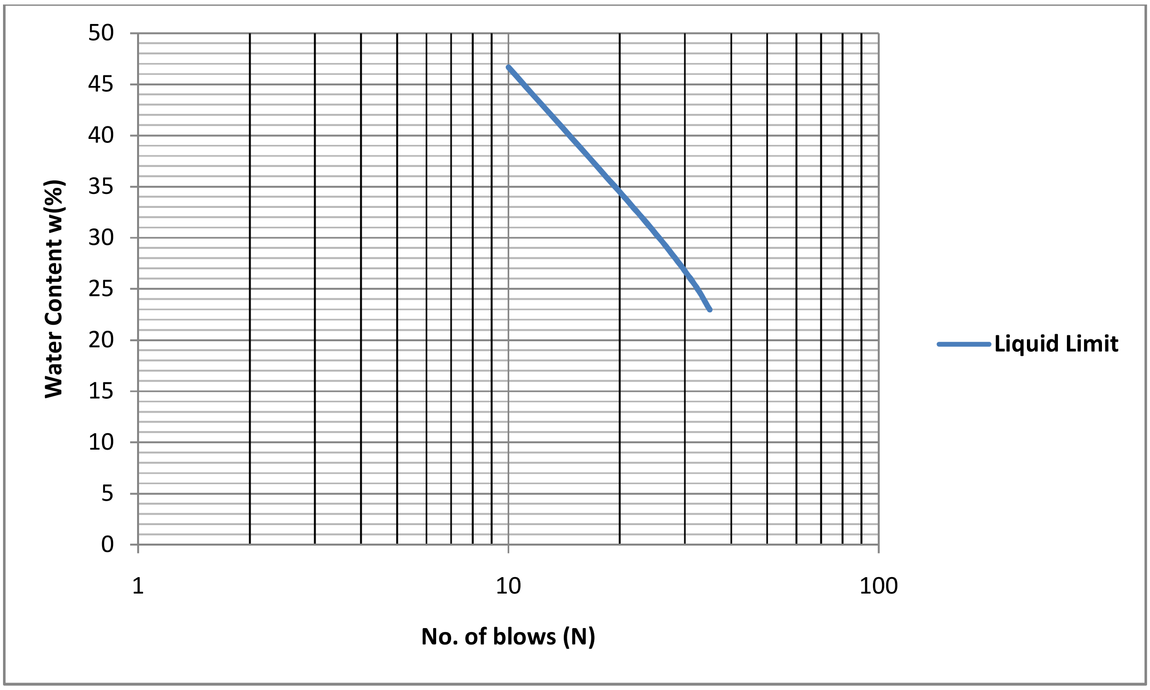


Figure 9: No. of blows v/s water content graph

Results:Liquid Limit W_L (from graph) = 30 %

4.4.2 Plastic Limit Test:

Plastic limit is the minimum water content at which a soil just begins to crumble when rolled into a thread of 3 mm in diameter. This water content in is between the plastic and semi-soil states of soil.

4.4.2.1Observations and Calculations:

Table 7: Calculation of plastic limit

Sl. No.	Observations and Calculations	Determination No.		
		1	2	3
1	Mass of empty container (M_1)g	6.54	6.01	5.37
2	Mass of container + wet soil (M_2)g	10.12	10.68	11.02
3	Mass of container + dry soil (M_3)g	9.46	9.89	10.05
4	Mass of water = $M_2 - M_3$ (g)	0.66	0.79	0.97
5	Mass of dry soil = $M_3 - M_1$ (g)	2.92	3.68	4.78
6	Water content = $\frac{(4)}{(5)} \times 100\%$	22.60	21.46	20.29

Result:

Average Plastic limit of soil = 21.45%

Plasticity Index = Liquid Limit – Plastic Limit = 8.55%

4.5Determination of compaction properties of soil layer

4.5.1 StandardProctor Test(IS 2720(VII):1980) The Proctor compaction test is a laboratory method of experimentally determining the optimal moisture content at which a given soil type will become most dense and achieve its maximum dry density. The test is carried out according to procedure mentioned in IS 2720 Part VII, 1980.

4.5.2 Calculations:

Volume of mould = 981.75 cu.cm

Specific gravity of soil particles = 2.69

Table No. 8: Optimum Moisture Content Calculation

Sl. No	**Observations and Calculations**	**Determination No.**			
		1	**2**	**3**	**4**
1	Mass of empty mould + base plate	4275g	4275g	4275g	4275g
2	Mass of mould + base plate + compacted soil	6010g	6190g	6235g	6220g
3	Mass of compacted soil (2)-(1)	1735g	1915g	1960g	1945g
4	Bulk Density ρ = M/V	1.76g/cc	1.95g/cc	2.0g/cc	1.98g/cc
5	Water Content (w)	8.7%	10.92%	13.91%	17.13%
6	Dry Density $\rho_d = \frac{\rho}{1+w}$	1.62g/cc	1.76g/cc	1.78g/cc	1.69g/cc
7	Void Ratio, $e = \frac{G\rho_w}{\rho_d} - 1$	0.63	0.5	0.48	0.56

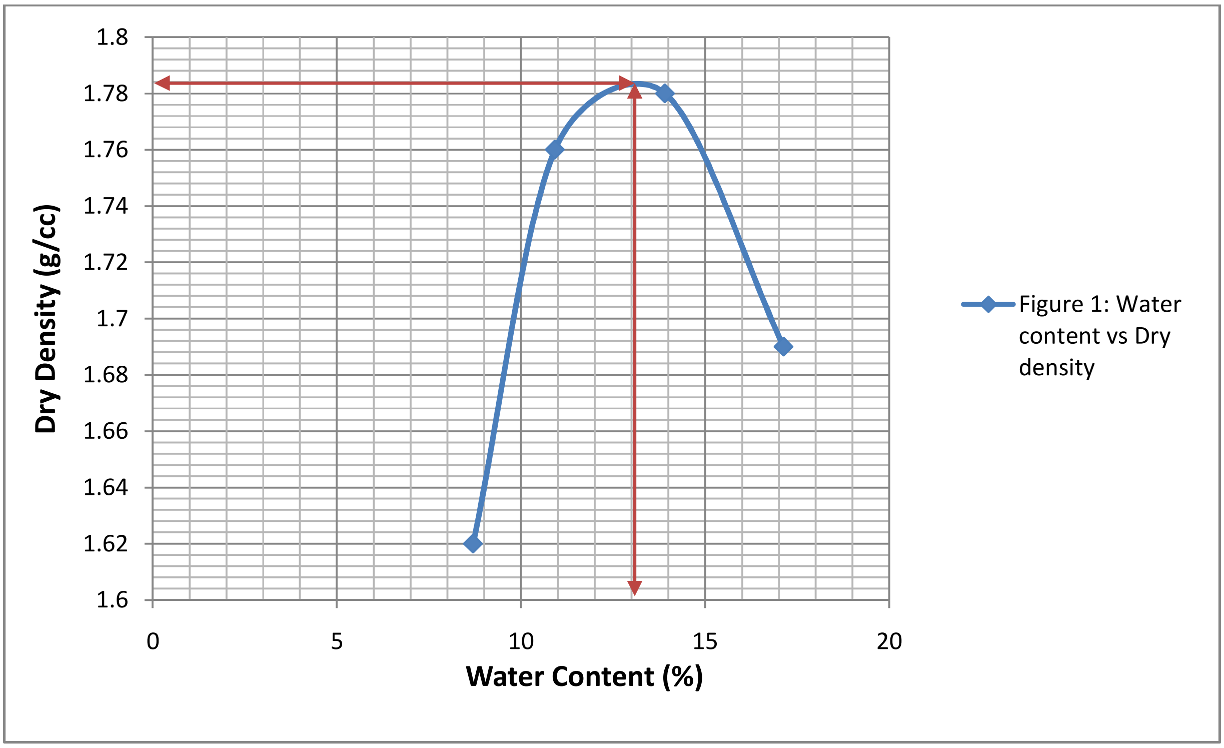


Figure 10: Water content v/s dry density graph

Thus, from the above drawn curve it is clear that:

(a). Maximum Dry Density of given soil layer is 1.78 g/cc = 17.46 kN/m^3

(b).Optimum Moisture Content (OMC) of soil layer is 13.91%

(c).Void Ratio of Compacted sand layer = e = (G. $_w$/ $_d$) -1 = 0.51

4.6 Direct Shear Test (IS 2720 Part 13, 1986)

The Direct shear test is a common test used to study the strength parameter of the soil. The test is carried out according to the provisions laid out in IS 2720 Part 13, 1986. The test is carried out at normal loads of 50kN/m^2, 100kN/m^2 and 150kN/m^2. The sample is sheared along the horizontal plane between the two halves. The test is continued until the specimen fails.

Table 9: Data Sheet for shear stress calculation under 50kN/m^2 normal loading

Sl. No.	Load in N	Horizontal displacement in mm	Normal stress in kN/m^2	Corrected area in cm^2	Shearing stress in kN/m^2
1	0	0	50	36.00	0
2	5	0.16	50	35.90	1.39
3	15	0.31	50	35.81	4.19
4	33	0.45	50	35.73	9.24
5	50	0.57	50	35.66	14.02
6	75	0.63	50	35.62	21.05
7	95	0.7	50	35.58	26.70
8	118	0.85	50	35.49	33.25
9	131	0.95	50	35.43	36.97
10	140	1.02	50	35.39	39.56
11	160	1.15	50	35.31	45.31
12	170	1.24	50	35.26	48.22
13	178	1.37	50	35.18	50.59
14	185	1.45	50	35.13	52.66
15	192	1.68	50	34.99	54.86
16	196	1.85	50	34.89	56.17
17	197	2.04	50	34.78	56.64
18	198	2.17	50	34.70	57.063
19	199	2.31	50	34.61	57.49
20	200	2.45	50	34.53	57.92
21	202	2.51	50	34.49	58.56
22	203	2.72	50	34.37	59.06
23	206	2.89	50	34.27	60.11
24	208	3.07	50	34.16	60.89
25	209	3.26	50	34.04	61.39
26	209.5	3.42	50	33.95	61.71

Table 10: Data Sheet for shear stress calculation under 100kN/m^2 normal loading

Sl. No.	Load in N	Horizontal displacement in mm	Normal stress in kN/m^2	Corrected area in cm^2	Shearing stress in kN/m^2
1	0	0	100	36.00	0
2	11	0.09	100	35.95	3.06
3	22	0.24	100	35.86	6.13
4	52	0.32	100	35.81	14.52
5	74	0.37	100	35.78	20.68
6	100	0.42	100	35.75	27.97
7	125	0.57	100	35.66	35.05
8	150	0.67	100	35.60	42.13
9	160	0.79	100	35.53	45.037
10	171	0.84	100	35.50	48.17
11	185	0.92	100	35.45	52.18
12	198	1.01	100	35.39	55.94
13	210	1.09	100	35.35	59.41
14	225	1.17	100	35.30	63.74
15	240	1.29	100	35.23	68.13
16	252	1.39	100	35.17	71.66
17	261	1.48	100	35.11	74.33
18	269	1.6	100	35.04	76.76
19	277	1.75	100	34.95	79.25
20	285	1.92	100	34.85	81.78
21	294	2.09	100	34.75	84.61
22	298	2.2	100	34.68	85.92
23	303	2.35	100	34.59	87.59
24	314	2.52	100	34.49	91.04
25	317	2.65	100	34.41	92.12
26	319	2.8	100	34.32	92.94
27	319.5	2.92	100	34.25	93.29

Table 11: Data Sheet for shear stress calculation under 150kN/m^2 normal loading

Sl. No.	Load in N	Horizontal displacement in mm	Normal stress in kN/m^2	Corrected area in cm^2	Shearing stress in kN/m^2
1	0	0	150	36	0
2	15	0.1	150	35.94	4.17
3	25	0.17	150	35.90	6.96
4	55	0.28	150	35.83	15.34
5	85	0.43	150	35.74	23.78
6	130	0.51	150	35.69	36.42
7	150	0.61	150	35.63	42.09
8	195	0.72	150	35.57	54.82
9	225	0.8	150	35.52	63.34
10	250	0.89	150	35.47	70.49
11	280	1.02	150	35.39	79.12
12	305	1.11	150	35.33	86.31
13	325	1.2	150	35.28	92.12
14	340	1.32	150	35.21	96.56
15	355	1.41	150	35.15	100.98
16	369	1.53	150	35.08	105.18
17	386	1.67	150	35.00	110.29
18	405	1.83	150	34.90	116.03
19	415	1.92	150	34.85	119.08
20	420	2.08	150	34.75	120.85
21	423	2.12	150	34.73	121.80
22	426	2.2	150	34.68	122.83
23	428	2.31	150	34.61	123.64
24	430	2.42	150	34.55	124.46
25	432	2.5	150	34.50	125.21
26	432	2.59	150	34.45	125.41
27	433	2.65	150	34.41	125.83

4.6.1 Observations and Calculations

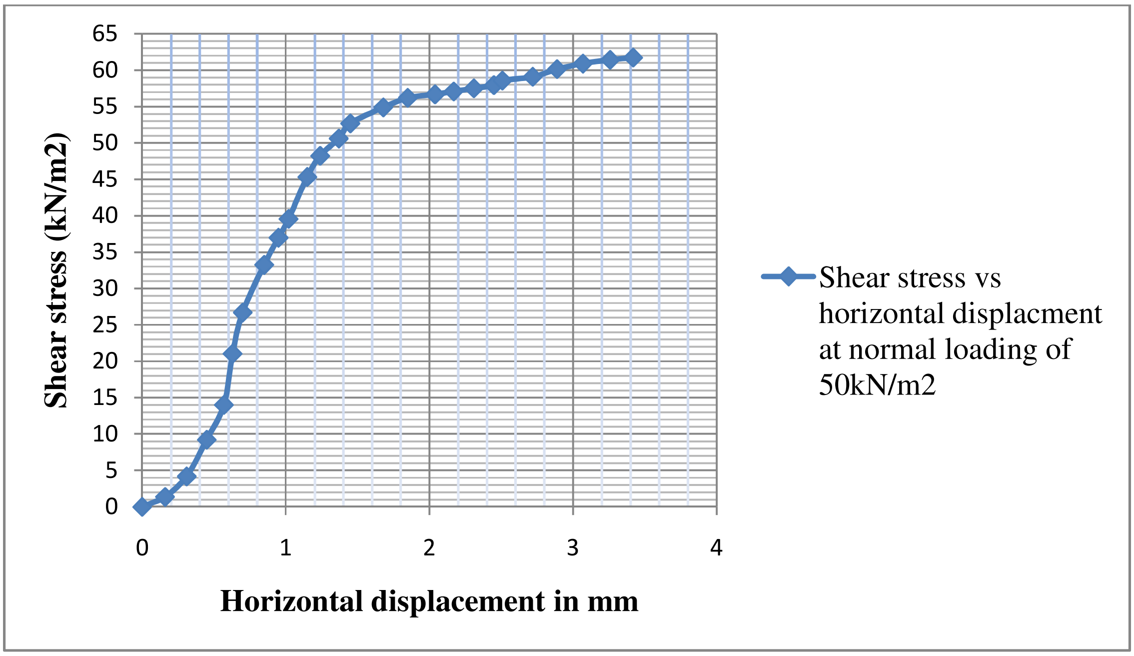


Fig 11: Shear stress v/s horizontal displacement graph for a normal loading of $50kN/m^2$

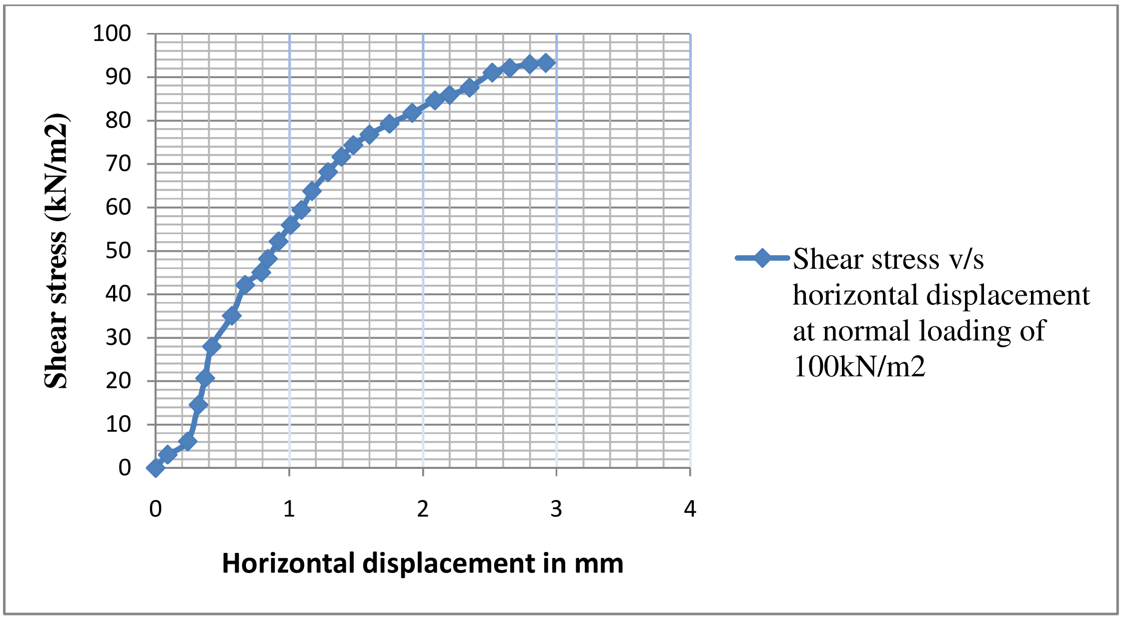


Fig 12: Shear stress v/s horizontal displacement graph for a normal loading of $100kN/m^2$

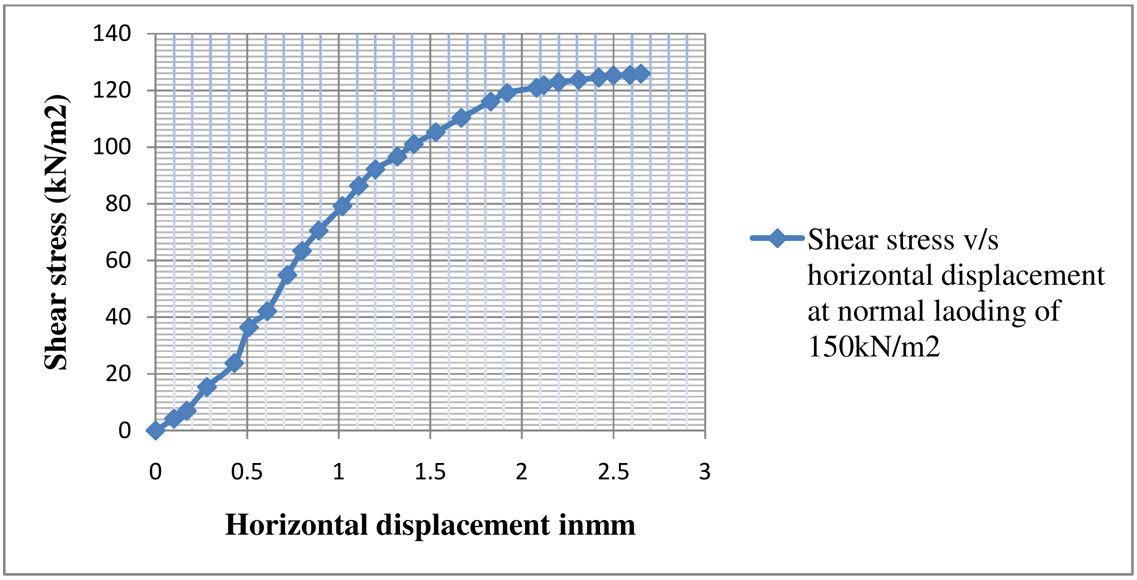


Fig 13: Shear stress v/s horizontal displacement graph for a normal loading of 150kN/m^2

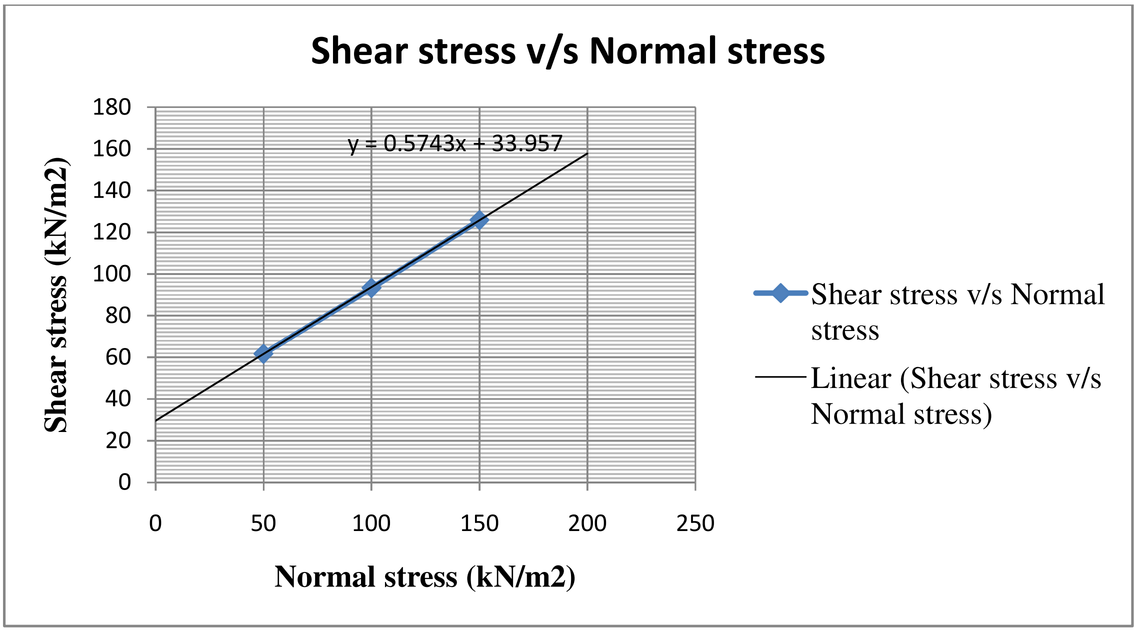


Fig 14: Shear stress v/s Normal stress graph

From graph, we have

$\Phi = 29.1^0$ and $c = 33.96$ kN/m^2

4.7 Unconfined Compressive strength test (IS 2720 Part 10)

The load transfer from foundation to sub-soil is dependent on the resistance offered by the underlying soil to shearing deformation or compressibility. The shearing strength is computed with the help of compression testing by application of axial load to the soil specimen until failure. The use of compression tests to investigate the shearing strength makes it possible to compute normal pressure and shearing stress as failure in such tests takes place by shear on one or more inclined planes. The unconfined compressive strength (qu) is the load per unit area at which the specimen of soil fails in compression. The procedure for testing is adopted using IS2720 Part 10

4.7.1 Observations and Calculations

Water content = 14%

Dry density(γ_d) from compaction curve = 17.46kN/m^3

Volume of soil sample taken = 8.61×10^{-5} m^3 (dia. 38 mm, length 76 mm)

Initial length L_o = 76mm,   Initial area A_o= 1133.54 mm^2

Table 12: Unconfined compressive strength determination

Dial Gauge Readings	Load (kN)	Displacement (Dial gauge Reading x 0.01)mm	Strain (ε)	Corrected Area (A), mm^2	Compressive stress (q), kN/m^2
0	0	0	0	1133.54	0
10	0.01	0.1	0.0013	1135.03	8.810
20	0.02	0.2	0.0026	1136.53	17.597
30	0.03	0.3	0.0039	1138.03	26.361
40	0.04	0.4	0.0053	1139.54	35.102
60	0.05	0.6	0.0079	1142.56	43.761
90	0.07	0.9	0.0118	1147.12	61.022
140	0.1	1.4	0.0184	1154.81	86.594
190	0.12	1.9	0.025	1162.61	103.216
240	0.13	2.4	0.0316	1170.50	111.063
290	0.14	2.9	0.0382	1178.51	118.794
340	0.13	3.4	0.0447	1186.63	109.554
390	0.12	3.9	0.0513	1194.85	100.431
440	0.11	4.4	0.0579	1203.20	91.423
490	0.1	4.9	0.0645	1211.66	82.531

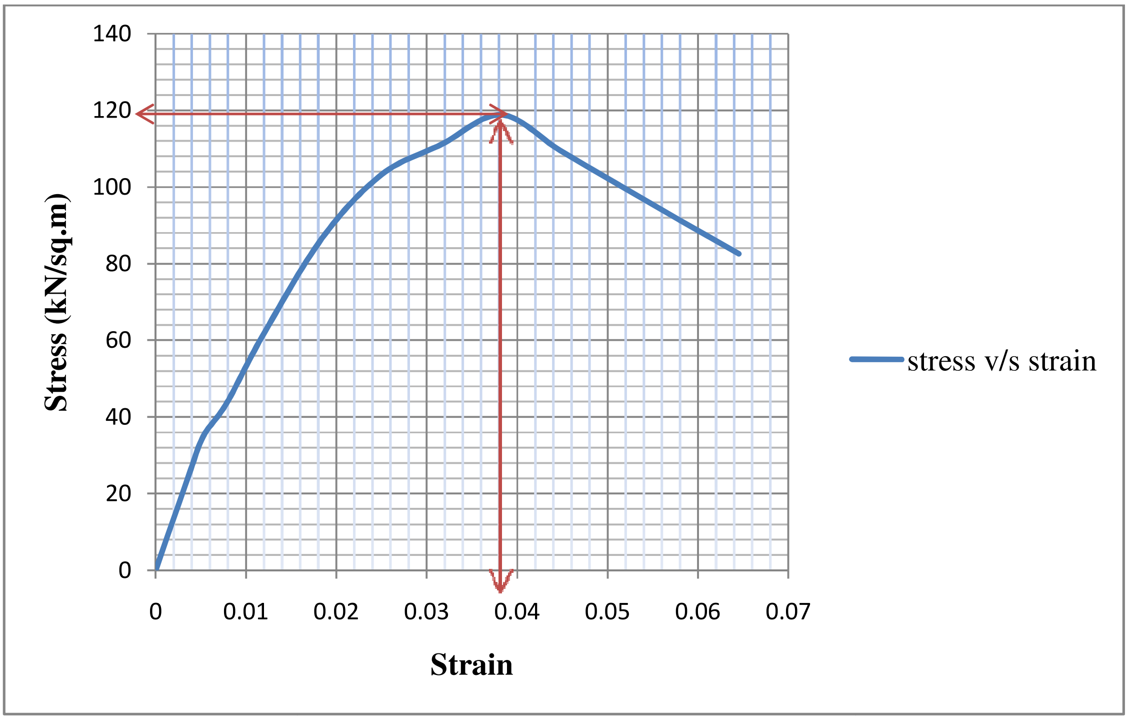


Fig. 15: Stress v/s strain plot of pure soil

From graph unconfined compressive strength is 118.8kN/m^2

And cohesion C $_=$ —

4.8 Specific gravity and Water absorption test on aggregates (IS 2386 Part 3, 1963)

The specific gravity of an aggregate is considered to be measure of the strength or quality of the material. Water absorption gives an idea of strength of aggregate. Aggregates having more water absorption are more porous in nature and are generally considered unsuitable unless they are found to be acceptable based on strength, impact and hardness test. Using the procedure given in IS 2386 Part 3, the tests are carried out.

Observations:

Specific gravity of coarse aggregate G = $\frac{a}{(b-c)}$

Absorption value of coarse aggregate = $\frac{(\mathrm{b}-\mathrm{a})\ 100}{a}$

Table 13: Table for specific gravity and water absorption test of coarse aggregate

Sl. No.	Particulars	Coarse aggregate
1	Weight of bucket in water (c1)	600 g
2	Weight of bucket + aggregate in water (c2)	1900 g
3	Weight of aggregate in water (c= c2 – c1)	1300 g
4	Weight of aggregate in saturated surface dry condition (b)	2044.8 g
5	Weight of oven dry aggregate (a)	2030.9 g
6	Specific gravity	2.72
7	% Absorption	0.68

From table, Specific gravity of coarse aggregates = 2.72

Percentage water absorption = 0.68 %

4.9 Particle size analysis of aggregates (IS 2386 Part 1, 1963)

Sieving is carried out on a 2000g aggregate sample for particle size distribution according to procedure mentioned in IS 2386 Part I, 1963

Table 14: Sieve analysis for coarse aggregate

Sieve size	**Weight retained (g)**	**Percentage weight retained (%)**	**Cumulative percentage weight retained**	**Percent passing**
40mm	30.2	1.51	1.51	98.49
20mm	405.6	20.28	21.79	78.21
12.5mm	1150.3	57.52	79.31	20.69
10mm	407.8	20.39	99.7	0.3
4.75mm	6.1	0.31	100	0
Pan	0	0	100	0

4.10 Aggregate impact test (IS 2386 Part IV, 1963)

The aggregate impact test is used to determine aggregate impact value which gives a measure of toughness of the aggregates. The test is carried out according to procedure laid out in IS 2386 Part IV, 1963.

Table 15: Observation table for impact test on coarse aggregate

Sl. No.	**Weight of container (g)**	**Weight of container + aggregate (g)**	**Weight of aggregate (g)**	**Weight of aggregate passing 2.36 mm sieve A(g)**	**Weight of aggregate retained in 2.36mm sieve B(g)**	**Impact Value $\frac{A}{B}100$**
1	904.62	1260.24	355.62	74.5	281.9	26.5%
2	904.62	1233.27	328.65	81.9	245.3	33.38%

From table, Aggregate impact value = (26.5+33.38)/2 = 29.95%

Chapter 5
Experiments on Silty Clay bed

5.0 Experiments on silty clay bed

The experimental study is carried out with the following objective of plotting the load v/s settlement plot of silty clay bed both with and without stone column reinforcement.

Soil was collected from Delhi Technological University campus by removing the surface vegetation. A total quantity of 1.5 cum. of soil was excavated. The soil was air dried and pulverized. The soil was passed through 4.75mm sieve. A total quantity of 1125 kilograms of soil was mixed with 10% water in small batches to obtain optimum moisture content. The dimensions of rectangular tank used for filling the soil are 1.5m×0.6m×0.9m. The tank was filled up to 0.75m height. The tank is made of steel plate of thickness 2mm with the joints welded. The application of load does not affect the walls of the stone column as the diameter of the stone column is so chosen so as to keep its effective area within safe distance from the walls of the rectangular tank.

Density of soil in tank = 16.15kN/m^3(By Core Cutter Method)

Picture 5: Rectangular tank soil filling (1.5m×0.6m×0.9m)

Picture 6: Mixing and weighing of soil with water in batches upto optimum moisture content

Picture 7: 1125 kilograms of soil filled in tank volume 1.5m×0.6m×0.75m

The tank was placed on top of reinforced concrete beams to provide a stiff base. The soil was placed in three layers and each layer was compacted with more than 100 blows to achieve uniform compaction. Care was taken to avoid entrapped air by tapping the clay layers gently with a wooden plank.

Loading is applied with the help of a hydraulic jack. At first loading is applied on soil bed unreinforced with stone column and the load versus deformation curve is plotted. The size of circular plate used is 300mm. The loading is increased until failure takes place.

Picture 8: Loading applied on silty clay bed

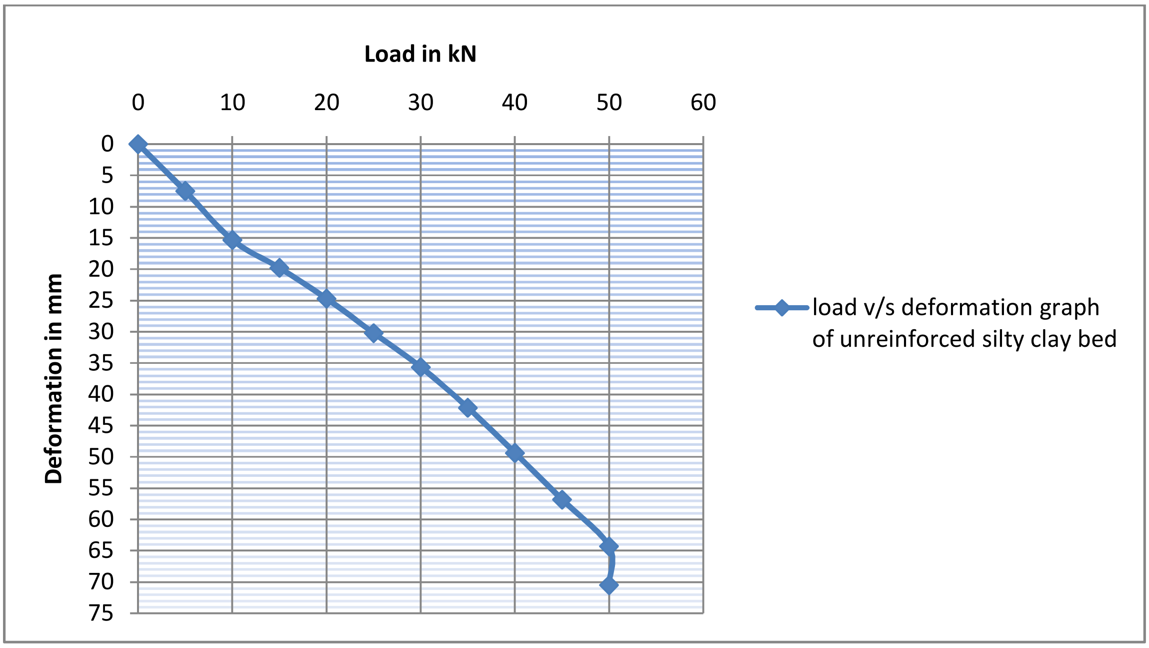


Fig 16: Load v/s deformation graph for unreinforced silty clay bed

Chapter 6
Experiments on Stone Column with and without Encasement

6.0 Tests on Stone Column

6.1 Experimental investigation

Tests were conducted on a single stone column of diameter 120mm for different length to diameter ratios on a loading frame as stress controlled test. The diameter of the circular steel plate of adequate thickness and rigidity is based on effective tributary soil area of the stone column as per the codal provisions given in IS 15824 (2003). The loading plate used in the test is circular having a diameter of 300mm which is more than twice the diameter of the stone column of 120mm. This size was used so that the loading plate will cover the equivalent circular effective area concentrically.

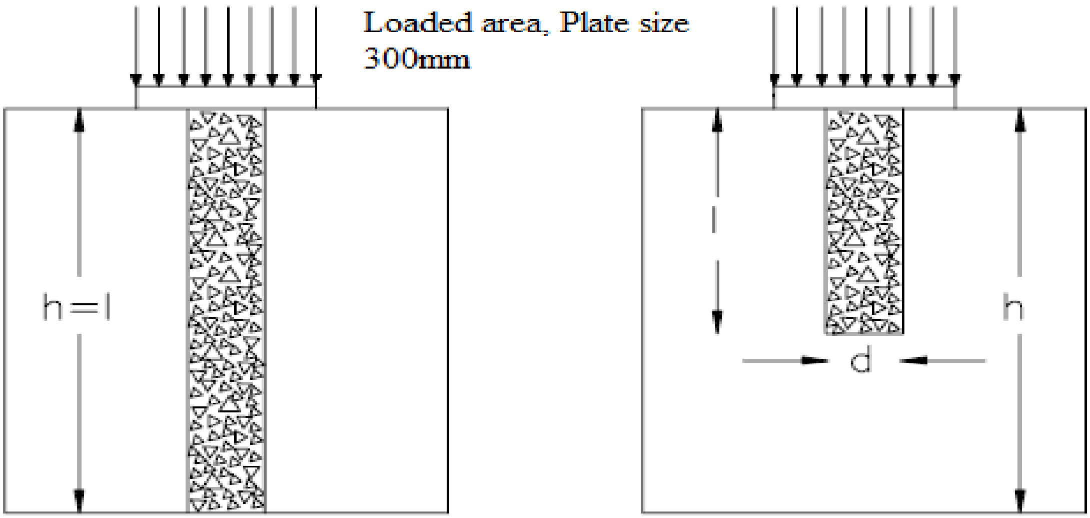
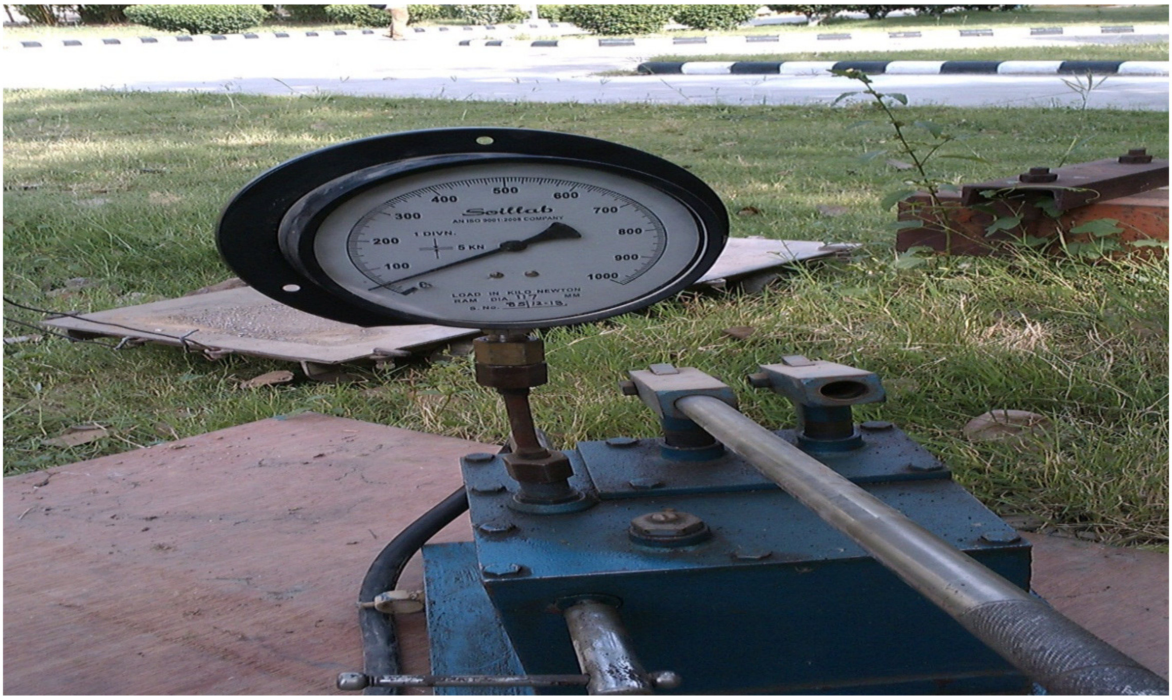


Fig 17: Loading of bed reinforced with stone column

The loading arrangement is shown in the figure above. Stone column was prepared by making a circular hole of diameter 120mm with length/diameter ratio 2.08, 4.16 and 5.83. The height of stone column prepared was 250mm, 500mm and 750mm respectively. Sufficient area was provided around the stone column so that there was no effect of loading on the tank walls. The size of aggregate used for the construction of stone column is 20mm, 10mm and a mixture of 10mm and 20mm aggregates in the ratio of 1:1 by weight. Load tests were also carried out on encased stone columns composed of 10mm, 20mm and a mixture of 10mm and 20mm aggregates in the ratio 1:1 by weight for full depth of 750mm. The total quantity of gravel placed for each length/diameter ratio was weighed and the dry density of the material as placed considering diameter of 120mm is shown in table below.

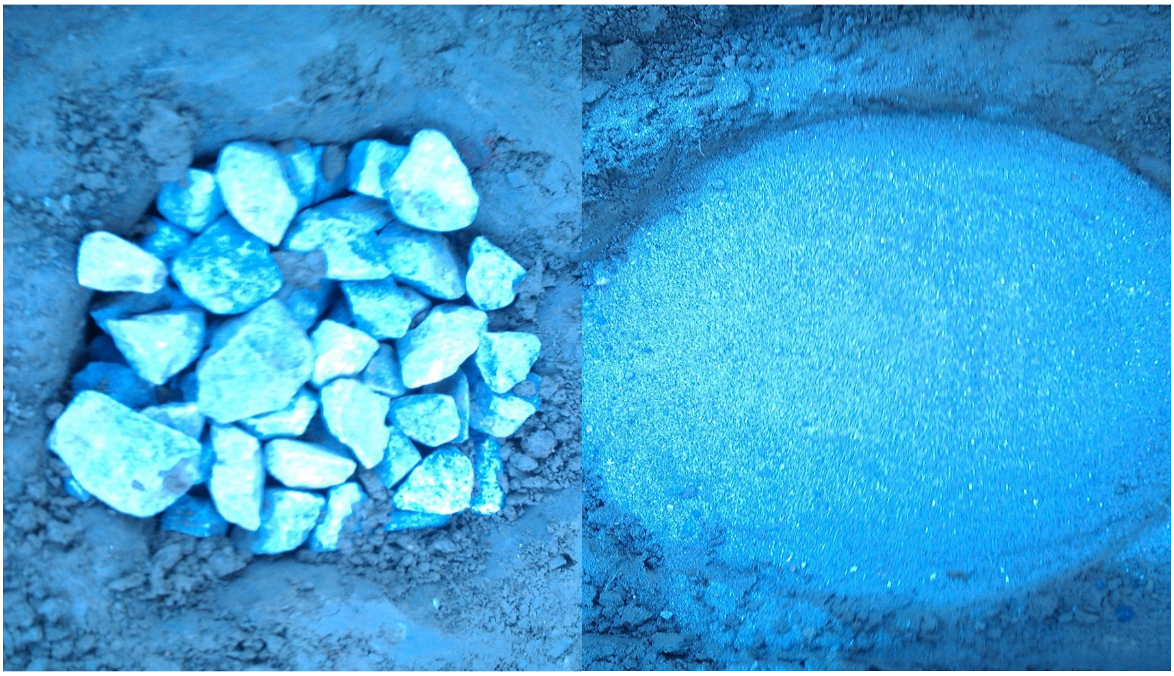
Table16: Density of stone for different gradation and depth of column

Sl. No.	Aggregate size	Length of column	Weight of aggregates	Diameter of column	Density
1	20mm	250mm	0.045kN	120mm	15.92kN/m^3
2		500mm	0.086kN	120mm	15.21kN/m^3
3		750mm	0.127kN	120mm	14.98kN/m^3
4	10mm	250mm	0.040 kN	120mm	14.15kN/m^3
5		500mm	0.084kN	120mm	14.86N/m^3
6		750mm	0.121kN	120mm	14.26kN/m^3
7	10mm+20mm (1:1 by weight)	250mm	0.043kN	120mm	15.21kN/m^3
8		500mm	0.084kN	120mm	14.85kN/m^3
9		750mm	0.125kN	120mm	14.74kN/m^3
10	20mm with encasement	750mm	0.129kN	120mm	15.21kN/m^3
11	10mm with encasement	750mm	0.124kN	120mm	14.62kN/m^3
12	10mm+20mm with encasement(1:1 by weight)	750mm	0.126kN	120mm	14.85kN/m^3



Picture 9: Load at failure in hydraulic jack

Picture 10: Sieves used for gradation of aggregates



Picture 11:Stone column with 20mm aggregate and sand layer on top of stone column

Picture 12: Stone column with 10mm aggregate and sand layer on top of stone column

Picture 13: Load application on stone column and stone column after failure

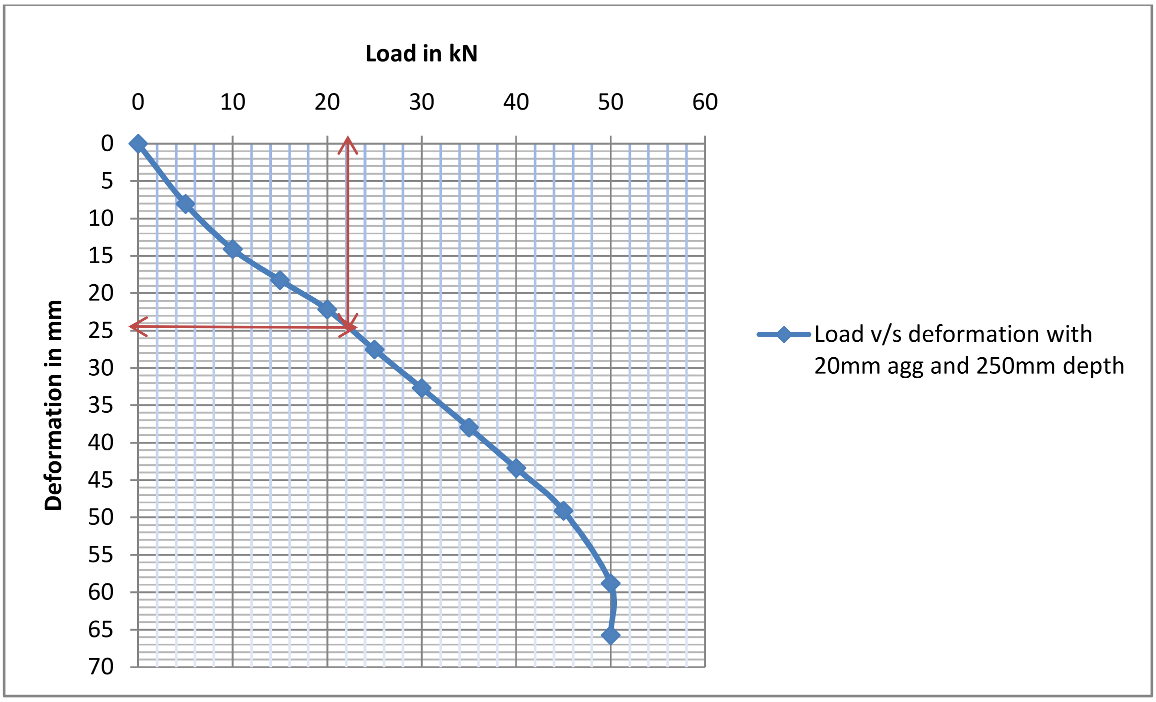


Fig 18: Load v/s deformation graph for stone column with 20mm aggregate and 250 mm depth

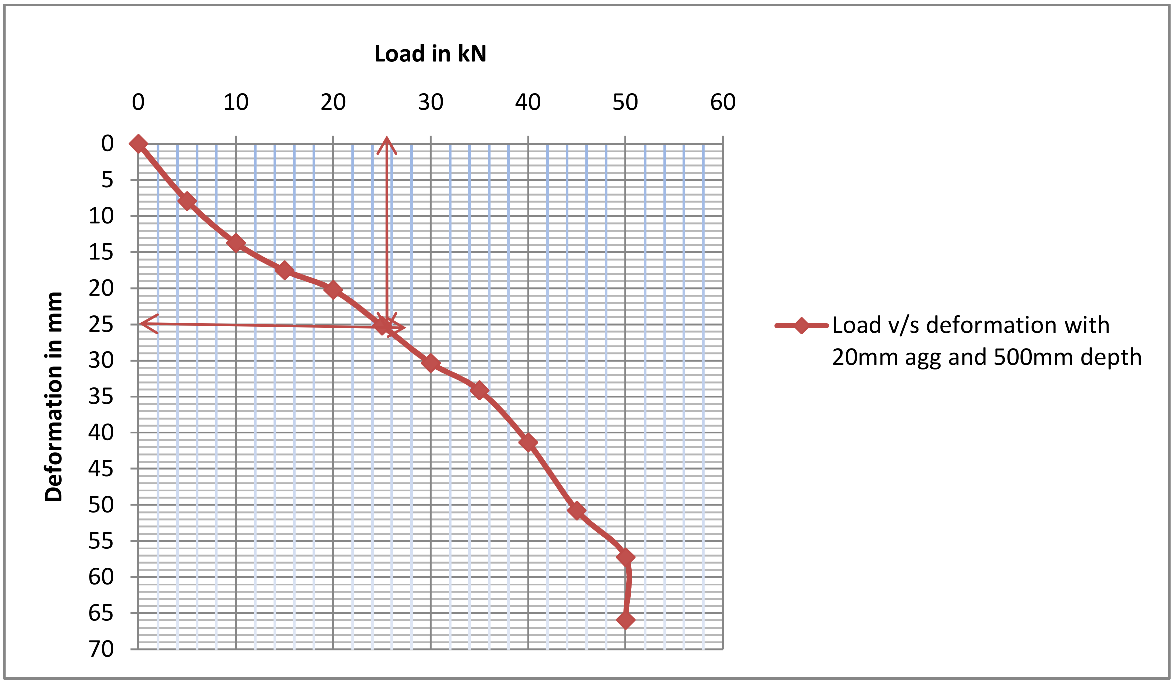


Fig 19:Load v/s deformation graph for stone column with 20mm aggregate and 500 mm depth

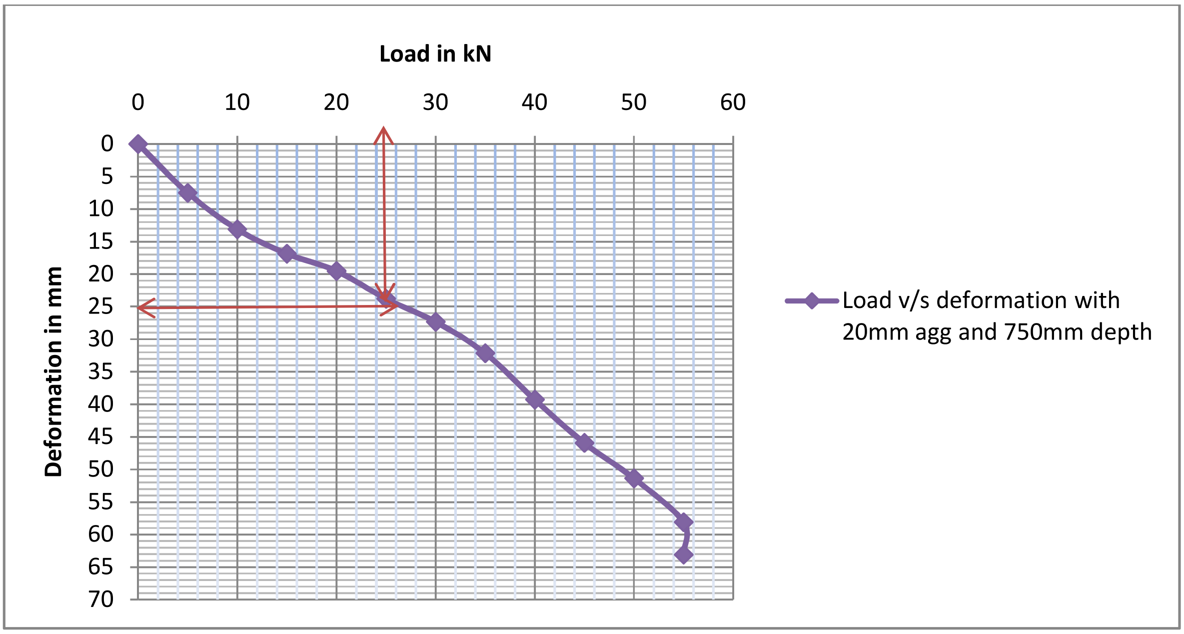


Fig 20:Load v/s deformation graph for stone column with 20mm aggregate and 750 mm depth

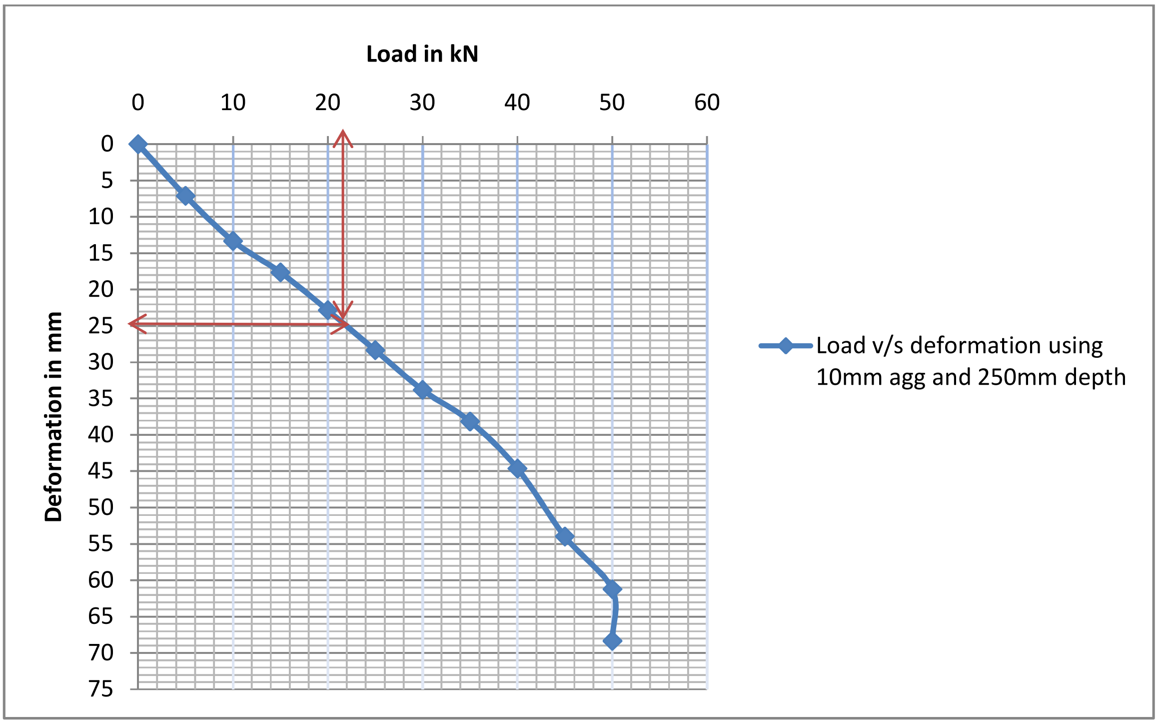


Fig 21:Load v/s deformation graph for stone column with 10mm aggregate and 250 mm depth

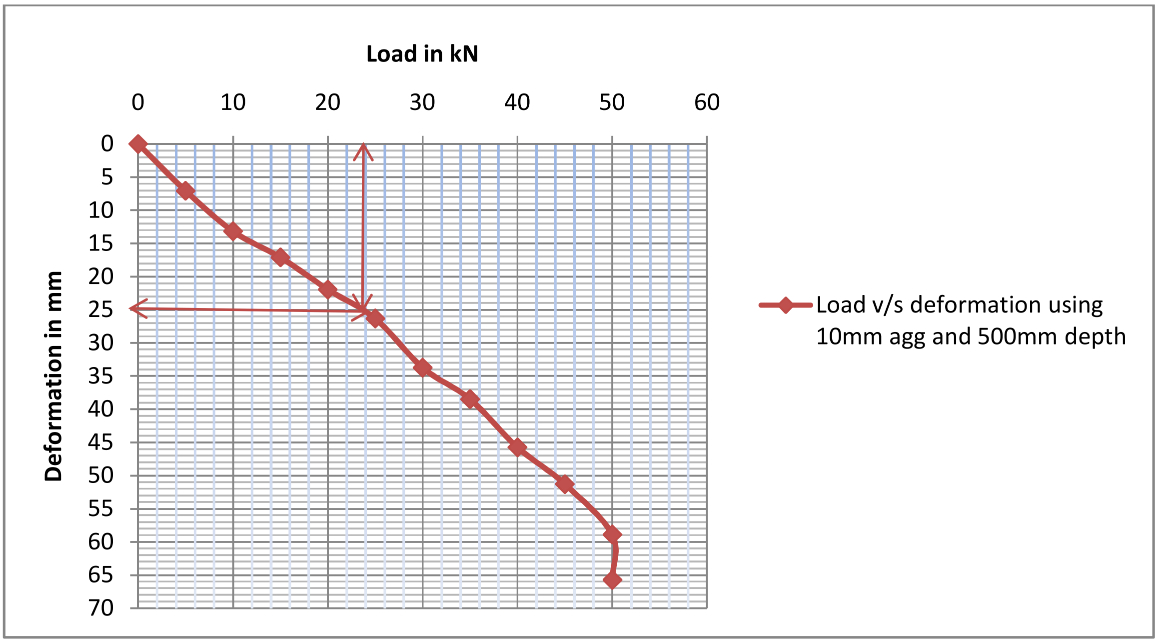


Fig 22:Load v/s deformation graph for stone column with 10mm aggregate and 500 mm depth

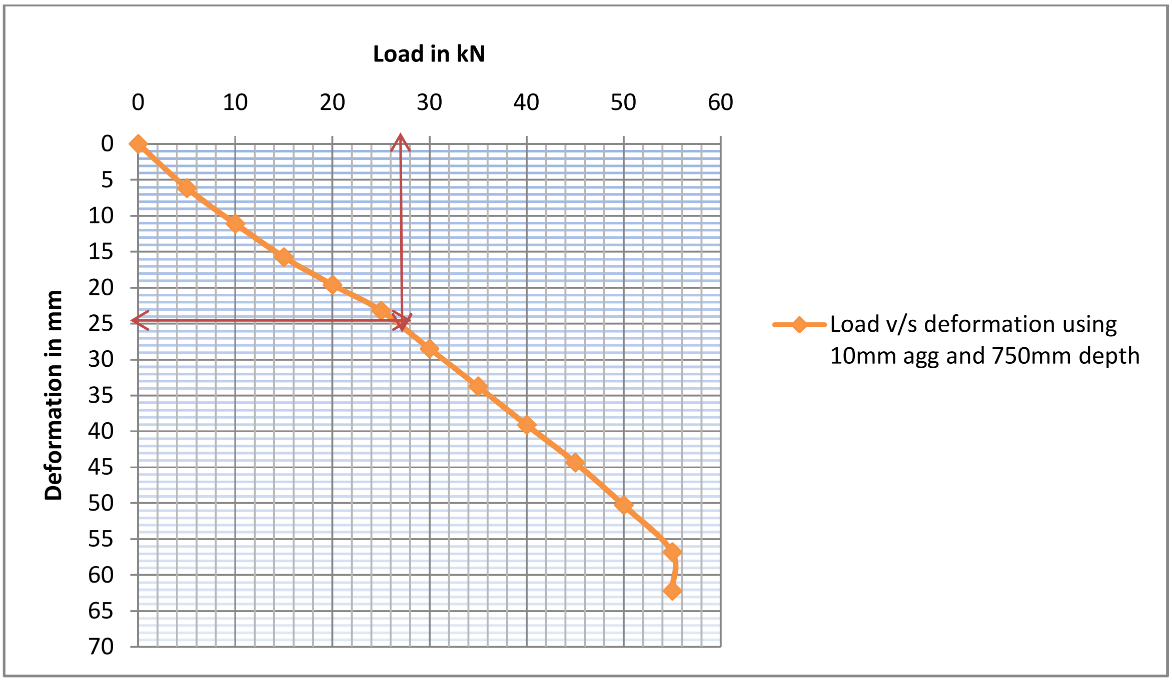


Fig 23: Load v/s deformation graph for stone column with 10mm aggregate and 750 mm depth

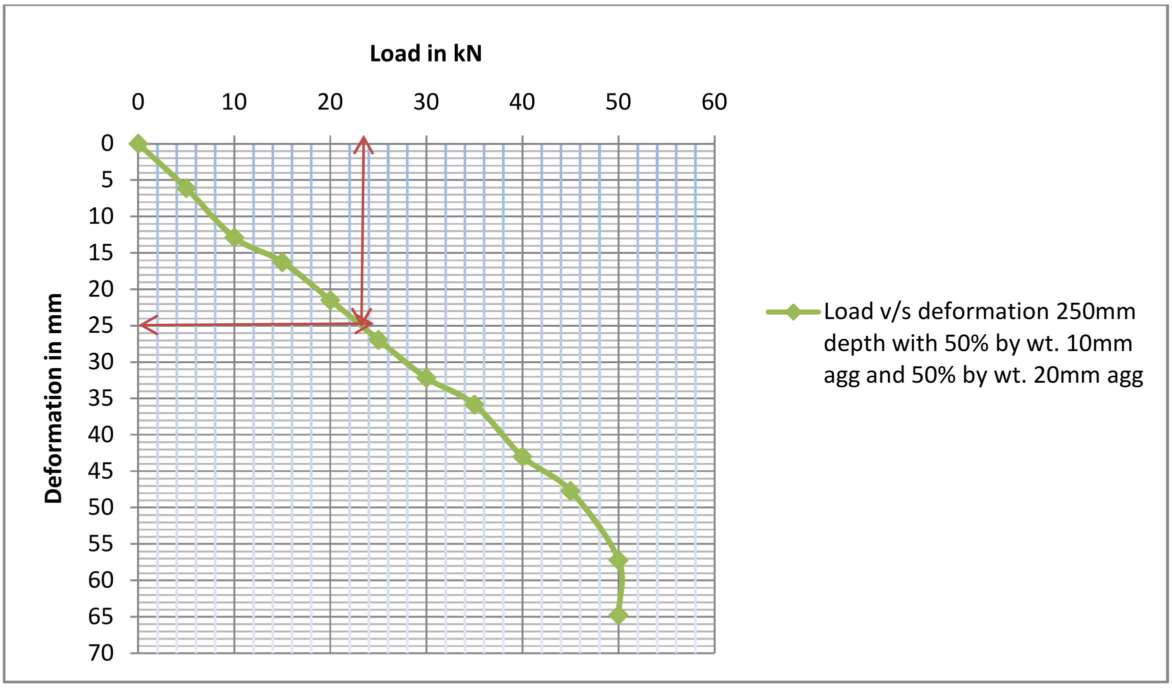


Fig24:Load v/s deformation graph for stone column with 10mm + 20mm aggregate in ratio 1:1 by weight and 250 mm depth

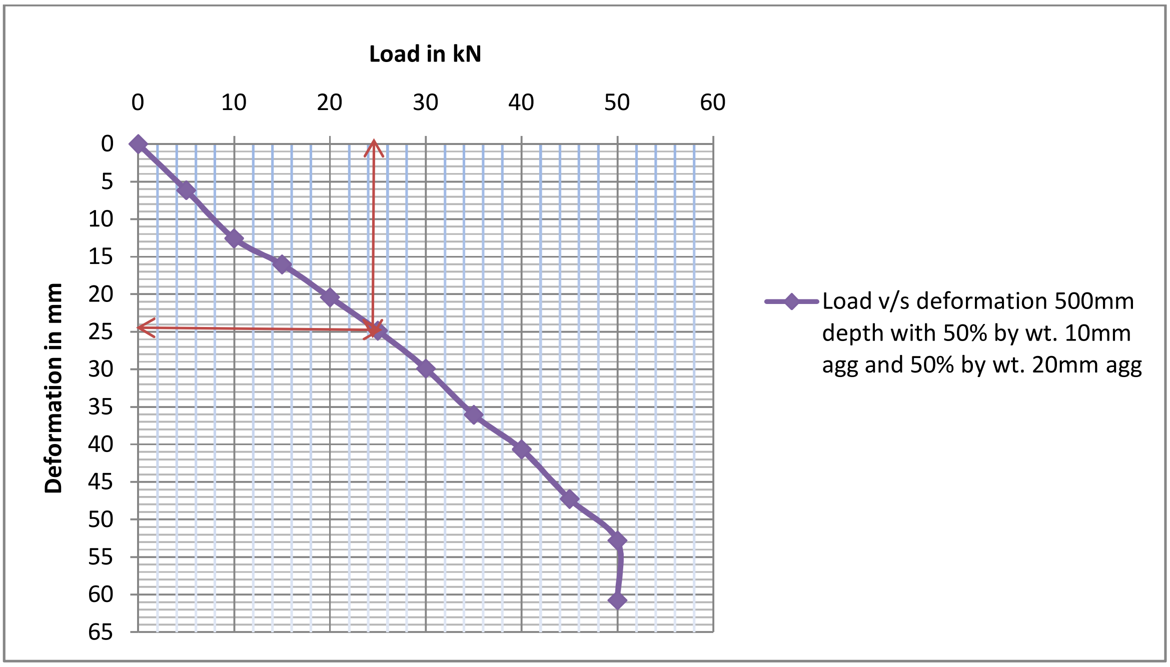


Fig 25:Load v/s deformation graph for stone column with 10mm + 20mm aggregate in ratio 1:1 by weight and 500 mm depth

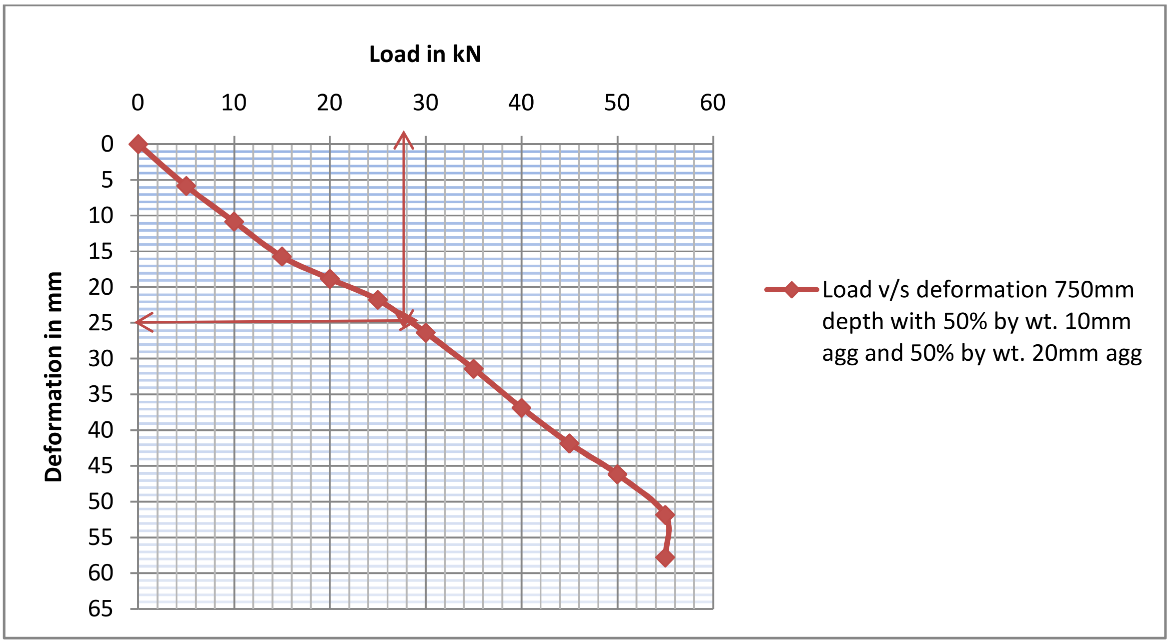


Fig 26:Load v/s deformation graph for stone column with 10mm + 20mm aggregate in ratio 1:1 by weight and 750 mm depth

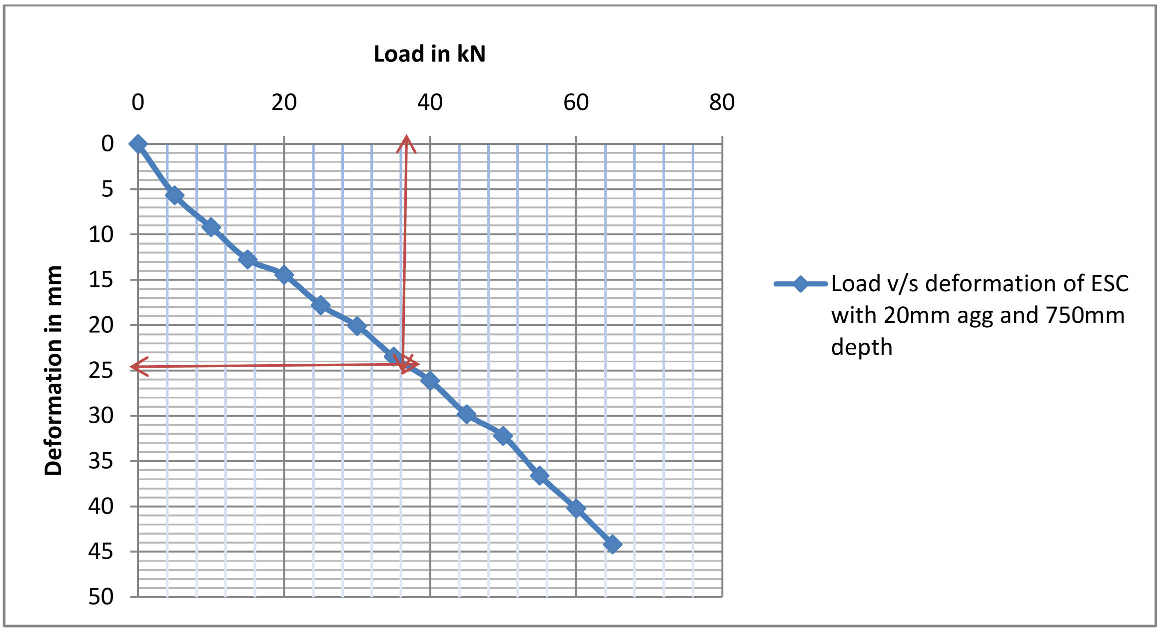


Fig 27:Load v/s deformation graph for encased stone column with 20mm aggregate and 750 mm depth

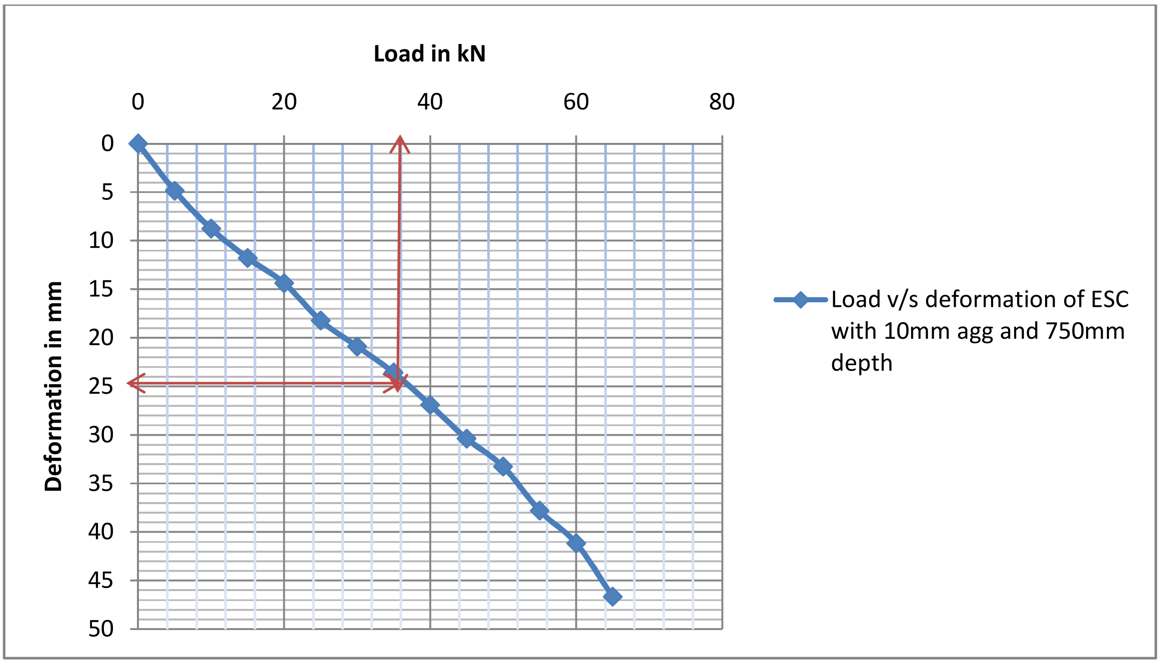


Fig 28:Load v/s deformation graph for encased stone column with 10mm aggregate and 750 mm depth

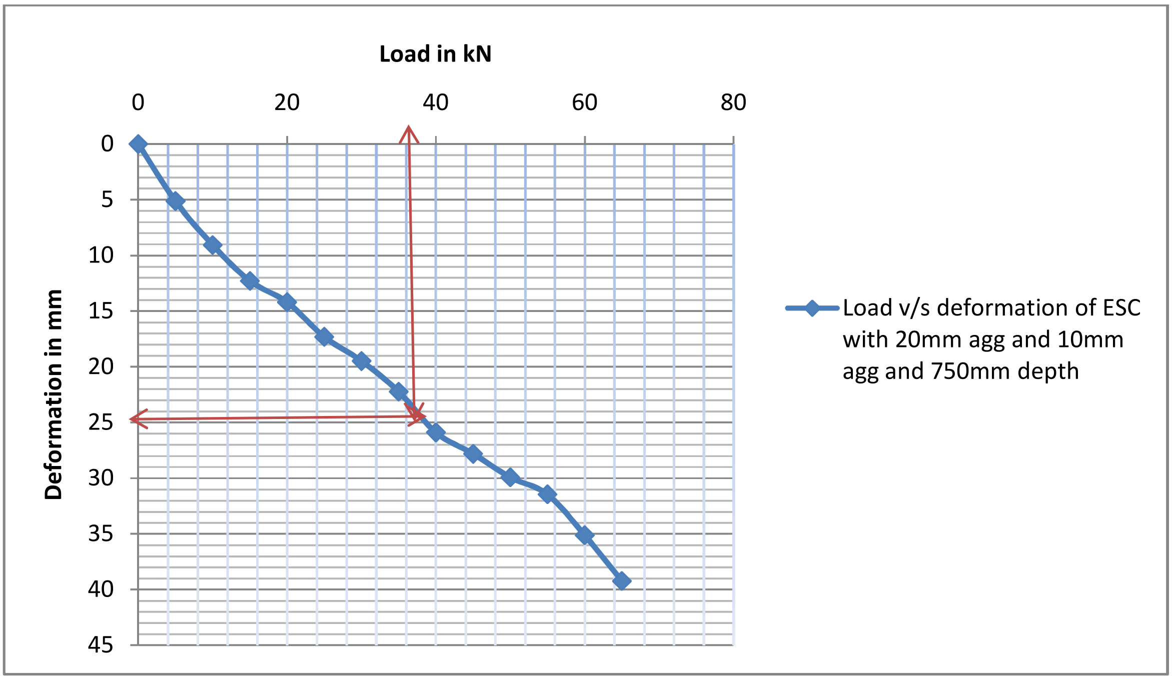


Fig 29:Load v/s deformation graph for stone column with 10mm + 20mm aggregate in ratio 1:1 by weight and 750 mm depth

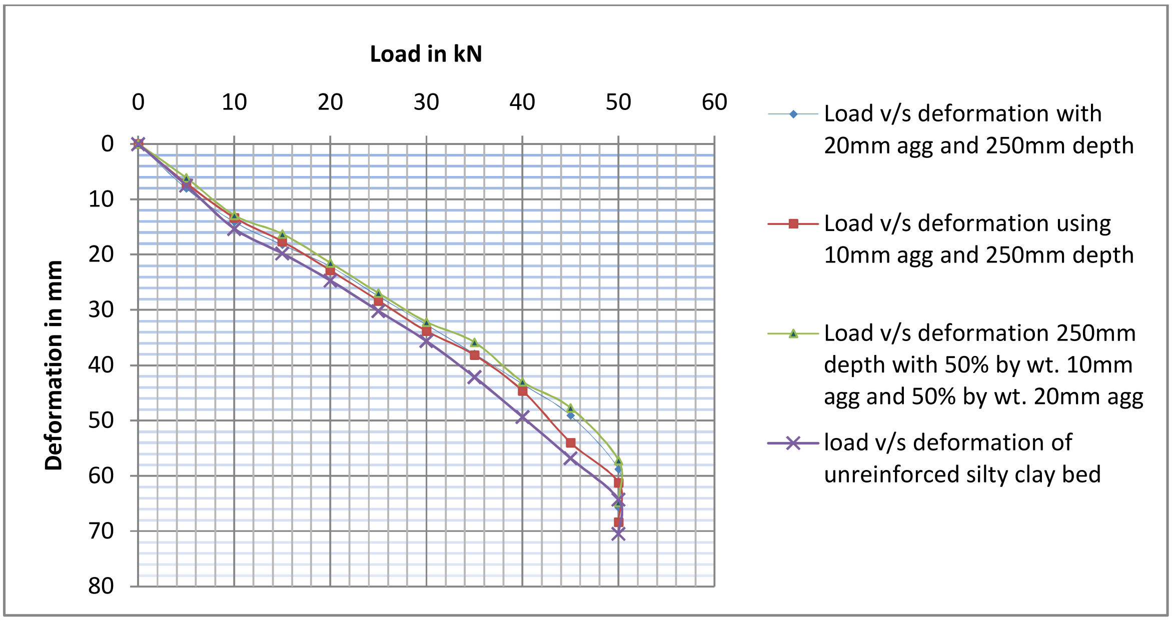


Fig 30: Comparison of load v/s settlement behaviour for 250mm depth stone column having different gradation

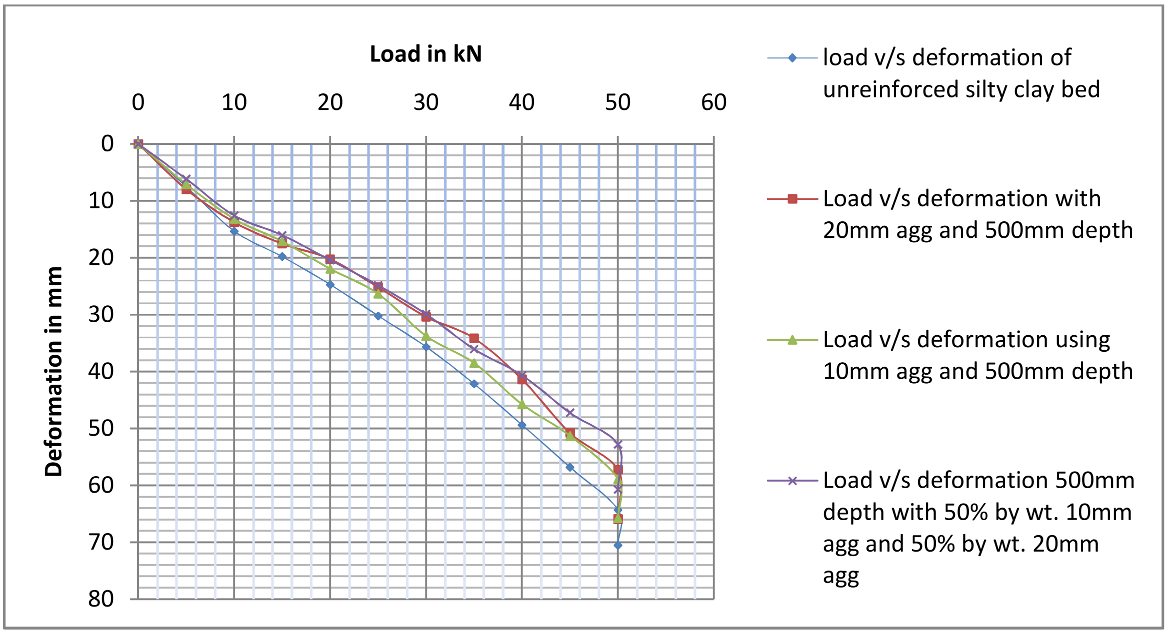


Fig 31: Comparison of load v/s settlement behaviour for 500mm depth stone column having different gradation

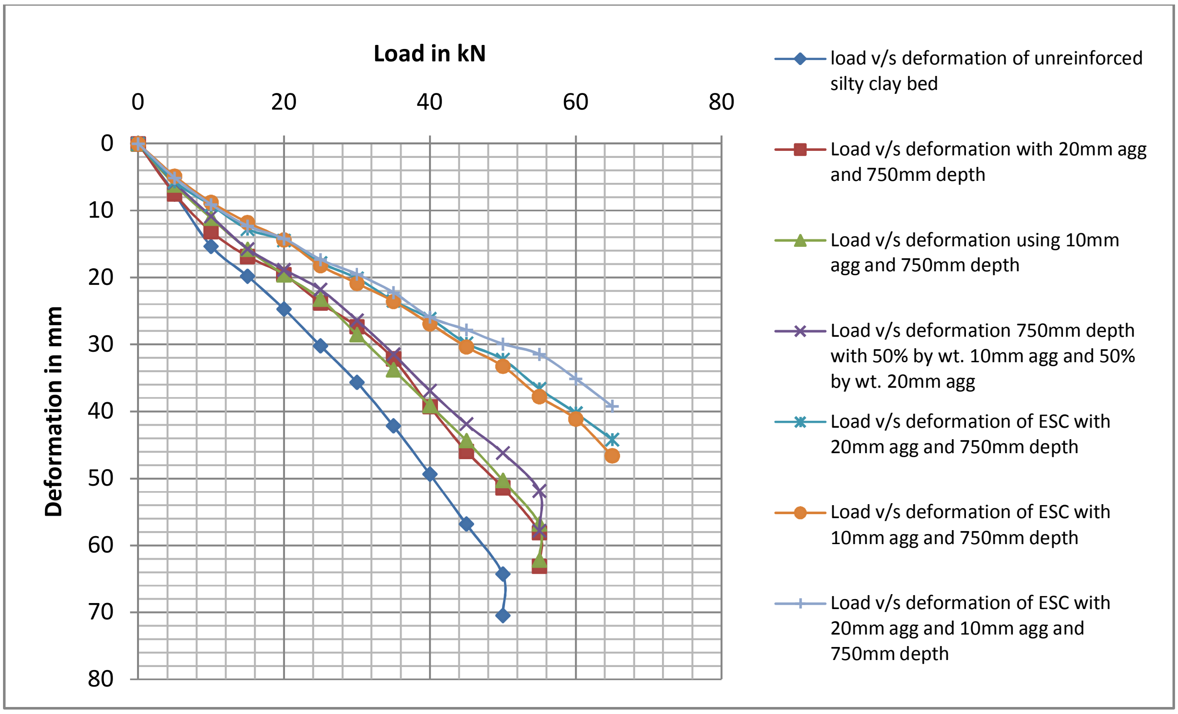


Fig 32: Comparison of load v/s settlement behaviour for 750mm depth stone column having different gradation with and without encasement

Table 17: Comparison of load carrying capacity at 25mm settlement for different stone columns

Depth (mm)	**Load at 25mm settlement**											
	Pure silty clay bed	**Silty clay bed with 20mm aggregates stone column**		**Silty clay bed with 10mm aggregates stone column**		**Silty clay bed with mixture of 10mm + 20mm stone aggregates.**		**Silty clay bed + encased stone column with 20mm aggregates (750mm depth)**		**Silty clay bed+ encased stone column with 10mm aggregates. (750mm depth)**		**Silty clay bed+ encased stone column with 10mm + 20mm aggregates (750mm depth)**	
	Absolute value in kN	**Absolute value in kN**	**% improvement**	**Absolute value in kN**	**% improvement**	**Absolute value in kN**	**% improvement**	**Absolute value in kN**	**% improvement**	**Absolute value in kN**	**% improvement**	**Absolute value in kN**	**% improvement**
250	20.27	22.64	11.7	21.96	8.34	23.22	14.55	37.89	86.92	37.14	83.22	38.78	91.31
500		24.99	23.3	23.47	15.78	25.15	24.08						
750		26.36	30.0	26.69	31.67	28.5	40.6						

Table 18: Comparison of settlements at 50kN load for different stone columns

Depth (mm)	**Settlement at 50kN load**												
	Pure silty clay bed	**Silty clay bed with 20mm agg. Stone column**		**Silty clay bed with 10mm agg stone column**		**Silty clay bed with mixture of 10mm + 20mm stone agg.**		**Silty clay bed with 750mm depth encased stone column made of 20mm aggregate**		**Silty clay bed with 750mm depth encased stone column made of 10mm aggregate**		**Silty clay bed with 750mm depth encased stone column having 50% 20mm + 50% 10mm aggregate by weight**	
	Absolute value in mm	**Absolute value in mm**	**Settlement reduction ratio (β)**	**Absolute value in mm**	**Settlement reduction ratio (β)**	**Absolute value in mm**	**Settlement reduction ratio (β)**	**Absolute value in mm**	**Settlement reduction ratio (β)**	**Absolute value**	**Settlement reduction ratio (β)**	**Absolute value in mm**	**Settlement reduction ratio (β)**
250	64.3	58.83	0.91	61.23	0.95	57.21	0.88	32.22	0.50	33.25	0.52	29.92	0.47
500		57.25	0.89	58.91	0.92	52.8	0.82						
750		51.37	0.80	50.28	0.78	46.19	0.72						

Table 19: Comparison of bearing capacities at 25mm settlement

<table>
<tr><th rowspan="3">Depth (mm)</th><th colspan="13">Measured bearing capacity at 25mm settlement = Load/Area of steel plate; (300mm dia. Steel plate)</th></tr>
<tr><th>Pure silty clay bed</th><th colspan="2">Silty clay bed with 20mm agg. Stone column</th><th colspan="2">Silty clay bed with 10mm agg stone column</th><th colspan="2">Silty clay bed with mixture of 10mm + 20mm stone agg.</th><th colspan="2">Silty clay bed + encased stone column with 20mm aggregates (750mm depth)</th><th colspan="2">Silty clay bed+ encased stone column with 10mm aggregates. (750mm depth)</th><th colspan="2">Silty clay bed+ encased stone column with 10mm + 20mm aggregates (750mm depth)</th></tr>
<tr><th>Absolute value in kN/m^2</th><th>Absolute value in kN/m^2</th><th>BCIF</th><th>Absolute value in kN/m^2</th><th>BCIF</th><th>Absolute value in kN/m^2</th><th>BCIF</th><th>Absolute value in kN/m^2</th><th>BCIF</th><th>Absolute value in kN/m^2</th><th>BCIF</th><th>Absolute value in kN/m^2</th><th>BCIF</th></tr>
<tr><td>250</td><td rowspan="3">225.2</td><td>251.56</td><td>1.11</td><td>243.94</td><td>1.08</td><td>257.97</td><td>1.15</td><td rowspan="3">420.96</td><td rowspan="3">1.87</td><td rowspan="3">412.75</td><td rowspan="3">1.83</td><td rowspan="3">430.93</td><td rowspan="3">1.91</td></tr>
<tr><td>500</td><td>275.87</td><td>1.23</td><td>260.87</td><td>1.22</td><td>279.43</td><td>1.24</td></tr>
<tr><td>750</td><td>296.87</td><td>1.32</td><td>296.65</td><td>1.31</td><td>316.72</td><td>1.41</td></tr>
</table>

Chapter 7
Theoretical Bearing Capacities

7.0 Theoretical bearing capacity values

Method I:IS 15284 Part 1(2003) lays down the following guidelines for calculation of ultimate bearing capacity of a single isolated stone column in mixed soil. The ultimate load capacity is computed using Bell's formula for passive pressure

$$\sigma_{rL} = P_p = \gamma z k_p + 2C_u\sqrt{k_p}$$

According to IS 15284 Part I, 2003, we can assume value of friction angle for aggregates in stone column within $38^0 - 42^0$

Assuming $\emptyset_{column} = 38^0$ and z = average bulge depth = 2× column diameter = 2×0.12 = 0.24m

Unit weight of soil surrounding the stone column, γ = 16.35kN/m^3

k_p =Passive earth pressure co-efficient of soil = $\frac{1+\sin\varphi_{soil}}{1-\sin\varphi_{soil}} = \frac{1+\sin 29}{1-\sin 29}$ = 2.89

So, Limiting radial stress, σ_{rL} = 16.35×0.24×2.89+2×33.96×$\sqrt{2.89}$ = 126.9 kN/m^2

Hence,

Limiting vertical stress, $\sigma_v = \sigma_{rl}k_{p(column)} = 126.9 \times \frac{1+\sin\varphi_{column}}{1-\sin\varphi_{column}} = 126.9 \times \frac{1+\sin 38}{1-\sin 38}$ kN/m^2

= 533 kN/m^2

Safe load = $\sigma_v \times \frac{(\pi/4)D^2}{2} = 896.65 \times \frac{(\pi/4)\times(0.12)^2}{2} = 3.01$kN

Method II: Hughes and Withers (1974)stated the following formula for calculation of ultimate bearing capacity:

$$q_{ult} = (\sigma_{r0} + 4C_u)\frac{1+\sin\varphi_{column}}{1-\sin\varphi_{column}}$$

σ_{r0} = initial effective radial stress = $K_0\sigma_{v0} = (1 - \sin\varphi_{soil}) \times \gamma \times 2D$

$= (1 - \sin 29^0) \times 16.35 \times 2 \times 0.12 = 2.02$ kN/m^2

$q_{ult} = (2.02 + 4 \times 33.96) \times 4.2 = 579.01$kN/m^2

Safe load = $\sigma_v \times \frac{(\pi/4)D^2}{2} = 579.01 \times \frac{(\pi/4) \times (0.12)^2}{2} = 3.27$ kN

Chapter 8
Results and Discussion

8.0 Results and Discussion

1. On the application of load, settlement of the plate is increasing non-linearly in case of pure silty clay bed.
2. Due to intrusion of stone column of various depths, the settlement increases due to increase in load. However, at any level of load, the settlement of soil with different types of stone column is less as compared to that of silty clay bed. This shows that there is reduction in settlement at any level of load and improvement in bearing capacity at any level of settlement. Hence, stone column proves to be a type of ground improvement technique
3. The load-settlement behavior is carried out for the stone column of same diameter and various depths such as 250mm, 500mm and 750mm and different gradation. Due to increase in depth of stone column, the load carrying capacity is increasing at every level of settlement. This may be due to different modes of failure of stone column such as bulging, shear and punching failure as explained in article 2.3. This proves that due to increase in depth of stone column, the load carrying capacity increases even without increase in diameter of stone column.
4. It is also observed that there significant increase in load carrying capacity and reduction in settlement in case of encased stone columns as compared to stone columns without encasement. This may be due to the fact that lateral expansion of stone columns is resisted by the encasement because the geotextile can provide tensile strength.
5. The percentage increase in load carrying capacity for 250mm, 500mm, and 750mm depth stone column are 11.7%, 23.3%, 30.0% for 20mm aggregates; 8.34%,

15.78%, 31.67% for 10mm aggregates and 14.55%, 24.08% and 40.6% for a mixture of 10mm and 20mm aggregates respectively. This may be due to the fact that mixture of 10mm and 20mm aggregates provide better grading and reduce value of void ratio

6. The percentage increase in load carrying capacity of encased stone columns for full depth i.e. 750 mm is 86.92% for 20mm aggregates, 83.22% for 10mm aggregates and 91.31% for a mixture of 10mm and 20mm aggregates respectively
7. The settlement reduction ratio for stone column of 250mm, 500mm, 750mm depth has been observed to be 0.91, 0.89, 0.86 for 20mm aggregates; 0.95,0.92,0.78 for 10mm aggregates; 0.88,0.82,0.72 for a mixture of 10mm and 20mm aggregates respectively.
8. The settlement reduction ratio for encased stone columns for full depth i.e. 750mm have been found to be 0.5 for 20mm aggregates, 0.52 for 10mm aggregates and 0.47 for a mixture of 10mm and 20mm aggregates respectively.

Chapter 9
Conclusion and Recommendation for future work

9.0 Conclusion:

The few experiments carried out on the local soil which is primarily silty clay have highlighted that for a single stone column or granular pile that there is marked decrease in settlement when soil is reinforced with stone column. It is observed that there is reduction of settlement and increase in bearing capacity with increase in depth of stone column. It is also observed that the decrease in settlement is more when the stone column is end bearing instead of floating. It is also found that for encased stone column the percentage increase in load carrying capacity and decrease in settlement is more than that of non-encased stone column.

9.1 Recommendation for future work

On the basis of experiments carried out in the present project, the following recommendations may be given for future work

1. The diameter of the stone column can be varied to check its effect on load carrying capacity.
2. The effect of number of stone columns in a group can also be observed.
3. The effect of encasement using different types of geosynthetic on load carrying capacity of stone column may also be observed.
4. The effect of pattern of arrangement of stone columns on load carrying capacity and settlement can also be observed
5. Some mathematical model can also be developed for the prediction of load carrying capacity of soil with stone column.
6. All the above points may be repeated for various types of soil and different materials used for the construction of stone columns.

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