Soil Stabilization with Vertical Piles Bearing Capacity and Settlement Characteristics of Cohesive Soil Reinforced with Sand, M Sand, Quarry Dust Piles

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Abstract
The cohesive soil deposits are unsuitable for construction activities because of their poor geotechnical properties such as less bearing capacity and high settlement. Modern urban development necessitate construction and development activities in available soft clay deposits. It is a usual practice to provide fine aggregate piles in soft clay deposits to improve its properties. Due to the unavailability of natural fine aggregate resources and critical environmental issues involved, the feasibility of using M sand and quarry dust as floating granular vertical pile material as a substitute to natural aggregate have been experimented in this work. For various footing sizes and various pile depths the California bearing ratio test was conducted. The settlement reduction factor and group pile efficiency are estimated by studying the load settlement characteristics. It is found that the M Sand reinforced piles increases the bearing capacity and reduces the settlement and recommended to be used for improving the bearing capacity of soft clay soils.
Keywords: Soil stabilization, Ground improvement technique, M Sand pile, Quarry dust pile, Settlement

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1. INTRODUCTION

Structures transfer their loads to the soil under foundation. Some soil deposits are not suitable for construction without giving proper treatment. These are problematic soils. Sandy soils, Silty or clayey sands and loose sandy soils are some of the problematic soils. Sandy soils are loose deposits which has less undrained shear strength. This may be less than 50kPa. Silty or clayey sands and loose sandy soils are classified as soft soil deposits. These soft clay deposits have poor strength and these are highly compressible. Since they are geologically young soil sediments. Black cotton soil is also one of the problematic soil. Since they tend to become soft under wet condition. When the moisture content increases in swells and when the moisture content decreases it shrinks. Numerous practices are referred in literatures to improve the properties of black cotton soil. Black cotton soil can be replaced with fill materials with high strength. Few researchers follow chemical stabilization. Using prefabricated vertical drains, Sand drains, reinforcing with geosynthetics, vacuum consolidation, pre consolidation, displacement of soft materials and piling are other techniques followed to improve the properties of black cotton soil deposits [1]. Deep pile foundations can also be provided instead of these ground improvement techniques. But behavior of pile foundation within different subsoil strata is always a challenging field for researchers [2]. Jayapal JP et al (2018) and Vinayagamoothy (2018) have stated that granular column is one of the promising ground improvement technique widely accepted as a solution to soft soil problems all over the world. Mohammed Y. Fattah et al (2015) have injected lime silica fume mix for stabilizing the soft soil.

Sometimes the geological strata of soft clay deposits extend beyond greater depth. Some of the solutions suggested above involve time and financial constraints in such cases. Hence they found to be unsuitable when clay deposit extend to large depth. Marine clays, marshy lands, loose sand, compressible soils, silty sand and clayey sand can be improved successfully and effectively by using stone columns or sand columns. AmitKumar et al, (2019) and Prasenjit et al (2017) have experimentally proved the efficiency of stone columns to strengthen the soft soils. When large settlements are to be arrested stone and sand columns are suitable solution. The techniques of improving soft clay deposits using stone column or granular column is under investigation level even though it is very effective in arresting settlements. Diameter of pile installed and length up to which the pile is to be installed are important parameters to be designed. The strength of granular pile material, strength of surrounding soil, method of installation of piles, relative rigidity of the footing and the number of granular piles to be installed beneath the footing are also important parameters to be designed. They decide the effectiveness and efficiency of the granular piling technique.

Mohammadreza et al (2014) have concluded that installation of lime mortar – well graded soil column improves the performance of soft soil tremendously. Seracettin et al (2016) have proved the effectiveness of floating and end bearing polymer columns in improving the load settlement behavior of soft clayey soil. Paramita Bhattacharya et al (2016) have concluded that increase in diameter of the granular column increases the efficiency. Ahmet Demir (2013, 2017) have used geogrid encasement also to prevent the lateral bulging of stone columns.

The objective of this research work is to perform laboratory level model tests on soft clay deposits to evaluate the effectiveness of granular piles. Using the multi cell concept floating piles and end bearing piles are driven on laboratory level models. The effectiveness of floating piles are compared to the end bearing piles. M Sand and Quarry dust piles are driven along with natural fine aggregate piles and their efficiency as a pile material is evaluated. The study was also conducted on clay deposits without pile reinforcement and the results were compared. H is denoted as the depth up to which soft soil deposit extends and L is denoted as the depth of pile reinforcement. The floating granular piles were installed for L/H ratios of 0.5, 0.75 and 1. Hence when L/H =0.5 the pile will be installed up to half the depth of clay deposit. And when L/H=1 they denote end bearing pile of full depth.

In this experimental work, the effect of surface area of footing plate on the depth of pile is also studied. Load settlement behaviour was studied using the California Bearing ratio test Apparatus. The granular piles were drilled and tested in the laboratory model level and they were related to field conditions to find the ultimate bearing capacity in field.

2. PROPERTIES OF CLAYEY SOIL

2.1 Liquid Limit and other properties

The soil sample has been collected from Maviduthikottai, near Devakottai, Tamilnadu, India. The following Tables 1 and 2 present the properties and consistency of clayey soil found as per IS 2720(reaffirmed in 2010). Liquidity index for soft clay is 100% and for stiff clay it may be zero (or) negative. The liquid limit plot is shown in Figure 1. The optimum moisture content for clayey soil is obtained as 14% as shown in Figure 2.

Table 1 Properties of clayey soil

Property	**Value**
Specific Gravity	2.144
Void Ratio	0.1286
Porosity	0.114
OMC	14%
Dry Density (g/cc)	1.703
Bulk Density (g/cc)	1.9415
IS Classification of Soil	CI

Table 2 Consistency of clayey soil

Property	**Value**
Liquidity limit(LL)	43.32%
Plastic Limit(PL)	27.38%
Plasticity Index(PI)	15.92
Flow Index(IF)	22.61
Toughness Index(IT)	0.70411
Consistency Limit	2.478%
Liquidity Index	-1.48%

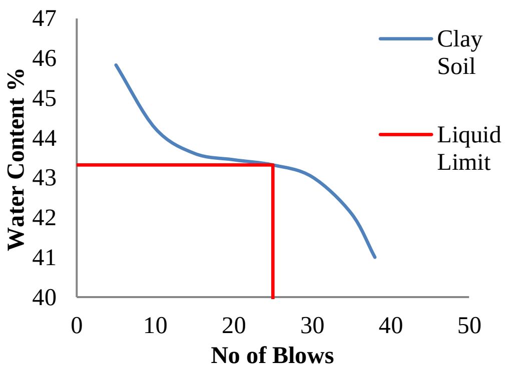

Figure 1 Liquid limit of clay soil

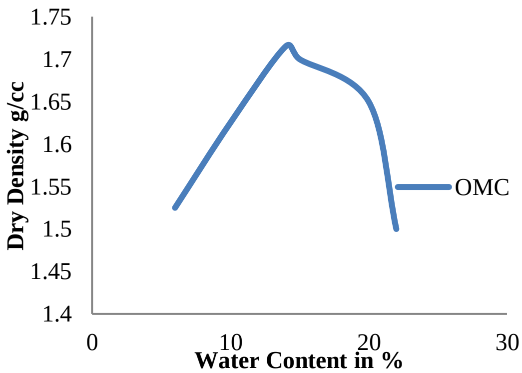

Figure 2 OMC for clayey soil

2.2 Particle size analysis on Clayey soil

The particle size distribution obtained for the clay sample as per IS1498 (reaffirmed in 2007) is shown in Figure 3 and Table 3.

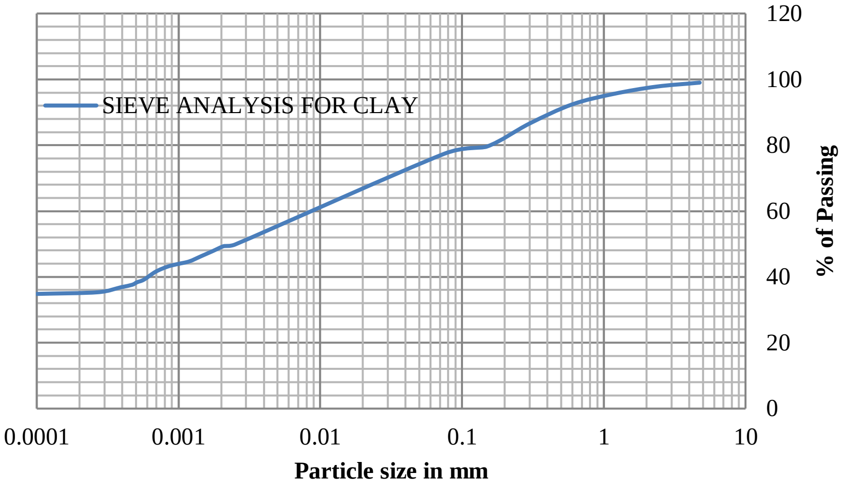

Figure 3 Particle size distribution curve for clayey soil

Table 3 Clay and Silt content

Sand Content %	22.60
Silt Content %	27.55
Clay Content %	49.84

2.3 Activity of Clay

The main factor that is concerned in designing foundation on clayey soil is its swelling potentials; shrinkage is also equally important. The volume change that results in shrinkage and swell depends on activity of the soil.

The activity of soil obtained from the test result is 0.31 which is less than 0.75 which means the soil is inactive soil i.e., the soil does not swell and shrink.

In order to check whether the soil is completely inactive, Free Swell Index (FSI) test has been conducted. The increase in volume in percentage observed in soil under submerged condition is termed as free swell index. The FSI obtained is 2.22% which is less than 20 hence the soil has low FSI. Hence the shrink and swell condition of soil is not taken into consideration for performing the tests.

3. PROPERTIES OF GRANULAR PILE MATERIALS

The Granular materials such as sand, M sand, quarry dust are used as pile materials. The pile materials were first dried in direct sunlight. Then they were sieved through the IS sieve 4.75mm size. The fraction of materials passing 4.75mm IS sieve were used and those retained are disregarded. Sieve Analysis was performed to determine particle size distribution of the granular pile materials. Specific gravity was determined for all three materials using pycnometer. Tests on minimum unit weight, maximum dry unit weight and relative density, were conducted on these granular materials. The sand is classified as poorly graded sand SP from the particle size distribution. The results of sieve analysis are given Figure 4. The properties of granular materials used are provided in Table 4.

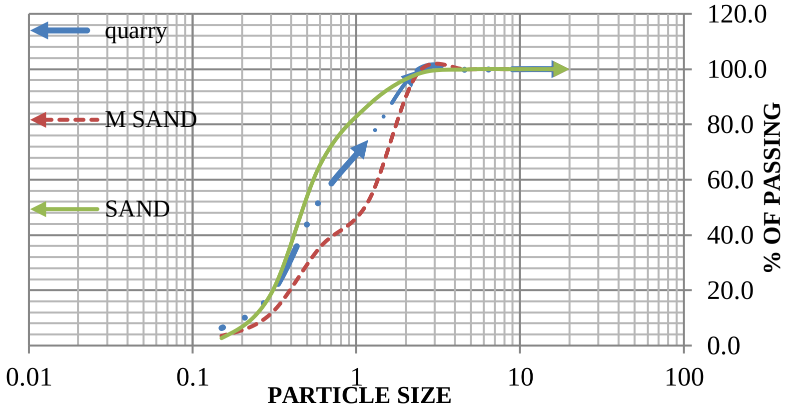

Figure 4 Sieve analysis graph for materials used as pile

Table 4 Properties of pile material

Property	River Sand	MSand	Quarry Dust
Silt & clay content (%)	2.70%	1.60%	6.35%
Sand content (%)	97.20%	98.50%	93.40%
Gravel content (%)	0.10%	0.10%	0.25%
D10	0.2	0.28	0.19
D30	0.37	0.54	0.27
D60	0.53	1.4	0.72
Coefficient of uniformity	2.65	5	3.789
Coefficient of curvature	1.292	0.744	0.533
Specific gravity	2.6	2.7	2.78
Void ratio of max density	0.449	0.455	0.157
Porosity	0.31	0.313	0.136

Cohesive resistance(C) and angle of internal resistance (Φ) of sand, M sand and quarry dust obtained from direct shear test are listed in the Table 5 given below.

Table 5 Direct shear test results

Materials used	**Cohesion resistance (C)**	**Angle of shearing resistance(ɸ)**
Sand	0.8	33.27
M sand	0.07	37.99
Quarry Dust	0	49.89

4. METHODOLOGY

The pressure settlement experiment on clayey soil reinforced with granular materials was conducted in California bearing ratio testing machine having an ultimate load carrying capacity of 10kN as per IS 2720-Part 16(reaffirmed in 2010). The test has been conducted in standard mould of diameter 15cm and height 15cm using surcharge weights each of 2.5kg. In this experiment two different steel plates have been used as footing plates with 13cm and 14cm diameter respectively having a thickness of 6mm. Figure 5 presents the pattern of installation of piles inside the standard mould.

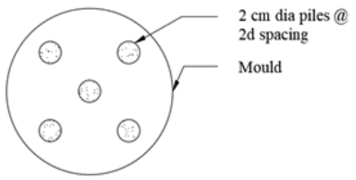

Figure 5a Plan view of piles

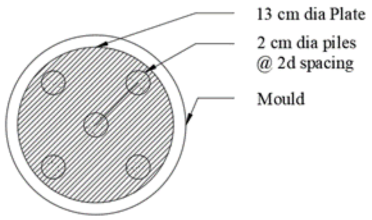

Figure 5b Plan view of piles with 13cm plate

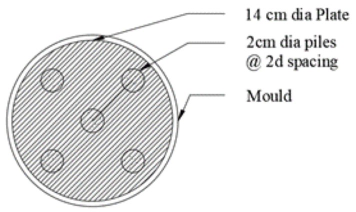

Figure 5c Plan view of piles with 14cm plate

Figure 5 Arrangement of piles with CBR mould with and without plates

A replacement procedure is followed to prepare granular pile reinforced clay bed. It was planned to reinforce the clay beds with 20.0mm diameter granular pile materials. Hence, a Poly Vinyl Chloride (PVC) pipe having outer diameter 20mm was used in this experimental study.

The PVC pipe was very thin and open ended having thickness 2mm. The outer surface of the PVC pipe was lubricated with oil in order to ensure smooth drilling of pipes. Then they were compressed against clay soil bed so that they form conduits for replacement method. The compression was given uniformly so that 2cm depth is driven for one turn. This procedure will ensure that suction pressure may not be developed by the existing soft clay deposit during removal of pipe from clay soil. This process will form a desired hole of required depth in the clay bed. The compression was applied as a static force manually into the PVC pipe in gentle manner. This procedure will ensure that the clay deposit will not be disturbed during this process. If the clay deposit get disturbed, this may change the properties of existing clay deposit after reinforcement. The granular vertical piles were formed in three layers each 20mm thick in case of 6cm depth pile. The three layers were lightly compacted using tamping rod made of steel to form piles with uniform compacted dry density. In order to avoid lateral bulging of vertical granular piles, only light compaction force was given. The same procedure was followed for 9cm and 12cm deep piles. The granular piles with various depths are sown in Figure 6.

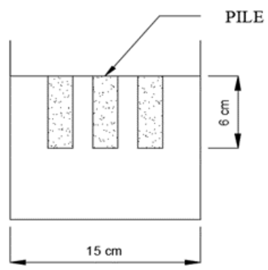

Figure 6a C/S of floating pile A

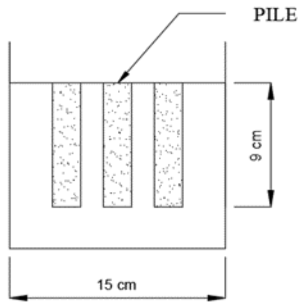

Figure 6b C/S of floating pile B

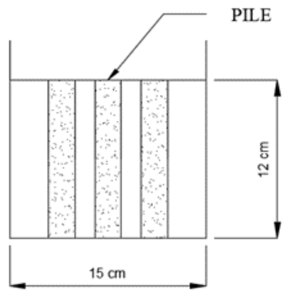

Figure 6c C/S of end bearing pile

Figure 6 Piles of varying depth installed in mould

Two types of floating piles one is of half the depth of clayey soil other is of three quarter the depth of clayey soil were tested. Then one end bearing pile which had depth up to the full depth of clay deposit was tested. All the tests were repeated for 13cm footing plate and 14cm footing plate. The tests were conducted for all three types of pile materials such as sand, M sand and Quarry dust. Boring of piles was done using PVC pipes as shown in Figure 7. The CBR test apparatus is shown in Figure 8.

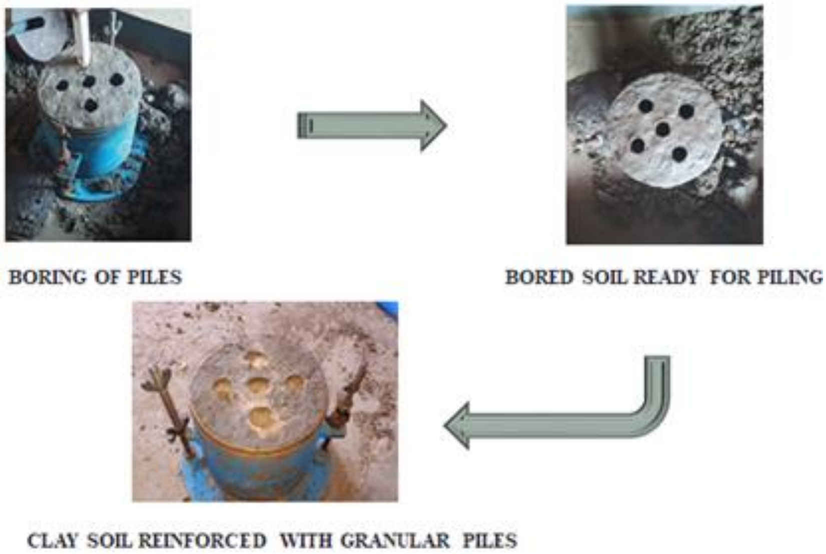

Figure 7 Method of installation of piles within the mould

Figure 8 CBR Test Apparatus

5. EXPERIMENTAL RESULTS

5.1 Bearing Capacity of Clayey Soil

The unconfined compressive strength of clayey soil is determined from UCC test and the test results are given in Table 6.

Table 6 Strength of Clay Soil

Property	Value
Shear strength of clayey soil	14.01 kN/m^2
Unconfined compressive strength of clayey soil	28.02 kN/m^2
Ultimate bearing capacity of clayey soil	79.857 kN/m^2
Allowable bearing capacity of clayey soil	26.619 kN/m^2

5.2 Load Vs Settlement Curve for Clayey Soil

The pressure settlement characteristics of footing were studied on clayey soil by using CBR instrument as shown in Fig 8. In which footing size is considered as the plate size. In this experiment two types of footing/plate sizes have been used on clay soil one is 13cm plate and another is 14cm plate. The thickness of the plate is a constant for both plates. Since the surface area is different for both the cases the pressure settlement behaviour is also different. Thickness of the plate is 6mm. Therefore the settlement of the soil corresponding to the thickness of the plate was found out. The results are shown in Figure 9 given below:

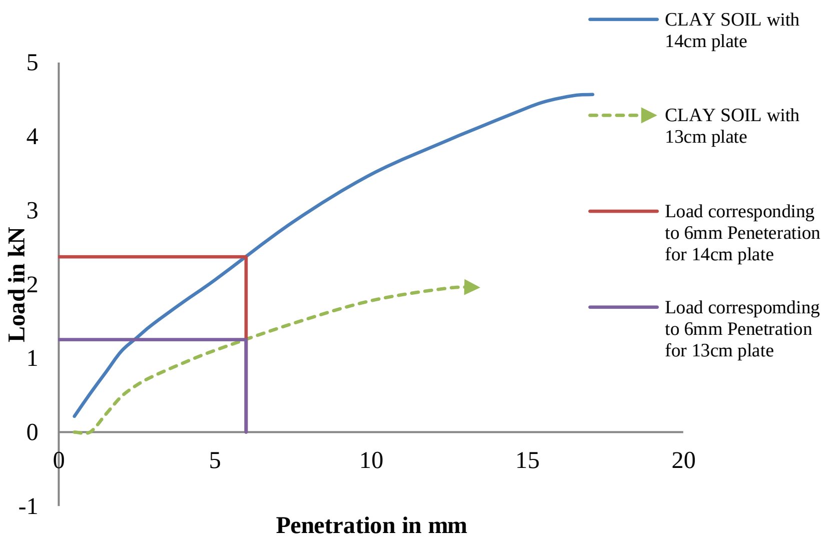

Figure 9 Load vs settlement curve for clayey soil

5.3 Load vs Settlement Curves with Granular Piles

After installing granular piles the pressure settlement behaviour with two footing plates, three granular pile reinforcement depth configurations and three pile materials were tested using CBR testing machine. The results are given below in the form of graph in Figure 10a to Figure 10f.

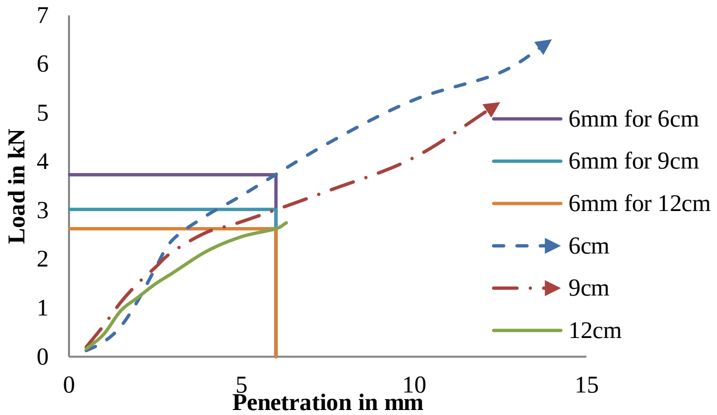

Figure 10a Load vs settlement curve for sand piles with depth of 6cm, 9cm, and 12cm on 14cm footing plate

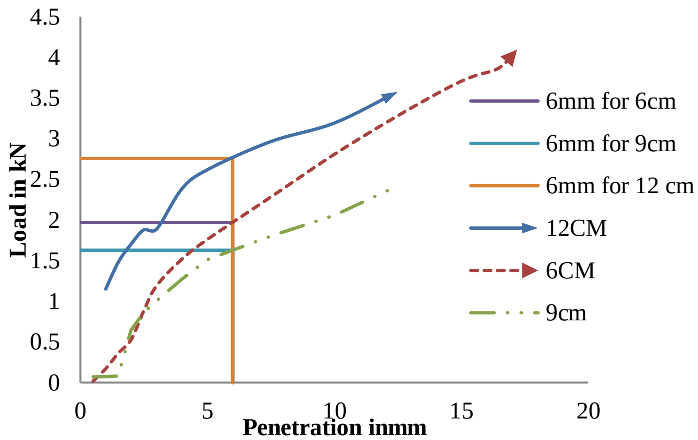

Figure 10b Load vs settlement curve for sand piles with depth of 6cm, 9cm, and 12cm on 13cm footing plate

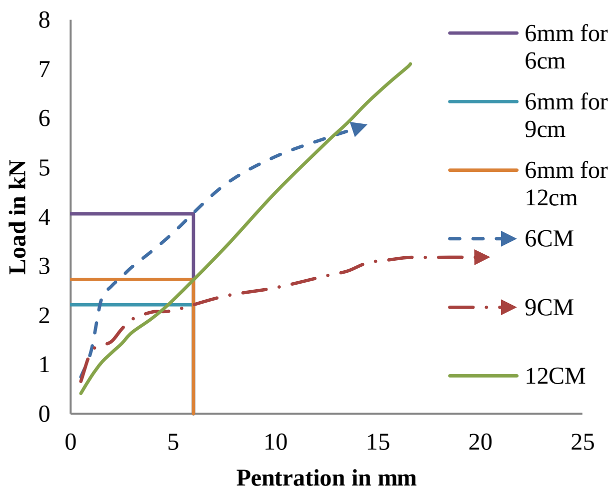

Figure 10c Load vs settlement curve for M sand piles with depth of 6cm, 9cm, and 12cm on 14 cm footing plate

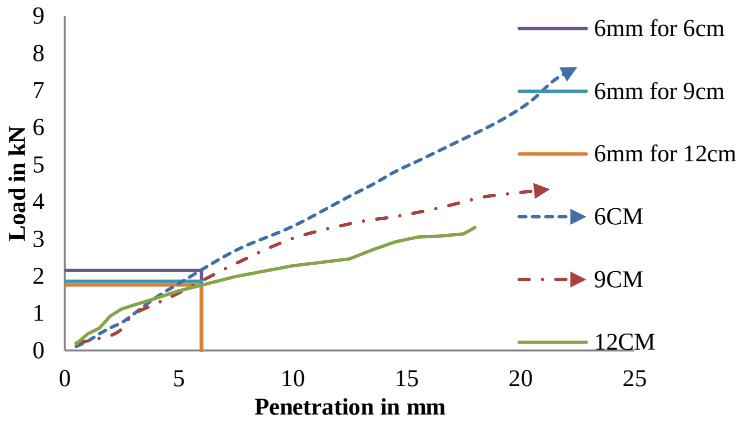

Figure 10d Load vs settlement curve for M sand pile with depth of 6cm, 9cm, and 12cm on 13cm footing plate

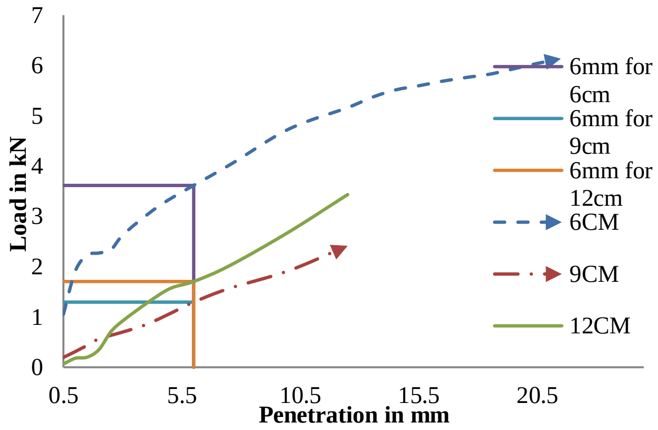

Figure 10e Load vs settlement curve for Quarry dust piles with depth 6cm, 9cm, 12cm on 14cm footing plate

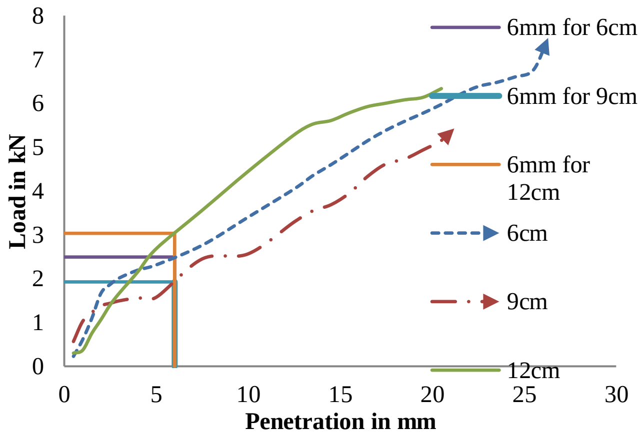

Figure 10f Load vs settlement curve for Quarry dust pile with depth 6cm, 9cm, 12cm on 13cm footing plate

From the series of tests conducted on clayey soil with different footing plates by reinforcing them with vertical granular pile materials such as M Sand and Quarry dust, it is concluded that end bearing piles had lesser settlement and were effective with footing plate of size 13cm. Floating pile which was reinforced up to half the depth soil layer (L/H=0.5) had lesser settlement and was effective with footing plate of size 14cm. Hence when the footing size is large the there is a reduction in the stress applied hence pile reinforcement depth can be reduced with increased footing sizes.

The settlement reduction in percentage and increase in load carrying capacity for end bearing piles with 13cm footing plate and floating pile up to half the depth of soil layer for 14cm plate are tabulated below in Table 7.

Table 7 Results of load vs settlement curve

Type of footing plate	**Material used as a pile**	**Settlement reduction in %**	**Increase in load carrying capacity after installation of piles**
13cm plate pile depth is 12cm	Sand	78.04	2.201
	M sand	48.68	0.699
	Quarry dust	74.99	2.423
14 cm plate pile depth is 6cm	Sand	48.075	1.573
	M sand	74.27	1.709
	Quarry dust	74.27	1.524

In this study it is found that the load carrying capacity increases up to a maximum of 2.4 times after installation of quarry dust piles. Manszur Rahman (2012) has also confirmed that the load carrying capacity increases up to 1.8 to 3.5 times depending on the variation of replaced area of clay by sand. The percentage reduction of settlement is almost constant for the M sand and Quarry dust reinforced granular pile system and the value is 74.27% in 14cm plate with L/H=0.5. Quarry dust reinforced ground has constant percentage of reduction in settlement which is around 74% for both 13cm and 14cm plates. The percentage reduction of settlement increases from 48.07% to 74.27% when the M Sand and Quarry dust are used as reinforcement for granular pile system in 14cm plate with L/H=0.5.

The 14 cm plate with the suitable dimensions filled with M sand floating pile of 6cm depth gives higher strength and this is considered as efficient comparing to sand and quarry dust piles. When the surface area of plate increases the load carrying capacity also increases and the corresponding settlements found to decrease as compared to small surface area plate.

5.4 Stress Concentration Ratio for Clayey Soil Reinforced with Granular Piles

The ground which is reinforced with granular vertical piles is composite in nature. When the ground composed of composite material is subjected to loading, there occurs a stress redistribution since the stiffness of the vertical column or pile material is different from adjacent clay soil deposit. But the vertical settlement of granular columns and clay deposit is almost equal. The stress on the granular column material is higher due to stress concentration. The stiffness of the granular layer is greater than the stiffness of the adjacent soil. Inside the unit cell, the distribution of vertical stress can be written in terms of the stress concentration ratio, n

$n = \sigma_s / \sigma_C$ (1)

σ_s = vertical stress on sand pile

σ_c = vertical stress on surrounding clay soil

The stress concentration ratio has been plotted against settlement and shown in Figure 11. Lesser the settlement better is the stability of the structure. From the graph it is clear that quarry dust piles have lesser settlement for smaller stress ratios 2 & 4 and M sand has lesser settlement for stress ratios from 6 to 10. When the stress ratio increased further sand has lesser settlement compared to other materials.

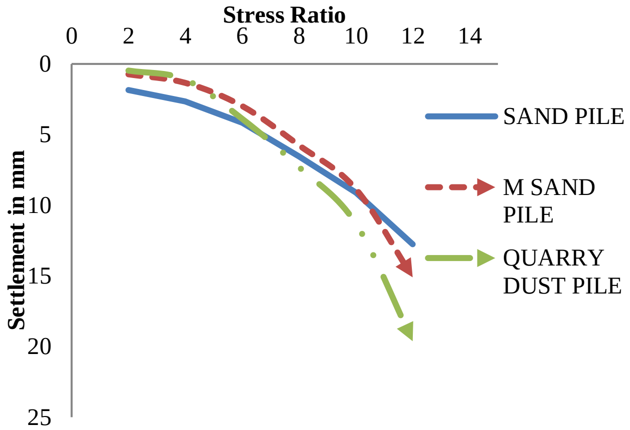

Figure 11 Stress improvement ratio and corresponding settlement for different materials

5.5 Stresses on C-Φ Soil

In this experimental work, the soil has been observed to undergo general shear failure. This laboratory experiment can be correlated to real field conditions. From theoretical considerations, the diameter and spacing of piles are found out on the basis of footing size. After installation of sand pile, settlement on clayey soil is reduced and corresponding load carrying capacity found to increase. By using the area replacement ratio, the area replaced from the clay soil is determined and from this settlement decrement factor and load increment factor are determined. Within the unit cell, applied load is collectively taken by the stiff sand pile and the tributary soft clay.

$$F = \sigma A = (\sigma_{s+} \sigma_c)A \quad (2)$$

$$\sigma_= \sigma_s a_s + \sigma_c(1-a_s) \quad (3)$$

where, F = applied external load

σ = average load intensity

σ_s = vertical stress on the sand pile

σ_c = vertical stress on surrounding soft clay

A, A_s, A_c = Cross sectional area of unit cell, sand pile and tributary clay respectively

a_s = stress concentration ratio

Due to the presence of sand pile it is understood that part of the stresses are carried by the sand piles hence there is a reduction in the stresses on clayey soil. The reduction in stress in the clayey medium is often expressed as stress reduction factor μ_c and stress increment factor μ_s. using the following equations 4 and 5.

$$\mu_c = \frac{\sigma c}{\sigma} = \frac{1}{1+(n-1).as} \quad (4)$$

$$\mu_s = \frac{\sigma s}{\sigma} = \frac{n}{1+(n-1).as} \quad (5)$$

For a C-Φ soil,

Active earth pressure, $\sigma_h = K_a\,\sigma_v - 2\sqrt{Ka\ C}$ (6)

Passive earth pressure, $\sigma_h = K_a\,\sigma_v - 2\sqrt{Kp\ C}$ (7)

Where,

$K_a = (1-\sin \Phi)/(1+\sin \Phi)$

$K_p = (1+\sin \Phi)/(1-\sin \Phi)$

C = Cohesion ; σ_h = Horizontal stress ; σ_v = Vertical stress

For pile reinforced clay ground, it is somewhat obvious to assume that the properties of c-material i.e clay are improved by Φ-material. The following details summarize the active and passive state of sand and clay respectively. [17, 18, 19]

For Sand (Φ-Material) C=0

Active state: $\sigma_h = ((1-\sin \Phi)/(1+\sin \Phi))\sigma_v$ (8)

Passive state: $\sigma_h = ((1+\sin \Phi)/(1-\sin \Phi))\sigma_v$ (9)

For Clay (C-Material) Φ=0

Active state: $\sigma_h = \sigma_v - 2c$ (10)

Passive state: $\sigma_h = \sigma_v + 2c$ (11)

Active and passive stress in cohesive soil are 60.022kN/m^2 and 81.564kN/m^2 respectively by using appropriate values from properties discussed previously. For C-Φ soil active and passive stress are 49.37kN/m^2 and 987kN/m^2 respectively. The percentage reduction in active earth pressure is observed to be 21.57% and the percentage increase in the passive earth pressure is observed to be 91.7%. Hence it is well understood that the installation of granular piles improve the characteristics of the soil.

5.5 Efficiency of Group Pile

Clayey soil reinforced with M Sand pile of 6cm depth (L/H=0.5) with 14cm plate size is considered as an efficient system. It gives higher strength as observed from the above test results. Therefore, group pile efficiency is determined for M Sand floating pile with L/H=0.5 for the 14cm diameter plate.

Group Pile Efficiency Factor (η_g)

$$\eta_g = 1 - \frac{\varphi}{90}\frac{(n-1)m+(m-1)n}{mn} \qquad (12)$$

$$\emptyset = \frac{d}{s} \qquad (13)$$

Where, m = number of rows

n = number of piles in a row

d = diameter of the pile

s = center to center spacing of pile

η_g is found to 0.754 and Barksdale et al (1984) developed an approach to determine the bearing capacity on a composite ground based on general shear failure of group piles. In this simplified mechanism, the failure surface is represented by two straight rupture surfaces. Analyzing the force equilibrium of the wedge formed by the two straight rupture surfaces produced the ultimate bearing capacity in the following form,

$$Q_{ult} = 2(1-a_s)c_u\tan\Phi + 0.5\gamma_c B\tan^3\Phi + 2c_u\tan\Phi \qquad (14)$$

$$\Phi = 45 + \tan^{-1}(\mu_s a_s \tan\Phi) \qquad (15)$$

The bearing capacity ratio has been calculated and presented in Table 8. And a graph is drawn between stress ratio in field and ultimate stress in field and it is shown in Figure 12. And it is found that the ultimate stress in field increases with stress ratio. Mohamed Hussein et al [21] have also concluded that the increase in area of replacement of granular piles directly reduces the effect of swelling characteristics of clayey soil.

Table 8 Bearing capacity ratio

Stress Ratio	**Stress in field**	**µs**	**φ**	**Q_{ult}**	**Ratio of field to Lab Q_{ult}**
2	53.25	1.84	48.6	80.25	1.51
4	106.5	3.17	51.14	91.98	0.86
6	159.75	4.17	52.99	102.23	0.64
8	213	4.95	54.4	117.17	0.52
10	266.26	5.58	55.49	118.99	0.45
12	319.51	6.1	56.37	125.88	0.39

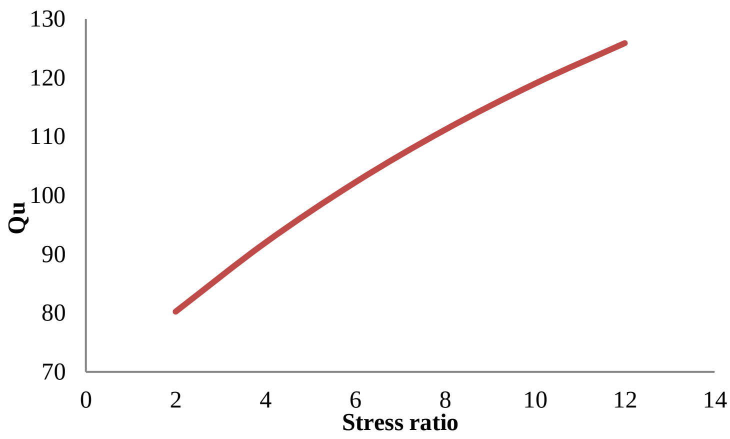

Figure 12 Graph between Stress ratio in field and ultimate stress in field for M Sand

5.6 Settlement Reduction Factor

For the Pile depth of 6cm the initial stress in soil before installation of piles is calculated to be 114.276 kN/m^2 and the initial stress in soil after installation of piles is calculated to be 152.368 kN/m^2. From this the settlement reduction ratio has been calculated and the same is given in Table 9. From the Table 9 it is clear that increasing the stress ratio there is reduction in settlement of the clay ground reinforced with sand piles compared to the normal clay ground [22, 23]. The reduction in settlement is mainly due to the increase in density which is also proved by Zillal et al[24].The following result is presented for clay soil reinforced with M sand piles up to a depth of 6 cm with 14 cm footing plate.

Table 9 Settlement reduction ratio

Stress Ratio	**Change in stress after installation of piles**	**Change in stress for clay**	**μc**	**Settlement after reinforcing piles**	**Settlement in clay**	**Settlement reduction ratio**
0	380.2	75.5	1.1	6.29	12.9	0.48
2	760.4	150.9	0.92	8.21	21.6	0.38
4	1520.8	301.8	0.79	10.4	33.1	0.31
6	2281.2	452.7	0.69	11.6	41.1	0.28
8	3041.6	603.7	0.62	12.4	47.1	0.26
10	3802	754.6	0.56	12.9	51.9	0.25
12	4562.4	905.5	0.51	13.3	56.1	0.24

5.7 Spacing of Piles in Field

Unconfined compressive strength of soil is work out to be 28.027 kN/m^2 and Shear strength of soil is 14.01 KN/m^2. Having the Unit weight of soil as 19.046 KN/m^3 and the area replacement ratio (As/Ac) as 0.088 the spacing of piles in field can be calculated [25,26,27]. In this calculation width of the footing is assumed as 1.4m and diameter of pile on the field is assumed as 1m. The Table 10 shows spacing of M sand piles in field according to the stress improvement ratio.

Table 10 Spacing of piles in field

Stress ratio	**Qu in field (kN/m^2)**	**Θ=tan^{-1}(D/S)**	**Spacing in m**
2	80.24619	-38.841	-1.24193
4	91.97923	16.16038	3.450929
6	102.226	35.28961	1.412894
8	111.1695	45.37737	0.986913
10	118.9989	51.78775	0.787269
12	125.885	56.31378	0.66657

The spacing of piles should be between 2D to 4D. From the Table 10 spacing corresponding to stress ratio 4 is approximately 3.5 times of diameter of pile. Decreasing the spacing below 2 times of diameter of pile and increasing the spacing beyond 4 times of diameter of pile is uneconomical. From the above results, it is clear that stress ratio 4 is the economical ratio and has improved the bearing capacity up to 91.98 kN/m^2 and has a settlement reduction factor of 0.38. Sanjaya Kumar Jain et al [28] have proved that the vertical stone column reinforcement improves bearing capacity and reduces settlement.

6. CONCLUSION

The clay soil deposits have low bearing capacity and hence the soil needs improvement or stabilisation. The technique of reinforcing the clayey soil deposits with vertical granular piles is experimented in this work. Three different materials such as sand, M sand and quarry dust have been used in this work. Installing piles up to half the depth and three quarter depth (L/H=0.5 and 0.75) will give economical results than end bearing piles of full depth (L/H=1). The surface area of footing plate is varied to study the effect of footing size on the load carrying capacity of soil. From the series of experiments conducted in the laboratory keeping the are replacement ratio as constant, it is inferred that when the surface area of footing is larger floating pile having depth equal to one half of the clay deposit is more efficient in reducing the settlement of clayey soil. When the surface area of the footing is comparatively lesser, the end bearing piles of depth equal to full depth of clay deposit is more efficient in reducing the settlement of clayey soil. Comparing the three materials used as pile reinforcements the following conclusions are arrived.

M sand reinforced piles increases the bearing capacity up to 71.16% and the settlement was reduced up to 74%. Whereas sand reinforced piles have increased the bearing capacity up to 57.38% and the settlement is reduced up to 45%. Quarry dust pile reinforced ground has shown an increase in load carrying capacity up to 52.42% and reduction in the settlement. Hence M sand reinforced piles are more efficient compared to other material piles. From this study it is recommended that, when the stress ratio i.e the ratio of applied stress in clayey soil with pile to applied stress in clayey soil without pile is increased settlement also increases correspondingly. Therefore stress ratio cannot be increased beyond certain limit since it causes several problems like bulging of soil and rapid settlement. The centre to centre spacing between piles can generally be from 2d to 4d, where d is the diameter of the pile. But this spacing limit is not suitable when the stress ratio is increased beyond 4. Hence it is concluded that the M Sand reinforced granular piles are more effective in reducing settlement and increasing bearing capacity.

7. Declarations

7.1. Author Contributions

Aarthi.K contributed to the conception and design of the study; Poongothai SP performed the experimental tests and numerical study and analysed the data; Poongothai SP wrote the first draft of the manuscript; Aarthi.K guided and supervised the research work and corrected and submitted the manuscript. All authors have read and agreed to the published version of the manuscript.

7.2. Data Availability Statement

The data presented in this study are available in article.

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7.5. Conflicts of Interest

The authors declare no conflict of interest.

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