LOAD SETTLEMENT RESPONSE OF GEOTEXTILE ENCASED GRANULAR PILE IN EXPANSIVE SOIL

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ABSTRACT - *This paper represents the study on expansive soil whose volume undergo a significant change with the change in water content. The volume change may be caused either by swelling or by shrinkage. In the present study, we used bentonite clay, and a trial is done to improve the ground condition and use the clay for construction purposes. For this we use granular pile encased with geotextile to improve the ground condition. As we know, granular material fails due to bulging in clay under compressive load so to avoid this it is encased with the geotextile. In the present study, the aim is to see the variation in load bearing capacity and settlement response of granular pile. Further a study is conducted to see the effect of gradation of granular material on load bearing capacity and settlement response on bentonite clay. Various tests were performed for the determination of Atterberg limits (liquid limit, plastic limit and shrinkage limit). Then sample was compacted at OMC and MDD was found out using modified Proctor test. CBR test was conducted to study the load bearing capacity and to see the settlement response. Specific gravity was found of both bentonite clay and granular material using pycnometer method. (Granular material < 4.75mm).*

Key Words:__Bentonite clay, granular material, geotextile.

1. INTRODUCTION

This project aims is to improve the ground condition of land having poor engineering properties like where we found expansive soil (clay soil) and construction becomes very difficult on such kind of soil. As we know soil is naturally formed so there is always a possibility of encounter such soil at the construction site.

In this fast-changing world and urbanization suitable land for construction encourages the geotechnical engineers for improving the properties of poor soil and for that there are numerous ground improvement technique and stone column technique is one of the better techniques to be adopted where we encountered with expansive soil not only in terms of ease in application but in terms of cost effective also.

As we know clay soil whose volume changes significantly due to the presence of smectite clay minerals including montmorillonite and bentonite which has the most shrink-swell capacity. This project presents one of the techniques to use the problematic soil for construction purposes by inserting granular piles in it. The granular piles generally fail due to bulging in soft clay due to compressive load, to counter this situation we use geotextile around the pile and try to further improve their performance. This study deals with the response of granular pile in the presence of Geotextile under compression.

1.1 Background/Literature Review

There are numerous ground improvement techniques are in practice to improve the properties of the ground. Stone column technique is a well-established technique and is best suited for improving soft clays, silts and also for loose sand deposits. The concept of Granular column was first adopted in France in 1830 to improve the properties of soil and later this is adopted all over the world to increase the bearing capacity, to reduce settlement, and also to increase the resistance to liquefaction (Barksdale and Bachus 1983, Alamgir et al 1996). Granular columns are constructed using an electric or hydraulic actuated vibrating probe. The probe was originally developed by Steuerman. The vibratory probe, essentially in cylindrical shape consists of a hydraulic or electric motor mounted within a cylindrical casing of 350mm-450mm in diameter and 2.0m – 4.5m in length. The motor powers a rotating eccentric weight which provides the lateral vibration and compaction. Granular columns are constructed using this vibrating probe either by wet process or dry process.

Failure mechanism of Granular columns

Granular column which is greater than three times the diameter, generally fails by bulging as shown in Figure 1. Similar to pile foundations Granular columns are installed in groups. A Granular column in a group has significantly higher ultimate load carrying capacity than an isolated Granular column, this is due to the fact that the interior columns are confined by the surrounding columns which resulted in the increased stiffness and in turn slightly increases the ultimate load carrying capacity. Hence a group of Granular columns installed in soft soils probably undergoes a combined bulging and bearing type failure and short column groups can fail by end bearing.

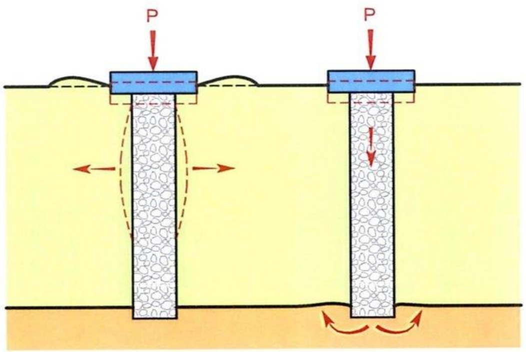


Fig.1 Failure of granular column due to bulging

General shear failure approach

Madhav and Vitkar (1978) considered the plane strain version of a granular pile as a granular trench and proposed the failure mechanism. The theory is developed for a c-φ soil. The ultimate capacity (Qult) is given as

$$Q_{ult} = C_2 N_c + 1/2\, \gamma_2 B N_\gamma + \gamma_2 D_f N_q$$

Where $N_c = (C_1/C_2) N_{C1} + N_{C2}$ and

$$N = (\gamma_1/\gamma_2) N_{\gamma 1} + N_{\gamma 2}$$

N_{c1}, N_{c2}, $N_{\gamma 1}$, $N_{\gamma 2}$ and N_q are dimensionless factors, depending on the properties of soil. C_1 and γ_1 are cohesion and density of the trench material, and C_2 and γ_2, those of soil.

Encased Granular columns

From the earlier studies it is proved that the Granular column has a proven record of experience, but these Granular columns becomes ineffective when it is installed in very soft clays, because of the very poor confinement offered by the surrounding soft clays. In such situations the Granular column tends to undergo excessive bulging and the soft clays enters into the voids of the aggregates (Mekema et al 1975). Hence the Granular columns itself needs an encasement for its improved performance. Geotextile has been used as an encasement for Granular columns successfully in recent years. Van Impe (1989), proposed the concept of encasing the Granular columns by warping with geosynthetics. Fig 2 shows the concept of encased Granular column.

This encasement provides additional restraint and it also brings several beneficial to the Granular columns like increase in stiffness of Granular columns, prevents intermixing of Granular columns and soil, improving drainage characteristics etc. (Malarvizhi, and Ilamparuthi 2007). Katti et al (1993) proposed a theory for improvement of soft ground using Granular column with geosynthetic encasement based on particulate concept (Dheerendra Babu et al, 2013). Several researchers focused on studying the behaviour of encased Granular columns in soft clays. Castro and Sagaseta (2010) reported that performance of Granular columns is improved by providing stiff encasement under moderate loads. Murugesan and Rajagopal (2007) performed laboratory model test on the Granular column installed in a unit cell tank, reported that the modulus of the encased Granular column plays major role in improving the strength of the encased Granular columns.

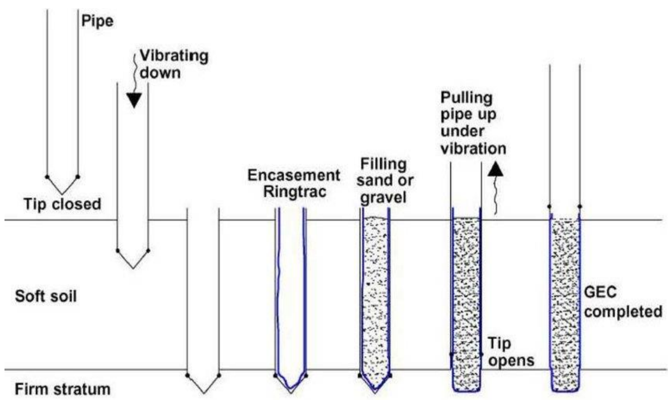


Fig.2 Encasing of granular pile with geotextile

2. OBJECTIVES

- To study the Load-settlement behaviour of clay soil, granular pile and granular pile with geotextile encasement.
- To see the effect of gradation of granular material on load bearing capacity and settlement response on clay soil.

3. MATERIAL USED AND METHODOLOGY

3.1 Material Used for the Study

3.1.1 Bentonite Clay

Bentonite clay used for the experimentation was Sodium bentonite which is naturally occurring hydrated aluminum silicate clay. It exhibits very high free differential swell index along with the high absorbency properties. Bentonite used in this study is sodium bentonite purchased from the industrial area of Mundka in Delhi. It was subjected to various laboratory tests as per IS codes to determine the various properties and results are as given in Table 1.

3.1.2 Granular Material

In the present study, we used fine aggregate as granular material whose size is less than 4.75mm and retained on 300µ. Specific gravity of granular material is calculated as per IS code. We make different mix proportion of fine aggregate and select the best sample which gives least settlement response against a corresponding load.

TABLE 1: Basic Properties of Bentonite after testing it as per different IS Codes

Observed Physical Properties	Values
Colour	Light brown with yellowish shade
Specific Gravity	2.666
Liquid Limit (WL)	234.48%
Plastic Limit WP)	62.41%
Plasticity Index (IP)	172.07%
Shrinkage Limit (%)	15.42%
Differential Free Swell index (FSI %)	670.07%
Optimum Moisture Content (OMC)	25%
Maximum Dry Density (MDD)	17.481 kN/m^3

TABLE 2: Basic Properties of Granular Material as per IS 2386-part 3 1963 code

Observed Physical Properties	Values
Colour	Light orange
Specific Gravity	2.50
Mean size	Size passing from 4.75mm and retaining on 300µ.

3.1.3. Geotextile

In the present study, to prevent bulging of granular material into the clay under compressive load, we encased it with Non-woven Polyester geotextile fibre which was brought from S H filter fabric, Delhi and all the properties of it were already tested, and it was provided by the supplier to us.

3.2 LABORATORY EXPERIMENTS CONDUCTED

3.2.1 Atterberg Limits

Consistency test is performed for determining the Liquid Limit and Plastic Limit is as per IS 2720 (Part 5) –1985. For that we prepare 5 different samples at different moisture content and kept the moisture content for 7 days. Then the sample was tested after thoroughly mixing. Both liquid limit and Plastic limit was determined as per above mentioned IS code and the result is shown in table1.

Fig.3,4 Determination of liquid limit and plastic limit

3.2.2 Specific Gravity (for Bentonite Clay)

For determining the specific gravity of Bentonite clay we use pycnometer method and 2-3 times trials need to be done in case of Bentonite clay. We use around 130-140g Bentonite clay for determining the specific gravity as per IS 2720 (Part 3/Sec 1) – 1980 and the results is shown in the above table 1.

Fig.5 Test for Specific gravity of clay.

3.2.3 Modified Proctor Compaction Test

We use heavy compaction test on Bentonite sample as per IS 2720 (Part 8) 1980. The compaction curves for bentonite was obtained and the OMC and MDD values are given in above Table 1. These OMC and MDD values obtained from laboratory compaction test provide a reference point while estimating the actual water content of the field-compacted soil liner.

Fig.6 Modified Proctor compaction test

3.2.4 Differential Free Swell Index

Free Swell Index of the Bentonite clay is determined as per IS 2720 (Part 40) – 1977. As we know Bentonite exhibit highly expansive nature, so we took proper care while determining its free swell index and took only 2g of the sample to get more precise results. Sample was putted into the distilled water to see the swelling property of bentonite and the same was putted into the Kerosene to compare the results. The results is mentioned in the above table1.

Fig.7 Free swell index test for clay

3.2.5 Shrinkage Limit

Shrinkage limit of the Bentonite is determined as per IS 2720(part 6)-1972. For that we use bentonite oven dry cake and use mercury to get the exact shrinkage value of the Bentonite. The results for the same is mentioned in the above table1.

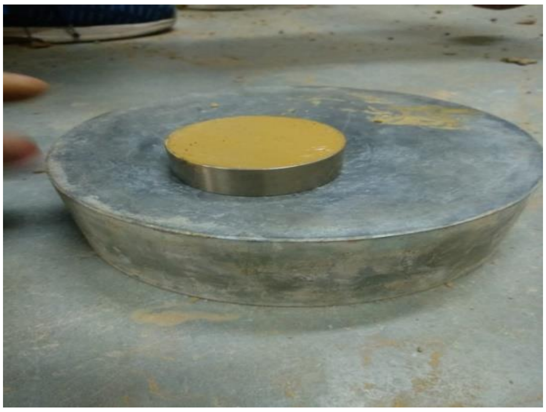


Fig.8 Shrinkage limit test for clay

3.2.6 Specific Gravity (for Granular material)

For determining the specific gravity of Granular material (less than 4.75mm) we use pycnometer method as per IS 2386(part 3)-1963. The results for the same is mentioned in the above table2.

Fig.9 Specific gravity for granular material

3.2.7 Sieve Analysis (for Granular material)

For finding out the best grade at which we get the minimum deflection corresponding to a particular load or maximum bearing capacity corresponding to a particular penetration, we had done Sieve analysis of the granular material as per IS 2386(part1)-1963. Sample was passed through 4.75mm, 2.36mm, 1.18mm, 600μ and 300μ in between them we got 4 different samples of granular material and those samples were used to get the best gradation.

Fig.10 Sieve analysis for granular material

3.2.8 California Bearing Ratio

The CBR tests was performed for the sample of bentonite clay for soaked condition with four days soaking (96hrs) in accordance with IS: 2720 (Part 16) –1987. Sample of Bentonite was first prepared at OMC and then the mould is prepared using heavy compaction and filled in 5 layers. Different moulds are prepared with different mix proportion of granular pile to get best gradation results i.e the least load settlement corresponding to a particular load. After getting the best gradation results the same was tested with encasing the granular material with geotextile. Proving ring used for the calibration is of the capacity of 10kN and it was properly calibrated before performing the CBR test. It's calibration factor or load factor was 1.38kg/division or 13.54N. The results are shown below in the result section.

4. RESULTS

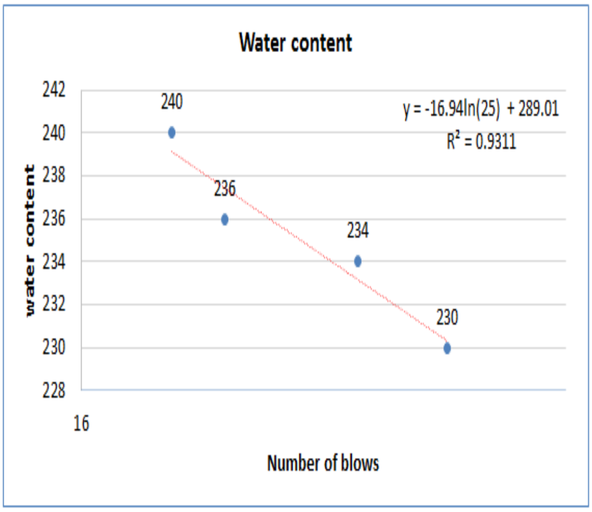
4.1 Atterberg Limits

4.1.1 Liquid Limit (W_L) = 234.48%

4.1.2 Plastic Limit (W_P) = 62.41%

4.1.3 Plasticity Index (I_P) = (W_L-W_P)

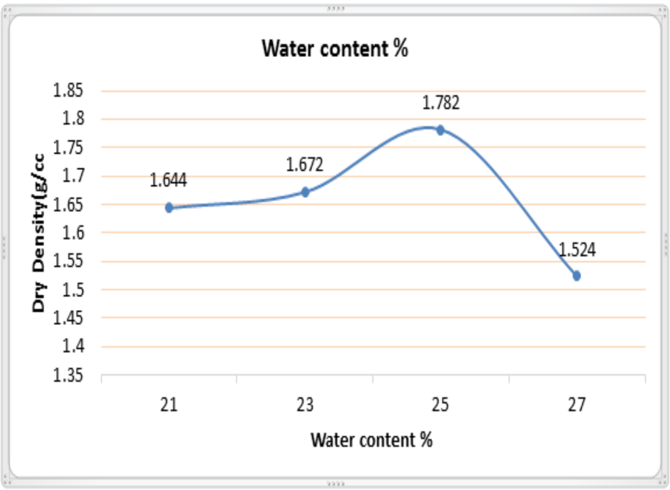
= 172.07%



Graph 1 Water content % VS No. of blows

Fig.11 Determination of moisture content

4.2 Modified Proctor Compaction Test



Graph 2 Dry density VS water content%

Optimum moisture content (OMC) = 25%

Maximum dry density (MDD) = 17.481kN/m³

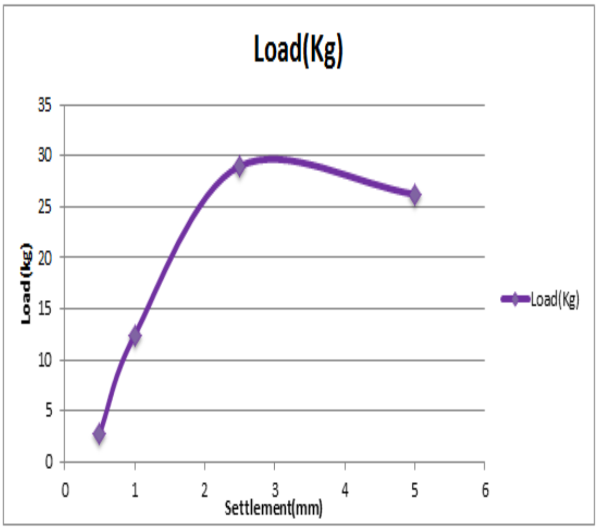
4.3 Shrinkage Limit and free swell index

Shrinkage limit= 15.42%

Free swell index= 670.07%

4.4 California Bearing Ratio

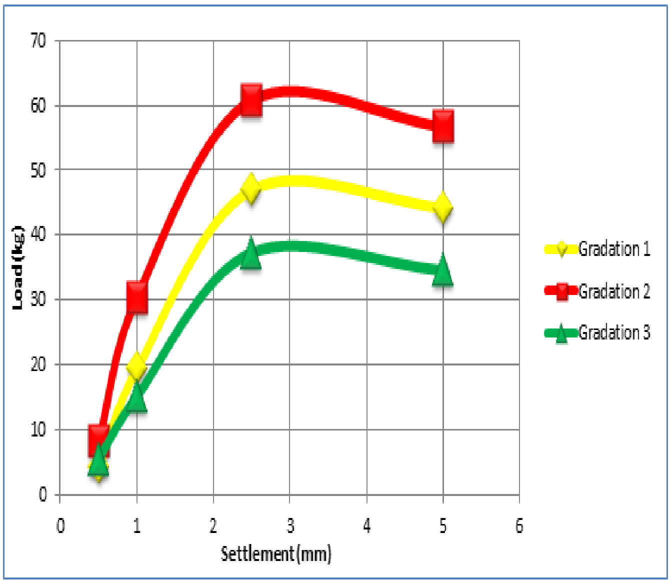
4.4.1 Load VS settlement behaviour of Pure Bentonite is shown below.



Graph 3

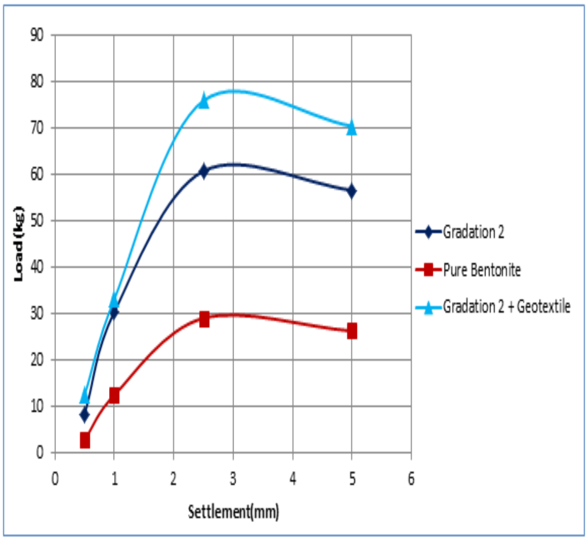
Fig. 12, 13 CBR test for pure clay soil

4.4.2 Load VS settlement behaviour of Bentonite clay with the insertion of granular pile of different mix proportion is shown below.



Graph 4

4.4.3 Load VS settlement behaviour of Bentonite clay with insertion of granular pile of best gradation mix proportion (Gradation 2) encased with non- woven geotextile is shown below.



Graph 5

Fig. 14 CBR mould prepared with granular pile

Fig. 15 CBR mould in soaked condition

Fig. 16 CBR mould prepared with granular pile encased with non-woven geotextile

Fig. 17 Testing of sample with granular pile encased with non -woven textile

5. DISCUSSION

After classifying the soil by performing various tests it was observed that the soil is highly plastic in nature. It exhibits high swelling properties which makes it unsafe for direct construction purposes and keeping all these things in mind it was tested under extreme condition like performing modified Proctor compaction test instead of Light Proctor test, CBR test is also performed in soaked condition(after 96hrs) instead of unsoaked condition so the worst scenario results get observed.

Granular material (fine aggregate) used for the insertion of pile is sieved as per IS 2386(part1)-1963. After sieve analysis different grades of the retained weights were prepared to get the best results i.e granular pile having least settlement against a load and having maximum bearing capacity.

Load settlement behaviour of clay

Graph 3 was plotted between settlement and load to see the settlement response of pure clay soil when acted upon by a compressive load. It can be concluded that the settlement increases as the compressive load increases. It can be seen that the volume decreases rapidly due to the presence of more void initially and after that it fails after a significant decrease in volume. It was observed that the clay bears more load at the settlement of 2.5mm as compare to settlement of 5mm because as the settlement increases the clay particle already be disrupted and it will be easier to break them as compare to fully packed strata.

Effect of Gradation

Graph 4 was plotted between load and settlement and the results were compared of different grading of granular material. It was observed that gradation 2 which is a mixture of all the four retained weights of granular material through sieve in equal proportion. It gives the highest bearing capacity at the settlement of 2.5mm among 3 different grading proportion so a significant effect can be observed if the grading is done in best composition. This significant effect may be due to the least voids formation and best homogeneous mixture when the granular material is mixed in equal proportion.

Effect of Encasing and Non encasing of granular material

Graph 5 shows the load and settlement response of pure clay soil, clay with granular pile without encasement of geotextile and granular pile with encasement of granular pile. It was observed that the granular pile without encasement of geotextile fails early and bears less load as compare to the granular pile with the encasement of geotextile which bears more load or have the more bearing capacity at the same settlement of 2.5mm. This is due to the failure of pile by bulging and dispersion into the clay or it can be said it is due to the confinement problem. When the pile is not able to resist the load, it fails. The granular materials are mixed into the soft soil. In the end, the pile is fully broken due to the mixing of the granular material into the clay.

Whereas this problem i.e bulging and confinement does not arise in case of encased granular pile and that's why it has more bearing capacity as compare to the pile without encasement of geotextile and gives less settlement against a corresponding load.

Improvement and estimation of load bearing capacity

In pure clay soil, the bearing capacity was found around 29.001kg at 2.5mm settlement and after the insertion of granular pile it increased upto 60.764kg. The load bearing capacity is increased nearly 109% in case of granular pile without geotextile. Further when the pile was encased with geotextile the load bearing capacity was found around 75.995kg at 2.5mm settlement i.e there is an increment of around 162% in load taking capacity as compare to pure clay and around 25% increase load taking capacity after encasing of pile with geotextile.

6. FUTURE WORK

- Similar work will be conducted by moderation in the shear strength of the clay soil and by using different types of granular materials as pile forming material.
- In the present study, single granular column is studied. Group of column in clay soil also needs to be studied.
- The swelling and shrinkage behaviour of Bentonite clay soil reinforced with granular pile or with some other material also needs to be studied.
- In the present study, the load was applied only on the granular material pile and not on the whole area which to study the composite material.

7. Conclusion

From the experiments conducted and the graphs shown above, following conclusion can be made.

[1] The load bearing capacity of clay soil (Bentonite) alone is very low as per the tests performed in soaked condition and the CBR value comes out to be nearly 2.11% at the penetration depth of 2.5mm.

[2] The load bearing capacity of clay soil (Bentonite) is found to be more after the insertion of granular material pile and it was found to be different on different mix proportion of granular material.

[3] It can be concluded that the load bearing capacity of the clay soil is affected due to different mix proportion of granular material pile and it was found to be maximum when all the 4 different retained weights are mixed in equal proportion.

[4] When the granular material pile of best gradation was encased with non-woven geotextile the bearing capacity of the clay is found to be maximum at a penetration depth of 2.5mm and the CBR value corresponding to that comes out to be 5.54%.

[5] Considering all the values of settlement under compressive load it can be concluded that in case of using geotextile encased granular pile there would be a considerable difference in load taking capacity as compare to pure clay soil and pile without encasement of geotextile.

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