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SOIL STABILIZATION USING FLYASH AND GEOGRIDS

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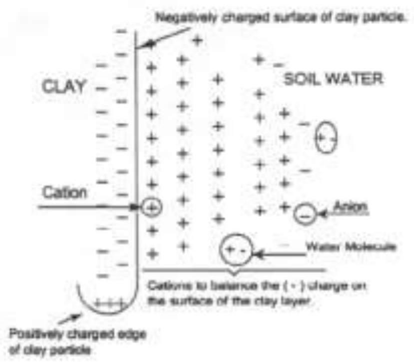
Abstract:- Sandy soils urge many problems for geotechnical engineers commonly, there low Share powers combined with the magnitude of the proposed loads require the soil to be secured. The granular soil deposits are very unbound and experience more settlement. Depending on the structural load auctioning and the depth of the sand layers, large settlement may happen in these deposits. Hence, the insitu soil is treated to adopt the proposed loads with maximum efficiency . various techniques are available for ground security eg, grouting, freezing, dewatering, compacting etc. Most however are site specific , often high priced reinforcing the soil below shallow foundations with geo synthetic reinforcement is one of the firmest growing techniques in the field of geotechnical engineers. Large progress in the bearing ability was observed in reinforced soil over the unreinforced soil by bringing in geogrid reinforcement . the decrease in the cast was 1.5% of total cost. Composition time for geogrid reinforcement bed was less than of a stone column and is expected to be more firmly fixed with time.

1.0 Introduction

1.1 Swelling: The mechanism of swelling in expansive clays is complex and is influenced by a number of factors. Expansion is a result of changes in the soil water system that disturb the internal stress equilibrium. Clay particles are platelets which have negative electrical charges on their surfaces and positive charge edges. The negative charges are balanced by captions in the soil water that become attached to the surfaces of the platelets by electrical forces. The electrical inter particle force field is a function of both the negative surface charges and the electrochemistry of the soil water . Van der Waals surface forces and adsorptive forces between the clay crystals and water molecules also influence the inter particle force field. The internal electrochemical force system must be in equilibrium with the externally applied stresses and capillary tension in the soil water.



Internal Electrochemical System of Soil

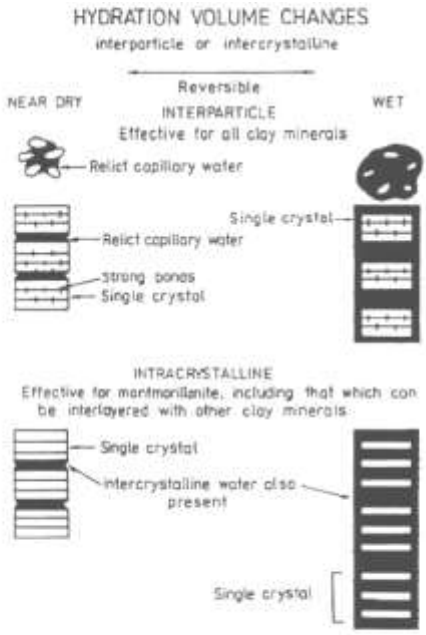
There are two basic mechanisms involved in swelling phenomena :

1.1.1 Inter Particle r Inter Crystalline Swelling:

It is effective for all kinds of clay minerals. In a nearly dry clay deposit relict water holds the particles together under tension from capillary forces. On wetting, the capillary tensions are relaxed and the clay expands.

1.1.2 Intracrystalline Swelling:

It is a characteristic of the Montmorillonite group of minerals. The layers that make up the individual single crystals of Montmorillonite are weakly bonded, mainly by water in combination with exchangeable cations. On wetting, water enters not only between the single crystals, but also between the individual layers that make up the crystals.

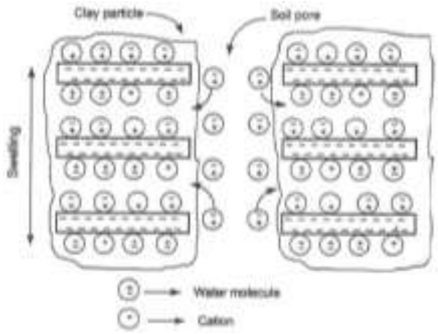


Mechanism of Swelling

There can be two factors causing intra crystalline swelling:

a) The unbalanced electrostatic charges on clay-particle surfaces draw water molecules into the area between silicate sheets, thus forcing them apart.

b) The captions attracted to the clay surfaces provide the other factor is swelling behavior. Because of the attraction of the negatively charged clay particle surfaces for cation, small spaces within or between clay particles may contain a higher concentration of cations than larger pores within the soil. These conditions show in diagram create an osmotic potential between the pore fluids and the clay-mineral surfaces. Normally, cations diffuse from a higher concentration to a lower concentration in ordesr to evenly distribute the ions throughout the solution. In expansive soils, because ions are held by the clay particles, water moves from areas of low ionic concentration (high concentration of water) to areas of high ionic concentration (low concentration of water) within clay particles or aggregates. This influx of water exerts pressure, which causes the clay to swell.



Swelling of Clay Rich Soil

International Journal of Information Movement Vol.2 Issue VII (November 2017)
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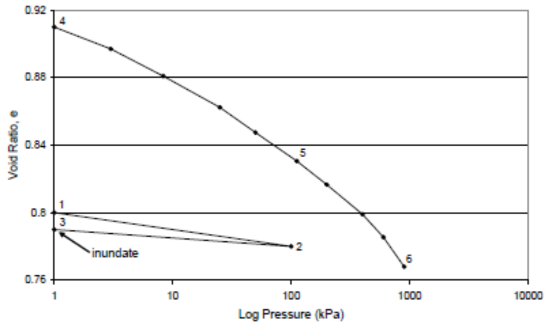
1.2 Method A

The specimen is inundated and allowed to swell vertically at the seating pressure applied by the weight of the top porous stone and the loading plate. Readings of swell are taken at 0.1, 0.2, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, and 30.0 minutes and 1, 2, 4, 8, 24, 48, and 72 hours. The specimen is left to swell till the end of primary swell . After completion of primary swell a vertical pressure of approximately 5, 10, 20, 40, 80, etc., kpa is applied until the specimen is recompressed to its initial void ratio/height. Method A can be modified to place an initial vertical stress, σ1, on the specimen equivalent to the estimated vertical pressure on the in situ soil within 5minutes of placing the seating pressure and securing the zero deformation reading.



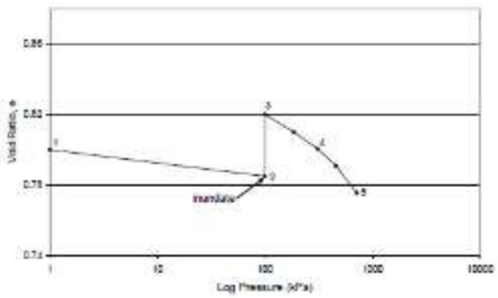
Log pressure curve for method A

Deformation is read within 5 minutes of placing the vertical pressure. Then the vertical stress is removed, except for the seating pressure. Deformation is recorded within 5 minutes after removal of σ1, the specimen is inundated, and the test is continued as explained in the receding paragraph.

This method measures (a) the primary swell, (b) present heave for vertical confining pressures up to the swell pressure, and (c) the swell pressure.

1.2.4 Method B

A vertical pressure exceeding the seating pressure is applied within 5minutes of placing the seating pressure. Deformation is read within 5 minutes of placing the vertical pressure .The specimen is inundated immediately after the deformation is read and deformation is recorded after elapsed times similar to Method A until primary swell is complete. The specimen is loaded vertically up to its initial void ratio/height as in Method A.

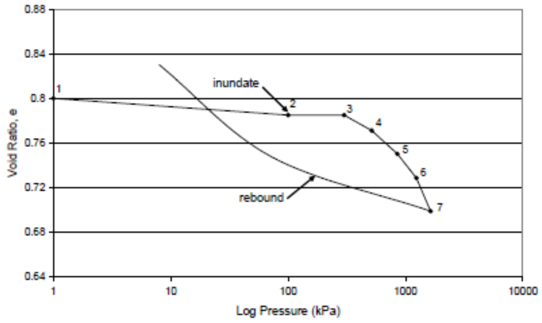


Void Ratio – Log Pressure Curve for Method B

This method measures (a) the present heave or settlement for vertical pressure usually equivalent to the estimated in situ vertical overburden and other vertical pressure up to the swell pressure, and (b) the swell pressure.

1.2.5 Method C

An initial vertical pressure, σ1, which is equivalent to the estimated vertical in situ pressure or swell pressure is applied within 5 minutes after placement of the seating pressure.



Void Ratio – Log Pressure Curve for Method C

Deformation is read within 5 minutes after placing σ1 and the specimen is immediately inundated with water. Increments of vertical stress as needed to prevent swell is applied .The specimen is loaded vertically as in Method A. The rebound curve following consolidation is determined.

This method measures (a) the swell pressure, (b) pre-consolidation pressure, and (c) present heave or settlement within the range of applied vertical pressures.

2.0 Experimental Investigation

2.1 Loading Arrangement and Test Tank

Tests were lead using a model tank having internal dimensions of 900mm in length 900mm in width and 900 mm in height figure 1 shows the schematic order of tests setup with model footing in situation. Dimension of wooden footing used in this one g model test is 150mm *150mm*20mm. The tank is made up of compact firm and it has high rigidness. The portal frame is sustained to the floor by means of base plate with anchor bolt arrangements. A stable beam acts as a stay\ fou8ndation for the pre calibrated proving ring and hydraulic loading jack arrangement. The sandy soil is levelled up to the head of soil. Isolated footing is placed at the centre of the tank, and then the proving ring is placed on top of the footing the piston in the loading frame reaches the proving ring with the help of hydraulic jack. Two dial gauges are settled on the footing to estimate the settlements.

2.1.1 Properties of Test Medium

Equally graded river sand was chosen as a test medium. Tests were carried on sand samples for gradiation, specific gravity, maximum and minimum dry density and strength of sand. Checked pounding in order to attain homogenous sand beds. For the unit weights elaborated in this investigation the required quantity of sand was weighted for every layer to determine the total weight of the sand bed. The unit weights were measured to be.14.4kn/m for soil of RD. Relative density of 17% and 16.6kn/m for soil of RD 57% and the corresponding friction angles were 33 degree and 37 degree respectively .

2.1.2 Properties Df Reinforcing Material

The thickness and aperture of the chosen geogrid choosed material were measured using the vernier caliper. For the explain some of the of tensile strength, geogrid sample of 100mm length and 200mm width is used and determined by wide width arrangement. Some of the commercially available grogrid materials are rolled below in table 1 and for the present investigation to geogrid of CE 111 was chosen. The specialty of extreme tensile strength of designed reinforcing material at maximum strain of 10% given by producers is tabulated in table 1.

Table 2.1 Properties Of Reinforcing Material

Materials ultimate tensile strength	Thickness	aperture size
CE111 2.00KN/m	2.8mm	2.8mm
CE121 7.68KN/m	3.2mm	3.2mm
CW121 5.80KN/	5.5mm	5.5mm

2.1.3 Fabrication of Geogrid Cages

The depth of effect of carrying ability is B and for settlement is 2B For an isolated footing. The geogrid circular pipes of 40cm diameters are adopted and used as vertical reinforcement by changing the interspace are shown in figure 2. The length of one geogrid cell used is 500mm and these are placed at 0.3b depth from te base footing. Where b is the width of footing .the no. rows of grogrid reinforcement is 4 and the no. Of columns of reinforcement is 4.

Figure 2 Geogrid Cages

2.2 Conclusions And Discussions

Tests were carried on model footing placed on sand bed of unbound density [14.4kn/m]and their achievement was studied in order to know the result of reinforcing the sand during the tests load strain Vs settlement manner of isolated footing settled over reinforced sand was enrolled the results acquired from the 1G model tests ar introduced and discussed below.

The load Vs settlement report gained for unreinforced sand is shown in fig. 3.the curves show that as the load increases the settlement also increases swiftly. The increase in load beyond certain limit, increase the settlement flying and resulting to hall shot.

2.3 Performance of Isolated Footing on Reinforced Soil

Fig4. Shows the load Vs settlement connection for the soil sample with and without grogrids. In this case the grogrid cage spacing is achieved as 5cm .the likeness shows that firstly as the load increases the settlement als increases slowly with increase in rate of settlement .from te results it is also argued that the reinforced soil exhibits more opposition to allotted load than the soil without reinforcement.

For instance ,at a load of 0.5kn the soil without grogrids show more settlement than the soil without grogrid.same remark is interpreted in fig. 5 for soil reinforced using gergrid cages of20cm c/c spacing.

2.4 Effect of Vertical Spacing of Geogrid Freinforcement

Retationship between loads Vs settlement for different interspace of geogrids is presented in fig. 6.from the results it is remarked that as the vertical spacing between the geogrid cages decreases the settlement occurred in specific load intensity also decreases resulting in the increases in extreme upholding ability of the soil. Even though the initial parts of the response curve illustrate minimal increase ,the upholding ability increase is significant in latter part of the curve for less spacing of reinforcement the percentage increases in upholding ability of soil due to reinforcement provided by the geogrids placed at 5cm,10cm,15cm,20cm spacing are 50%,45%,35% and 29% respectively.

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Hence this response proved the vertical spacing of geogrid cages assist prisely in enhancement of uploading ability.

3.0 Numerical Modelling

A finite element model using pilax 2D is simulated to examine the effect of geogrid reinforcement in increasing the upholding capacity of granualar soils. The footing is discretised using plate elements were as the soil medium (i,e test medium) is discretised using 15 node veg elements. The material of footing is idealised with straight elastic material quality. Mohar – columb plasticity material is used to idealise electroplate response of sand . Fig 7 shows the typical model simulated for numerical analysis.

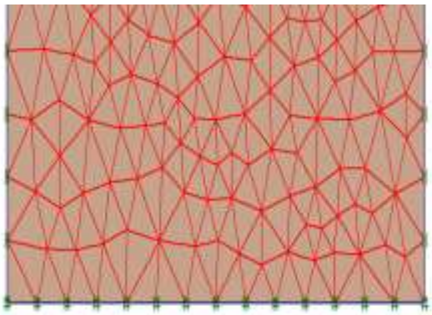
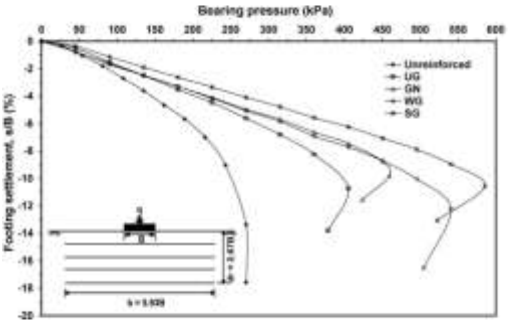


Figure 7 Finite Element Model

4.0 Comparision B/W Experimental and Numerical Analysis

Fig. 8 exhibits the comparison b/w the land settlement response curve gained through numerical analysis and from the laboratory trials for unreinforced case. It is clear that for specific load strain plaxis undervalues the settlement by nearly 80% but load settlement design is almost similar. This shows that the remarks gained from numerical analysis soundly good with the tested results. However numerical analysis forecasts inferior settlement values for all load intensities.



Bearing capacity Improvement for different geosynthetic products (Madhavi Latha and Amit Somwanshi 2009)

Fig 9 shows the similarity of experimental and finite element results for soil reinforced with geogrid cages of vertical distance 5cm. Results shows that the settlement increases experimentally with increase in load, which compared reasonably well with the experimental results. This response proves that the material model considered and cases pretended in the FE model predicts the response of footing soundly

Stress versus strain curves for different confining pressures from 50 KPa to 150KPa were plotted from the tests results of triaxial compression test performed on the sand and sand reinforced with different combinations of geogrid sheets. These plots are shown in Figs.3 (a-c). Because preparation of identical samples of geogrid reinforced sand at 20 % relative density beyond 5 layers of geogrid sheets was not possible hence the present investigation was restricted to 5 layers of geogrid sheets only. The improvement in strength parameters (stiffness modulus and shear strength parameters) of sand reinforced with 1 layer and 2 layers of geogrid sheets was not significant and hence it is non included in this report for discussions. The values of stiffness modulus ($\sigma d/\epsilon$) for geogrid reinforced sand having different combinations of geogrid sheets were computed from stress-strain curves and are shown in Table 3. The modified failure envelopes of sand reinforced with different combinations (3 layers, 4 layers and 5 layers) of geogrid sheets were drawn which are shown in Fig.4. The shear strength parameters (c and ϕ) of reinforced sand were measured from the modified failure envelopes and are shown in Table 4.

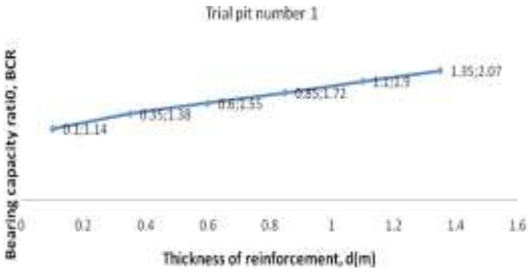


Figure 4.01: Relationship between bearing capacity and thickness of geogrid material on a soil

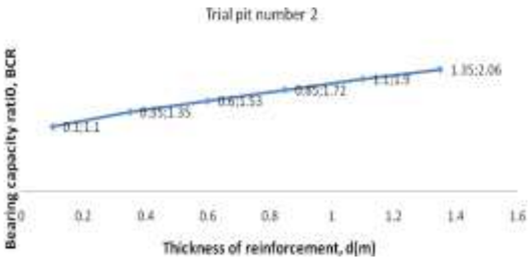


Figure 4.02: Relationship between bearing capacity and thickness of geogrid material on a soil

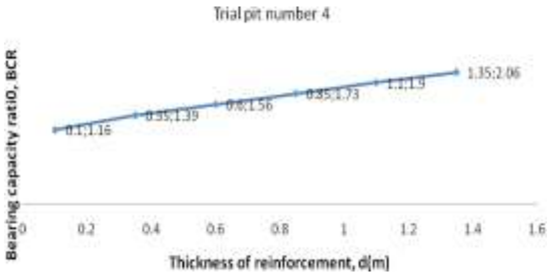


Figure 4.02: Relationship between bearing capacity and thickness of geogrid material on a soil.

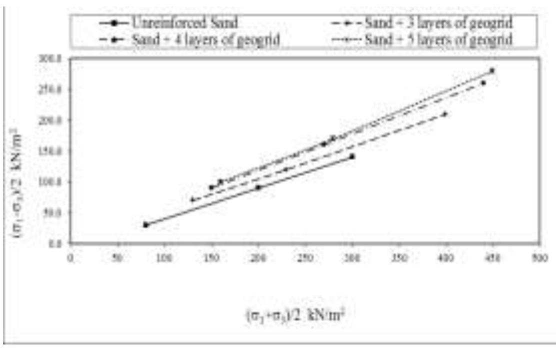


Fig. 3(c): Stress versus Strain Curves of Geogrid Reinforced Sand under Confining Pressure of 150 kPa

5.0 Conclusion

5.1 Effect Of Geogrid Sheet

Based on the results of tri axial compression tests performed on geogrid reinforced sand for different combinations of geogrid sheets such as 3 layers, 4 layers and 5 layers, the computed values of stiffness modulus and shear strength parameters of reinforced sand are shown in Table 3 and Table 4 respectively. It is observed from Table 3 that the stiffness modulus of reinforced sand increases with the increase in confining pressure and the number of geogrid sheets. The results of column 5 of Table 3 show the values of stiffness modulus of reinforced sand corresponding to different confining pressures and with different combinations of geogrid sheets. It is clear from the values of stiffness modulus that it increases with the increase in confining pressure and this aspect is observed for all the combinations of geogrid sheets. This is due to the fact that frictional capacity of reinforcements is directly proportional to major principal stress.

As major principal stress increases with the increase in confining pressure, frictional capacity of reinforcements, therefore increases with the increase in confining pressure. It is further observed from column 6 of Table 3 that the average increase in stiffness modulus of reinforced sand increases with the increase in number of layers of geogrid sheet and this trend is observed for all the combinations of geogrid sheets. For instance the average stiffness modulus of unreinforced sand is 3387. When 3 layers of geogrid sheet is added to the sand, the average stiffness modulus of reinforced sand increases to 5263 i.e. improvement in stiffness modulus of sand is 55 % (last column of Table 3).Similar trend is observed from the results of Table 3 for 4 layers and 5 layers of geogrid sheets also and the maximum improvement in average stiffness modulus of sand is 92 % over plain sand for 5 layers of geogrid sheet.

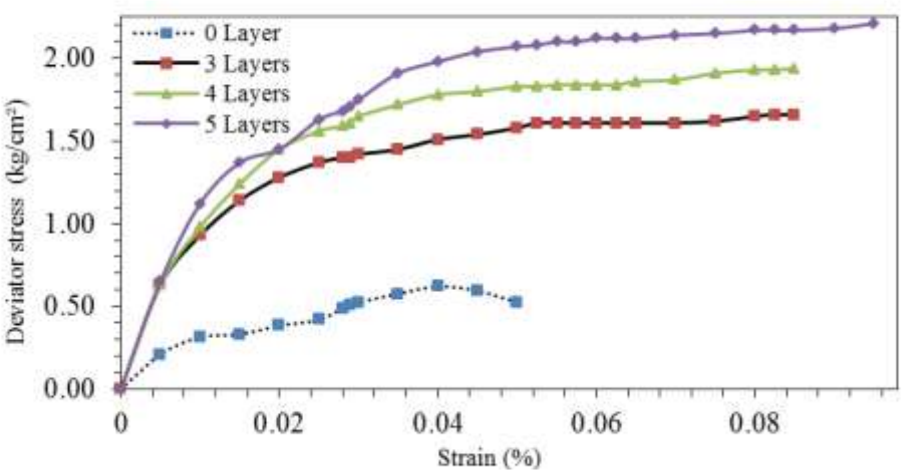


Fig 4: Modified failure Envelopes of Geogrid Reinforced Sand

The significant increase in average stiffness modulus of reinforced sand due to addition of geogrid sheet improves the load-settlement characteristics of sand and the amount of immediate settlement will be reduced significantly. It is observed from Table 4 that the shear strength parameters (c and ϕ) of reinforced sand increases with the increase in number of layers of geogrid sheets and this aspect is observed for all the combinations of geogrid sheet. Column 2 and Column 3 of Table 4 show the values of angle of internal friction (ϕ) and cohesion (c) of reinforced sand. Column 4 and Column 5 Because the cohesion (c) of unreinforced and reinforced sand for 3 layers and 4 layers is zero, hence improvement in cohesion (c)component of shear strength of sand has not been shown. The cohesion component of shear strength of reinforced sand for 5 layers of geogrid sheets is observed to be 25 kPa which indicates that there is a substantial improvement in shear strength parameters of reinforced sand for 5 layers of geogrid sheets. The results of Column 5 of Table 4 clearly show that the percentage increase in (ϕ) values of reinforced sand are 14, 29 and 36 over plain sand for 3 layers, 4 layers and 5 layers of geogrid sheet respectively.

The significant increase in shear strength parameters of sand due to addition of geogrid sheet improves the load carrying capacity of sand and geogrid reinforced sand can be used for supporting the heavier loads of civil engineering structures. The increase in stiffness modulus and shear strength parameters of reinforced sand is due to the fact that inclusion of geogrid sheet into sand improves its load deformation behaviour by interacting with the sand particles mechanically through surface friction and also by interlocking. The function of bond or interlock is to transfer the stress from sand to the geogrid sheet by mobilizing its tensile strength. Thus, geogrid sheet works as frictional and tension resisting element. Further, addition of geogrid sheet makes the sand a composite material whose strength and stiffness is greater than that of unreinforced sand. An in situ system of stabilization using geogrid in the form of geogrid cell was organised and the increase in the upholding ability is observed from both the experimental and numerical study. The findings of the study are under The extreme upholding ability of the reinforced soil increases with decrease in vertical distance B/W the geogrid cages. The maximum %age increases in the upholding ability of the reinforced soil is observed as 50% for geogrids placed at 5cm distance and 29% for geogrid placed at 20 cm spacing. The increase in the load bearing ability of the reinforced soil is due to the extensional sticky shear. The remarks of FEM analysis suits properly with the results of model test carried.