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(71)**Name of Applicant :**
1)Dr.A. Hemalatha
Address of Applicant :Professor/Head NPR College of Engineering and Technology, Dindigul- 624401 Tamil Nadu, India Tamil Nadu India
2)Dr. K.Basker
3)Dr.K.Mohan Das
4)Dr. C. Selin Ravi Kumar
5)Mr.D. Kanakaraju Yadav
6)Dr. S. M. Subash
(72)**Name of Inventor :**
1)Dr.A. Hemalatha
2)Dr. K.Basker
3)Dr.K.Mohan Das
4)Dr. C. Selin Ravi Kumar
5)Mr.D. Kanakaraju Yadav
6)Dr. S. M. Subash

(57) Abstract :
Granular fill material is used to improve the bearing capacity and liquefaction behavior of soil. In many cases the depth of replaced granular fill becomes very thick, thus lead to the excessive cost. An experimental investigation was initiated to evaluate the effect of geogrid reinforcement. It was observed that the extensile force of the geogrids gradually contributes to the improvement of the reinforced specimens shear strength and the extensile force increased with the increase in the number of Geogrid layers. The findings conclude that the geogs influence the shear behaviour of granular filling and improve the interlocking strength of the fill, thus improving its shear strong. The extensible force provides better interlocking property to the granular filled, leading to the decrease in the pore water pressure.

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FORM 2
THE PATENTS ACT, 1970
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&
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COMPLETE SPECIFICATION
(*See* section 10 and rule 13)

1. TITLE OF THE INVENTION

Effects of Geogrid Reinforcement on The Static Liquefaction Behavior of Granular Fill by Triaxial Test Method

2. APPLICANT(S)

a)	Name	:	Dr.A.Hemalatha
b)	Nationality	:	Indian
c)	Address	:	Professor & Head NPR College of Engineering and Technology, Dindigul- 624401,Tamil Nadu, India
a)	Name	:	Dr. K.Basker
b)	Nationality	:	Indian
c)	Address	:	Professor CMR College of Engineering and Technology, Hyderabad- 501401, Andhrapradesh, India
a)	Name	:	Dr.K.Mohan Das
b)	Nationality	:	Indian
c)	Address	:	Professor CMRCET(Autonomous) Kandlak oya, Medchal Road,Hyderabad-501401, Telangana, India
a)	Name	:	Dr.C.Selin Ravi Kumar
b)	Nationality	:	Indian
c)	Address	:	Associate Professor Malla Reddy Engineering College(Autonomous), Maisammaguda, Kompally, Secunderabad-500100 Hyderabad, Telangana, India
a)	Name	:	Mr.D. Kanakaraju Yadav
b)	Nationality	:	Indian
c)	Address	:	Assistant Professor Malla Reddy Engineering College (Autonomous),Maisammaguda, Kompally, Medchal, Hyderabad-500100, Telangana, India
a)	Name	:	Dr.S.M.Subash
b)	Nationality	:	Indian
c)	Address	:	Associate Professor & Head, Department of Civil Engineering,

Guru Nanak Institute of Technology,
Ibrahimpatnam, Ranga Reddy,
Hyderabad-501506, Telangana, India

3. PREAMBLE TO THE DESCRIPTION

COMPLETE

The following specification particularly describes the invention and the manner in which it is to be performed

4. DESCRIPTION

FIELD OF INVENTION

The liquefaction behaviour of soil is commonly associated with the large earthquakes. The sudden increase in pore water pressure cause subsidence of foundations and damage to earth structures. Silty gravel obtained from Karaikudi, Sivagangai District, Tamil Nadu, India was used as a granular fill material in this study. Tri-axial compression tests were performed on reinforced and un- reinforced granular fill with different layers of geogrids.

Background of Invention

The liquefaction behaviour of soil is commonly associated with the large earthquakes and has been severely damage the various buildings, roads and other structures. Static liquefaction of loose and very loose saturated sands is a modern classical mechanic’s subject and the sudden increase in pore water pressure cause subsidence of foundations and damage to earth structures. Therefore, it is very important to consider the liquefaction potential of dams, embankments, slopes, foundation materials and placed fills in addition to that a new stabilisation method should be identified to efficiently combat this problem. The current trend is to improve the engineering properties of the native soil using various soil stabilisation techniques, neither mechanical nor chemical stabilisation techniques. Replacing the existing soil by granular fill material is one of the conventional stabilisation techniques to improve the bearing capacity of the soil. In order to satisfy the required bearing capacity and the allowable settlement, in many cases, the depth of replaced granular fill becomes very high, thus lead to the excessive cost and over exploitation of granular fill. In recent years, reinforcing the soil using geosynthetic reinforcement has been proven as an effective alternative to enhance the strength properties of the native soil. Placing of geosynthetic reinforcement layers in between the soil, provides more tension and lateral confinement to the soil, thus significantly increase the strength properties of the soil and the transform the soil to effectively sustain the applied loads at lower depths. Compared to mixing the discontinuous fibers with a soil mass, reinforcing of soil using geogrid is very simple and the primary advantages of the geogrid are providing lateral and vertical restraint to the soil mass and significantly reduce the settlement. Furthermore, the introduction of geo-synthetic reinforcements could reduce the pavement thickness by 20% to 50%. Past few decades, the application of geogrids in soil reinforcing has been widely carried out and reported. Alawaji studied the effects of width and depth of the geogrid on the behaviour of collapse settlement, deformation modulus and bearing capacity of collapsible soil. The increase in geogrid width and decrease in depth, increase the efficiency of the geogrid system. Liu et al. conducted a large-scale shear test to study the interface shear strength of different soils (sand, gravel, and laterite) against PET-yarn geogrids of various tensile strengths and the test results were shown that the soil/PET- yarn geotextile interface has significantly lower shear strength than soil strength. Phanikumar et al. conducted a series of laboratory plate load tests on fine, medium and coarse sand beds reinforced with different layers of circular geogrids of 120 mm diameter. Test results were shown that the increase in the number of geogrid layers and the decrease in space between them improve the load–settlement response and Load Improvement Ratio (LIR) further. The large-scale direct shear test on geogrid reinforced fresh and fouled ballast was indicated that the geogrid considerably increases the shear strength and apparent angle of shearing resistance. Field test using seven different footing diameters and different granular fill layer thicknesses was conducted by Murat Ornek et al. The test results were indicated that the use of granular fill layers over natural clay soil has a considerable effect on the bearing capacity characteristics. Ahmet Demir et al. carried out sixteen field tests to evaluate the effects of replacing natural clay soil with a stiffer granular fill layer and single-multiple layers of geogrid reinforcement. The test results were shown that use of granular fill and geogrid for reinforced soil footings (RSF) have considerable effects on the subgrade modulus and bearing capacity. Discussion on the design of a geocell foundation based on the experimental investigation and geotechnical problems can be found in Sitharam and Hegde. The results of previous research demonstrated that the geo-synthetic composite enhance the engineering properties of the coarse sub soil significantly. Experimental investigation was carried out to evaluate the beneficial effect of geogrid reinforcement on the static liquefaction resistance of granular fill obtained from Karaikudi, Sivagangai District, Tamilnadu, India was investigated. Triaxial compression tests were performed to evaluate the influence of geosynthetic composite on the static liquefaction resistance of granular fill. The experimental parameters were number of geogrid layers and confining pressures; 100, 150, and 200 kPa. The obtained test results were compared with one another to evaluate the influence of different reinforcement layer on the static liquefaction resistance behaviour of granular fill.

Experimental Program

Granular Fill Material

Silty gravel obtained from Karaikudi, Sivagangai District, Tamilnadu, India was used as a granular fill material in this study. The conventional laboratory tests were conducted to obtain the engineering properties of the granular fill. The specific gravity value of the granular fill was about 2.64. From the Standard Proctor Compaction test the optimum moisture content and maximum dry unit weight were obtained and the values were about 7 % and 21.7 kN/m3 respectively which is shown in Figure 1. The direct shear test was performed and the obtained internal friction angle and the cohesion of the granular fill were 430 and 15 kN/m2 . In order to keep the homogeneity granular fill passing through 4.75 mm was used in both laboratory and field test.

Geogrid

Netlon 121 CE was used as horizontal geogrid reinforcement in this study. It is a bidirectional polypropylene sheet having a thickness of 4 mm . The maximum tensile strength of the sheet was 15kN/m with a square aperture size of 100 mm2. The typical geogrid sheet is shown in Figure 2. The physical and mechanical properties of the geogrid provided by the manufacturer are summarized in Table 1.

Sample Preparation and Test Procedure

In order to investigate the influence of geogrid on the liquefaction resistance of granular fill, a tri-axial compression tests were performed on reinforced and un- reinforced granular fill with different layers of geogrids. All tests were performed on cylindrical specimens with the size of 40mm diameter and with the aspect ratio of 2 (around 80mm). The test specimens are prepared by technique suggested by Ladd, and this technique provides conservative results. A cylindrical rubber membrane was put inside a cylindrical prefabricated mould and it’s both ends were secured. Suction force was applied to the space between the membrane and the mould. The mould was then placed over the Perspex disc. Initially the required amount of oven dried granular fill and water required (optimum moisture content 7 %) for each layer was calculated; then the granular fill and the water were mixed well using a counter current mixer. Followed by the granular fill divided into five parts and the weight of each part were predetermined depending on the desired relative density. Subsequently, the granular fill was placed in the mould by layer by layer and the layer was compacted to the predetermined height to achieve the desired density. When the granular fill reached the preferred depth, a layer of geogrid was placed then the compaction was continued until the granular fill reached its desired height. After placing five layers of sample parts, a single filter paper, a porous stone and over that Perspex disc with a hole for top drainage and a groove for loading ram were placed above the specimen. Finally, all the specimens were tested in tri-axial compression with three different confining pressures; 100, 150, and 200 kPa at a strain rate of 1.25 mm/min. Figure 3 shows the schematic diagram of samples of sand reinforced with different forms of reinforcement. During testing, the shear stress, shear strain, pore water pressure, and specimen failure shapes were observed.

Results and Discussions

Failure Patterns

Three types of failure patterns such as shear band, bulging in the middle and remarkable bulging at the top, were observed and the typical failure modes corresponding to the specimens were summarized in Table 2 and presented in Figure 4. It was observed that the introduction of geogrid in the granular fill modifies the failure mode of the granular fill from shear band to bulging. In the case of nonreinforced granular fill, shear band was observed at the mid-height of the specimens. The similar failure mode was observed in specimen reinforced with one layer of geogrid. Nevertheless, in the case of specimen reinforced with two layers of geogrid, bulging was initiated at the mid- height of the specimen, with the increase in the pressure the bulging was propagated to the top. The failure pattern of the granular fill reinforced with three layers of geogrid exhibited remarkable bulging at the top and decreased towards mid height. The extensile force of the geogrid gradually contributes to the improvement of the reinforced specimens shear strength and the extensile force increased with the increase in the number of geogrid layers, as a result the failure mode changed from shear band to bulging. The failure modes observed in this study were fairly consistent with the previous researches of Xiaobin Chen et al. From the above observation, it can be inferred that the introduction of geared more than one layer will provide considerable extensile force for the improvement of shear strength.

Stress–Strain Behaviour

The experimental observations specifically principal stresses and principal strains were recorded under different confining pressures and the results were summarized in Table 2. The deviatoric stresses-axial strain behaviour of all specimens reinforced under different confining pressures is presented in Figure 5. It was observed that the installed geogrid layers in the granular fill improved the stress–strain response in terms of increase in peak deviatoric stress and decrease in failure strains. Figure 5 shown the unreinforced granular fill exhibited a strainsoftening trend under low confining pressures, nevertheless, the granular fill reinforced with geogrid exhibited strain hardening behaviour. From this observation, it can be inferred that the magnitude of this strain hardening is possibly related to the extensile force provided by the geogrid. During shear under various confining pressures, the influence of the geogrid is not obvious when the total axial strain is less than 1% ($\varepsilon a < 1\%$), and the curves of all reinforced specimens were very close to the un-reinforced specimens, irrespective of the confining pressures. However, the effect of geogrids becomes more obvious when the axial strain is larger than 1% ($\varepsilon a > 1\%$), which can be evident from Figures 5, 6 and 7. For instance, under a confining pressure of 100 kPa and at the respective axial strain of 5%, the deviatoric stress of the un- reinforced specimen was about 155.1 kPa, whereas the granular fill reinforced with the two and three layers of geogrid achieved the deviatoric stress of 237.5 kPa and 275.12 kPa, respectively, which is 53.32% and 77.41%, higher than that of the un- reinforced specimen.

This is a result that the installed geogrid restricts the lateral deformation of the granular fill by its extensile force, leading to the shear contractions and enhancement in shear strength. As a result, the shear stress capacity of the reinforced granular fill increased with the increase in the number of layers. From Figure 5 to 8, it can be understood that the divorce stress of the reinforced specimens increased with the increase in the confining pressure. For instance, the specimen GF-100-2L achieved a ultimate stress of 292 kPa, nevertheless, the specimens GF- 150-2L and GF-200-2L achieved a ultimate stress of 342 kPa and 390 kPa, respectively and which are 17.12% and 33.56 % higher. From the above observation, it can be inferred that the geogrids considerably influence the shear behaviour of granular fill, and the geogrid reinforcement improves the interlocking strength of the granular fill, thus improving its shear strength.

Effects on Pore Water Pressure Behaviour

The pore water pressure of all the specimens under different confining pressures was measured using a hydraulic pressure gauge, and the curves of pore water pressure's development and dissipation were presented in Figures 9, 10 and 11. From Figures 9, 10 and 11 , it can be understood that, irrespective of the confining pressure, the propagation of the pore water pressure curves is similar for all specimens. There is a sharp development phase was observed until the axial strain value of 4%, followed by a slow dissipation phase observed (from 4% to 12%) during the whole shear procedure as shown in Figures 9, 10 and 11 . The pore water pressure development is mainly derived from the shear behaviour, including the particles movement and rearrangement in the earlier phase (0 % to 4 % axial strain). In the dissipation procedure, the main shear patterns are the rotation and crushing of coarse particles, from which the new porosity is derived. So, the pressure decreased slowly and pore water moved through the new porosity induced by shear patterns (4 % to12 % axial strain). From Figures 9, 10 and 11, it can be understood that the introduction of geogrid in granular fill decreased the pore water pressure that could cause liquefaction, in addition the pore water pressure causing liquefaction decreased with the increase in the number of geogrid layers. This is a result of the fact that the installed geogrid restricts lateral deformation of the granular fill by its extensile force and provides better interlocking property on granular fill, arranged between the geogrid, leading to the shear contractions and easy dissipation of pore pressure along the sample length. Under a confining pressure of 150kPa the pore water pressure of the un-reinforced specimen was about 117 kPa, whereas the granular fill reinforced with one, two and three layers of geogrid achieved the pore water pressure of 97.5 kPa, 76.5 kPa and 49.5 kPa, respectively, which i s 20.12%, 52.94% and 136.36%, lower than that of the un-reinforced.

In a similar manner, the granular fill reinforced with one, two and three layers of geogrid achieved the pore water pressure of 118 kPa, 92 kPa and 52 kPa, respectively, under a confining pressure of 200 kPa, which is 25.42%, 60.86% and 184.61%, lower than that of the un-reinforced granular fill. From the above observations and from Figure 12, it can be inferred that the liquefaction resistance of the reinforced granular fill decreased with the increase in confining pressure and this behaviour was fairly agreed with findings of Boominathan and Hari. Under the confining pressure of 100 kPa, the granular fill reinforced with one, two and three layers achieved a pure water pressure of 70 kPa, 56 kPa and 40 kPa, whereas the same specimens were achieved the pore water pressure of 118 kPa, 92 kPa and 52 kPa, under the confining pressure of

200 kPa. It can be inferred that the introduction of geogrid layer in improving the liquefaction resistance of the granular fill, in addition, the more improvement in liquefaction resistance can be achieved with the increase in the number of layers. Furthermore, the confining pressure influenced on the liquefaction resistance of granular fill, at low confining pressures, the more improvement in liquefaction resistance can be achieved in granular fill reinforced with geogrid.

Summary of Invention

The influence of geogrid on the static liquefaction resistance of granular fill obtained from Karaikudi, Sivagangai District, Tamil Nadu, India was experimentally investigated. . It was observed that the extensile force of the geogrid gradually contributes to the improvement of the reinforced specimens shear strength and the extensile force increased with the increase in the number of geogrid layers, as a result the failure mode changed from shear band to bulging. The findings conclude that the geogrids considerably influence the shear behaviour of granular fill, and the geogrid reinforcement improves the interlocking strength of the granular fill, thus improving its shear strength.

5. Detailed Descriptions of Figures

The detailed description should refer to the following drawings in which they refer to:

Figure 1. Standard Proctor Compaction test curve for granular fill

Figure 2. Netlon 121 CE-Geogrid

Figure 3. Details of triaxial test specimens

Figure 4. Ultimate deviatoric stress of all specimen’s comparison

Figure 5. Pour water pressure of all specimens-comparison