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*Research Article*

# Prediction of Effects of Geogrid Reinforced Granular Fill on the Behaviour of Static Liquefaction

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The experimental investigation on the effects of granular fill and geogrid reinforced granular fill on the behaviour of the static liquefaction potential of the subsoil is reported in this study. A series of plate load test were carried out with different thickness of the granular fill, number of geogrid layers, and size/dimension of the footing. The test results were presented in terms of bearing capacity and subgrade modulus for the settlement of $\delta_{10}$, $\delta_{15}$, and $\delta_{20}$. The experimental results revealed that the introduction of granular fill significantly increases the bearing capacity and effectively control the settlement behaviour of the footing. The introduction of geogrid in granular fill enhanced the Percentage of Control in Settlement and Bearing Capacity Ratio by a maximum of 328.54% and 203.41%, respectively. The introduction of geogrid in granular fill interrupts the failure zone of the granular fill and enhances the subgrade modulus of the footing by a maximum of 255.55%; in addition subgrade modulus of the footing was increased with an increase in the number of geogrid layers. Based on the test results it is suggested that the footing with large size has beneficial improvement on the reinforced granular fill.

## 1. Introduction

The liquefaction behaviour of soil during earthquakes has severely damaged the various buildings, roads, and other structures. Static liquefaction of loose and very loose saturated sands is a modern classical mechanics subject and the sudden increase in pore water pressure causes 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 [1] in addition to the fact that a new stabilization method should be identified to efficiently combat this problem. The traditional improvement methods to prevent the liquefaction of granular soils are draining and densification techniques such as dynamic compaction, vibro techniques, stone columns, and compaction grouting [2]. Another technique to enhance the strength of the soil is using chemical admixtures like calcium carbonate powder, cement, lime, fly ash, and so forth [3, 4]. Even though these techniques have been shown to be successful in practice, densification and draining of deep soil profiles are often ineffective [5] and they required heavy equipment. Recently reinforcing of soil using geocomposites has been emerged and the tension and lateral confinement provided by reinforcement significantly increase the strength properties of the soil and also the reinforced soil effectively sustains the applied loads [6]. The main advantages of mixing discontinuous fibers with a soil mass are the absence of potential planes of weakness that can develop parallel to oriented reinforcement [7]. However, compared with these, 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 [8]. In the past few decades, the application of geogrids in soil reinforcing has been widely carried out and reported.

Alawaji [9] 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. The geogrid having a width

of four times the diameter of loaded areas and a depth of 0.1$D$ is recommended as an efficient and economical reinforcement. Liu et al. [10] 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 had shown that the soil/PET-yarn geotextile interface has significantly lower shear strength than soil strength. Phanikumar et al. [11] 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 had shown that the increase in the number of geogrid layers and decrease in spacing 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 indicated that the geogrid considerably increases the shear strength and apparent angle of shearing resistance [12]. Field test using seven different footing diameters and different granular fill layer thicknesses was conducted by Ornek et al. [13]. The test results indicated that the use of granular fill layers over natural clay soil has a considerable effect on the bearing capacity characteristics. Demir et al. [14] 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 had shown that use of granular fill and geogrid for reinforced soil footings (RSF) has considerable effects on the subgrade modulus and bearing capacity. Very recently Kolay et al. [15] investigated the improvement in the bearing capacity of silty clay soil with thin sand layer on top and placing geogrids at different depths. The test results revealed that the bearing capacity for the soil increases with the increase in number of geogrid layers.

The research carried out so far focused on increasing the shear strength capacity of the soil using geogrid and also none of these studies demonstrated the influence of geogrid sheets in improving liquefaction resistance of granular soil. In addition, studies that are related to large scale field test on bearing capacity of geogrid reinforced soil are very limited. The main objective of this study is to experimentally investigate the effects of granular fill and geogrid multiple layers in the behaviour of bearing capacity and static liquefaction resistance. Plate load tests were conducted using model footing to evaluate the granular fill and geogrid multiple layers in liquefaction resistance of soil in terms of bearing capacity and settlement behaviour. The experimental parameters were thickness of the granular fill (0.30$D$, 0.45$D$, 0.60$D$, 0.75$D$, and 0.90$D$), number of geogrid layers (one and two), and size/dimension of the footing (0.5$D$ and 0.75$D$).

## 2. Experimental Program

### 2.1. Material Properties

*2.1.1. Subsoil Characterization.* The entire testing program was carried out in the coastal area located in Nagapattinam Taluk, Thanjavur District, Tamilnadu, India. Geotechnical site investigation both laboratory and in situ tests were conducted to explore the soil properties in the experimental test area [16]. In situ tests such as borehole drillings and test pit excavation were performed to identify the soil profile of the experimental test area and four types of subsoil were identified [17]. A layer of transported soil having a depth of 1 m was observed initially. After that a layer of silty clay marl was observed between the depths of 1 m and 5 m. Stiffened clay later was observed below the silty clay marl between the depths of 5 m and 9 m. After 9 m depth, the soil layer was changed from clay to hard strata. The excavation continued up to the depth of 12 m, a layer of limestone profile was observed at the depth of 12 m. The standard penetration test [18] was conducted during the drilling process and "$N$" value was varied in the range of 5$N$ to 50$N$ as shown in Figure 1. The specific gravity values of the soils were determined from the laboratory test [19] and the value varies in the range of 2.6 to 2.71 along the entire depths which is shown in Figure 2. Unconfined shear strength ($c_u$) of the soil samples was evaluated [20] through an unconfined compression test and the value varies between 60 and 75 kN/m$^2$ as shown in Figure 3.

Table 1: Properties of geogrid.

| | | |
|---|---|---|
| 1 | Structure | Bidirectional |
| 2 | Aperture shape | Square |
| 3 | Mesh aperture size | 10 × 10 mm |
| 4 | Raw material | Polypropylene |
| 5 | Colour | Black |
| 6 | Thickness of sheet | 4 mm |
| 7 | Tensile strength | 15 kN/m |
| 8 | Elongation at nominal strength | 22.7% |

*2.1.2. 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 [21]. The specific gravity value [19] of the granular fill was about 2.64. From the standard proctor compaction test [22] the optimum moisture content and maximum dry unit weight were obtained and the values were about 7% and 21.7 kN/m$^3$, as shown in Figure 4. The direct shear test [23] was performed and the obtained internal friction angle and the cohesion of the granular fill were 43° and 15 kN/m$^2$. In order to keep the homogeneity granular fill passing through 4.75 mm was used in both laboratory and field tests.

*2.1.3. 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 15 kN/m$^2$ with a square aperture size of 100 mm$^2$ (10 × 10 mm). The typical geogrid sheet is shown in Figure 5. The physical and mechanical properties of the geogrid provided by the manufacturer are summarized in Table 1.

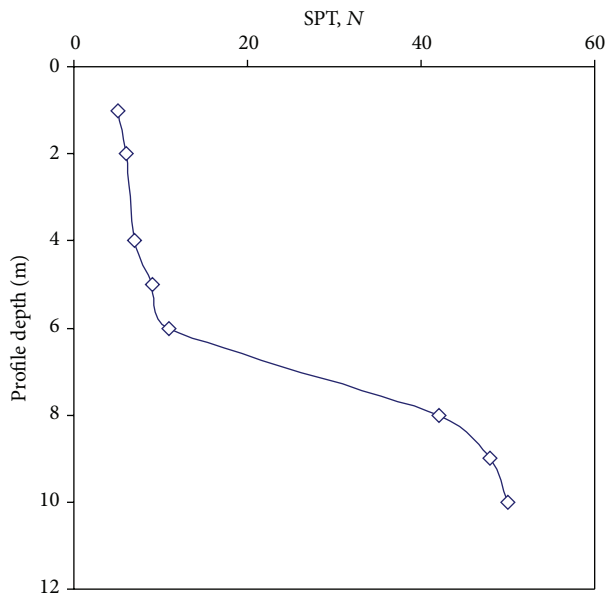


Figure 1: Typical SPT ($N$) values obtained from borehole drillings.

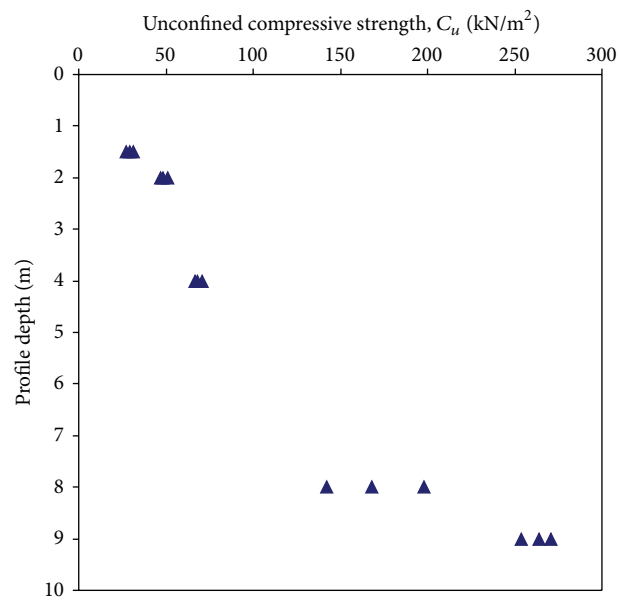


Figure 3: Unconfined shear strength distribution of subsoil.

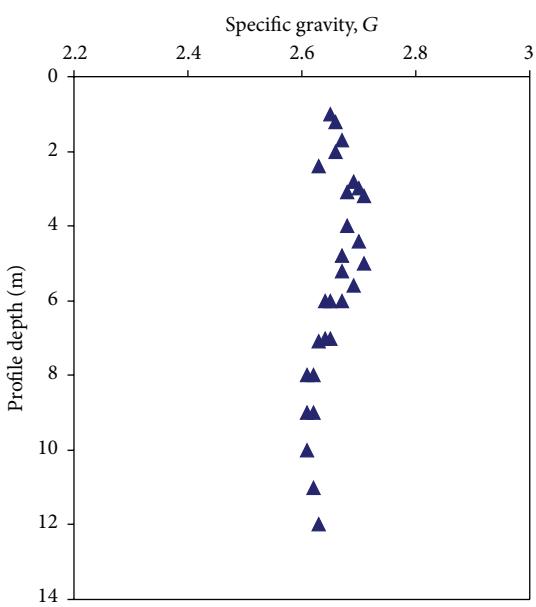


Figure 2: Specific gravity distribution of subsoil.

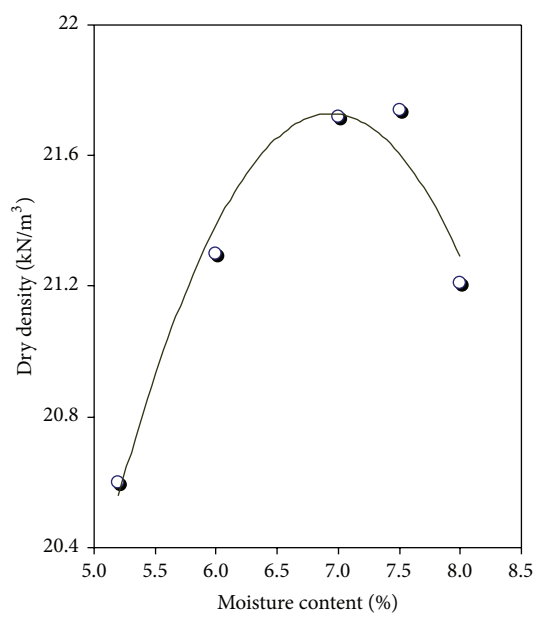


Figure 4: Standard proctor compaction test curve for granular fill.

*2.2. Experimental Setup and Procedures.* After the laboratory and in situ test, the reaction loading frame was installed at the site. After that the 1.6 m depth of soil was removed and the surface of the test area was leveled. For granular fill initially, the amount of granular fill and water required for each layer was calculated then the granular fill and the water was mixed well using a counter current mixer. After that the granular fill was placed layer by layer and the layer was compacted using an earth rammer to the predetermined height to achieve the desired density. The size of the foundation of 2.1 m × 2.1 m was kept constantly in the entire test and the depth of the granular fill or thickness of granular fill ($H$) was varied according to the footing diameter (0.30$D$, 0.50$D$, 0.70$D$, and 0.90$D$). For each reinforced condition, the granular fill was prepared in the above same manner and the layer was compacted to the predetermined height. When the granular fill reached the preferred depth, a layer of geogrid was placed and then the compaction was continued until the granular fill reached its desired height. The footing was positioned on the ground, taking care to ensure that its centerline was exactly in line with the axis of the hydraulic jack and the loading frame. The downward load was applied to the footing or base plate by a hydraulic jack and the axial deformation of the footing was measured by using two linear voltage displacement

Figure 5: Netlon 121 CE, geogrid.

transducers (LVDT) which was kept at the top surface of the footing. The hydraulic jack and the two linear variable displacement transducers (LVDT) were connected to a data logger. The testing program was performed according to the ASTM D 1196-93 (ASTM, 1997) and the testing program was stopped; neither the applied vertical load was obviously reduced nor the settlement of the footing was considerable for the small increment in the applied load. The experimental setup is shown in Figure 6. The influence of granular fill with and without geogrid on the behaviour of bearing capacity and liquefaction was studied in this research. Three series of tests such as Series I, Series II, and Series III were performed. Series I consisted of various diameters ($D$) of the footing (0.50$M$ and 0.75$M$) on the surface that is without granular fill and geogrid. Series II is very similar to Series I, except the footing was placed on the top of the granular fill with various thicknesses ($H$) according to the footing diameter (0.30$D$, 0.50$D$, 0.75$D$, and 0.90$D$). Series III consisted of footing on the granular fill with geogrid reinforcement. The experimental parameters are number of geogrid layers ($N$) (one and two) and the distance between the bottom surface of the footing to the top of the geogrid layer ($H_{\mathrm{FTG}}$) (0.1$D$ and 0.2$D$). Throughout the test the distance between the two geogrid layers was kept constant as 200 mm ($H_{\mathrm{GG}}$).

## 3. Description of Tests

To identify the tests easily, sixteen tests in three series were designated with names such as RT-0.50$M$ and RT-0.75$M$ (Series I); 0.50-$G$(0.30$D$), 0.50-$G$(0.50$D$), 0.50-$G$(0.75$D$), 0.50-$G$(0.90$D$), 0.75-$G$(0.30$D$), 0.75-$G$(0.50$D$), 0.75-$G$(0.75$D$), and 0.75-$G$(0.90$D$) (Series II); 0.50-$N$1-0.1$D$, 0.50-$N$1-0.2$D$, 0.50-$N$2-0.1$D$, 0.75-$N$1-0.1$D$, 0.75-$N$1-0.2$D$, and 0.75-$N$2-0.1$D$ (Series III). For example, in test RT-0.50$M$, first character "RT" indicates the reference test and the second character refers to the footing diameter. In test 0.75-$G$(0.30$D$), number "0.75" refers to the footing diameter and the following character "$G$(0.30$D$)" indicates the granular fill having a thickness of 0.30 of diameter of the footing. In test 0.50-$N$1-0.2$D$, number "0.50" refers to the footing diameter and next character indicates the number of geogrid layers (one and two) and the last "0.2$D$" indicates the depth of the geogrid layer from the bottom of the footing.

## 4. Results and Discussion

*4.1. Series I: without Granular Fill and Geogrid.* A total of two tests with the footing size of 0.50 m and 0.75 m (RT-0.50$D$ and RT-0.75$D$) were conducted in Series I without any soil improvement. It is the reference test which is used to compare the test results of Series II and Series III. The Load-settlement behaviour of the unimproved soil for both footing sizes is shown in Figure 7. In all reference tests, from the initial stage, the settlement of the footing is directly proportional to the applied pressure, and the load-settlement behaviour of the tests in Figure 8 is clear evidence for the local shear failure of the subsoil. In addition to that, in both tests the ultimate bearing capacity of the soil was not clearly identified. From the load-settlement plots, the subgrade modulus of the soil was calculated by (1) and the subgrade modulus were calculated for the settlement of $\delta_{10}$, $\delta_{15}$, and $\delta_{20}$, since determining the bearing capacity and subgrade modulus would have been more appropriate for the case where allowable settlement controls performance. The results were shown and listed in Figure 9 and Table 2, respectively:

$$k = \frac{q}{\delta}, \tag{1}$$

where $k$ is subgrade modulus, $q$ is the bearing capacity, and $\delta$ is the footing settlement. Figure 8 clearly shows that the subgrade modulus of the soil increased when increasing the footing size and the subgrade modulus was decreased with the increase in footing settlement. The results were discussed in terms of settlement to diameter ratio ($s/D$). Figures 7 and 8 clearly show that the experimental results revealed that the bearing capacity of the soil was increased when increasing the size of the footing. This is a result of the fact that the increase in resistive pressure area exerted by the soil and the development of resistive pressure is directly proportional to the footing area. The bearing capacity value of both tests was obtained at the $s/D$ ratio of 3% [13], and the bearing capacity value of the test RT-0.50$M$ was 216 kN/m$^2$ which is 47.5% less than the test RT-0.75$M$. In addition the $s/D$ ratio of 2% and 4% was used throughout the test to discuss the enhancement in bearing capacity.

*4.2. Series II: with Granular Fill and without Geogrid.* The influence of granular fill and the increase in granular fill thickness on the behaviour of bearing capacity and the settlement of the footing were evaluated [24] in Series II. A series of tests with the footing diameter of 0.50M and 0.75M was conducted on the granular fill bed and the thickness of the granular fill was varied according to the footing diameter (0.30$D$, 0.50$D$, 0.75$D$, and 0.90$D$). The applied load and the corresponding settlement for various thicknesses of granular fill in both cases are shown in Figures 9 and 10. From Figures 9 and 10, it can be seen that, for a small load range, the relation

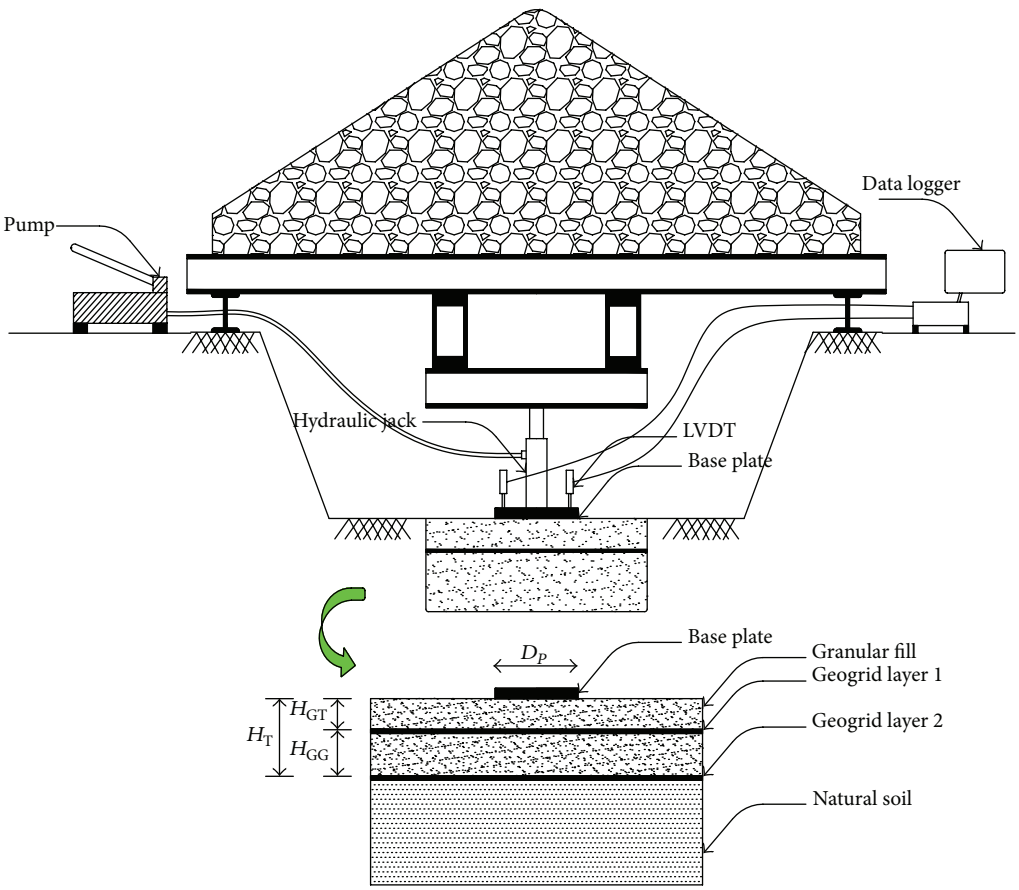


Figure 6: Experimental setup.

Table 2: Bearing capacity and subgrade modulus for Series I, II, and III.

| Test series | Test names | Bearing capacity (kN/m$^2$) | | | Subgrade modulus (MN/m$^3$) | | |
|---|---|---|---|---|---|---|---|
| | | $\delta$—10 mm ($\delta_{10}$) | $\delta$—15 mm ($\delta_{15}$) | $\delta$—20 mm ($\delta_{20}$) | $\delta$—10 mm ($\delta_{10}$) | $\delta$—15 mm ($\delta_{15}$) | $\delta$—20 mm ($\delta_{20}$) |
| Series I | RT-0.50D | 172 | 216 | 240 | 17.20 | 14.40 | 12.00 |
| | RT-0.75 | 244 | 292 | 324 | 24.40 | 19.47 | 16.20 |
| Series II | 0.50-G(0.30D) | 320 | 388 | 428 | 32.00 | 25.87 | 21.40 |
| | 0.50-G(0.50D) | 368 | 442 | 488 | 36.80 | 29.47 | 24.40 |
| | 0.50-G(0.75D) | 448 | 518 | 556 | 44.80 | 34.53 | 27.80 |
| | 0.50-G(0.90D) | 488 | 556 | 588 | 48.80 | 37.07 | 29.40 |
| | 0.75-G(0.30D) | 340 | 416 | 456 | 34.00 | 27.73 | 22.80 |
| | 0.75-G(0.50D) | 410 | 484 | 518 | 41.00 | 32.27 | 25.90 |
| | 0.75-G(0.75D) | 474 | 536 | 570 | 47.40 | 35.73 | 28.50 |
| | 0.75-G(0.90D) | 542 | 618 | 658 | 54.20 | 41.20 | 32.90 |
| Series III | 0.50-N1-0.1D | 686 | 768 | 818 | 68.60 | 51.20 | 40.90 |
| | 0.50-N1-0.2D | 578 | 656 | 698 | 57.80 | 43.73 | 34.90 |
| | 0.50-N2-0.1D | 760 | 828 | 836 | 76.00 | 55.20 | 41.80 |
| | 0.75-N1-0.1D | 716 | 796 | 842 | 71.60 | 53.07 | 42.10 |
| | 0.75-N1-0.2D | 595 | 678 | 728 | 59.50 | 45.20 | 36.40 |
| | 0.75-N2-0.1D | 784 | 850 | 890 | 78.40 | 56.67 | 44.50 |

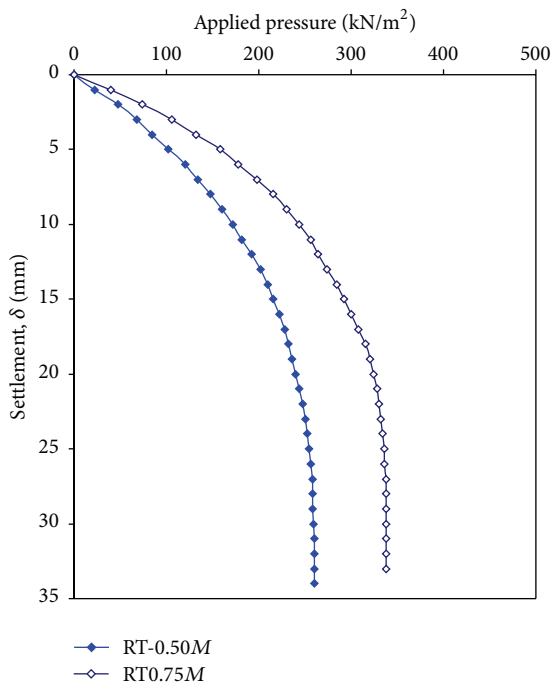


Figure 7: Load-settlement curve for reference tests, comparison.

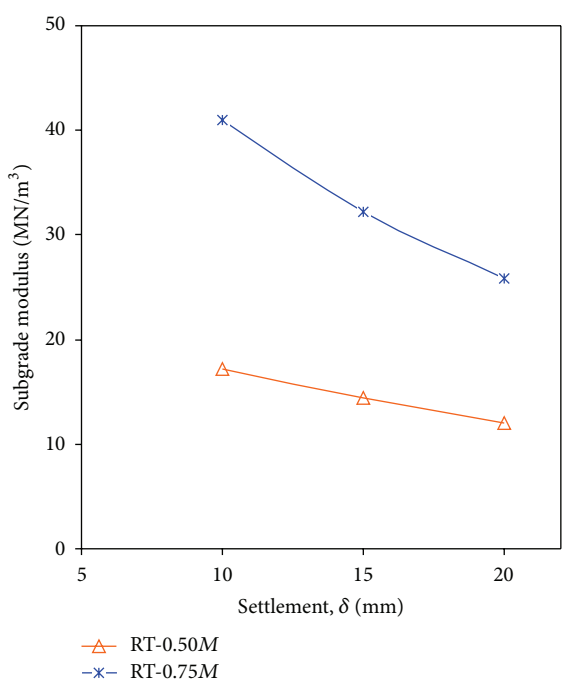


Figure 8: Subgrade modulus for reference tests, comparison.

between the applied load and settlement exhibited the linear behaviour, loading further the load-settlement behaviour has become nonlinear and in addition the peak load was not clearly identified. The failure mode observed in all test of Series II was a local shear failure. The experimental results revealed that the introduction of granular fill significantly increases the bearing capacity and effectively control the

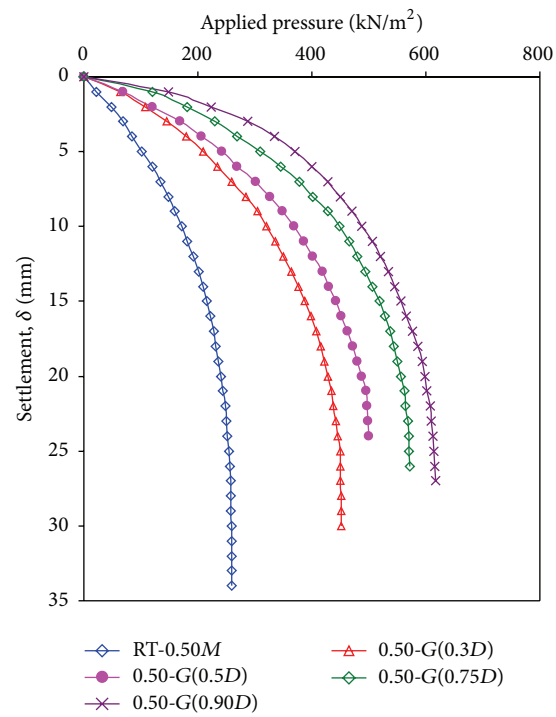


Figure 9: Load-settlement curve for Series II ($D$ = 0.50$M$), comparison.

static liquefaction of the subsoil and in addition the load-settlement behaviour becomes very stiffer with the increase in the $H/D$ ratio, namely, increase in granular fill thickness. The placing of granular fill decreases the pore water pressure developed because of a decrease in interstitial pressure distribution. The performance improvements of bearing capacity due to the granular fill are expressed in terms of Bearing Capacity Ratio (BCR) [14] and the following equation was used for evaluating the BCR:

$$\mathrm{BCR} = \frac{q_G}{q_0}, \tag{2}$$

where $q_G$ is the bearing capacity of the soil reinforced with granular fill and $q_0$ is the bearing capacity of the unreinforced or reference soil. The BCR ratio was obtained for the $s/D$ ratio of 3%. The relation between BCR and $H/D$ ratio is shown in Figure 11. From Figure 12, it can be observed that the bearing capacity and the BCR value of the footing have increased with the increase in granular fill thickness in all cases. However, the BCR has decreased when increasing the footing diameter. Figure 11 clearly shows that the correlation between the granular fill thickness and BCR was too strong for all footing diameters ($R^2$ = 0.985). Figures 9, 10, and 11 clearly show that the bearing capacity of the footing was significantly increased for the granular fill having the thickness of $H/D$ = 0.75; however, a close observation of Figures 9, 10, and 11 exhibits that the enhancement of bearing capacity is not significant beyond the $H/D$ value of 0.75. From the results, the granular fill having a thickness of $H/D$ = 0.75 was selected as the most advantageous fill height and the same

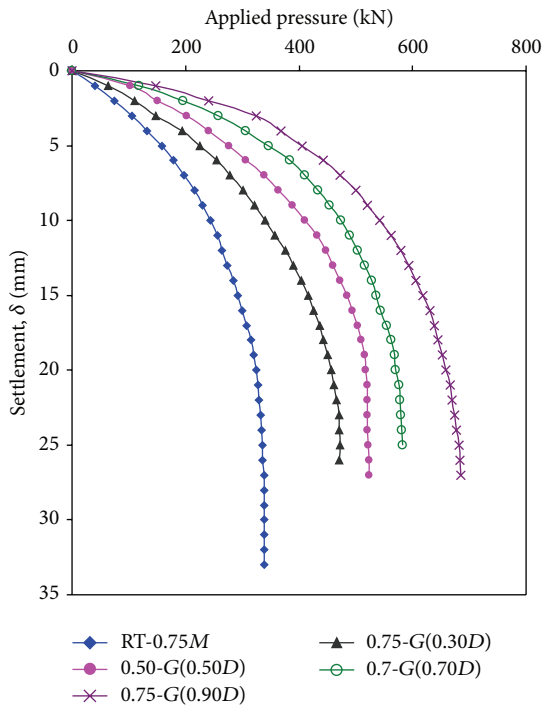


Figure 10: Load-settlement curve for Series II ($D = 0.75M$), comparison.

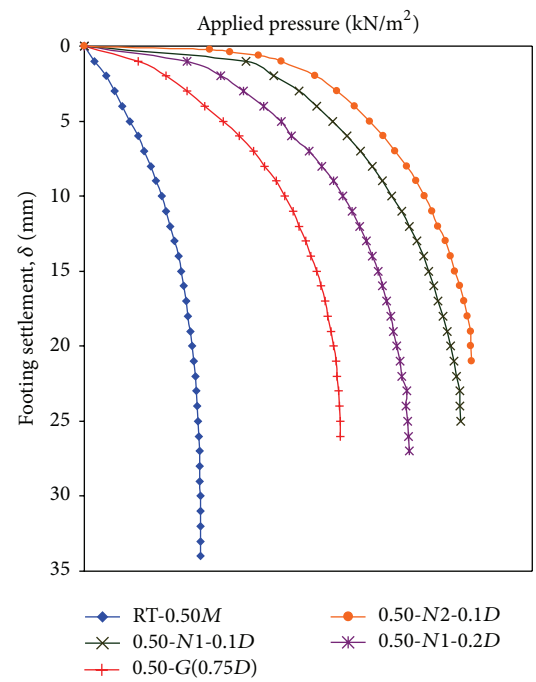


Figure 12: Load-settlement curve for Series III ($D = 0.50M$), comparison.

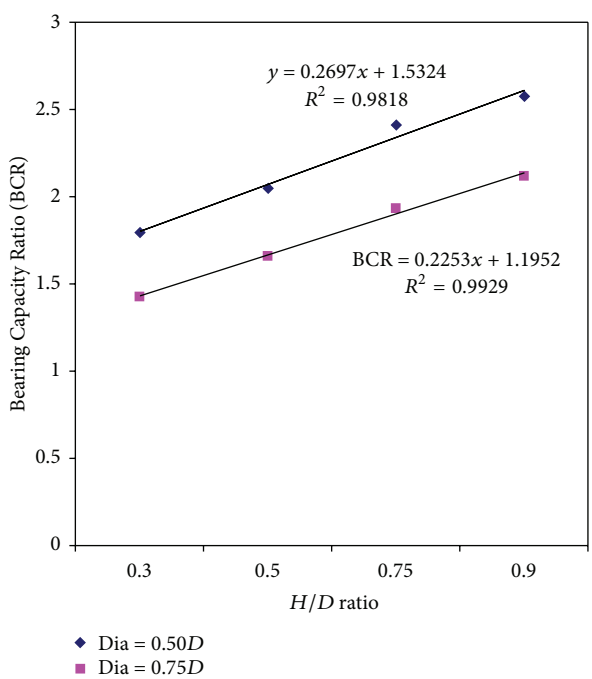


Figure 11: Bearing Capacity Ratio (BCR) for Series I, comparison.

thickness was introduced in Series III. Compared to the RT-0.50$M$, test 0.50-$G$(0.75$D$) increased its load bearing capacity by 30.21%, in similar manner test 0.75-$G$(0.75$D$) increased its bearing capacity by 32.68% than that of RT-0.75$M$.

*4.3. Series III: Geogrid Reinforced Granular Fill.* The main objective of this research is to effectively evaluate the influence of geogrid reinforced granular fill on the behaviour of bearing capacity, subgrade modulus, and BCR. From the test results obtained from Series II, the thickness of granular fill was maintained as $H = 0.75D$ throughout the test. Figures 12 and 13 show the load-settlement behaviour of different footing with different geogrid layers. As expected the introduction of single geogrid layers at the distance of 0.1$D$ from the bottom surface of the footing in both footing cases comparatively improves the liquefaction behaviour of the soil, load-settlement behaviour, and the bearing capacity which is shown in Figures 12 and 13. This is a result of the fact that the introduction of geogrid interrupts the failure zone of the granular fill and the stress sharing area has been increased at a geogrid depth due to the wider dispersion of stress affected by horizontal geogrid reinforcement [11]. As a result, the stress in the granular fill considerably reduced, resulting in a smaller amount of settlement. Another possible reason is that the tensile stress in geogrid is produced by the applied load fully defied by the geogrid resulting in improved load-settlement behaviour of the system. The results in Figures 12 and 13 show that, when lifting up the depth of geogrid from 0.1$D$ to 0.2$D$ (test 0.50-$N$1-0.2$D$), the test results have not shown any considerable improvement in the load-settlement behaviour and in addition the bearing capacity was moderately low when compared to the test 0.50-$N$1-0.1$D$. From the observation, it can be understood that the geogrid very near to footing will provide considerable improvement in the load-settlement behaviour and bearing capacity and the introduction of geogrid at profound depths

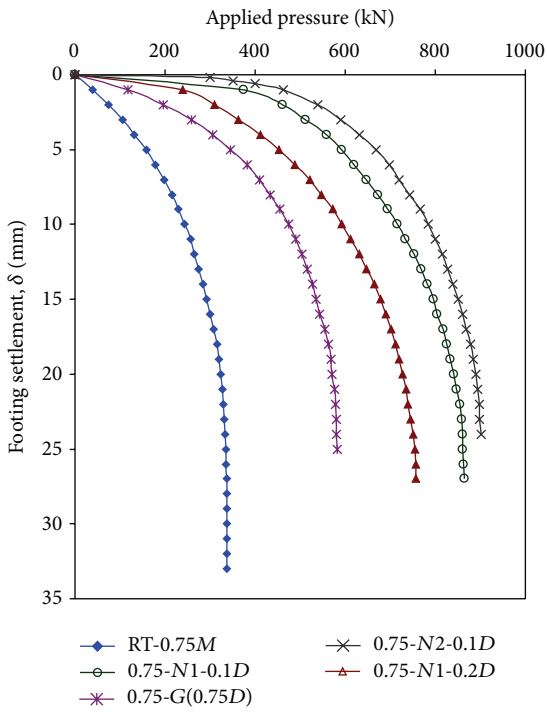


Figure 13: Load-settlement curve for Series III ($D = 0.75M$), comparison.

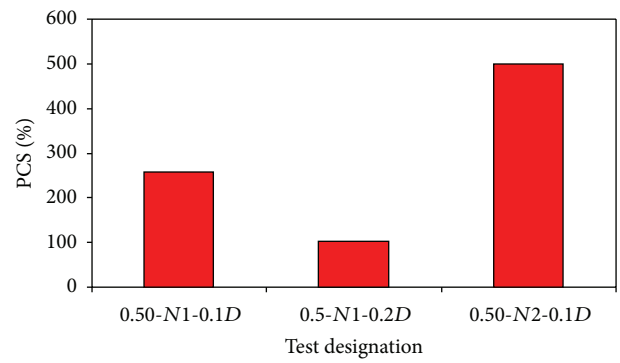


Figure 14: Percentage of Control in Settlement (PCS) for Series III ($D = 0.50M$).

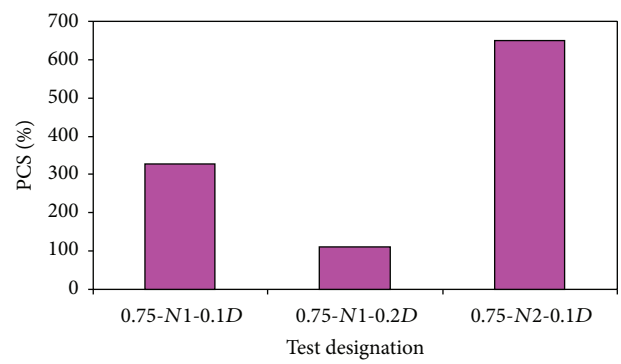


Figure 15: Percentage of Control in Settlement (PCS) for Series III ($D = 0.75M$).

is not advisable. The decrease in performance may be due to the depth of reinforcement considerably deep, and resulting work efficiency of the geogrid has decreased. The above similar finding was observed in footing with the diameter of $0.75D$. The introduction of another geogrid layer with the $H_{GG}$ of 200 mm (0.50-$N$2-0.1$D$) in the granular fill, showing further improvement in the load-settlement behaviour and the beneficial effect, is much greater than that of a granular fill with single layer in both footing cases ($0.50M$ and $0.75M$). This is a result of the fact that during loading the interlocking between the geogrid and granular fill has greatly improved and the geogrid reinforced granular fill started to behave like a geogrid composite plate. This composite plate action significantly resists the applied pressure and slows down the soil failure and also significantly restraints the load-settlement behaviour. The influence of the geogrid reinforced granular fill on the behaviour settlement compared to the unreinforced footing evaluated through the parameter called Percentage of Control in Settlement (PCS). The PCS were calculated at the settlement of $\delta_{15}$ or for the $s/D$ ratio of 3% using the following equation:

$$\text{PCS} = \frac{\delta_{15} - \delta_R}{\delta_R} \times 100, \tag{3}$$

where $\delta_{15}$ is the settlement of the unreinforced footing and $\delta_R$ is the settlement of the reinforced footing at the footing pressure of unreinforced one having a settlement value of $\delta_{15}$. Figures 14 and 15 show the comparison of PCS factors with respect to the footing with granular fill (0.50-$G$(0.75$D$) and 0.75-$G$(0.75$D$)). Compared to 0.50-$G$(0.75$D$), the test 0.50-$N$1-0.1$D$ and 0.50-$N$1-0.2$D$ increased its PCS by 257.14% and 101.25%, respectively, as shown in Figure 14. Compared to 0.70-$G$(0.75$D$), tests 0.75-$N$1-0.1$D$ and 0.75-$N$1-0.2$D$ improved their PCS by 328.54% and 111.54%, respectively, which is shown in Figure 15. Compared to the single layer of geogrid, the PCS of granular fill reinforced with two layers of geogrid was outperformed and the tests 0.50-$N$2-0.1$D$ and 0.75-$N$2-0.1$D$ enhanced their PCS by 67.15% and 81.74%, respectively, when compared to the tests 0.50-$N$1-0.1$D$ and 0.75-$N$1-0.1$D$, respectively.

The Bearing Capacity Ratio (BCR) for each test was calculated from (1) for the settlement of $\delta_{10}$, $\delta_{15}$, and $\delta_{20}$. From Figure 16, it can be understood that the BCR increases with the increase in the number of geogrid layers. However, Figure 16 clearly shows that the BCR value of the reinforced granular fill has decreased with increase in settlement in all cases. This may be attributed to the composite layer of geogrid not able to affect the dispersion of stress at large settlement and not able to defy the tensile stress induced by the applied pressure. The tests 0.50-$N$1-0.1$D$ and 0.50-$N$1-0.2$D$ increased their BCR by 203.41% and 157.26%, respectively, when compared to 0.50-$G$(0.75$D$); their BCR values were 3.55 and 3.07, respectively, at the $s/D$ ratio of 3%. Also tests 0.75-$N$1-0.1$D$ and 0.75-$N$1-0.2$D$ improved their BCR by 115.87% and 87.15% than that of 0.75-$G$(0.75$D$) and the BCR values were 2.89 and 2.32 which is shown in Figure 17.

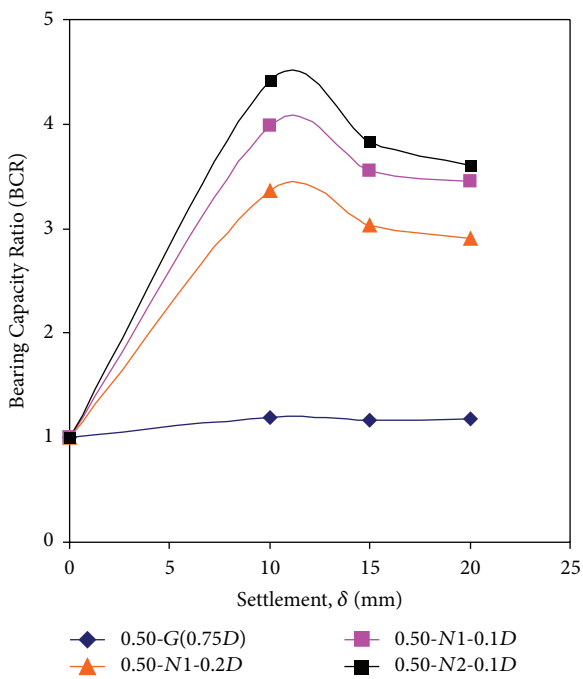


FIGURE 16: Bearing Capacity Ratio (BCR) for Series III ($D = 0.50M$), comparison.

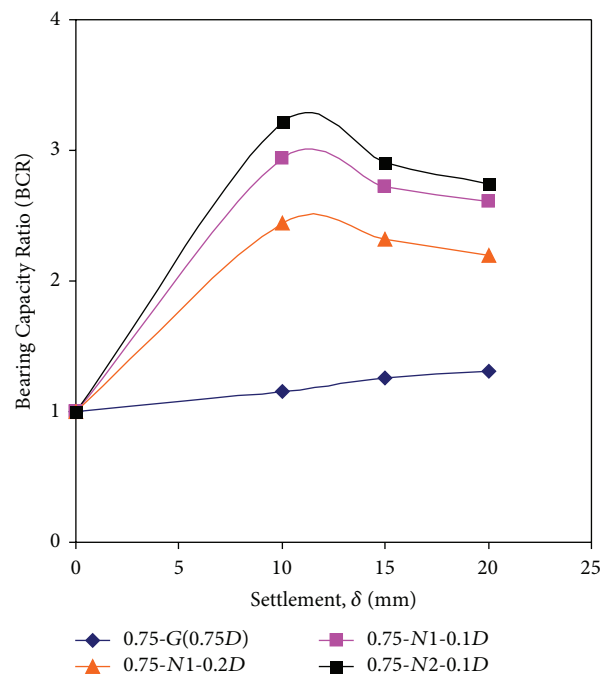


FIGURE 17: Bearing Capacity Ratio (BCR) for Series III ($D = 0.75M$), comparison.

The comparison of subgrade modulus obtained from series III at the settlement of $\delta_{10}$, $\delta_{15}$, and $\delta_{20}$ is shown in Figures 18 and 19 and presented in Table 2 Figures 18 and 19 mean that the subgrade modulus of the footing decreased with the increase in settlement and in addition the introduction of geogrid in granular fill enhances the subgrade modulus and reduces the footing settlement. At the $s/D$ ratio of 3% the subgrade modulus of the tests 0.50-$N$1-0.1$D$, 0.50-$N$1-0.2$D$, 0.75-$N$1-0.1$D$, and 0.75-$N$1-0.2$D$ was 51.21 MN/m$^3$, 43.73 MN/m$^3$, 53.67 MN/m$^3$, and 45.21 MN/m$^3$, respectively, which is 255.55%, 203.68%, 176.507%, and 132.97%, respectively, higher than that of an unreinforced footing (RT-0.50$D$ and RT-0.75$D$). On the whole geogrid reinforced granular fill resulted in improved load-settlement behaviour, load bearing capacity, and decrease in internal pore water pressure resulting in liquefaction control and in addition provided a more economic advantage. Based on the test results it is suggested that the footing with large size has beneficial improvement on the reinforced granular fill.

## 5. Conclusion

A series of field tests were carried out to understand the influence of granular fill and the geogrid reinforced granular fill on the behaviour of static liquefaction potential, bearing capacity, and load-settlement behaviour of the subsoil in the coastal area located in Nagapattinam Taluk, Thanjavur District, Tamilnadu. Based on the test results the following conclusion can be made.

(i) In all reference tests, the failure mode of the subsoil was a local shear failure and in addition the ultimate bearing capacity of the subsoil was not clearly identified.

(ii) The introduction of granular fill significantly increases the bearing capacity and effectively controls the static liquefaction of the subsoil

(iii) Compared to the reference test, the introduction of granular fill, with the depth of 0.3$D$, 0.50$D$, and 0.75$D$, increased its load bearing capacity of the footing by 6.84%, 16 20.54%, and 30.21%, respectively, and the bearing capacity values were 388 kN/m$^2$, 442 kN/m$^2$, and 518 kN/m$^2$, respectively.

(iv) The introduction of geogrid very near to footing will provide considerable improvement in the load-settlement behaviour and bearing capacity and the introduction of geogrid at profound depths is not advisable.

(v) Compared to 0.50-$G$(0.75$D$), the tests 0.50-$N$1-0.1$D$ and 0.50-$N$1-0.2$D$ increased their PCS by 257.14% and 101.25%, respectively.

(vi) The subgrade modulus of the footing decreased with the increase in settlement and in addition the introduction of geogrid in granular fill enhances the subgrade modulus and reduces the footing settlement.

(vii) The geogrid reinforced granular fill resulted in improved load-settlement behaviour, load bearing capacity, and decrease in internal pore water pressure resulting in liquefaction control and in addition provided a more economic advantage. It is suggested that the footing with large size has beneficial improvement on the reinforced granular fill.

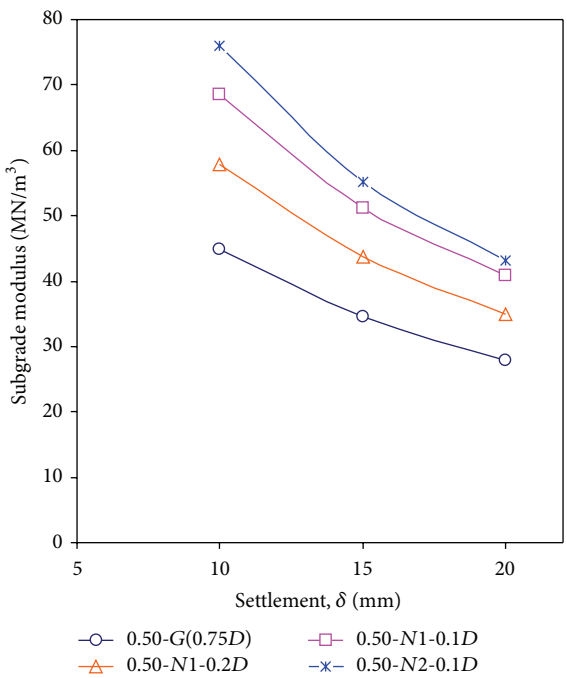


Figure 18: Subgrade modulus for Series III ($D = 0.50M$), comparison.

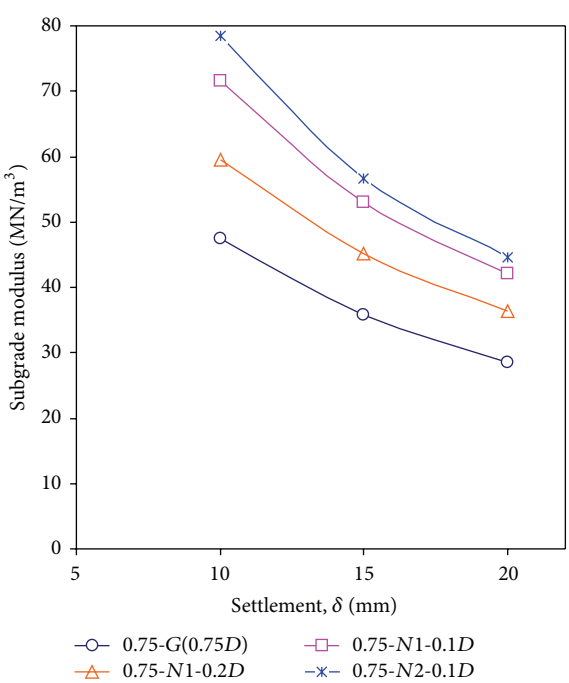


Figure 19: Subgrade modulus for Series III ($D = 0.75M$), comparison.

## Conflict of Interests

The authors declare that there is no conflict of interests regarding the publication of this paper.

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