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Performance of geosynthetics-strengthened unconnected piled raft foundations under seismic loading

A.G. Jihad[1,2], M. Karkush[1]

[1]*Civil Engineering Department, University of Baghdad, Baghdad, Iraq*

[2]*Road and Transport Engineering Department, Al-Qadisiyah University, AL-Qadisiyah, Iraq*

mahdi_karkush@yahoo.com

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Abstract. Over the past 20 years, geosynthetic reinforcement has been widely used in earthen constructions. For constrained building projects and challenging ground conditions, geosynthetic-reinforced-pile-supported (GRPS) embankments are regarded as dependable options. There has been significant research into ways to improve the stability of pile foundations, and there has also been some research into the viability of using unconnected piled raft under seismic load. This study investigates the use of geosynthetics to improve the performance of unconnected pile foundations under seismic load. A series of reduced-scale physical model experiments conducted on a shaking table was used to assess the seismic soil-structure interaction behaviors of geosynthetics-reinforced pile foundation systems. The study makes use of disconnected, closed-end aluminum piles with different thicknesses of cushion layers of sand soil and different spacing between piles are used in the study. The performance of disconnected footing was examined using a shaking table to determine the impact of the geosynthetics reinforced. These model findings are used to identify and thoroughly describe the contradictions in the present design methodologies. This study shows the settlement of the foundation under seismic loading decreased by reinforcing the cushion layer with one and two layers of geogrid materials, increasing the thickness of the cushion layer led to increasing the settlement of the foundation under seismic loading, where the settlement decreases with decreasing the value of spacing between piles. However, with further increases in either the cushion thickness or cushion stiffness or both, the effect of improvement would decrease.

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1. *Introduction*

One of the key functions of geotechnical engineering is the design of the foundation system. A safe, effective, and cost-effective project will undoubtedly result from the proper selection and design of the foundation system. A foundation system can use piles for two main reasons: to increase bearing capacity and/or to limit predicted settlements to acceptable levels. The general assumption made by traditional pile foundation design techniques is that the piles would support the entire weight of the superstructure. Over the past few decades, a more modern strategy has been used, one that uses piles as settlement reducers in what is known as a piled raft foundation. Davis and Poulos [1] introduced the concept of piled raft foundation in 1972. After that, many researchers have been involved in the design and analysis of such type of foundation. Among those [2–8] and others, the foundation of a piled raft should only contain the minimum number of piles necessary to make the settlement at a reasonable level. A portion of the loads that are transmitted from the superstructure to them via the raft is also carried by these piles. In other words, the loads are shared between the raft and piles. High axial stress concentration at the pile head may occur [9, 10].

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In addition, if the foundation is planned as a structurally connected piled raft in seismically active zones or places prone to high winds, large shear forces in addition to overturning moments may arise in the pile's head due to the lateral dynamic load. For all of that, the ultimate capacity may be governed by the structural capacity of the pile rather than the geotechnical capacity. In these situations, more likely than soil failure is the structural failure of a foundation. Therefore, to avoid structural damage to the foundation, the dimensions of piles should be increased and the result is an uneconomical design of the foundation. The use of disconnected piles and the use of an intervening compacted fill layer known as a cushion layer between the raft and the piles have recently been suggested as alternatives to the traditional method for dealing with constraint responses. The piles in such a foundation system are considered soil reinforcement members rather than structural components [11–13]. In unconnected piles, the carrying loads could be much higher than that carried by structurally connected piles, and still, the economic benefits have been gained. Moreover, there should be no possible damage to structural connections. In addition, because of the mobilized adhesion force along the soil-raft interface, the horizontal loads can be effectively transmitted as shown in Figure 1 [11,14,15].

In this study, the propagation and degradation of the sandy soil in the cushion layer that separates the raft and unconnected piles were investigated in detail by conducting several tests using a shaking table. The cushion layer used in this study was reinforced by several layers of geosynthetic materials at a different cushion layer thickness and change the space between piles, all the raft footing was tested under seismic load.

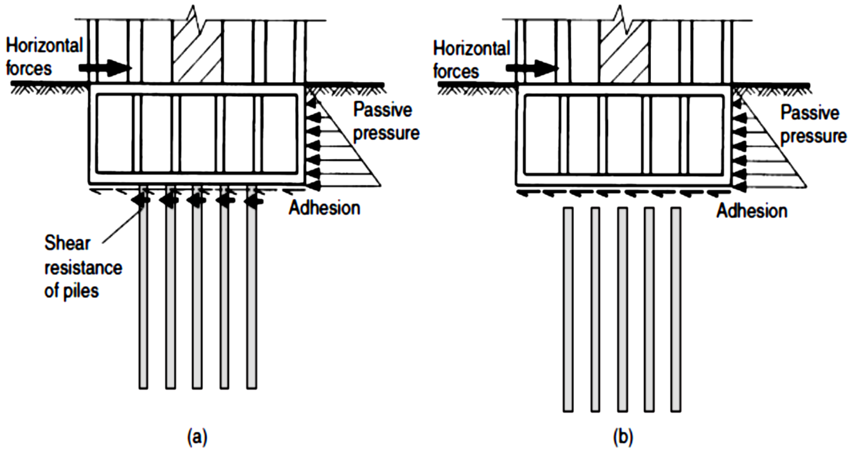


Figure 1. Transfer mechanism for horizontal loads: a) piles structurally connected to raft and b) piles structurally unconnected from raft [11].

2. Materials and Methods

2.1. Shaking Table

Using a shaking table device, the uniaxial sinusoidal motion was adopted with various frequencies as shown in Fig. 2. The shaking table was produced in the local market which has a dimension of 1×1 m size with 1.25 tons loading capability. It was supported by a heavy-duty steel frame that had 4 rollers that could move smoothly in one direction. All parts of the shaking plate were fixed to a rigid steel frame by 8 bolts to ensure rigidity and eliminate any motion, vibration, or sliding of the shaking table during the controlled seismic loading. In addition, the AC driver is contacted by the computer to control the time and frequency of the shaking. To adjust the exact frequency magnitude a calibration process must be done, this process was executed as follow: for a specific period, the number of cycles was recorded compared to the AC frequency, and using the accelerometer in the base of the shaking table the results were recorded, the result is the frequency in Hz by change the velocity of the motion a new value of frequency would appear as shown in Fig. 3. Thus, for a specific value of frequency, it is easy to select the correct number in the AC drive to determine the speed of the motor to get the required acceleration in the base of the shaking table.

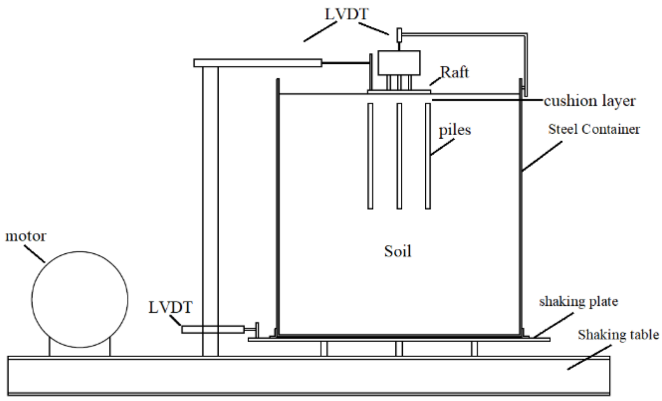


Figure 2. Testing model scheme.

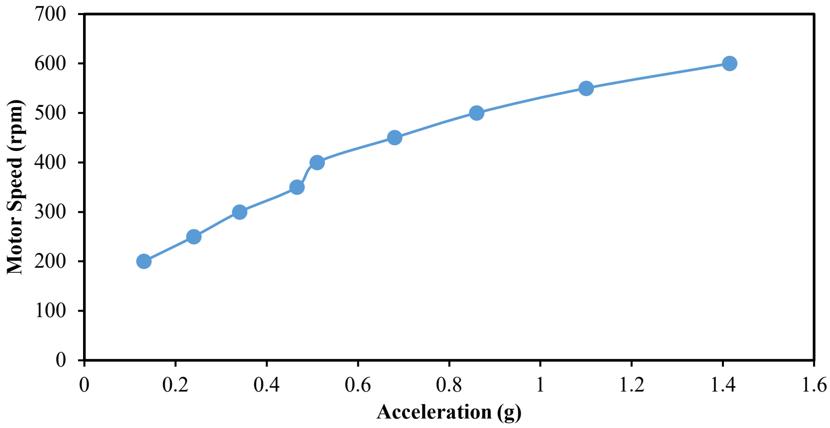


Figure 3. The calibration of the motor speed (rpm) and the acceleration.

2.2. Steel Container

A steel container had a depth of 850 mm and 800×700 mm cross-section and was made of a 5 mm plate thickness. The steel container was set up on a mobile frame base. In every test, the container's side dimension was greater than three times that of the raft side. The piles' end tips were about 25D or more above the container's base. It was made out of a steel frame of L-shape steel angles of 75x75 mm in size. Three sides of the steel container were covered with a steel plate 5 mm thick, and the fourth side was covered with strengthened transparency glass which help to watch the behavior of the foundation during the test. One of the improvements applied was the flexible rubber joints at the box corners which permit motion within a specific limit, this movement was limited to 20 mm and could occur at the same time alternately for each opposite side. The purpose of this flexible joint was to lessen the effect of the fixed joint on the behavior of the soil sample during shaking, which resulted in unanticipated behavior by wave reversals, damping, and other factors.

2.3. Pile Model

The pile model used in the current study was made of aluminum alloy of hollow square section of 14 mm in outside dimension and 11 mm inside dimension. The embedded pile length of 350 mm was used in all experimental model tests. The slenderness ratio (length to diameter ratio (L/d)) was 22.15.

2.4. Raft Model

The square steel plate with side dimensions 210×210 mm and a thickness of 12 mm was used as a model raft. The modulus of elasticity (E_r) and Poisson's ratio (ν_r) of the steel plate were 210×10^6 kPa and 0.30, respectively. The total weight of the suggested footing was 18 kg which is equivalent to the ordinary building weight.

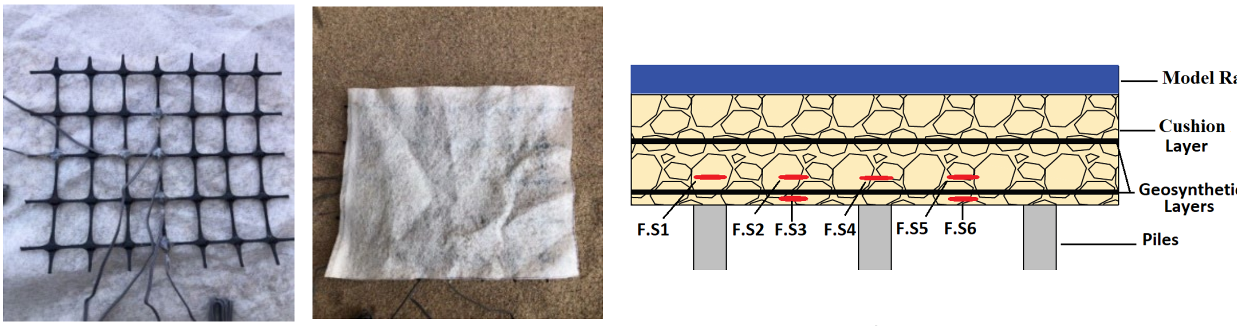
2.5. Geosynthetics

This study looked into how soil degradation was affected by geosynthetic reinforcement. A non-woven geotextile sheet was laid on top of a biaxial geogrid in the reinforced fill tests. The geosynthetic characteristics and forms and its location are presented in Table 1 and Fig. 4, respectively. Given that the fill material was sand, the non-woven geotextile was used over the geogrid to both transmit the load from the foundation to the geogrid and prevent sand from flowing through the geogrid apertures.

Table 1. Physical characteristics and geometry of geosynthetic materials.

Geosynthetic	Properties	Units	Values
Biaxial geogrid	Aperture dimensions	mm	25×33
	Minimum rib thickness	mm	0.76
	Tensile strength @ 2% strain	kN/m	6.6
	Tensile strength @ 5% strain	kN/m	13.4
	Ultimate tensile strength	kN/m	19
Non-woven geotextile	Mass/unit area	gm/m^2	96
	Thickness	mm	1.04
	Wide-width tensile strength	kN/m	7.23
	Wide-width tensile elongation	%	54
	Puncture strength (CBR)	N	1218
	Opening size	mm	0.15

Note: geosynthetic properties are based on the manufacturer-provided datasheet.



Biaxial geogrid **Non-woven geotextile** **Locations of geosynthetic layers**

Figure 4. Geosynthetic materials used in the experimental work.

2.6. Instruments

A 25 mm accurate LVDT (Linear Variable Differential Transformer) was placed on the center of the foundation system to measure the vertical settlements of the foundation model. Through metal rods fixed to the horizontal beam at the top of the box. The system was supported by metal rods and fixed to the base of the shaking table. The horizontal movement for the foundation and the shaking table was measured by LVDTs of lengths (400 and 500 mm) respectively shown in Fig. 5.

The acceleration of the shaking table, inside the soil, and on the plate and the foundation was measured by using an analog accelerometer sensor ADXL335, which can measure three directions of acceleration excitation as shown in Fig. 5. Five sensors were used; one on the shaking table, three within the soil column at different depths (each 20 cm depth apart), and the last one was fixed on the foundation. The selected sensor ADXL335X was a simple breakout board that allows easy, direct, and rapid evaluation of the performance of the ADXL335 accelerometer. The used sensor was a 3-axis analog-output accelerometer with a ±3 measurement range. The small size (25.4×25.4 mm) of the accelerometer made it easy to maneuver and fix on the different systems without needing additional hardware. All the accelerometers were connected to the data acquisition system channel.

a **b**

Figure 5. a) Accelerometer and b) LVDTs used in the tests.

Data acquisition with LabVIEW Application was used to record and save all the readings of LVDTs and accelerometer. LabVIEW (Laboratory Virtual Instrumentation Engineering Workbench) is a platform and development environment for a visual programming language from National Instruments. It is commonly used for data acquisition, instrument control, and industrial automation on a variety of platforms including Microsoft Windows, various flavors of UNIX, Linux, and Mac OS X. The data logger is supplied with 32 channels for connecting the transducers for all the instruments where 5 channels are used for accelerometers and 3 channels are used for LVDTs, as well 4 channels are used for AC drive control and internal connecter. The problem was represented by disturbance of the instrument's signals by the magnetic radiations from the motors of the shaker leading to incorrect readings. To solve this problem, two actions were adopted as described in the following points. First using ground connections for all the equipment and connecting them to a copper rod that has been previously installed in the ground soil to ensure the removal of any charges generated that may cause noise in the reading, the second use of a transformer for electricity equipped for all equipment to ensure the elimination of any noise caused by the electric current on the readings.

2.7. Soil Box Model and Bedding Soil

A relatively uniform dry sand of grain size between 0.35 and 1.18 mm was utilized in this investigation as foundation soil. The maximum and minimum dry unit weight of the tested sand were found 17.44 kN/m^3 and 15.17 kN/m^3, respectively. Table 2 shows the physical properties of the used soil. According to the Unified Soil Classification System (USCS), the soil is classified as poorly graded sand (SP).

Table 2. Physical Properties of the tested soil.

Property	Value	Standard Specification
Specific gravity	2.64	[16]
Sand (0.35–1.18 mm), %	100	
Coefficient of curvature	1.05	
Coefficient of uniformity	1.52	
D_{10}, mm	0.5	[17,18]
D_{30}, mm	0.6327	
D_{60}, mm	0.7597	
USCS soil type	SP	
Maximum dry unit weight, kN/m^3	17.44	[19]
Minimum dry unit weight, kN/m^3	15.17	[20]
Used dry unit weight, kN/m^3	16.627	
Relative density, %	50	

To reconstitute or prepare the sand samples for experimental model tests, several ways were used such as raining, vibration, or tamping. In general, as it was reported by [21–25], to provide relatively homogenous sand samples and to get the required relative density, the raining procedure is considered the best. The adopted raining technique consisted of a steel hopper of 35 cm height with a valve at the bottom to control sand raining manually. A sieve of 4.75 mm opening size was attached to the bottom via a funnel connected to a hopper to distribute the sand uniformly. The hopper was hanging by a pulley to adjust the height of the rain. Many trials were performed to draw a relationship between the falling height and the corresponding dry density as shown in Fig. 6. From the curve, it is clear that the falling height should be about 200 mm to get the required dry density of 16.627 kN/m^3.

The inside walls of the steel container were divided into 8 intervals 10 cm each to ensure that the desired dry density was obtained properly. To produce a homogenous sample overall in the container, the required quantity of sand for each layer was weighted such that the designed dry density could be obtained. After the completion of the raining of the soil for each layer, the sand layer was equalized by a steel plate of section area (175×200 mm) to an average of one blow per unit area for grain fixation and insurance of the required layer height. However, the target relative density of the sand sample was 50% ± 2%.

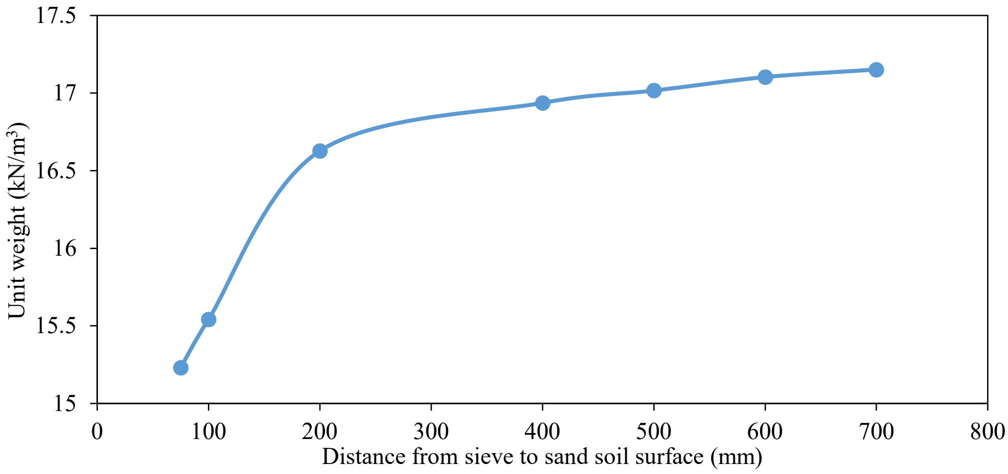


Figure 6. The relationship between the height of falling and dry density of sandy soil.

2.8. Seismic loading

Seismic loading, which is known as an application of seismic oscillation to a structure, can be considered one of the most essential basics of earthquake engineering. It depends on seismic hazard, geotechnical site parameters, and structural natural frequency which occurs at the surface of the structure and in contact with the ground or an adjacent structure. In the years 2017 and 2018, seismic activities were recorded with different strengths in Iraq. During the last two months of the year 2017, more than seventy earthquakes hit Iraq. As shown in Table 3, the magnitude of these earthquakes ranged from 4.0 to 7.3 on the Richter scale, with depths ranging from 6.21 to 42.32 km.

Table 3. History of some earthquakes in Iraq.

Year	Magnitude (Richter scale)	Location
1960	6.0 to 6.7	Halabja City which locates at northeast of Baghdad
1967	6.1	100 km south of Halabja City
2013	5.6 and 5.8	Two earthquakes hit about 60 km south of Halabja City, northeast of Baghdad
2017	7.3	Series of earthquakes 30 km south of Halabja City, northeast of Baghdad, Iraq
2018	4.0 to 4.5	A series of earthquakes hit northeast of Baghdad

Halabja City is located northeast of Baghdad (Iraq) and near the international border between Iraq and Iran. Halabja earthquake was used in this study. The acceleration of this earthquake was 0.24 g, and seismic loading conditions have been applied to the model as acceleration is shown in Fig. 7.

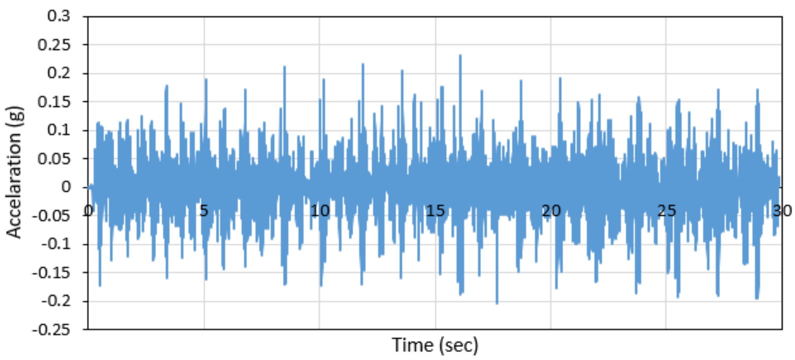


Figure 7. The acceleration time history of the applied Halabja earthquake.

2.9. Testing procedure

The test procedure followed in this investigation consisted of many steps such as:

1. The preparation of each model test was commenced by weighing the required quantity of sand to get the target density, then it was poured into the container by rainfall method into 8 layers each of 100 mm thickness.

2. Sand raining was paused when reaching the level above the pile tip by 50 mm. The pile was inserted 50 mm into the soil bed. This level was calculated previously to ensure proper seating of piles into the sand. The reached sand surface was leveled with a straight edge, and the steel mesh of square spaces of 70 mm was used to support the piles and ensure their verticality as shown in Fig. 8.

a b c d

Figure 8. a) Piles fixation by steel mesh, and b) horizontal leveling of upper head of piles, c) vertical leveling of piles, and d) final level of soil and unconnected piles.

3. Continuing the raining of soil to reach the middle of the pile level, stop the raining work and remove the steel mesh because the pile was fixed by the soil, through that the top level and side of the unconnected piles checked carefully by water bubble in longitudinal, transverse, and diagonal directions as shown in Fig. 9. Although the pile installation method utilized in this study is not the same as in practice, it was chosen instead of the pushing or driving method because using these methods may affect the unit weight of the nearby soil of the piles and consequently, the modulus of elasticity and the wires of the strain gauges in the pile affected the soil in the pushing method.
4. Through the last steps, the instruments fixed in their location. The accelerometer number one in the base plate of the shaking table and the numbers two, three, and four was put at three levels in the body of soil every 200 mm apart as shown in Fig. 9.
5. Sand surface should be leveled carefully such that it should be at the same level as the top pile's head in order to ensure that the applied cushion was definitely at one level with both the top head of the piles and the sand surface as shown in Fig. 9.
6. The cushion layer is raining in a special mold, the special mold used in this step depends on the dimension side and the thickness of the cushion. After that, the soil around the cushion prepared by raining to the level required and then carefully lifting the plate mold to ensure the density did not change.
7. In the reinforcement cushion layer tests, the geosynthetic (biaxial geogrid and non-woven geotextile) was added to the cushion layer in single or double layers. The first layer was at the head piles level and in case of double reinforcement the second layer was in the medium of the cushion layer.
8. The raft was placed on the cushion and the special loading was seated on it; the accelerometer number 5 was fixed in the raft previously.
9. The LVDTs were placed in the designated location, LVDT number one was fixed on the plate of the shaking table to determine the horizontal movement of the shaking plate, while LVDTs number two and three are fixed on the raft to measure the horizontal and vertical movement of the raft as shown in Fig. 9. To prevent the inclination in the LVDT2 that may be happened because of the vertical displacement of the raft, therefore sliding channel was used to allow vertical and horizontal movement at the same time.
10. After that, the instruments were connected to the data acquisition including all the instruments such as accelerometers and LVDTs. The wires extend through the piles and within the soil at the least possible effect on the test.
11. After assurance that all the measurement devices were connected correctly to the data acquisition system and connected to the computer in order to record and save all the readings automatically.
12. Starting the test by operating the computer and starting the shaking table work, the computer recorded and save all the readings automatically.
13. Removing the raft model and all the instruments, discharging sand from the model box, and keeping it in containers (water withdrawal and air drying the soil in case of saturated test).
14. Repeating the procedure mentioned above points to make another test.

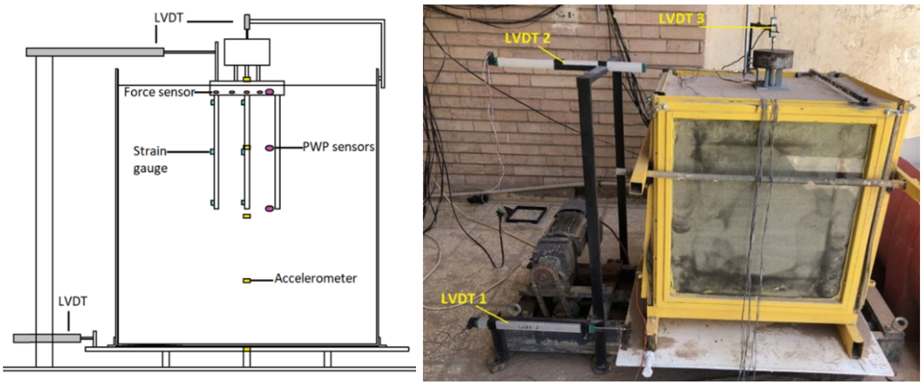


Figure 9. Schematic diagram of model test and instrument's location.

3. Results and Discussion

In this study, the settlement of the unconnected pile was experimentally measured by changing the thickness of the cushion layer, spacing between the piles, and reinforcement of the cushion layer by geosynthetics (biaxial geogrid and non-woven geotextile) using one and two layers. The thickness of the cushion used in the study was 2d, 3d, and 4d where d is the pile diameter. The spacing between piles used was 3d, 4.5d, and 6d. The settlement was determined at the end of the test after 30 seconds after starting the seismic load. The notations of conducted tests in this study are shown in Table 4.

Table 4. Notations and description of conducted shaking table tests.

Test no	Cushion thickness	Spacing between piles	Reinforcement
T1	2d		
T2	3d	3d	
T3	4d		
T4	2d		
T5	3d	4.5d	Without reinforcement
T6	4d		
T7	2d		
T8	3d	6d	
T9	4d		
T10	2d		
T11	3d	3d	One layer consists of biaxial geogrid and non-woven geotextile
T12	4d		
T13	2d		
T14	3d	4.5d	
T15	4d		
T16	2d		
T17	3d	3d	Two layers, each consisting of biaxial geogrid and non-woven geotextile
T18	4d		
T19	2d		
T20	3d	4.5d	
T21	4d		

The results of experimental tests showed that the interposed cushion layer between the raft and piles had caused a downward relative displacement of the soil-pile system. This relative displacement is maximum at the pile heads and extends to a certain depth below the pile heads. The relative displacement becomes zero at a certain depth which is known as the depth of the neutral plane. Initially, the surrounding soil and the raft settle greater than the piles which produce negative skin friction along the upper part of the piles (above the neutral plane). Thus, the piles loaded through their heads as well as through the negative skin friction. Because of the load on the pile head and the negative skin friction, the pile is settling and a mobilized positive skin friction occurring at the lower part of the pile shaft, as well as resistance at the lower tip. It is found that this mechanism was governed by the thickness and stiffness of the cushion layer.

Fig. 10 shows the variation of settlement of the tested soil models without reinforcement for several spacings between piles 3d, 4.5d, and 6d where d is the equivalent pile diameter. Also, the thickness of the cushion layer varies from 2d to 4d. The settlement increases with increasing the thickness of the cushion layer for a small spacing ratio of 3d but increasing the spacing between piles to 4.5d reflects a different trend of behavior. For this ratio of spacing (4.5d), increasing the thickness of the cushion layer to 3d reduces settlement more than 2d, but still, 4d gives higher settlement than the foundation with cushion layers of 2d and 3d. In case of spacing between piles is 6d, the settlement is decreased with increasing the thickness of the cushion layer, but still cushion layer of thickness 4d gives settlement higher than 3d. Generally, reducing the spacing between piles increases the stresses in the foundation and consequently increases the settlement, therefore the thickness of the cushion layer must be matched with spacing between piles to get the optimum benefit of cushion layer thickness. The difference in measured settlement due to changing the thickness of the cushion layer ranged between 0.19 to 9.75%.

To study the effects of changing the spacing between piles on the variation of settlement with time three ratios (3d, 4.5d, and 6d) have been studied as shown in Fig. 11. In each case, the thickness of the cushion layer is assumed constant. The settlement increased with increasing the spacing between piles, but the variation in the final value of settlement after 30 seconds of shaking ranged from 12.75 to 14.56%. Generally, the trend of settlement variation with time is almost the same for all tested cases. It can be seen that most of the settlement 90% occurred during the first 20 seconds of dynamic loading, then a slight increase noticed in the value of the settlement continues to the end of the loading period.

The stiffness of the cushion layer has an important role in increasing the efficiency of the unconnected piled raft foundation in reducing the settlement. However, if the stiffness of the cushion layer is not high enough, the performance of the unconnected piled raft in reducing the settlement is lower but it's better than the unconnected piled raft without reinforcement. Increasing the thickness of the cushion layer without reinforcement led to increasing the value of settlement because the seismic load decreases the angle of internal friction of soil used in the cushion layer and that makes the cushion layer creeps out from the cushion space, but this case is reversed in case of using reinforcement in the cushion layer whether using one or two layers of geogrid, so the settlement decreases with increasing the thickness of reinforced cushion layer. The geosynthetic materials used in reinforcing the cushion layer helps in the redistribution of the load between the pile and increases the stiffness of the cushion layer. Fig. 12 shows the effect of using reinforcement in the cushion layer on the variation of settlement with time for tested soil models. The efficiency of using reinforcement appears significantly when the thickness of the cushion layer is 3d and more and for any spacing between piles. Also, using one layer of reinforcement reduces the settlement of footing better than the cushion layer without reinforcement, and using two layers of reinforcement in the cushion layer reduces the settlement better than using one layer of reinforcement as shown in Fig. 12.

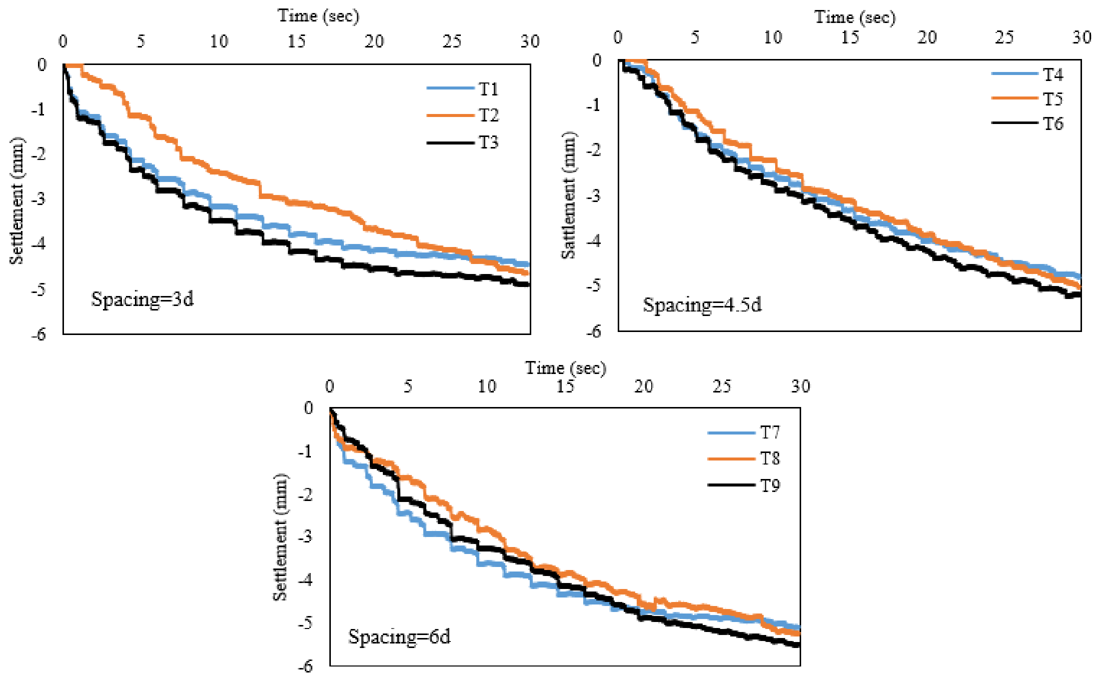


Figure 10. Variations of the settlement with time for soil samples without reinforcement of cushion and change cushion thickness.

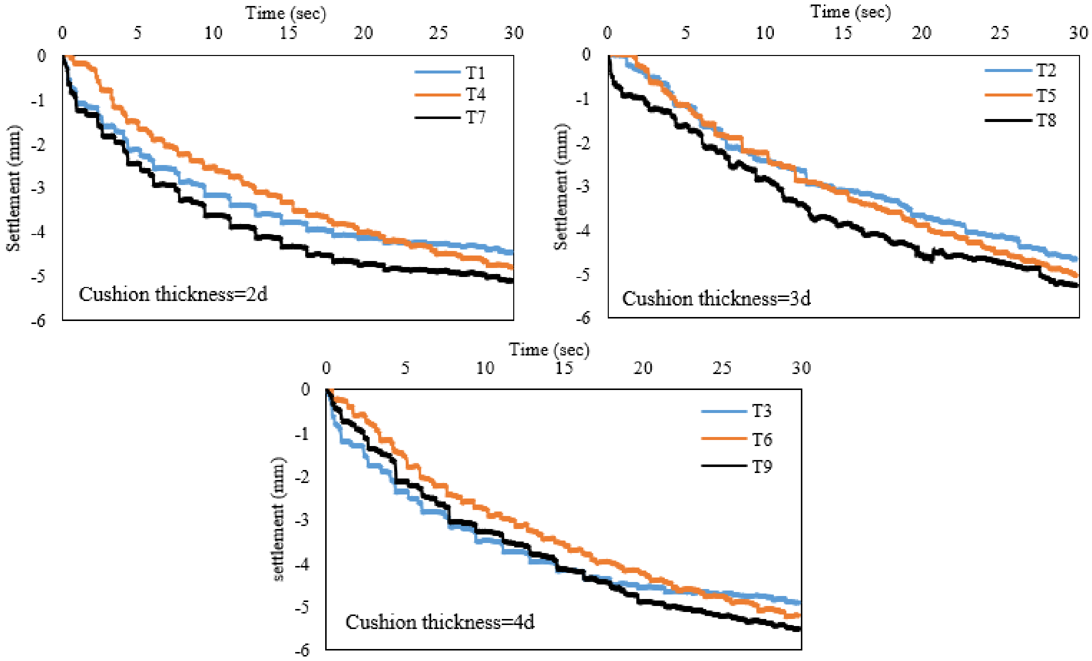


Figure 11. Variations of the settlement with time for soil samples without reinforcement of cushion and change spacing between piles.

The mechanisms of the geosynthetic material in soil work as a tensioned membrane and help to stabilize the soil. The interlocking of geosynthetic-soil particles applied additional horizontal stress to the soil particles under loading and in turn increased the soil shear strength thus creating more stable soil. The vertical component of the tensioned membrane reduced the pressures on the soil between the piles and subsequently increased the pressures on the stationary supports (piles), so the tensioned membrane effect was more effective at the displacement. Fig. 13 and 14 show the effects of using one and two layers of geogrid on the settlement of soil model subjected to dynamic loading with constant spacing between piles. The results presented in these figures exhibited decreasing the settlement with increasing the number of reinforcement layers and this reduction was associated with increasing the thickness of the cushion layer.

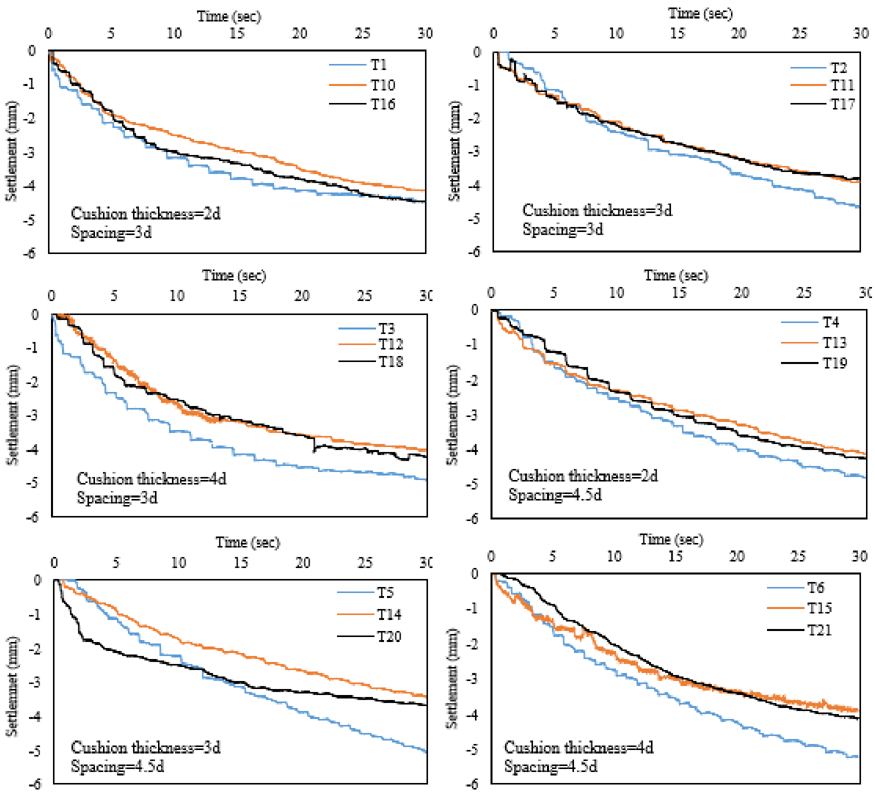


Figure 12. Variations of the settlement with time for soil samples with changing the number of reinforcement layers.

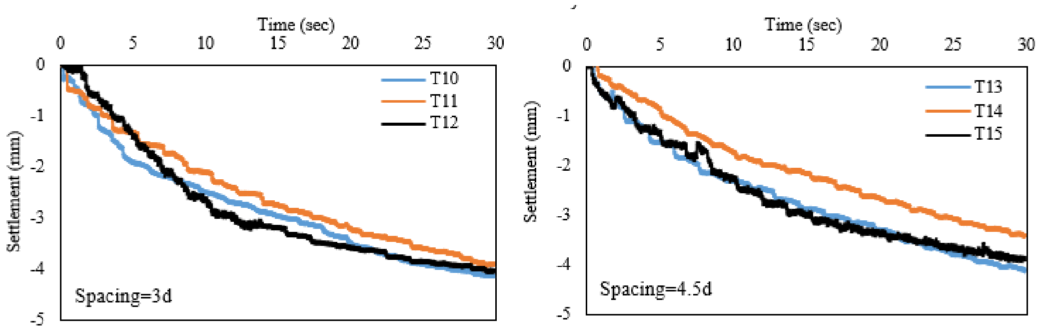


Figure 13. Variations of the settlement with time for soil samples of cushion layer reinforced with one layer of geogrid.

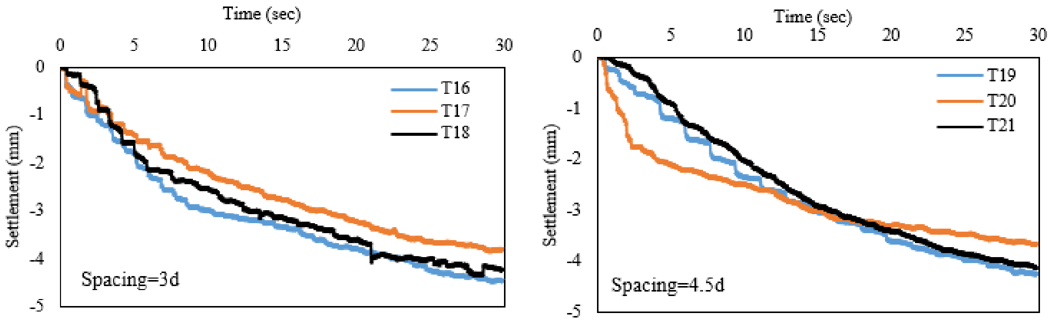


Figure 14. Variations of the settlement with time for soil samples of cushion layer reinforced with two layers of geogrid.

4. Conclusions

This work is devoted to investigating the effects of improvement of the cushion layer in unconnected piles subjected to seismic loading using one and two layers of geogrid. The lateral restraint and the tensioned membrane effect of geosynthetic materials helped transfer the more vertical load onto the supports. The following points can be concluded from the results of the experimental work conducted in this study:

- The settlement of the foundation under the seismic loading decreased by reinforcing the cushion layer with one and two layers of geogrid materials. The improvement in settlement by reinforcement of the cushion layer was between 8 and 27%. There is no significant difference between using one and two layers of reinforcing the cushion layer but using two layers of reinforcement is better than using one layer.
- The reinforcement and the thickness of the cushion layer govern the amount of settlement. Increasing the thickness of the cushion layer led to increasing the settlement of the foundation under seismic loading in the case of using a cushion layer without reinforcement, the settlement increased between 4 and 10% with an increase in the thickness of the cushion from 2d to 3d and 4d. Especially in the case of the small thickness of the cushion layer and spacing between piles.
- The variation of the spacing between piles led to a change in the settlement, where the settlement had decreased with decreasing the value of spacing, the value of the settlement had decreased between 7 and 14% when decreasing the spacing from 6d to 4.5d and 3d. Reduction of the distance between piles makes the foundation system works as a block and that reduces the settlement.
- For a constant thickness of the cushion layer using geosynthetic materials in the reinforcement of the cushion layer reduces the settlement of the unconnected piled raft system.
- The results of tests indicated that there had been a considerable role for the thickness and the stiffness of the cushion layer on the loading sharing ratio between soil, raft, and piles. However, with further increases in either the cushion thickness or cushion stiffness or both the effect of adjustment will decrease.
- It was observed the variation of the spacing between piles led to a change in the settlement, where the settlement had decreased with decreasing the spacing between piles which had made the foundation system work as a block consequently leading to reduce the settlement.

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Information about authors:

Mahdi Karkush, *Doctor of Technical Sciences*
ORCID: https://orcid.org/0000-0003-1304-0303
E-mail: mahdi_karkush@yahoo.com

Amer G. Jihad,
E-mail: amer.ghani@qu.edu.iq

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