Investigation of the behavior of shallow machine foundation resting on a saturated layered sandy soil subjected to a dynamic load

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Abstract. In liquefying soils, Shallow foundations may experience an increase in settlement and displacement due to dynamic loading. Therefore, the machine footing may settle and tilt excessively. In this paper, the settlement of shallow foundation and inner displacement on liquefiable of unreinforced and reinforced saturated multi-layered sandy soils (medium-dense sand MD) will be studied. The relative density of the first and second layers is 50% and 85% corresponding to medium sand soil and dense sand soil respectively. The tests have been carried out on 20 models. The amplitudes of the applied harmonic load are 0.25, 1 and 2 tons with frequencies of 0.5, 1 and 2 Hz. The used foundation was with dimension 200*200*20 mm and the geogrid was used as reinforcement material. For each amplitude and frequency of load, the sand models were tested without and with reinforcement in various configurations (0.5B, 1B using single reinforcement layer and 0.5B, 1B using double reinforcement layers) where B is the square footing width. The results showed the high susceptibility for liquefaction and its potential which increased with increasing amplitude load and amplitude frequency. Also, the surface settlement and displacement were increased in the first layer and reduced with increasing relative density. The results also showed the high effect of reinforcement material and its configuration and number of layers by maintaining the little settlement values when fixing other parameters.

1. Introduction

The design of footing for dynamic loads (as machine footing) various from the design for static loads. Machine footing should supply upholding for machinery and its other parts, and supply sufficient stiffness to maintain machinery in its site with finite vibration amplitudes through ordinary working. Since this high-speed machine must meet high strict vibration standards, the rigidity requirements rather than strength are often controlled in the foundation design [1]. Dynamic loads can arise from various sources such as employing machinery, building work, motion loads on bridges, shocks, and earthquakes [2]. So, the soil periodically undergoes shear stress on site in many conditions. Typical tests have been reported to determine the permanent settlement of a shallow foundation subject to different types of dynamic loads had been a statement by [3]. The outcomes of the dynamic load against the displacement of strip lying on the dense sand are presented in reference [4]. The circular foundation on the granular soil undergoes to vertical vibrations outcomes offered from the laboratory model on the constant displacement [5].
The state of soil of the drainage is impossible, sand has been ready to be liquefied and has no shear strength, and water pressure in pore \( u \) increases with total pressure \( \sigma_v \) constant until \( \sigma_v = u \), then the effective pressure \( \sigma'_v \) will be equivalent to zero. For the reason of altitude stress inside pore water, the contact force among soil particles turn into extremely small or zero, also, in an extreme state, the particle-to-particle contact can break when the excess pore water pressure has been increased for this great rates. In several states, the soil will show slight or non-shear strength with behavior most similar a viscous liquid [6].

Assessment of the efficiency of geogrids in enhancing soil settlement and bearing capacity has been done in the latest years by many studies, due to engineers' desire to optimize geotechnical structures. Geosynthetics materials widely used, its economy, easy of installed, the using of reinforced soils in applications of geotechnical engineering have resulted in performance and depended ability as in the construction of road, embankments of railway, slopes steadiness and weak ground amelioration, etc. for example [7-8].

This paper presents the results of laboratory model tests to exam the liquefying soils, inner displacement and the settlement of rigid square machine foundation resting on the surface of unreinforced and reinforced multilayered sand soil with different densities subjected to dynamic loads in different amplitude loads (0.25, 1 and 2) tons and different frequencies (0.5, 1 and 2) Hz.

2. Geometry of the Problem

In many practical engineering causes, it may be necessary to lay shallow foundations on stratified deposits. A layer of deposits below shallow footing affecting bearing capacity is named an underground. The thickness of underground by a simplified analysis can be shown as Eq. 1 by [9]:

\[
H = \frac{b}{2} \left( \tan \theta + \frac{\phi}{2} \right)
\]

(1)

Where \( B \) is a width of a shallow foundation and \( \phi \) is the angle of soil internal friction. The subsoil displays a layered structure if the thickness of the deposit surface layer is less than \( H \). In the most practical problems, the subsoil is two-layered as shown in figure 1.

![Figure 1. Geometry of the problem, h1 depth for top layer, h2 depth for lower layer.](image)

3. Procedure of Testing

The tests conducted in a total number of 20 models. All the model tests of saturated sand under dynamic loads with a relative density equal to 50% and 85% respectively corresponding to medium-dense sand MD multi layers soil (\( h_1 = 15 \) cm for 1st layer and \( h_2 = 55 \) cm for 2nd layer) as shown in figure 1. The sand models testing without reinforcement and with reinforcement of three series of reinforcement depth from
the model surface (0.5B and 1B and 0.5B,1B where B is the width of foundation 200×200×20 mm), the foundation at the surface. In this study, Karbala sand is used and the physical properties of sand accomplished through the standard tests. Table 1 shows the physical properties of sandy soil.

Table 1. Physical properties of used sandy soil.

| Index Properties                           | Value   | Specification          |
|--------------------------------------------|---------|------------------------|
| Specific gravity                           | 2.6     | ASTM D 854 (2006)      |
| D10 (mm), D30 (mm), D60 (mm)               | 0.15, 0.2, 0.5 | ASTM D 422 and 2487 (2006) |
| Coefficient of Uniformity (Cu)             | 3.3     |                        |
| Coefficient of Curvature (Cc)              | 1.2     |                        |
| Soil classification (USCS)                 | SP      |                        |
| Minimum Void ratio                        | 0.49    | ASTM D4254 (2014)      |
| Maximum Void ratio                        | 0.33    | ASTM D4254 (2014)      |
| Maximum dry unit weight (kN/ m³)           | 19.5    | ASTM D 4253 (2006)     |
| Minimum dry unit weight (kN/ m³)           | 17.4    | ASTM D 4254 (2006)     |
| Angle of internal friction (RD =50%)       | 40°     | ASTM D 3080 (2006)     |
| Angle of internal friction (RD =85%)       | 44°     | ASTM D 3080 (2006)     |

By using a steel tamping hummer sand deposit was prepared. To attain the relative density calculated the weight required since the volume and unit weight of the sand are calculated before also. The soil of each layer was compacted to predetermine depth then complete the soil deposit, through that reinforcement placing in wanted depth, finally, top surface scrap and level by a ruler to get a flat surface. The square foundation (200×200) mm lead in touch with upper face of the model.

4. Reinforcement Material

The reinforcement material used in this study was geogrid manufactured by Al-Latifya Factory for plastic mesh, which are made by uni-axially or bi-axially drawing a perforated polymeric sheet in a technique that produces a mesh structure with uniform apertures, as shown in figure 2. The engineering properties provided by the manufacturing company [10-12] are given by Table 2.

Table 2. Technical properties of geogrid used [10].

| Property                     | Test Method | Unit per (m) length | Data* |
|------------------------------|-------------|---------------------|-------|
| Tensile strength at 2 %      | ISO10319    | kN/m                | 4.3   |
| Tensile strength at 5 %      | ISO10319    | kN/m                | 7.7   |
| Peak tensile strength 1      | ISO10319    | kN/m                | 13.5  |
| Yield point Elongation       | ISO10319    | %                   | 20.0  |
5. Devices and Design Model

To examine liquefying soils, inner displacement with the settlement of rigid square machine foundation resting on the surface of unreinforced and reinforced multilayered sand soil will be investigated. It is needful to imitate the conditions closest to that occur in field. To realize this goal, apparatus for special testing with another parts were created by [13]. The apparatus can apply various dynamic loads and various frequencies. The device overview is shown in figures 3. The device is composed from next parts: steel frame load, shaft encoder, model footing, acquisition data, ADXL345 digital accelerometer, axial loading system, and steel container. An electro-mechanical shaft encoder is an apparatus utilized to convert the movement of the shaft to a digital code. The additional output provides from encoder movement that is processed to get information for, settlement, revolution per minute (rpm), speed, and position. The model test configurations used in this work are shown in Table 3.

6. Dynamic Loading Test

After placing the foundation on the sand layer surface, the dynamic load was applied for a preset sequence. The dynamic download application has continued for 20 minutes. The dynamic load function is represented by Eq. 2:

$$F(t) = a_o \cdot \sin \omega t$$

Where $a_o =$ amplitude of load, $\omega =$ frequency of load, $t =$ time, and $T =$ period. The shape of the dynamic applied load wave is of the form close to the sinusoidal compressive type as shown in figure 4.

7. Results and discussion

7.1. Dynamic Settlement for Surface Foundation

Figures 5 to 9 show exemplary correlation of settlement against the number of cycles of different amplitudes of load with and without reinforcement. Generally, it shows same trend the curves followed, and as the dynamic loading increases, increases of the settlement at surface. Also, noticed the range of settlement magnitudes in dense sand less than the range in medium sand, because a rise in particle pressure. These figures showed sudden settlement in the first layer, then gradual increase until reach approximately stable of sample, that means the second layer of the soil depends on the existence of reinforcement and the other studied parameters, (dense layer resists load amplitude until it fails, that very clear with amplitude load 1
ton and 2 tons). Also, fewer values of settlement can be obtained where amplitude of load be 0.25 ton with amplitude frequency equal to 0.5 Hz, but that rises with rising the amplitude of frequency and load. Furthermore, the outcomes exhibited the percentage of vertical settlement ratio can be reduced by about (5-28%) when using reinforcement at a depth equal to (0.5B), while the percentage was (2-30%) when using reinforcement at a depth equal to (1B), but when using two layers of reinforcement the better values of enhancement can reached between (11-47%). From that, it can be seen that the less less enhancement in the settlement vales in the first soil layer (medium sand) because of the less density and the effect of loading. The effect of reinforcement more explains when its lays at a depth equal to B. When the use of two layers of reinforcement, effective enhancement is achieved, this is because reinforcing entire layer by the grid as an entity to a specific depth or region.

Table 3. Model tests configuration using.

| No. | Configuration of test | Load (ton) | Frequency (Hz) | No. of reinforcement layers | Configuration of reinforcement layers |
|-----|-----------------------|------------|----------------|-----------------------------|--------------------------------------|
| 1   | 0L 0.25T 2Hz          | 0.25       | 2              | -                           | -                                    |
| 2   | 1L 0.5B 0.25T 2Hz     | 0.25       | 2              | 1                           | 0.5B                                 |
| 3   | 0L 0.25T 1Hz          | 0.25       | 1              | -                           | -                                    |
| 4   | 2L 0.5B,1B 0.25T 2Hz  | 0.25       | 2              | 2                           | 0.5B, 1B                             |
| 5   | 1L 0.25T 1Hz          | 0.25       | 1              | 1                           | 0.5B                                 |
| 6   | 1L 0.5B 0.25T 1Hz     | 0.25       | 1              | 1                           | 0.5B                                 |
| 7   | 1L1B 0.25T 1Hz        | 0.25       | 1              | 1                           | 0.5B                                 |
| 8   | 2L 0.5B,1B 0.25T 1Hz  | 0.25       | 1              | 2                           | 0.5B, 1B                             |
| 9   | 0L 0.25T 0.5Hz        | 0.25       | 0.5            | -                           | -                                    |
| 10  | 1L 0.5B 0.25T 0.5Hz   | 0.25       | 0.5            | 1                           | 0.5B                                 |
| 11  | 1L 1B 0.25T 0.5Hz     | 0.25       | 0.5            | 1                           | 0.5B                                 |
| 12  | 2L 0.5B,1B0.25T 0.5Hz | 0.25       | 0.5            | 2                           | 0.5B, 1B                             |
| 13  | 0L 1T 2Hz             | 1          | 2              | -                           | -                                    |
| 14  | 1L 0.5B 1T 2Hz        | 1          | 2              | 1                           | 0.5B                                 |
| 15  | 1L 1B 1T 2Hz          | 1          | 2              | 1                           | 0.5B                                 |
| 16  | 2L 0.5B,1B 1T 2Hz     | 1          | 2              | 2                           | 0.5B, 1B                             |
| 17  | 0L 2T 2Hz             | 2          | 2              | -                           | -                                    |
| 18  | 1L 0.5B 2T 2Hz        | 2          | 2              | 1                           | 0.5B                                 |
| 19  | 1L 1B 2T 2Hz          | 2          | 2              | 1                           | 0.5B                                 |

Figure 4. Dynamic load wave.
Figure 5. Settlement of surface footing for reinforcement and non-reinforcement MD deposit with 0.25 ton load and 2 Hz frequency.

Figure 6. Settlement of surface footing for reinforcement and non-reinforcement MD deposit with 0.25 ton load and 1 Hz frequency.

Figure 7. Settlement of surface footing for reinforcement and non-reinforcement MD deposit with 0.25 ton load and 0.5 Hz frequency.

Figure 8. Settlement of surface footing for reinforcement and non-reinforcement MD deposit with 1 ton load and 2 Hz frequency.

Figure 9. Settlement of surface footing for reinforcement and non-reinforcement MD deposit with 2 ton load and 2 Hz frequency.
7.2. Displacement under foundation

Measurements of the footing displacement magnitudes achieved at three levels (0.5B, 1B and 1.5B) for each test with explained some of these in figures 10 to13. The displacement was measured by using a ADXL345 Digital Accelerometer sensors to measure acceleration and then computed displacements by using Deepsoil program. As shown in the result of the tests, amplitude of displacement for saturated sand with time, it can be observed that the trend of the relationship of the Displacement-Time for all test results is unique, its appeared a high measurement values at start and end test for low amplitude load, referred to the conditions of test and response of the dynamic behavior of soil. So, when applied low load amplitude the high displacement magnitude measured then stable along time of test until load removed, appeared significantly change in magnitude of displacement. Therefore, when increasing amplitude load the change in displacement values continuous a long of time test, clearly. From that, one may conclude that the high percentage of loading indicates the high values of displacement and settlement.

Figure 10. Displacement variation of surface foundation for non-reinforcement MD deposit at depths 0.5B, 1B, and 1.5B with time under 0.25 ton amplitude load and 2 Hz amplitude frequency.
Figure 11. Displacement variation of surface foundation for reinforcement 1 layer at 0.5 B MD deposit at depths 0.5B, 1B, and 1.5B with time under 0.25 ton amplitude load and 2 Hz amplitude frequency.

Figure 12. Displacement variation of surface foundation for reinforcement 2 layer at 0.5 B and 1B MD deposit at depths 0.5B, 1B, and 1.5B with time under 1ton amplitude load and 2 Hz amplitude frequency.
Figure 13. Displacement variation of surface foundation for reinforcement 2 layer at 0.5 B and 1B MD deposit at depths 0.5B, 1B, and 1.5B with time under 2 ton amplitude load and 2 Hz amplitude frequency.

7.3. Liquefaction Potential Evaluation

Liquefaction factor of safety (FL) is assessed as ratio of cyclic resistance, it is indicated as (CRR) / ratio of stress cyclic, it is indicated as (CSR) i.e., the loading caused by earthquake divide by resistance of the soil to liquefaction. CSR represents the pressure caused by the forces of earthquake, CRR called the identical soil column resistance [15]. Hazard of liquefaction may be makes groups as liquefaction not happen (FL ≥ 2.0), reasonable (1.5 ≤ FL < 2), rise (1 ≤ FL < 1.5), and very rise (FL <1.0) [14-18]. The next steps are the suggested way to calculate the safety factor against liquefaction. The suggested procedure is suitable for load of dynamic terms of this study:

i. Calculating the dynamic force from equation (3) [6]:

\[ F_{\text{dynamic(max)}} = A_2 \sqrt{k^2 + (c\omega)^2} \]  
\[ k = \frac{4Gr}{1-\nu} \]  
\[ c = \frac{3A}{1-\nu} \cdot \frac{r^2}{\sqrt{G/2g}} \]
\[ G = \frac{E}{2(1+\nu)} \]  

(6)

Where: \( A_z \): vibration amplitude of footing, \( k \): constant of spring, \( c \): coefficient of damping, \( \omega \): circular frequency = \( 2\pi f \) (\( f \) = vibration frequency), \( G \): at surface modulus of shear, \( E \): Young’s modulus, \( \nu \): ratio of Poission, \( r \): foundation radius, square foundation radius Equivalent = \( r_o = \sqrt{\frac{B}{\pi}} \), \( \gamma \): the soil unit weight of, \( g \): gravity acceleration.

ii. Computing stress of the dynamic load by used equation (7) which acts the pressure caused by the forces dynamic influence, represented by CSR.

\[ \sigma_d = \frac{F_{dynamic}}{area} \]  

(7)

iii. Calculation CRR, it is equal soil column resistance, it seems by the soil shear strength:

\[ \tau = \bar{c} + (\bar{\sigma} + \sigma_d)\tan\phi \]  

(8)

Where: \( \bar{c} \) = cohesion of soil at surface.

iv. By using equation (9), computing of the safety factor:

\[ F_L = \frac{CRR}{CSR} \]  

(9)

Table (4) presents the computing of foundation and soil properties for multi-layer sandy soil, while Table (5) shows the computing of surface foundation liquefaction potential for different soil models, various amplitude loads and various amplitude frequencies. From Table (5), can see the factors of safety against liquefaction of surface footing is various from test to test and the outcomes appear that the liquefaction happen in most cases and the very high hazard of liquefaction. The causes for few magnitude of the safety factor less than one is that the load capacity is too high.

**Table 4. Properties of soil and surface foundation.**

| Relative Density % | Unit weight \( \gamma \) (Kn/m3) | Equivalent radius of footing \( r_o \) (m) | Poisson ratio \( \nu \) | Modulus of elasticity \( E \) (kpa) | Shear modulus \( G \) (kpa) | Spring constant \( K \) (kn/m) | Damping coefficient \( c \) (kn-s/m) |
|-------------------|--------------------------|-------------------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| 50                | 18.52                    | 0.113                   | 0.3             | 35000           | 13462           | 8693            | 9.87            |
Table 5. Potential of liquefaction for foundation at surface resting on medium-dense MD sand.

| Test name (table 3) | K (kn/m) | Damping c (kn.s/m) | ω (rad/sec) | Az (m) × 10⁻³ | Fd (kN) | Dynamic stress σd (kPa) | τ (kPa) | FL |
|---------------------|----------|-------------------|-------------|---------------|--------|----------------------|--------|----|
| 0L 0.25T 2Hz        | 8693     | 9.87              | 12.56       | 1.082         | 9.40   | 235.16               | 202.72 | 0.862 |
| 1L 0.25T 0.25T 2Hz  | 8693     | 9.87              | 12.56       | 0.084         | 0.73   | 18.25                | 20.72  | 1.135 |
| 2L 0.25T 0.25T 2Hz  | 8693     | 9.87              | 12.56       | 0.075         | 0.65   | 16.30                | 19.08  | 1.170 |
| 0L 0.25T 1Hz        | 8693     | 9.87              | 6.28        | 0.098         | 8.54   | 213.63               | 184.65 | 0.864 |
| 1L 0.25T 0.25T 1Hz  | 8693     | 9.87              | 6.28        | 0.167         | 1.45   | 36.29                | 35.86  | 0.988 |
| 1L 0.25T 1Hz        | 8693     | 9.87              | 6.28        | 0.367         | 3.19   | 79.76                | 72.32  | 0.906 |
| 2L 0.25T 1Hz        | 8693     | 9.87              | 6.28        | 0.08          | 0.69   | 17.38                | 20     | 1.150 |
| 0L 0.25T 0.5Hz      | 8693     | 9.87              | 3.14        | 2.614         | 22.7   | 568.09               | 482.03 | 0.848 |
| 1L 0.25T 0.25T 0.5Hz| 8693     | 9.87              | 3.14        | 1.155         | 10.0   | 251.01               | 216.0  | 0.860 |
| 1L 0.5B 0.25T 0.5Hz | 8693     | 9.87              | 3.14        | 0.655         | 5.69   | 142.34               | 124.84 | 0.877 |
| 2L 0.5B,1B 0.25T 0.5Hz| 8693  | 9.87              | 3.14        | 1.643         | 14.2   | 357.06               | 304.99 | 0.854 |
| 0L 1T 2Hz           | 8693     | 9.87              | 12.56       | 0.007         | 0.06   | 1.521                | 6.68   | 4.394 |
| 1L 0.5B 1T 2Hz      | 8693     | 9.87              | 12.56       | 2.138         | 18.5   | 464.68               | 395.28 | 0.850 |
| 1L 1B 2Hz           | 8693     | 9.87              | 12.56       | 4.138         | 35.9   | 899.38               | 759.99 | 0.845 |
| 2L 0.5B,1B 2Hz      | 8693     | 9.87              | 12.56       | 0.26          | 2.26   | 56.51                | 52.82  | 0.934 |
| 0L 2T 2Hz           | 8693     | 9.87              | 12.56       | 1.677         | 14.5   | 364.64               | 311.34 | 0.853 |
| 1L 0.5B 2T 2Hz      | 8693     | 9.87              | 12.56       | 3.643         | 31.6   | 791.79               | 669.72 | 0.845 |
| 1L 0.5B,1B 2T 2Hz   | 8693     | 9.87              | 12.56       | 2.289         | 19.9   | 497.50               | 422.81 | 0.849 |

8. Conclusion

- The settlement of medium-dense (MD) soil be in its maximum values in the start of a test, then try to be more stable.
- The noticeable reducing of settlement values when using geogrid as reinforcement material especially when used multi-layers. The settlement decrease of about 45% for the soil incorporated geogrid material.
- When the load amplitude increases, the displacement amplitude increases.
- The amplitude of displacement decrease when the depth increases and relative density increases.
- The potential of liquefaction in multi-layered sandy soil increase with the increasing of load amplitude and frequency.
- The oscillating loading of water pressure seems to severely affect the liquefaction and the liquefaction rate increases with the decrease of sand strength or increase the loading force.

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