INVESTIGATION OF THE END BEARING LOAD IN PILE GROUP MODEL IN DRY SOIL UNDER HORIZONTAL EXCITATION

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pile group, horizontal shaking, dry sand, end bearing

Abstract
A series of 94 laboratory tests were conducted to measure the response of pile foundation when subjected to dynamic loads. Eight tests were conducted on single pile in dry soil at relative density 30 % (loose) and 50 % (medium); 66 tests on group of piles with different spacings and patterns. All tests were carried out under operating frequencies 0.5, 1 and 2 Hz under horizontal shaking. All tests were achieved with one embedment ratio (L/d = 30). These tests were grouped in three different numbers of piles; 2 piles in row and line patterns, 3 piles and 4 piles; and three pile spacing ratios (s/d = 3, 4 and 5).

The results of dry soil indicating the mechanism of dynamic response of piles and soil subjected to dynamic horizontal shaking include the variation and distribution of acceleration with time in different states of soil in addition to the vertical and horizontal displacements, end-bearing load, peak acceleration and the peak velocity of foundation.

It was concluded that for a dry soil bed, the acceleration amplitudes increase with frequency for both soil relative densities (loose and medium) and different pile patterns (number; single or group and different spacing ratios s/d). The maximum acceleration in the foundation is lower than in the soil bed for all operating shaking frequencies, pile spacing ratios and soil states. The decreasing of the maximum acceleration recorded in the foundation as

PREISKAVA NOSILNOSTI KONICE V MODELU SKUPINE PILOTOV V SUHIH TLEH OB VODORAVNEM VZBUJANJU

Izveden je bil niz 94 laboratorijskih preizkusov za merjenje odziva temeljenja na pilotih, ki so bili izpostavljeni dinamičnim obtežbam. Osem preizkusov je bilo izvedenih na posamičnem pilotu v suhi zemljini z relativno gostoto 30 % (rahlo) in 50 % (srednje gostoto stanje); 66 testov na skupini pilotov z različnimi razmiki in vzorci. Vsi preizkusi so bili izvedeni pri delovnih frekvencah 0.5, 1 in 2 Hz pri horizontalnem trešenju. Vsi preizkusi so bili izvedeni z enim razmerjem vpetosti (L/d = 30). Ti preizkusi so bili združeni v tri skupine z različnimi števili pilotov; 2 pilota v vrsti in linijskem vzorcu, 3 pilota in 4 pilota; in tri razmerja razmika med piloti (s/d = 3, 4 in 5).

Rezultati za suho zemljino kažejo mehanizem dinamičnega odziva pilotov in zemljine, izpostavljenih dinamičnemu vodoravnemu trešenju, dodatno k vertikalnim in vodoravnim premikom, nosilnostim na konici, največjem pospešku in največji hitrosti temeljenja vključujejo še spreminjanje in porazdelitev pospeška s časom v različnih stanjih zemljine. Ugotovljeno je bilo, da se za suho zemljinsko osnovno plast amplitudo pospeševanja s frekvenco povečujejo tako za rel ativni gostoty zemljine (rahlo in srednje gostoto stanje) kot za različne vzorce pilotov (število; posamični pilot ali skupina pilotov in različna razmerja razmika s/d). Pri vseh delovnih frekvencah trešenja, razmerjih razmikov med piloti in stanji zemljine je največji pospešek v temelju nižji kot v zemljinski osnovni plasti. Zmanjšanje največjega pospeška,
compared to that in the soil bed is between 10-100 % for loose and medium state of soil, and the decrease in loose state is more than in medium state. This means that there is damping effect or attenuation of vibration waves. The amplitudes of recorded acceleration in the pile cap are much higher than in the soil bed for single pile and pile group with different pile spacing ratios, also these amplitudes are increasing with increase of shaking frequency and relative density of the soil.

1 INTRODUCTION

The pile foundations response under dynamic loading is comparatively more complicated than that for static loading. Unfortunately till now, the piled foundation system performance subjected to cyclic loads or during earthquakes is not completely understood [22]. The soil-pile-foundation interaction is one of the main vital issues in the dynamic analysis which needs more understanding. However, a very little information is available on observed dynamic behavior of pile foundations which is due to difficulties in performing such tests involving the several variables related to both soils and piles.

Numerous shaking table tests have been conducted on sand soils, with and without the pile models. These practical examinations include tests to study ground responses and behavior of the sand, and model pile tests to evaluate the pile behaviors and soil-pile interactions under shakings. Further tests and analyses of the shaking test data are required for a better understanding of behaviors of soil, soil-pile interaction, and combining the generated movement and pile responses under earthquake shakings. The shaking table test data may be used for verification of analysis methods and numerical modeling for ground responses and soil-pile interactions during earthquake shakings.

Full-scale tests are generally assessed to submit the most reasonable results but, are limited because of their high costs. The limitations highlight the two main difficulties of carrying out field full-scale tests on piles subjected to earthquake excitation, essentially the difficulty in modeling bedrock excitation and providing enough energy to excite large pile groups in addition to their significant cost, time and effort. However, a number of studies dealt with centrifuge and shaking table model testing. The most common previous studies on dynamic testing of piles have been achieved with full scale piles are; Maxwell et al. [19], Novak and Grigg [20], Gúi [17]; on small scale tests; Fattah et al. [14][15].

Brown [12] studied dynamic and static lateral loading on pile groups using different full-scale field tests carried out on groups of piles of six to 12 piles, both driven and bored, in relatively cohesionless and soft cohesive soils. All the pile groups were loaded laterally statically reaching large deflections, and instrumented pipe pile groups were also tested under dynamic loads reaching large deflections, equivalent to those that might the pile be subjected to due to major impact of ships and earthquakes. Dynamic loading was exerted by a number of impulses of increasing magnitude adopting a horizontally mounted Statnamic device. While this loading did not cover the characteristics of lateral loading and ground motion that may cause development of high pore water pressures, it did cover the damping that occurs at high levels of pile deflections and the inertial effects of the structure.

Banerjee [9] presented the behavior of pile foundations under earthquake loading. The study investigated the interaction between soil behavior, pile stiffness and superstructure inertial loading on pile response during earthquake. It was noted that the soil around the piles does not just support the piles, it also exerts inertial loading on the piles. Pile head loading cannot replicate this effect. Experimental data have been deduced from a centrifuge modeling, which then used as a basis for validating and calibrating numerical analyses. The research involved four major components: (1) characterization of the dynamic properties of kaolin clay through element testing using the cyclic triaxial and resonant column apparatus; (2) dynamic centrifuge testing on pure kaolin clay beds (without structure) followed by 3-D finite element back-analyses; (3) dynamic centrifuge testing on clay-pile-raft systems and the corresponding 3-D finite element back-analyses and (4) parametric studies leading to the derivation of a semi-analytical closed-form solution for the maximum bending moment in a pile under seismic excitation.

Boominathan et al. [11] carried out a study by full-scale lateral dynamic pile load testing to determine the dynamic characteristics of soil-pile system. The results of
two full-scale field dynamic lateral pile load tests carried out at two different sites in India (Chennai and Hazira) and the results of a nonlinear three-dimensional finite element analysis of piles under dynamic lateral loads using the program ABAQUS were presented. The non-destructive technique known as Multichannel Analysis of Surface Waves (MASW) was used for determination of the shear wave velocities of different layers up to a depth of 12.5 m below the ground level based on the average SPT-N value and for evaluation the stiffness (maximum dynamic shear modulus) of the subsurface required finite element analysis. A steady state sinusoidal force was generated with a 5-tonne capacity mechanical oscillator. The forced vibration response of the piles was measured using two acceleration transducers fixed at the mid height of the pile cap, and at the pile cut off level. After every steady state lateral vibration test, the eccentricity of the oscillator was increased to raise the dynamic force and the test was repeated to cover a wide range of lateral displacements expected during a typical dynamic loading of the pile.

Janalizadeh and Zahmatkesh [18] applied a pseudo-static method for estimation of the response of pile during dynamic loading. The geometry and the soil modeling parameters have been defined, and then the numerical model was verified by means of the centrifuge test. Next, the behaviors of piles were studied with the effects of various parameters such as soil layering, kinematic and inertial forces, boundary condition of pile head and ground slope. A method for analysis of piles in liquefiable soil under seismic loads has been presented. Three cases were considered to evaluate the effects of variations in stiffness and lateral resistance of the p-y curves on the testing results.

The dynamic response of pile foundation in dry sandy soil excited by two opposite rotary machines was considered experimentally by Fattah et al. [15]. A small scale physical model was manufactured to accomplish the experimental work in the laboratory. The physical model consists of two small motors supplied with eccentric mass (0.012 kg) and eccentric distance (20 mm) representing the two opposite rotary machines, an aluminum shaft as the pile, and a steel plate a pile cap. The experimental work was achieved taking the following parameters into considerations: pile embedment depth ratio ($L/d$, where $L$ is the pile length and $d$ is its diameter) and operating frequency of the rotary machines. All tests were conducted in medium dense fine sandy soil with 60 % relative density. To predict precisely the dynamic load that will be induced from the rotary machines, a mini load cell with a capacity of 100 kg was mounted between the aluminum plate (the machine base) and the steel plate (pile cap). The results revealed that, before machine operation, the pile tip load was approximately equal to the static load (machine and pile cap), whereas during machines’ operation, the pile tip load decreased for all embedment depth ratios and operating frequencies. This reduction was caused by the action of skin friction that was mobilized along the pile during operation, and as a result the factor of safety against pile bearing failure increases. For all operating frequencies and pile lengths, the factor of safety against bearing failure increased during machines’ operation, where the pile tip load became less than its value before starting operation. During operation, the skin friction resistance mobilized along pile length led to decrease the bearing load.

The objectives of the present study are determination of the frequency independent dynamic response of both single pile and group of piles to lateral vibration for different patterns and spacings, calculation of a velocity and acceleration-time history in addition to displacement - time history of pile groups subjected to earthquake excitation, and investigating the effect of soil confinement due to pile spacing on the load transfer in pile groups.

2 TESTING APPARATUS AND METHODOLOGY

The testing device (the manufactured model) is a metal structure, which consists of three main interrelated parts. All these parts have the ability to slide (slip) one against the other by means of ball bearings, which can work together giving a relative horizontal motion between them as shown in Figure 1a. The two parts have been linked by a piece of metal connected by steel screws to fix these movable sliding parts.

In the second part (slide II), a metal holder (with dimensions 800 mm wide and 400 mm long) is mounted which is also being slided by ball bearings along the longitudinal axis with a distance more than 600 mm in the two directions (sides). But in this work, this distance was limited to only 50 and 60 mm.

A steel piece (plate) of L-shape (with dimensions of 900 mm wide, 1000 mm long and 300 mm high) is mounted while strengthening its edge and base by three triangular stiffeners to avoid any rush or slippage of interior or exterior parts as shown in Figure 1b. Another two ball bearings with internal diameters of 45 mm are mounted within the bracket base in which a connectivity and installation screw (PIN) enters to get a reciprocating motion as shown in Figure 1c. A decentralized source motion must be generated and connected via an arm to the L-shaped base plate which is installed on the
metal holder. Then, a linear reciprocating movement must be determined at distances of 50 mm and 60 mm that means a drift from the center by 25 mm or 30 mm radius from every direction as illustrated in Figure 1d. Two decentralized Cama which is a rotating pin for translation of movement (with diameter of 95 mm) have been manufactured with downward drift distance of 25 mm or 30 mm from the center as shown in Figure 2c. They were mounted on three-phase engine, with capacity of 3 horsepower. A rotation speed of 1450 rpm was used as an incentive for the rotational motion as shown in Figure 2.

This decentralized Cama rotates inside the bearing ball (needle bearing) which is linked by a 400 mm long connecting arm (or connecting rod) to the eccentricity installed by a pin as shown in Figure 2. As the test in this research requires obtaining the reciprocating motion at different speeds and frequencies, so an electric current controller (AC Inverter from Hyundai Company) for different rotational speeds of the engine was chosen. The inverter is used to determine the type and speed of rotation. To get the required velocities in this research, the inverter has been linked to a gear box (Configuration Gear Box through the shaft) to reduce the speed by around 3 folds.

3 STEEL BOX

The other part of the manufactured device is the steel box, which is used for model tests. Its dimensions are (800 × 800) mm for its base and 1000 mm height, it is connected with the L-shaped steel plate by four screws M12 for installation and to prevent any movement as shown in Figure 3. A side slot 400 mm wide and 700 mm high of the steel box has been made to facilitate the process of discharging sand or soil as shown in Figure 3. A steel angle has been installed at the top of the steel box to make a platform for the devices and sensors used in the test as shown in Figure 3.

During tests different velocities are used from slow to rapid motion (1 up to 14) Hz, the motion is slow without any strong vibrations. But, with increasing the rotational velocity, the motion is converted to be a linear speed accompanied with the appearance of a direction change after the cycle end for outgoing and return giving unacceptable vibrations due to the great moving mass, which generates a high momentum and high inertia (I).

A little change of soil mass for different model tests has a limited effect on (I) but the speed is of greatest value in increasing the value of (I), so when the velocity is incre-
ased, the linear mass starts to move quickly making it difficult to change the direction smoothly, therefore this problem has been solved by adding operating dampers to absorb the surging mass momentum at the end of the half, then give the initial speed in the opposite direction of the movement after the arrival of the CAMA to the tipping point as shown in Figure 4.

It is worthy to mention that in order to make the damping value variable in correspondence to the change (increase or decrease) in the linear speed; the dampers have been connected with source of pneumatic pressure from the compressor tank added to the system. The compressor contains a regulator valve for air pressure to provide air to the dampers at different pressures according to the selected speeds as shown in Figure 4.

4 RAINING TECHNIQUE

To obtain a homogeneous fill of sand with specific relative densities inside the steel box, a sand raining technique device had been manufactured with dimensions (700×700×200) mm³. The device is supplied with perforated cone holes distributed in an adequate way in correspondence to the speed of the sand falling, height of fall and the required relative density. This technique regulates the mechanism of sand fall, filling method, and the homogeneity of distribution, Figure 5. This box is fitted from its four corners by hooks and steel chain.

Figure 3. Steel box.

Figure 4. Pneumatic dampers system.

Figure 5. Sand raining box and meshes.

Mesh of conical opening
through a mechanism allowing its vertically upward and downward movement for the required distances of sand fall. The sand box is supplied in its bottom by mechanical gates. Wherever the height reached, the gates are opened simultaneously. The main job of this gate is to open the holes and allowing sand fall and vice versa.

The “raining technique” is used to deposit the soil in the testing tank at a known and uniform density, and in preparing the tested soil. The device consists of a steel tank, with dimensions of 700 mm length, 700 mm width and 200 mm height.

5 DATA ACQUISITION SYSTEMS

The system of data acquisition was utilized so that all data could be scanned and recorded automatically, this system consists of the following:

1. Strain gauge data logger
2. Pile’s tip load data system
3. LVDT data system
4. Accelerometer Data System
5. Vibration data system

6 MODEL PILES

The model pile used has a diameter of 18 mm. The size of pile’s model was chosen after reviewing the literature about the suitable pile size that could be considered representative. Although, Vesic [23] stated that “scale effects will be complex for model of piles smaller than 35 mm in diameter”, many researchers used smaller diameters in their tests: Al-Mhaidib [1] used steel piles with 30 mm in diameter, Boominathan and Lakshmi [10] used aluminum pile with a diameter of 19 mm and Al-Mhaidib [2] used steel piles with (25 mm) in diameter. The dimensions of the pile model that will be used in this study were also selected to minimize the boundary effects of the soil container in the experimental setup.

The ratio between the equivalent ground plane diameter of tank and the structural plane size of the test object (pile) was taken equal to 44. This equivalent diameter is large enough, so as the circumferential circle radius exceeds the extent far beyond the zone of primary compaction around the pile in sand; therefore, the effect of lateral boundaries of container is minor and could be ignored.

7 SOIL

In this work, poorly grained fine to medium dry sand taken from one of the sites middle of Baghdad city at a depth of 10 to 15 m was used to study the responses of piles subjected to dynamic actions. The soil properties are given in Table 1.

### Table 1. Physical properties of sandy soil used for testing.

<table>
<thead>
<tr>
<th>Standard of the test</th>
<th>Value</th>
<th>Property</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective size, $D_{10}$ (mm)</td>
<td>0.14</td>
<td>ASTM D 422 and ASTM D 2487 (2007)</td>
</tr>
<tr>
<td>Mean size, $D_{50}$ (mm)</td>
<td>0.22</td>
<td>ASTM D 422 and ASTM D 2487 (2007)</td>
</tr>
<tr>
<td>Coefficient of uniformity, $C_u$</td>
<td>1.70</td>
<td>ASTM D 422 and ASTM D 2487 (2007)</td>
</tr>
<tr>
<td>Coefficient of curvature, $C_c$</td>
<td>0.96</td>
<td>ASTM D 422 and ASTM D 2487 (2007)</td>
</tr>
<tr>
<td>Classification (USCS)*</td>
<td>SP</td>
<td>ASTM D 422 and ASTM D 2487 (2007)</td>
</tr>
<tr>
<td>Specific gravity, $G_s$</td>
<td>2.69</td>
<td>ASTM D 854 (2006)</td>
</tr>
<tr>
<td>Dry unit weights</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum, $\gamma_{d(max)}$ kN/m$^3$</td>
<td>15.2</td>
<td>ASTM D 4253 - (2000)</td>
</tr>
<tr>
<td>Minimum, $\gamma_{d(min)}$ kN/m$^3$</td>
<td>13.2</td>
<td>ASTM D 4254 - (2000)</td>
</tr>
<tr>
<td>Maximum void ratio, $\varepsilon_{max}$</td>
<td>0.99</td>
<td></td>
</tr>
<tr>
<td>Minimum void ratio, $\varepsilon_{min}$</td>
<td>0.74</td>
<td></td>
</tr>
<tr>
<td>Initial dry unit weight, $\gamma_{d(test)}$</td>
<td>13.74,14.13</td>
<td></td>
</tr>
</tbody>
</table>

8 PILES

The pile’s model used in the present study is made of smooth aluminum tube having outer diameter of 18 mm and inner diameter of 15 mm covered with plastic sleeve to protect strain gauges. Pile-embedment ratio (depth-to diameter) ($L/d$) used in testing single and group piles was (30). four different arrangements of the pile groups (2×1,1×2, triangle and 2×2) with different spacing ratios ($S/d$) (3, 4 and 5) are used for testing a group of piles. The mechanical properties of pile used are shown in Table 2.

### Table 2. Mechanical properties of aluminum pile used.

<table>
<thead>
<tr>
<th>Embedded length (mm)</th>
<th>Outer diameter (mm)</th>
<th>Wall thickness (mm)</th>
<th>Bending stiffness, $E_p l_p$ (kNmm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>540</td>
<td>18</td>
<td>1.5</td>
<td>0.18 × 10$^6$</td>
</tr>
</tbody>
</table>

The piles were instrumented with 8 pairs of strain gauges on each pile attached along the shaft to measure bending
strain by pasting (8) electrical-resistance-half bridge type strain gauges at distances of (0 L, 1/4 L, 1/2 L, and 3/4 L) from the top of the pile for single and group of piles tested in dry soil.

Each strain gauge has a length of 5 mm, gauge factor of 2.10, and resistance of (350 Ω), and was fixed with its axis corresponding to the pile axis. The wires of strain gauges were passed on a longitudinal pile covered with waterproof past to avoid damage.

9 LOAD CELLS

Mini load cell with a diameter of 20 mm and a capacity of 100 kg was mounted at the tip of the pile, in a way to prevent the load cell from splitting from the pile and to predict the end-bearing load as shown in Figure 6.

10 PILE CAP AND PAYLOAD

Two steel plates with dimensions of (100 × 100 × 10) mm³ and (150 × 150 × 10) mm³ were used to simulate the pile caps, Figure 7. The purpose of using steel plate rather than aluminum plate is to ensure the rigidity of the pile cap with respect to the piles. Also another steel plates were used as payloads on the pile foundation.

A set of measurements have been performed after applying different loads which were induced by static payload which represent the vertical allowable load carried by pile.

11 MODELS PREPARATION

Preliminary experiments were conducted to determine the uniformity of the process and how sand density changes with raining from different heights. Results from raining the sand from different elevations over known volume molds placed on the model floor demonstrated that uniform sand density could be achieved across the width of the model. By adjusting the pluvation height, different sand densities were obtained. The trial results suggested that the distance between the raining box and top of the sand should be 550 mm and 850 mm in order to produce uniform loose sand of relative density 30-50 %.

In order to attain the selected relative densities of (30 %) and (50 %), the heights of the free fall were found to be 55 mm and 85 mm, respectively. While the sand tank parts were placed together, sand layers were prepared so that the sand layers were not disturbed and consequently any change to the required density of the sand did not happen. After filling the raining box (tank) with sand and choosing the proper height of drop, the sand was poured into the test tank by moving the box.
forth and rear. The soil layer was prepared in 10 layers with 100 mm constant height for each one to attain the last elevation of 1000 mm from the bottom of container.

To minimize the influence of buried instruments on the soil deformation, miniature earth pressure cells (EPCs) and miniature accelerometers were used. The active EPCs diameter to grain size ratio ($D/d_{50}$) is 12, which is twice the recommended minimum to ensure a continuum response between the soil and the EPC active face [24]. Details of the instrumentation used in this work are shown in Figure 8. The EPCs were used to measure the total pressure on the soil at the target locations and for estimating the total stresses. Instrumentation records were acquired and recorded by multi data acquisition systems model.

12 RESULTS AND DISCUSSION

12.1 Variation of the vertical displacement of foundation with time

Figures 9 to 11 illustrate the variation of vertical displacement and settlement (permanent displacement when shaking is stopped) with time of the single pile and group of piles with $s/d = 3$ ratio, different number of piles and different frequencies and soil states.

Generally, it can be observed that the vertical displacement increased with frequency for all cases regardless the soil type and pile number and spacing. The rate of settlement increase in loose sandy soil is greater than that in dense sandy soil. From the results, it can be seen that the vertical displacements slightly increased with time with short shaking period then stayed constant to the end of the test, then the settlements increased significantly. After that, the values are increasing gradually and slightly with time in all cases.

For all cases and at different frequencies, the records of the first period (around 50-100 sec.) show sharp increase in the vertical displacement with time. While, gradual increase (or slight increase to steady) is observed in the rest times. The reason for these cases may be due to that the soil gradually densified during shaking which provided low settlement and more response.

It is also observed that some records particularly under 2 Hz frequency using pile group (triangle and 2×2) show sharp increase with time (more than 25 mm). In comparison of single pile with pile group, the vertical displacement values are lower in group with respect to single pile foundation and decrease with the increase of pile spacing in groups. Also, the values decreased with increasing the soil relative density.
Figure 9. Variation of vertical displacement of foundation with time of (single) pile in loose dry sand under different frequencies.
During the shearing, a granular material will typically have a net gain or loss of volume. If it had been originally in a dense state, then it typically, gains volume, a characteristic known as a Reynolds dilatancy. If it had originally been in a very loose state, then compaction may occur before the shearing begins or in conjunction with the shearing. It was observed that the increasing of shaking frequency leads to reduction in the oscillation of wave propagation values recorded due to densification of soil during shaking.

In general, it was noticed that the sandy soil rebound represents small part of settlement as the device shut down. Therefore, it is important to mention that the plotted values of total settlement represent the settlement taken simultaneously as the shaking stops. Thus, the residual or rebound displacements represent
the settlement of the foundation. As well as, the rate of settlement increase in loose sandy soil is greater than that of dense sandy soil.

It is worth mentioning that the (+) sign means that the displacement into the left and upward for the horizontal and vertical moving, respectively and vice versa, also the net displacement means the displacement for dynamic shaking to the end of test (without initial static case).

12.2 Variation of the end-bearing load of the piles with time

The net (due to the dynamic effect only) bearing loads of piles were measured and recorded using miniature load cells with diameter 20 mm placed at tip of piles. The variations of the end bearing load in single pile and some groups of piles with time for \( s/d = 3 \) ratio and different number of piles and different
frequencies and states of soil are illustrated in Figures 12 to 17. After careful examination of these figures, it can be seen that at the end of shaking, some piles maintain their values while others reset to zero load. The end bearing load values increase with frequency for both loose and medium soil states. Also, the end bearing load values increase with increasing number of piles for both states of soil.

In general, the end bearing load values increase when $s/d$ ratio increases from 3 to 4 while they decrease with $s/d = 5$. For triangle group, an increase in end bearing load is observed with increasing the pile spacing. When the spacing ratio is below $s/d = 5$, the pile group behaves as one mass, therefore, the inertia load becomes high which reflects the high percent of load transferred to the pile ends.
Figure 12. Variation of end bearing load of foundation with time of (single) pile in loose dry sand under different frequencies.

Figure 13. Variation of end bearing load of foundation with time of pile group (1×2) in loose dry sand under different frequencies.
Figure 14. Variation of end bearing load of foundation with time of pile group (2×2) in loose dry sand under different frequencies.
Figure 15. Variation of end bearing load of foundation with time of pile group (single) in medium dry sand under different frequencies.
Figure 16. Variation of end bearing load of foundation with time of pile group (1×2) in medium dry sand under different frequencies.

Figure 17. Variation of end bearing load of foundation with time of pile group (2×2) in medium dry sand under different frequencies.
The figures show that the behavior of the net bearing load at the tip of piles seems to be constant for a long period of test. On the other hand, the values of the net bearing load are 0.25 to 3.5 N in single pile for models in loose and medium sand, respectively.

At the start of vibration, there will be a rapid mobilization of skin friction along piles, then after a short period of time, the skin friction and end bearing reach a plateau and no noticeable change in the components was recorded.

The most important result is that, the pile tip load (net end bearing load) during shaking operation increased or decreased depending on many factors: number of piles, pile spacing, operating frequency, as well as, the state of soil.

From all figures, it is noted that, the oscillation of the frequented values decreases with increasing the shaking frequency.

When the frequency of vibration is high, there will be no enough time for sand particles to move over each other, so that the oscillation in the measured displacements and end bearing load decreases.

Fattah et al. [16] found that the final settlement of the foundation increases with increasing the amplitude of dynamic force, operating frequency and degree of saturation. Meanwhile, it is reduced with increasing the relative density of sand, modulus of elasticity, and embedding inside soils.

During operation, the skin friction resistance mobilized along the pile length due to increasing in settlement and densification (increase in pile soil interaction effect of soil) led to increase in the bearing load. This increase becomes clear as the spacing between piles increases.

A small settlement occurs due to low operating frequency, which means a small skin friction resistance along the pile mobilizes, which reduces more due to the interaction effect of piles, and that mobilized resistant is less than the induced dynamic load. As a result, the pile’s tip load increases. When increasing the operating frequency, the settlement increased, which means that increasing the mobilized resistance, and the increasing in pile’s tip load became less. In general, the increasing in pile tip load is not more than the applied dynamic load divided by the number of piles.

During shaking, when the end bearing load reaches a steady condition, the pile group behaves in a manner similar to the system under static loads; the load applied on the pile group is divided between the skin resistance and end bearing and no contribution is observed to the inertia effects.

Ercan [13] concluded that load developed in outer piles is about 1.25 times the load developed in inner piles. On the other hand, lateral deflection increased considerably as pile spacing decreased from 5D to 2D. However, this behavior was seen more clearly in the first two row piles. Pile spacing affects load distribution in pile groups significantly. As pile spacing increases, pile load decreases. As pile spacing increases, maximum bending moment occurred decreases under the same load applied.

12.3 Variation of the peak acceleration of the foundation with time

The variation of peak acceleration of the foundation in the axis of shaking was measured using a vibration meter (vibrometer) in addition to the previous measurements of acceleration of the foundation and soil bed using accelerometer. Figures 18 to 23 display the variation of the acceleration of the single pile and some groups of
Figure 18. Variation of peak acceleration of foundation with time of (single) pile in loose dry sand under different frequencies.
Figure 19. Variation of peak acceleration of foundation with time of pile group (1×2) in loose dry and under different frequencies.
Figure 20. Variation of peak acceleration of foundation with time of pile group (2x2) in loose dry sand under different frequencies.

Figure 21. Variation of peak acceleration of foundation with time of pile group (single) in medium dry sand under different frequencies.
Figure 22. Variation of peak acceleration of foundation with time of pile group (1×2) in medium dry sand under different frequencies.
piles with time for $s/d = 3$ and 4 and different number of piles and operating frequencies for both states of soil.

From these figures, in general it can be seen the increase in acceleration measured by vibration meter with time and frequency for both states of soil and for different number of piles in the group and different spacings. The figures also show the slight decrease in the rate of vibration with increasing $s/d$ ratio of the pile group. Concerning the effect of type of soil, the rate of vibration, in general, or the acceleration decreases when the soil is of higher relative density (medium).

The horizontal acceleration of foundation (pile cap) ranges between (0.65-8) m/sec$^2$ and (0.65-9) m/sec$^2$ for single pile in loose and medium sand, respectively for shaking frequency (0.5-2) Hz. For group of piles, the acceleration ranges from (0.45-7.5) m/sec$^2$ and (0.45-7.5) m/sec$^2$ for loose and medium sand respectively for the same shaking frequency. This means that there is a small attenuation of vibration due confinement offered by pile groups.

The above observation indicates that there is a small attenuation of vibration due confinement offered by pile groups.

12.4 Variation of the peak velocity of the foundation with time

The variation of velocity of the foundation measured using a vibration meter was also detected and recorded. Figures 24 to 29 display the variation of the velocity of some groups of piles with time for $s/d = 3$ and 4, different number of piles, operating frequencies and states of soil. It can be shown from these figures that the peak velocity measured using the vibration meter increases with time and frequency for both soil and different number of piles and pile spacing. The figures also show a decrease in the rate of vibration with increasing $s/d$ ratio and relative density.

13 CONCLUSIONS

In the light of experimental tests on model piles in sand and analysis of the results and other observations during the experimental approach, the following major conclusions drawn from the test are summarized as follows:

1. For a soil bed in dry state, the acceleration amplitudes increase with frequency for both soil relative
Figure 24. Variation of peak velocity of foundation with time of pile group (1x2) in loose dry sand under different frequencies.
Figure 25. Variation of peak velocity of foundation with time of pile group (1×2) in loose dry sand under different frequencies.
Figure 26. Variation of peak velocity of foundation with time of pile group (2×2) in loose dry sand under different frequencies.

Figure 27. Variation of peak velocity of foundation with time of (single) pile in medium dry sand under different frequencies.
Figure 28. Variation of peak velocity of foundation with time of pile group (1×2) in medium dry sand under different frequencies.

Figure 29. Variation of peak velocity of foundation with time of pile group (2×2) in medium dry sand under different frequencies.
densities (loose and medium) and different pile patterns (number; single or group and different spacing ratios \(s/d\)). The maximum acceleration in the foundation is lower than in the soil bed for all operating shaking frequencies, pile spacing ratios and soil states. The decreasing of the maximum acceleration recorded in the foundation as compared to that in the soil bed is between 10-100 % for loose and medium state of soil, and the decrease in loose state is more than in medium state. This means that there is damping effect or attenuation of vibration waves. The amplitudes of recorded acceleration in the pile cap are much higher than in the soil bed for single pile and pile group with different pile spacing ratios, also these amplitudes are increasing with increase of shaking frequency and relative density of the soil.

2. The increase in shaking frequency leads to reduce the oscillation of wave propagation values recorded due to densification of soil during shaking.

3. The pile tip load (net end bearing load) during shaking operation increases or decreases depending on number of piles, pile spacing, operating frequency and the soil state. During operation, the skin friction resistance mobilized along the pile length due to increasing in settlement and densification (increase in pile soil interaction effect of soil) led to increase the bearing load.

4. There is increase in acceleration and peak velocity of the foundation measured by vibration meter with time and frequency for both states of soil and for different pile numbers and spacings.

5. The most important result is that, the pile tip load (net end bearing load) during shaking operation increased or decreased depending on many factors: number of piles, pile spacing, operating frequency, as well as, the state of soil. The oscillation of the frequented values decreases with increasing the shaking frequency.

6. The pile group deflects significantly more than the isolated single pile when loaded to similar average load per pile. Moreover, the row position had an effect on the efficiency of the individual piles. The front row (leading row) piles exhibited stiffer responses than the trailing rows (second and third row). The pile spacing is an important indicator that affects the acceleration and time frequency characteristics of the displacement at pile top. With the increasing of \(S/D\), the internal forces are slightly reduced.

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