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DYNAMIC BEHAVIOR OF PILE GROUP MODEL IN TWO – LAYER SANDY SOIL TO LATERAL EARTHQUAKE EXCITATION

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ABSTRACT
The behavior of pile foundations under earthquake loading is an important factor affecting the performance of such structures. Observations from past earthquakes have shown that piles in stiff soils generally perform well, while those installed in soft or liquefiable soils are more subjected to problems arising from ground amplification or excessive soil movements.

A series of laboratory tests were conducted to measure the response of pile foundation when subjected to dynamic loads. A special vibration box was manufactured. The accompanied measurements include vertical and horizontal displacement and settlement of pile cap, acceleration in three dimensions in both soil and cap and earth pressures. The model pile used has an outer diameter of 18 mm and inner diameter of 15 mm. A group of (2x2) piles was tested.

It was concluded that for soil bed in dry state, the acceleration amplitudes increase with frequency for both soil relative densities (loose and medium). The maximum acceleration in the foundation is lower than in soil bed for all operating shaking frequencies 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.

INTRODUCTION
The design and analysis of foundation system include the relationship of a broad variety of related disciplines and sciences such as seismology, geology, soil and rock dynamics, and applied mechanics. Therefore, the foundation engineers need to pay attention for the advances and technical developments in these fields, or to be well advised in these fields in order to achieve cost-efficient and safe designs. For shallow and deep foundations, there is still much to do in the development of procedures to evaluate seismic bearing capacity of, and earthquake-induced permanent displacements in such structures. Thus, to determine foundation seismic loading for practical applications, simple and uncomplicated procedures are required, which can derived or developed from the accessible numerical modeling to reproduce perfectly the seismic response of manufactured prototypes (Romo et al., 2000).

Recent seismic activities associated with destructive earthquakes in many regions around the world pay the attention for the importance of pile foundations and their impact on the response of the supporting structures. In spite of the importance of static loading in pile foundation design, the dynamic loading still represents the greatest challenge in designing pile foundations due to additional forces on pile foundation excreted by dynamic loading (including axial and lateral) (Bentley, 1999).

The nature of the loading and soil-pile responses are quite different for different sources of dynamic loading. The type of dynamic loading in soil or the foundation of a structure depends on the source producing it. Loads vary in their magnitude, direction, or positions with time which may involve different types of variation of forces are called dynamic loads. While, the special type of load that varies in magnitude with time and repeats itself at regular intervals is called periodic load, for example, operation of a reciprocating or a rotary machine (Das and Ramana, 2011).

The total earthquake-induced loads on the pile comprise (Ghosh et al., 2012):

- Inertial loads imposed by the superstructure to the pile head. This is a function of frequency of the superstructure and the input motion and varies as the stiffness of the soil changes. This is normally greatest in the initial part of the shaking, before the onset of liquefaction.
- Kinematic forces acting along the embedded length of the pile due to the movement of the soil. If there is non-liquefied material above liquefied soil, the combination of stiffer, non-liquefied material and large movements due to the underlying layer are particularly onerous.

Brown et al. (1988) carried out a study to determine lateral load behavior of pile group in sand. A full-scale test was conducted on a 3x3 pile group in medium sand underlain by very stiff clay. The pile group was spaced at 3D on centers. Both pile group and a single isolated pile were subjected to two-way cyclic lateral loading. It was concluded that the pile group "was observed to deflect 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). However, no pattern was observed of the pile position within a given row. The "shadowing" effect was more considerable in sand compared to the clay. However, when piles were under two-way cyclic loading, group effects were still significant in sand, unlike the reduced significance of "shadowing" with cyclic loading that was observed in clay.

Brown et al. (2001) studied static and dynamic lateral loading of pile groups using several full-scale field tests conducted on pile groups of six to 12 piles, both bored and driven, in relatively soft cohesive and cohesionless soils. All of the groups were loaded laterally statically to relatively large deflections, and groups of instrumented pipe piles were also loaded dynamically to large deflections, equivalent to deflections that might be suffered in major ship impact and seismic events. Dynamic loading was provided by a series of impulses of increasing magnitude using a horizontally mounted dynamic device. While such loading did not capture the aspects of lateral loading and ground shaking that may generate high pore water pressures, it did capture the damping that occurs at very large pile deflections and the inertial effects of the problem.

Rollins and Cole (2006) conducted another study to investigate group interaction effects with respect to the pile spacing on laterally loaded pile groups. Full-scale cyclic lateral load tests were performed on 3x5, 3x4 and 3x3 pile groups in stiff clay with 3.3D, 4.4D and 5.65D pile spacing, respectively. The soil profile generally consisted of stiff clay layers with sand layers that were in a medium compact density state (Dr=60%), to a depth of 5 m. These soils were underlain by sensitive clay, silty clay and sand layers. Similar to the other studies, lateral load tests were performed on single piles in order to provide comparison to the pile group test results. For the tests, closed-end steel pipe piles were chosen. Rollins and Cole (2006) concluded that, lateral load resistance was a function of pile spacing. While decreasing the pile spacing, group interaction effects became progressively more important. Furthermore, the leading row (1st row) piles in the group carried the greatest load, while the trailing row piles (second, third, fourth and fifth row piles), carried smaller loads for the same displacement level. For these pile groups driven in clay, row location within the group had more significant effect on the lateral resistance than the location within a row.

Boominathan et al. (2015) carried out a study by full-scale lateral dynamic pile load testing to determine the dynamic characteristics of soil-pile system. This aspect is considered as vital issue in the design of pile foundations under dynamic/seismic loads. 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 prgram ABAQUS. Neither the potential for liquefaction nor the dilatational effect of clays and the compaction of loose sands in the vicinity of piles was accounted for, in the analysis.The non-destructive technique known 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 for the finite element analysis. A steady state sinusoidal force was generated with a 5-tonne capacity mechanical oscillator. The forced vibration response of the piles were 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.

The computed values of the amplitude of displacement corresponding to the pile cut-off level at each frequency for different eccentricities of the oscillator were plotted as frequency response curves. It was concluded that the resonant frequency of soil-pile system at Site-I reduces from 14 Hz to 10.5 Hz as the magnitude of the dynamic force increases, indicating a non-linear response of the soil-pile system due to the degradation of soil stiffness.

The present study is focused on dynamic response of piles to lateral shaking in order to predict the lateral dynamic responses of foundations in sandy soil under earthquake loading (during shakings). Thus, such soil will be tested on shaking box to produce the seismic ground shaking under different conditions (frequencies, amplitudes and relative densities. The detailed objectives of the present study are as determination of the frequency independent dynamic response of group of piles to lateral vibration and calculation of acceleration - time history in addition to displacement - time history of pile groups subjected to earthquake excitation.

TESTING APPARATUS AND METHODOLOGY

The laboratory test model was utilized to detect the responses of group of piles that were used to support foundation (inducing shaking) inserted in sandy soil. The embedment depth ratio; length to diameter (L/d) of the piles was 30. The tests were conducted in sandy soil with loose and medium relative densities (30 and 50%). The accompanied measurements include vertical and horizontal displacement and settlement of pile cap, acceleration in three dimensions in both soil and cap and earth pressures.

BOUNDARY EFFECTS OF THE SAND TANK

The stress and displacement patterns in the sand can be affected by the side boundaries of the soil container. In addition, due to the friction between the container walls and soil grains, the vertical stress in the sand can be reduced with depth (Kraft, 1991). To avoid side friction of walls, the ratio between the container height and its diameter must be equal to or less than one (Garnier, 2001 and 2002). Due to loading or installation of the pile, the soil around the pile will be affected (disturbed), and the zone of this disturbance varies with pile installation method and soil density; however, the previous studies reveal that the zone of disturbance is in the range of 3 to 8 of pile diameters (Robinsky and Morrison, 1964).

Based on the argument of previous studies, the use of 400 mm and 350 mm between the pile and the tank sides and underneath the pile tip for pile diameter of 18 mm is supposed to be adequate for the loose to medium sand samples used in this study, as shown in Plate 1.

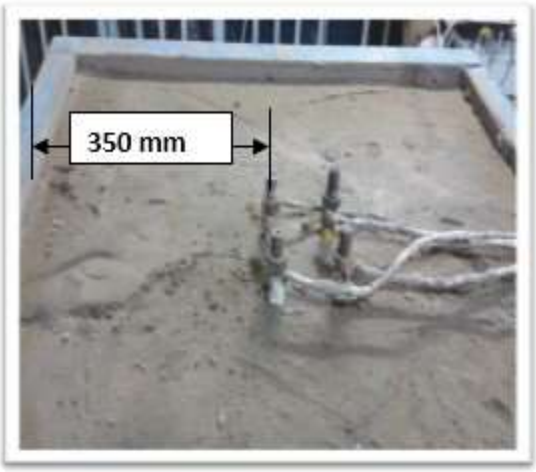

***Plate 1: Clear distance between the pile and tank side*.**

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SET-UP OF MANUFACTURED SHAKING BOX

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 Plate 2.

In the second part (slide II), a metal holder which is 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.

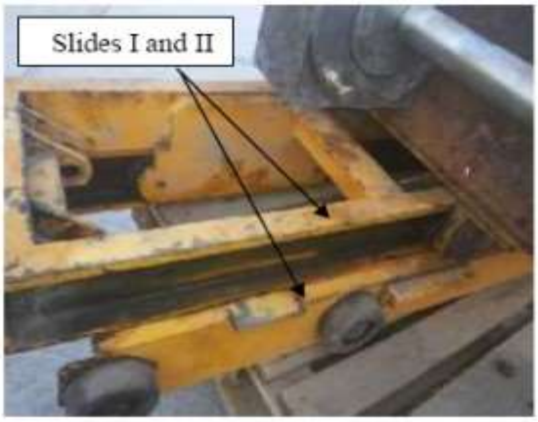
A steel piece (plate) of L-shape (with dimensions of 900 mm wide, 1000 mm long and 300 mm high) is mounted while strengthening its edge and base by three triangular stiffeners to avoid any rush or slippage of interior or exterior parts as shown in Plate 2b. Another two ball bearings with internal diameters 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 Plate 2c. 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 Plate 2d.

a

b

c

d



e *f*

Plate 2: Slides and base bracket system during manufacturing of shake box.

Two decentralized Cama (with diameter of 95 mm) have been manufactured with downward drift distance of 25 mm or 30 mm from the center as shown in Plate 3c. 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 Plate 3.

This decentralized Cama rotates inside the bearing ball (needle bearing) which is linked by a 400 mm long connecting arm (or connecting rod) tothe eccentricity installed by a pin as shown in Pate 3.

a ***b***

c ***d***

Plate 3: CAMA and pin connected during manufacturing of shaking box.

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 Plate 4. A side slot of 400 mm wide and 700 mm height of the steel box has been made to facilitate the process of discharging sand or soil 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 Plate 4.

Plate 4: Steel box.

With increasing the rotational velocity, which is converted to be a linear speed, the motion is accompanied with the appearance of a direction change problems 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:

$$I = MV^2 \quad (1)$$

where: M = moving mass;
V = velocity.

It is noted in the above equation that a little change of soil mass type to be other tests depending on the value of square but speed is of greatest value is in effect increasing the value of (I), so when the velocity is increased, 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.

Raining technique

To obtain a homogeneous fill of sand with specific relative densities inside the steel box, sand raining technique device had been manufactured with dimensions (700 × 700 × 200) mm. 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 as shown in Plate 5. This box is fitted from its four corners by hooks and steel chain through a mechanism allowing it’s 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.

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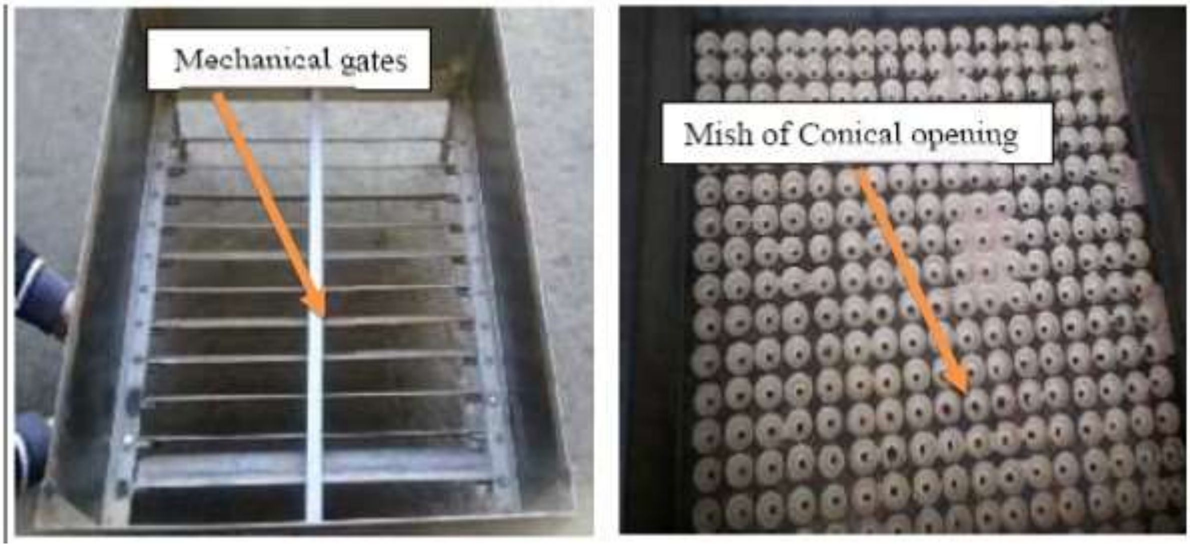

Plate 5: Sand raining box and meshes.

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 long, 700 mm wide and 200 mm high. Different meshes were used to achieve the required relative density, one as timber plate (20 mm thick) which is perforated as a conical shape with pores as patterns 5 mm c/c in each direction, the other shape as diffuser made of steel angle at space 5 mm groove hole which is ended with mechanical gate to pour the testing material from different heights.

DATA ACQUISITION SYSTEMS

The system of data acquisition was utilized so that all data could be scanned and recorded automatically, this system consists the following:
1. LVDT data system; 2. Accelerometer data system; 3. Vibration data system; 4. Earth pressure data system.

Model piles

The model pile used has an outer diameter of 18 mm and inner diameter of 15 mm. The size of pile's model was chosen after reviewing the literature about the suitable pile size that could be considered representative. The dimensions of the pile model that is 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. Pile-embedment ratio (depth-to diameter) (L/d) used in testing single and group piles was 30. Two steel plates with dimensions of (100 × 100 × 10) mm and (150 × 150 × 10) mm were used to simulate the pile caps Plate 6. 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 the another steel plates were used as a payloads on pile foundation.

Plate 6: Steel plate used as pile cap and payloads.

Materials Used

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

Models Preparation

Majority of laboratory tests on granular soils, such as clean sand and gravels, are performed on reconstituted specimens because obtaining samples of these materials in their undisturbed state or natural state is very difficult due to lack of 'Cohesion' (Pathak and Dalvi, 2011). Various sample preparation methods have been developed depending on moisture condition of the soil (dry and wet), the method of soil placement (pluviation, spooning or flowing) and medium through which the soil is placed (air or water).

The raining technique was used to pour the sand in the test tank. In order to achieve a uniform layer with a desired density, a special raining device was designed. The height of drop and the rate of discharge of the sand mainly affect the density of the sand layer in the raining method (Turner and Kulhawy, 1987).

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

Property	**Value**	**Standard of the test**
Grain size analysis		
Effective size, D_{10} (mm)	0.14	ASTM D 422 and ASTM D 2487 (2007)
Mean size, D_{50} (mm)	0.22	ASTM D 422 and ASTM D 2487 (2007)
Coefficient of uniformity, Cu	1.70	ASTM D 422 and ASTM D 2487 (2007)
Coefficient ofcurvature, Cc	0.96	ASTM D 422 and ASTM D 2487 (2007)
Classification (USCS)*	SP	ASTM D 422 and ASTM D 2487 (2007)
Specific gravity, Gs	2.69	ASTM D 854 (2006)
Dry unit weights		

Maximum, γ_d(max.)kN/m^3	15.2	ASTM D 4253 (2000)
Minimum, γ_d(min.) kN/m^3	13.2	ASTM D 4254 (2000)
Maximum void ratio, e_{max}	0.99	-------------------
Minimum void ratio, e_{min}	0.74	------------------
Initial dry unit weight, γ_d (test)	13.74, 14.13	-------------------

*USCS: Unified Soil Classification System.

To prepare the sand specimens, the container was fixed in its vertical position and sand was dry-rained (air-pluvation) into the container from specific heights to produce uniform beds of sand. The raining apparatus (perforated plate and diffuser) were suspended from an overhead crane girder that allowed continuous adjustment of its height to maintain a constant gap between the soil surface and the outlet. Preliminarily 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 rainer and top of the sand should be 55 mm and 85 mm in order to produce uniform loose sand of relative density 30-50%.

Pile installation

By using one embedment ratio or length to diameter, L/d ratio, the sand bed was made constant in each test of about 500 mm thick. A steel plate of straight edge was used to level the sand bed; a steel 25 mm square hollow section was located on the top of the test tank in middle distance and fastened at the position of the model piles, Plate 7.

a *b*


c *d*

Plate 7: Pile installation and leveling.

Thereafter, the model piles were located in their positions and were held vertically using tropical device. When the pile is located at its position vertically, the pile cap was removed. When the height of the sand in the test tank reached a specified depth, then the sand raining was stopped, the rod section was lifted up, and the sand surface was leveled. All sensors were put in their locations while the tank is filled with sand. After filling the tank with sand and leveling the surface, the zero loading reading of the load cell connected to the pile tip was recorded. Then, pile cap was connected to the single pile or group of piles, and static reading was recorded again, as shown in Plate 8.


Plate 8: Images of LVDTs, vibration meter, and accelerometer setup.

PRESENTATION AND DISCUSSION OF TEST RESULTS

Two accelerometers (measuring 3-components; in X, Y and Z directions) were used to detect acceleration, one of them was inserted in the soil bed nearby the pile tip, while the other one was attached to the foundation surface to detect acceleration in the soil and foundation. The inspections were achieved for all cases of tests. Figures 1 and 2 present the responses of (2x2) group of piles with s/d=3 ratio, for different number of piles, operating frequencies and states of soil.

By examining acceleration time history in the soil bed in all these figures together, it can be seen that the

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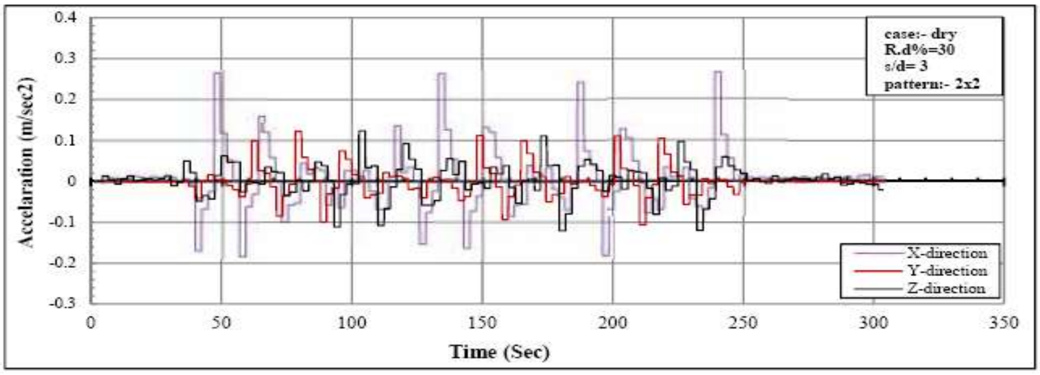
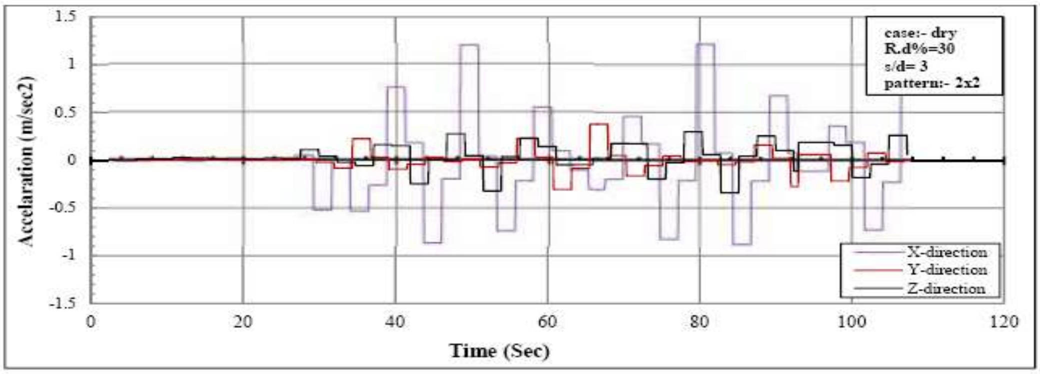
acceleration amplitudes increase with frequency for both soil relative densities (loose and medium).

It is worth mentioning that the acceleration in the soil bed in the vertical direction is positive for the direction of the gravity. These results can be discussed based on the findings of Qu and Shi (2008) who concluded that for pile group, while the number of piles and its layout scheme are settled, S/D is an important indicator whose change has certain impact on the acceleration and time-frequency characteristics of the displacement at pile top.

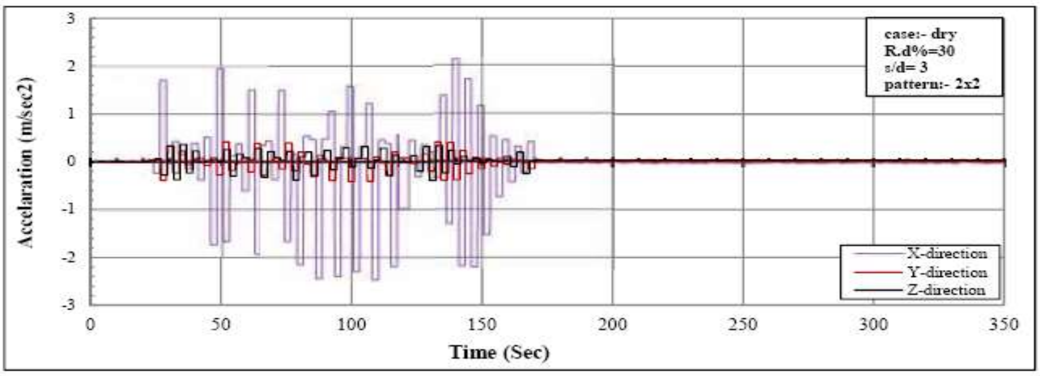
It is noticed that the amplitude of acceleration increases by about (25 – 100)% with increase of sand density depending on the frequency of loading.



a: *f* = 0.5 Hz

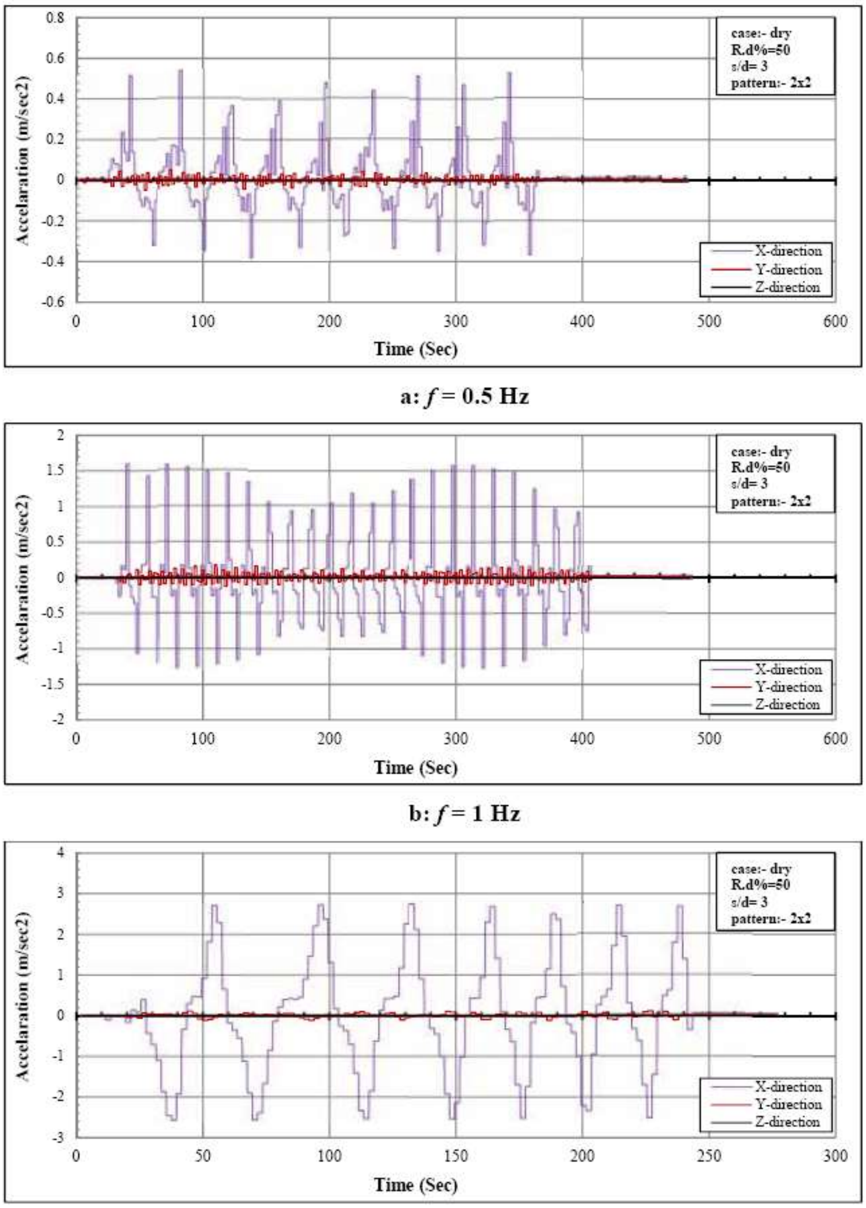


b: *f* = 1 Hz



c: *f* = 2 Hz

Figure 1: Acceleration time history in soil bed of group pile (2x2), s/d= 3 in loose dry sand under different frequencies.



a: *f* = 0.5 Hz

b: *f* = 1 Hz

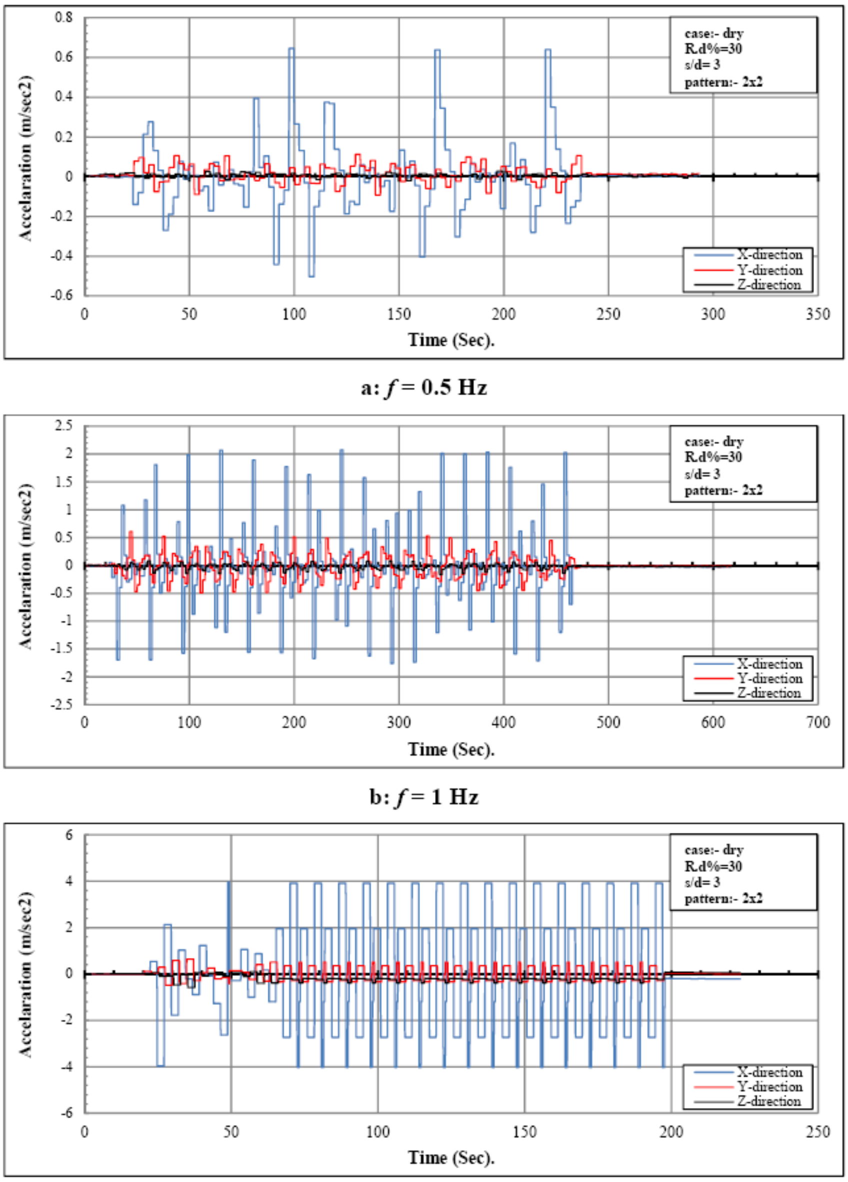
c: *f* = 2 Hz

Figure 2: Acceleration time history in soil bed of pile group (2×2), in medium dry sand under different frequencies (a.0.5 Hz, b.1 Hz, c.2 Hz).

In addition to the acceleration measured in the soil bed, the time history of acceleration in the foundation was also recorded when shaking starts by putting the accelerometer on the pile cap. Figures 3 and 4 display the responses of (2x2) group of piles with s/d ratio =3, for different operating frequencies and states of soil. As in case of the soil bed, the same acceleration behavior is noticed for the foundation. It is also seen that the acceleration increases with increasing the shaking frequency with different soil states.

In order to compare the maximum acceleration recorded in the foundation for loose compared with medium state, it can be seen that the rate of decreasing of acceleration is between (3-18) % approximately for (2x2) pile group.

The stresses inside the soil medium before and after shaking were measured too. The earth pressures were recorded using vertical earth pressure cells placed at three levels; pile tip, pile mid and pile mid at the corner. Figures 5 and 6 present the variation of the net total soil pressure with time from the shaking start to the end of test group of piles with s/d=3 ratio, different operating frequencies measured for different states of soil (loose and medium states). It is important to mention that the net total soil pressure presents the excess pressure (pressure minus overburden pressure). By examining the figures, it can be stated that the soil pressure shows slight change with increasing frequency. In general, the values of soil pressure decrease with increasing frequency and relative density. The maximum values of soil pressure were recorded at pile tip and pile mid then pile mid at corner, since the wave is coming from the model base. The confinement provided by the piles in the group inhabits the transfer of stress waves between piles.

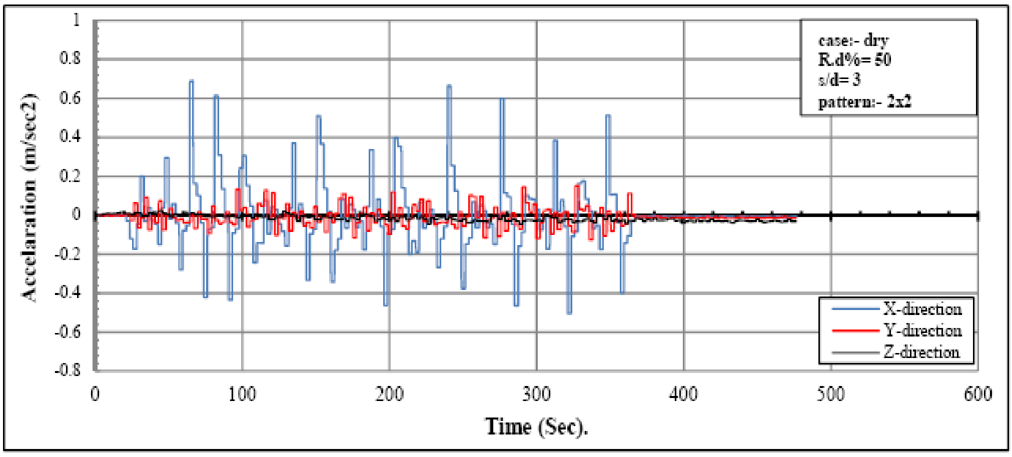


a: *f* = 0.5 Hz

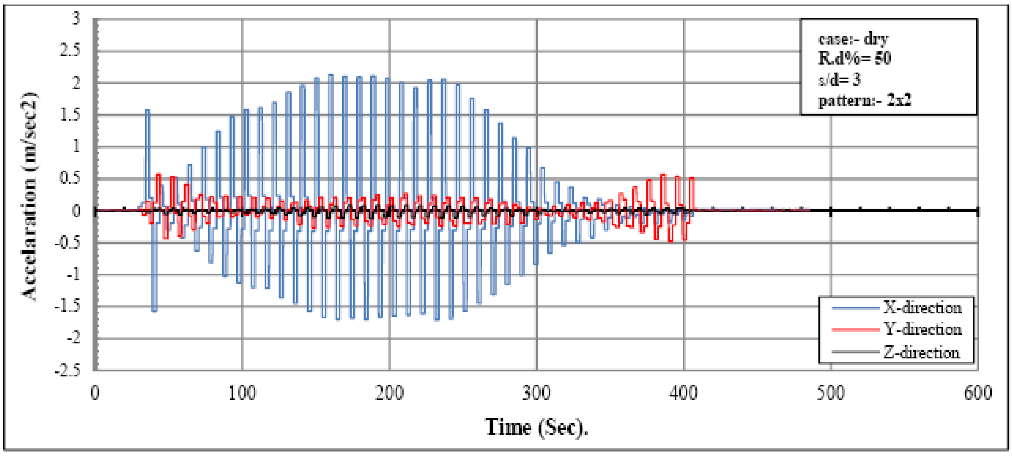
b: *f* = 1 Hz

c: *f* = 2 Hz

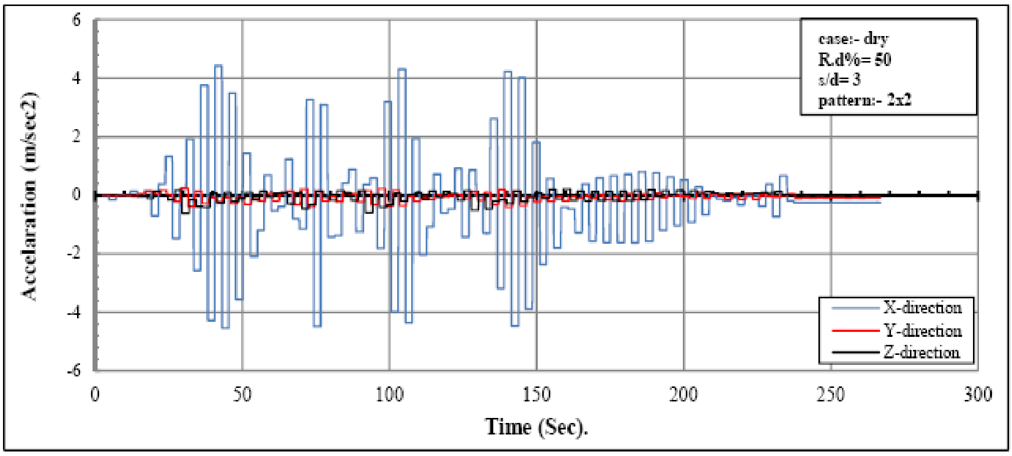
Figure 3: Acceleration time history in foundation of group pile (2×2), s/d= 3 in loose dry sand under different frequencies.



a: *f* = 0.5 Hz

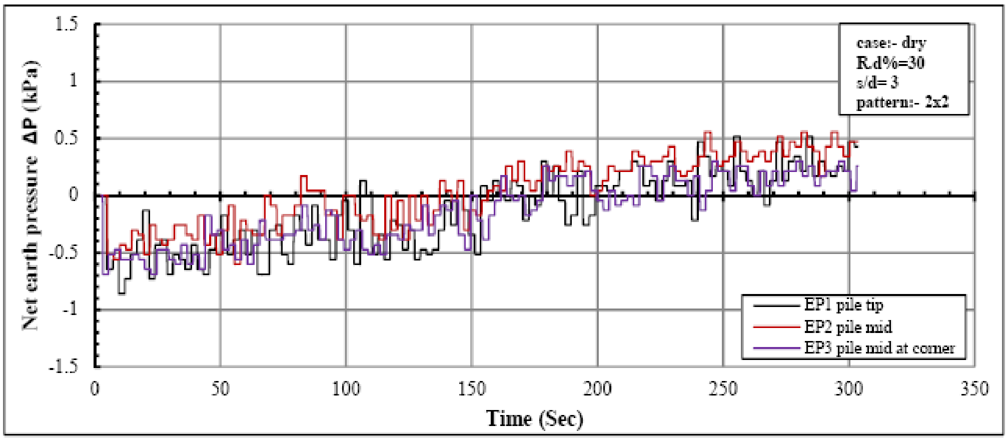


b: *f* = 1 Hz

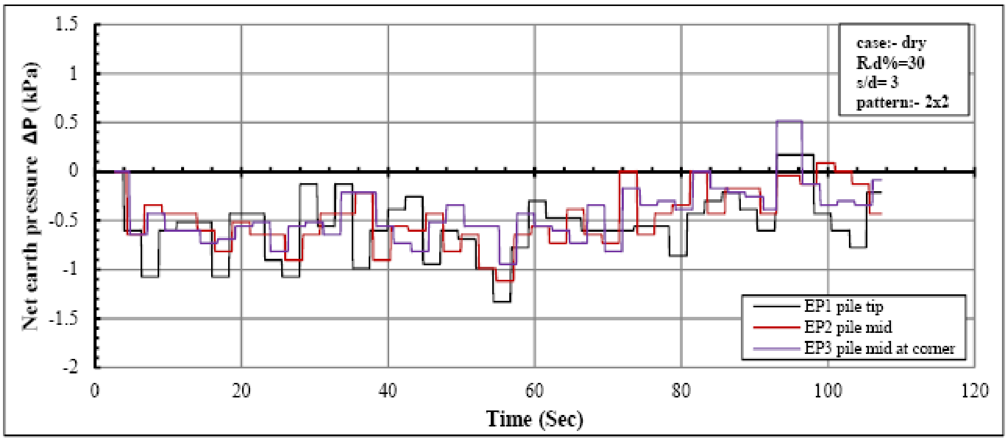


c: *f* = 2 Hz

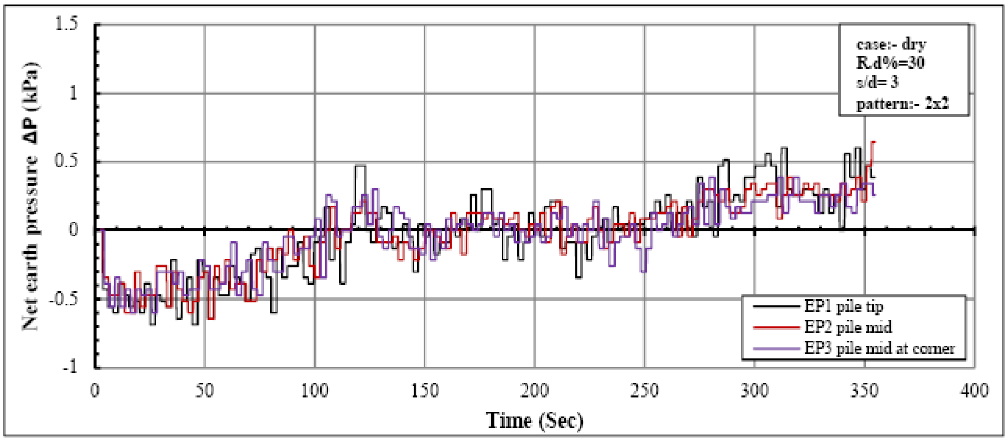
Figure 4: Acceleration time history in foundation of group pile (2×2), s/d= 3 in medium dry sand under different frequencies (a.0.5 Hz, b.1 Hz, c. 2 Hz).



a: f = 0.5 Hz



b: f = 1 Hz



c: f = 2 Hz

Figure 5: Soil pressure (ΔP) time history of pile group (2×2) in loose dry sand under different frequencies.

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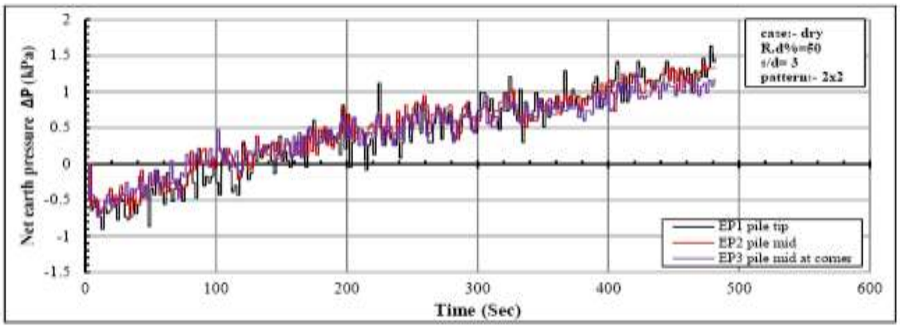
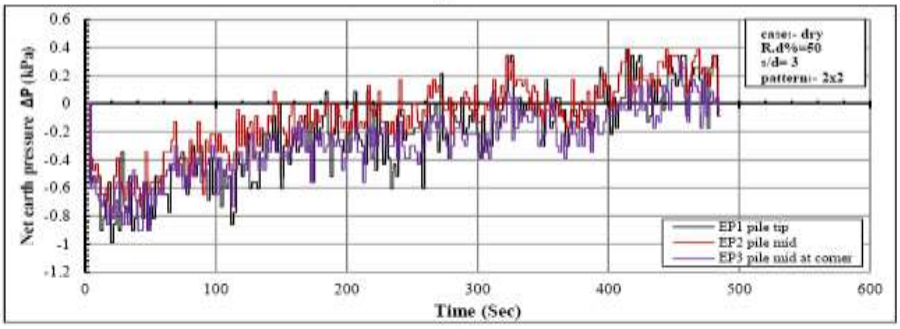
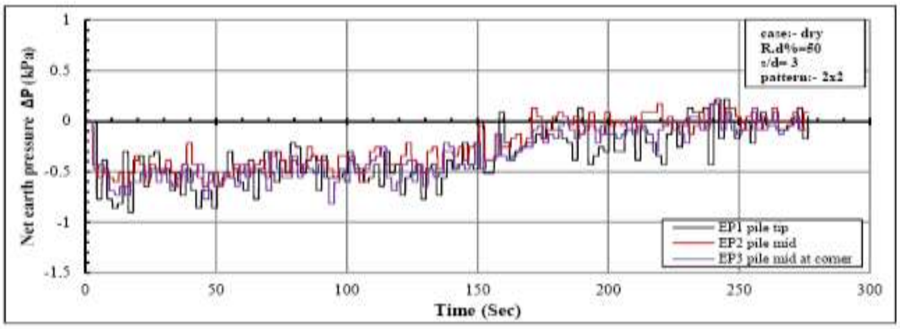


a: f = 0.5 Hz



b: f = 1 Hz



c: f = 2 Hz

Figure 6: Soil pressure (ΔP) time history of pile group (2×2) in medium dry sand under different frequencies.

PILE GROUPS IN TWO-LAYER SOIL SYSTEM

Several studies were conducted to compare the responses of both soil and foundation with different cases. This case consists of illustrating double layers of soil for 2×2 pile group pattern in which the lower layer is dense sand. The piles were embedded in dense sand for a distance of 100 mm.

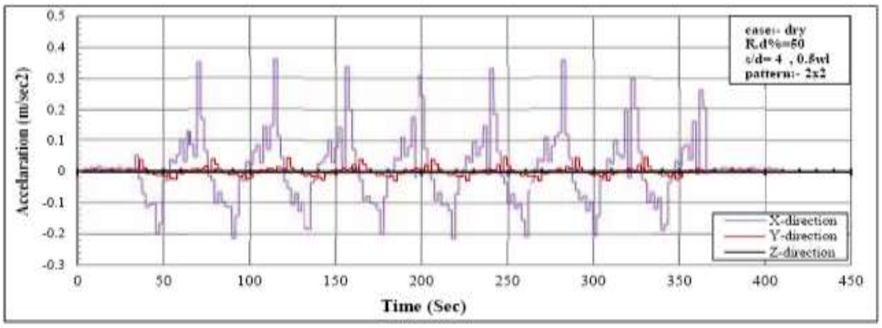
Regarding the acceleration curves, their values increased with increasing frequency for both soil bed and foundation regardless the soil type. No significant difference is observed in acceleration values between single and double layer of soil. The variations of acceleration are presented in Figures 7 to 10. The embedment of piles partly in dense sand layer has little effect on the measured acceleration.

It can be stated that the net earth pressure values at the middle of pile are significantly greater than those in position at the pile tip or at the corner of the steel box away from piles which are clearly shown in Figures 11 and 12.

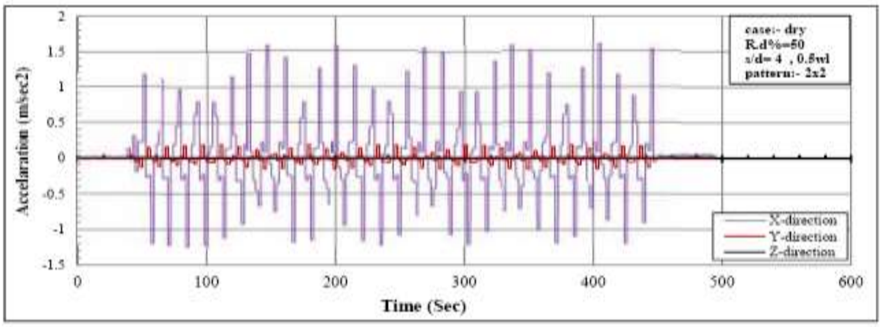
The static axial load applied on the seismically loaded pile has a considerable effect on the transmitted pressures at different levels of the pile length. The net earth pressure decreases by about 50% when the static applied axial load is halved.

It is shown that pile-to-pile interaction effects are significant mainly in the inertial loading creating a strong dependence on frequency of the group efficiency. In seismic loading, the interaction effects between piles in homogeneous structures are very small and can be neglected. Makris and Gazetas (1992) concluded that, for a homogeneous stratum, pile-to-pile interaction effects are far more significant under head loading than under seismic excitation.

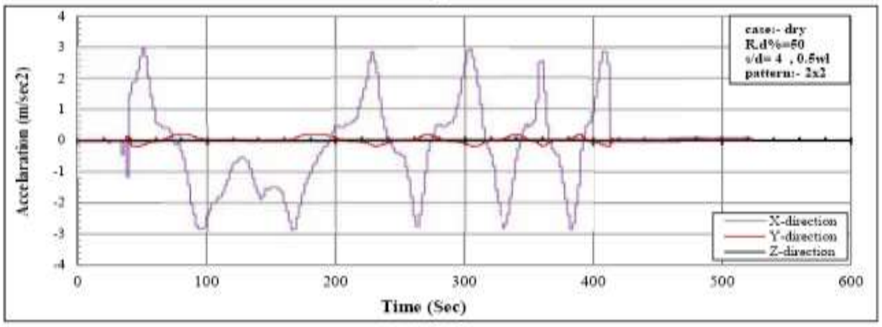
Under low frequency excitation, pile response is dominated by non-linear soil behavior. However, as the frequency increases, the component of soil reaction due to radiation damping increases and the non-linear characteristics of the responses become less dominant.



a: f = 0.5 Hz

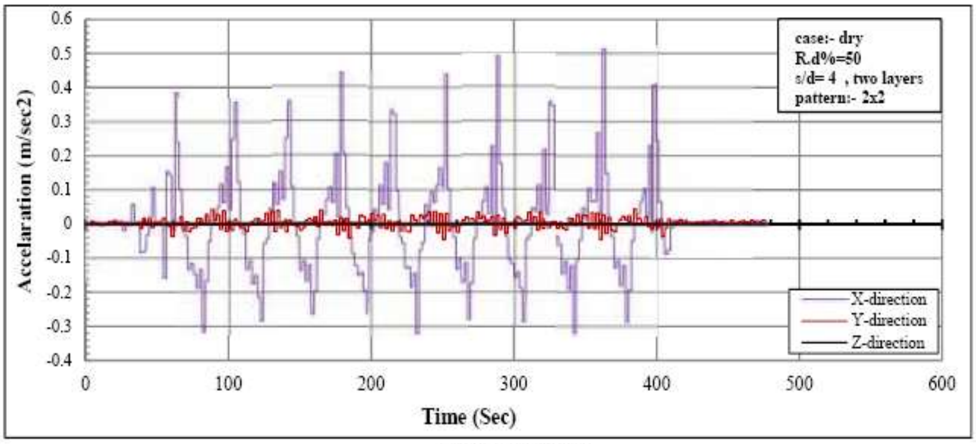


b: f = 1 Hz

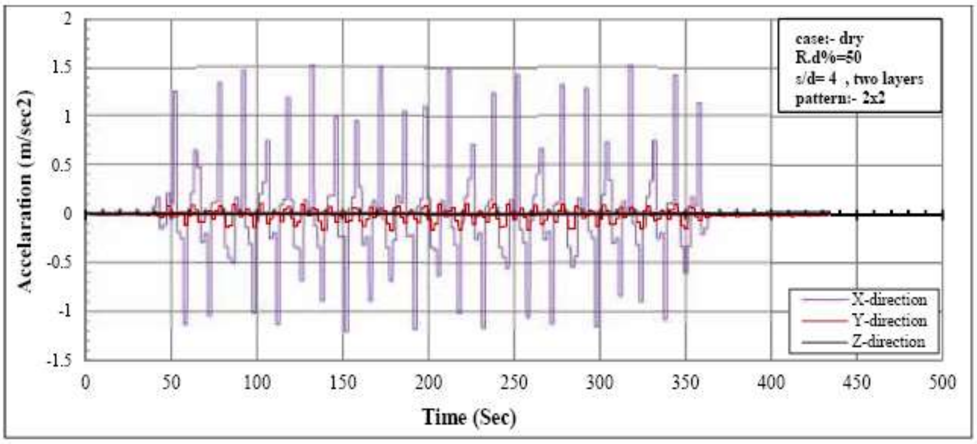


c: f = 2 Hz

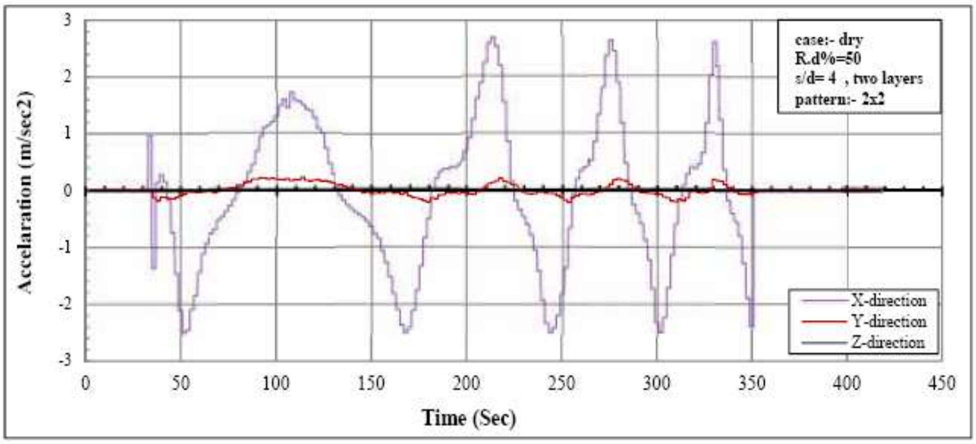
Figure 7: Acceleration time history in soil bed of pile group (2×2), with half-axial load for s/d =4 in medium dry sand under different frequencies.



a: *f* = 0.5 Hz

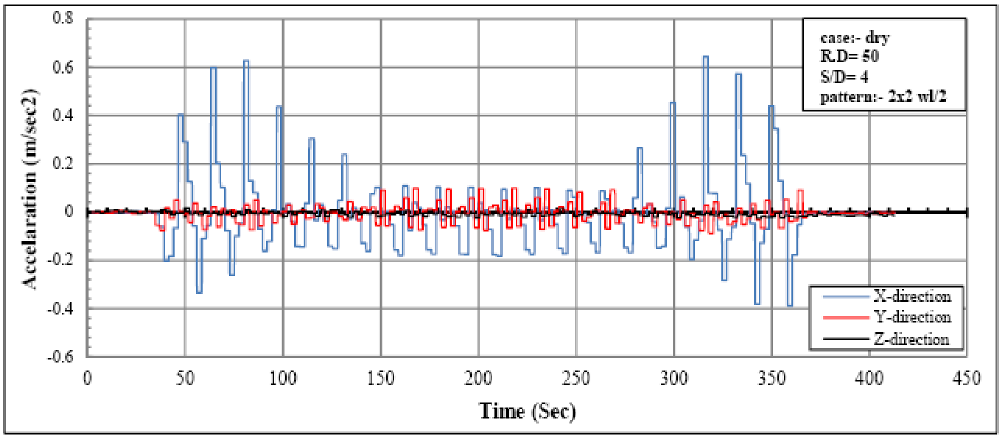


b: *f* = 1 Hz

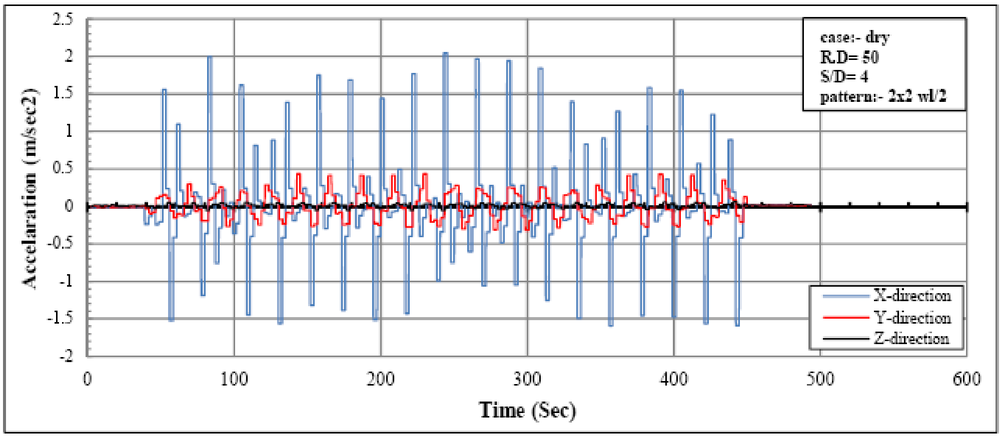


c: *f* = 2 Hz

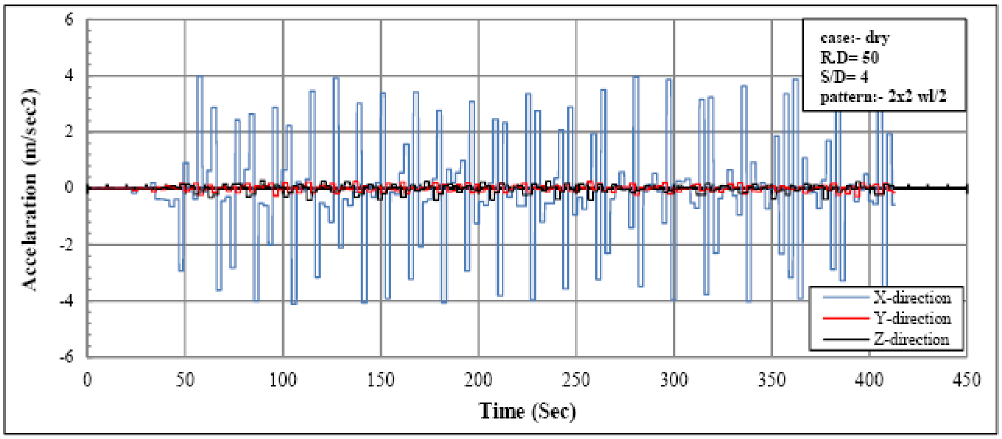
Figure 8: Acceleration time history in soil bed of pile group (2×2), with double layers of sand for s/d =4 in medium dry sand under different frequencies.



a: *f* = 0.5 Hz

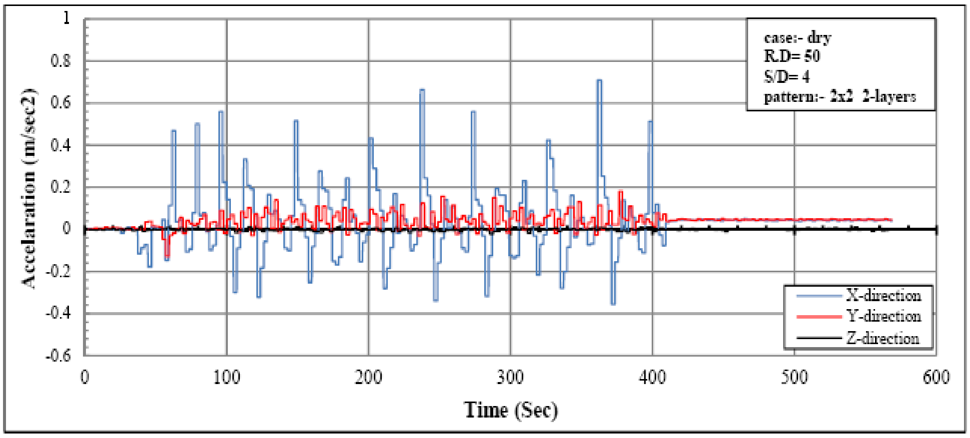


b: *f* = 1 Hz

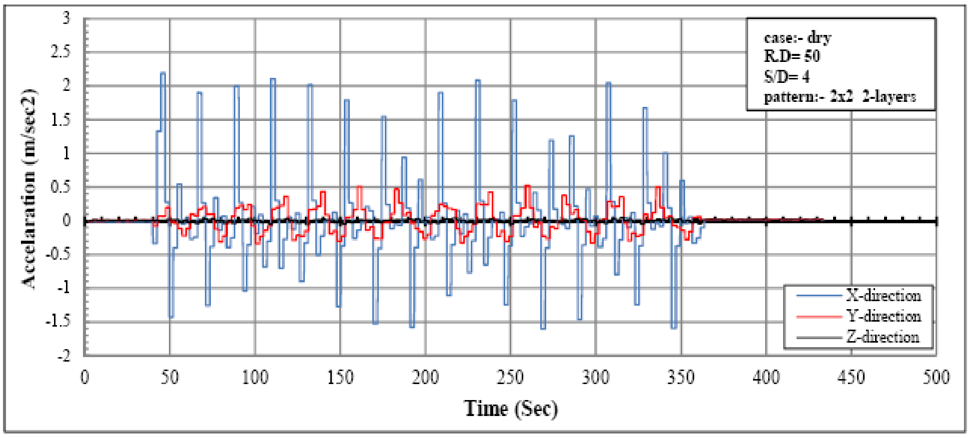


c: *f* = 2 Hz

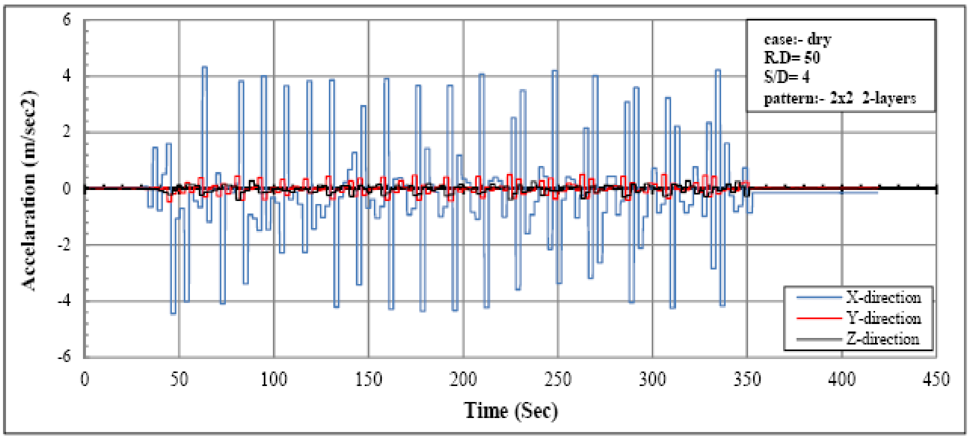
Figure 9: Acceleration time history in foundation of pile group (2×2), with half-axial load for s/d =4 in medium dry sand under different frequencies.



a: f = 0.5 Hz

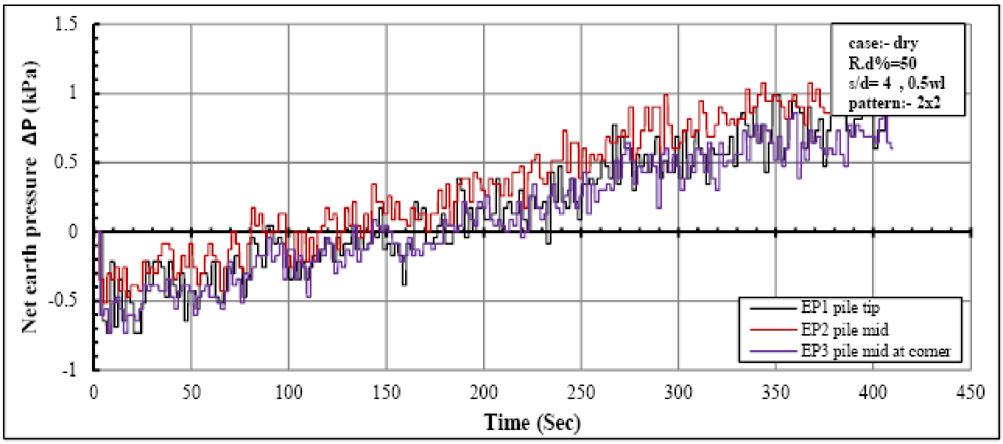


b: f = 1 Hz

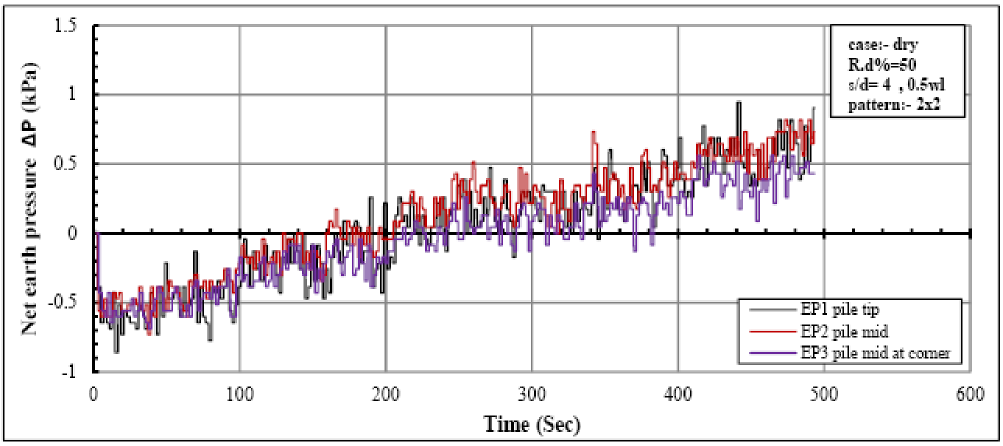


c: f = 2 Hz

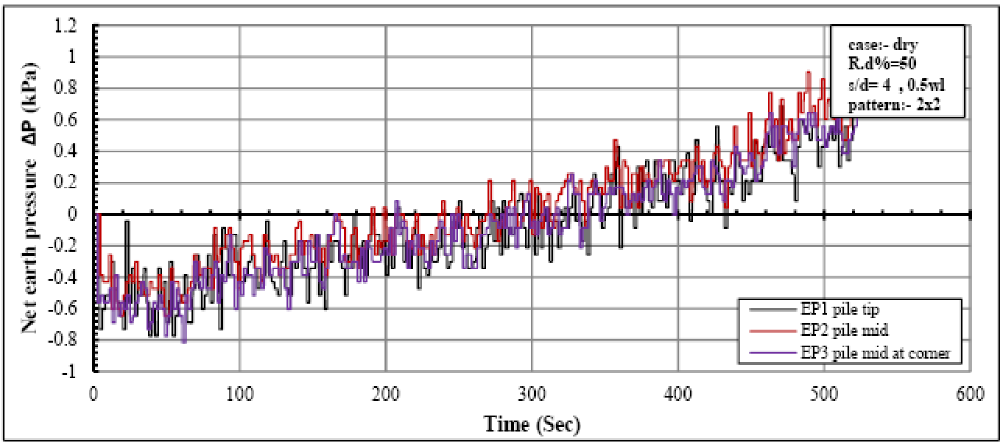
Figure 10: Acceleration time history in foundation of pile group (2×2), with double layers of sand for s/d =4 in medium dry sand under different frequencies.



a: *f* = 0.5 Hz

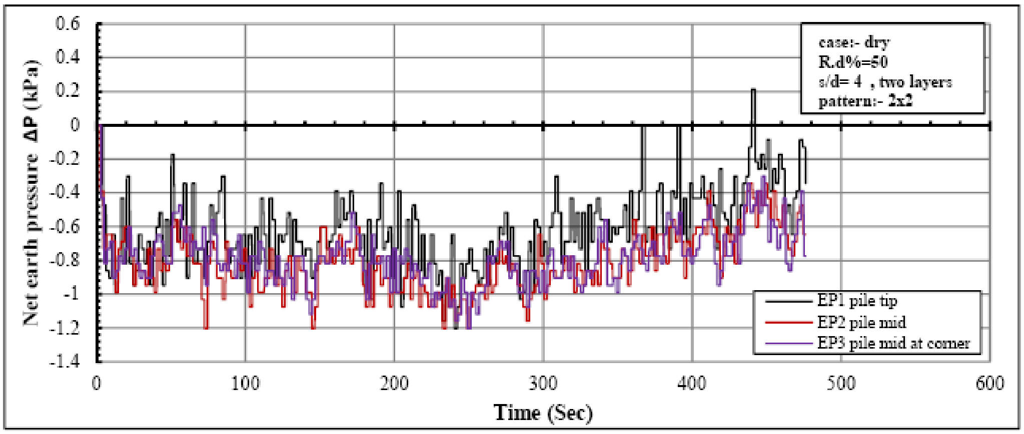


b: *f* = 1 Hz

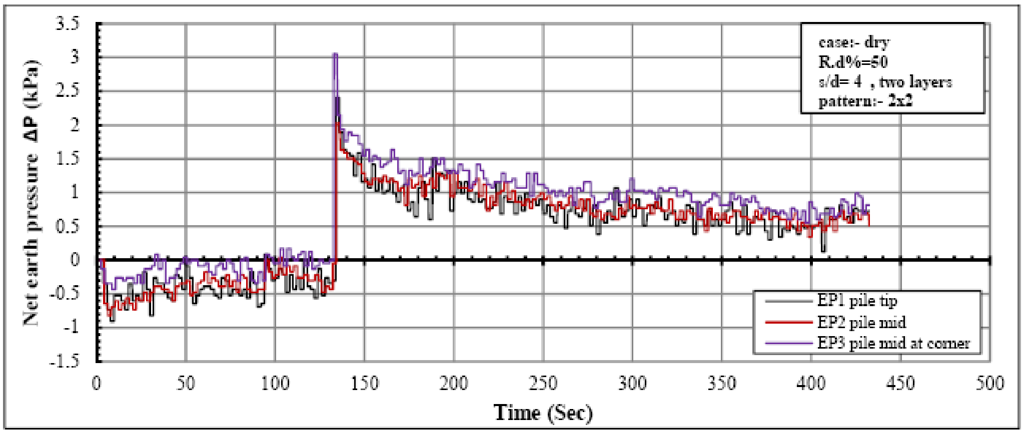


c: *f* = 2 Hz

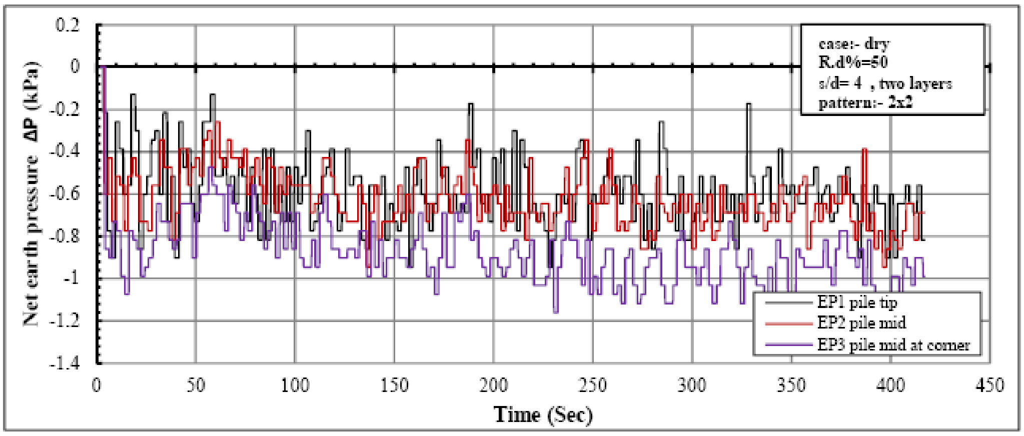
Figure 11: Soil pressure (ΔP) time history of pile group (2×2), with half-axial load for s/d =4 in medium dry sand under different frequencies.



a: *f* = 0.5 Hz



b: *f* = 1 Hz



c: *f* = 2 Hz

Figure 12: Soil pressure (ΔP) time history of pile group (2×2), with double layers of sand for s/d=4 in medium dry sand under different frequencies.

CONCLUSIONS

In the light of experimental tests and after the extensive testing and analysis of the results and other observations during the experimental analysis, the following major conclusions drawn from the test are summarized as follows:

1. For soil bed in dry state, the acceleration amplitudes increase with frequency for both soil relative densities (loose and medium).
2. The maximum acceleration in the foundation is lower than in soil bed for all operating shaking frequencies and 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.
3. The amplitudes of recorded acceleration in the pile cap are much higher than in the soil bed, also these amplitudes are increasing with increase of shaking frequency and relative density of the soil.
4. In general, the values of soil pressure decrease with increasing frequency and relative density. The soil pressure shows slight change with increasing frequency. The maximum values of soil pressure were recorded at pile tip and pile mid then pile mid at corner, since the wave is coming from the model base. The confinement provided by the piles in the group inhabits the transfer of stress waves between piles.
5. The embedment of piles partly in dense sand layer has little effect on the measured acceleration.

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