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# Mutual Seismic Interaction between Rigid Pile Inclusions and Surrounding Soft Soil Formation

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## Cover Page Footnote

I would like to express my genuine gratitude to Prof. Dr. Ayman Lotfy Fayed, Professor of Geotechnical Engineering, Structural Department, Faculty of Engineering, Ain Shams University for his kind supervision, fruitful comments, and valuable advice. My grateful appreciation is, also, extended to Dr. Tamer Sorour, Associate Professor of Geotechnical Engineering, Structural Department, Faculty of Engineering, Ain Shams University for his helpful advice, irreplaceable support, patience, guidance, useful suggestions, overall supervision and encouragement throughout this research. Lastly, I would like to express my deepest thanks to my family for their unwavering support and crucial guidance during my ups and downs throughout my postgraduate studies journey.

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# Mutual Seismic Interaction between Rigid Pile Inclusions and Surrounding Soft Soil Formation

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**Abstract - This study aims to investigate the mutual interaction between rigid Pile Inclusions (PI) and the surrounding soft formations, such as soft to medium stiff silty Clay. The studied area is located at New Mansoura City, Nile Delta, Egypt, where the soil formation consists of a surface silty Sand layer with a depth of 4m, followed by 15m-thick Soft Clay formation. The third layer is medium dense to dense sand extending to a depth of 40m, followed by a deep layer of very dense sand.**

**The Finite Element Method (FEM) is utilized to simulate this sophisticated seismic geotechnical problem, using PLAXIS-2D software and the embedded beam row feature was used to model the PI. The seismic analysis is performed in the time domain, using artificial time history (20.0 sec.).**

**Considering the generated dynamic bending moment along PI, an extensive parametric study is carried out to investigate the effect of the PI penetration length into Sand layer, PI diameter, Peak Ground Acceleration (PGA) and Shear Strength of Soft Clay Formation. According to the conducted study, results show that the penetration length has a significant effect up to a certain depth while both the pile diameter and PGA are directly proportional with the generated bending moment. In addition, the relative stiffness between the soft clay formation and the underlying sand layer highly affects the results of the study. As a general conclusion from all considered studies, edge piles were found to be more critical than middle piles.**

***Keywords-*** **rigid pile inclusion; soft clay soil; seismic analysis; finite element; artificial earthquake.**

## I. INTRODUCTION

During the last decades, civil engineering and the construction industry witnessed a remarkable development and evolution of construction technologies and innovative engineering solutions. Regarding the geotechnical engineering field, soil improvement techniques provided a competitive and cost-effective solution to enhance the bearing capacity of subsoil and reduce potential deformations.

Both in seismically active and inactive zones, soil improvement methods are often implemented at projects where the existing subsoil profile is likely to result in unfavourable behaviour. As mentioned earlier the deficiency can be in many forms covering all problems associated with unacceptably large deformations of soil. These deformations might be horizontal or vertical and/or both and it could occur in correspondence with and/or after the shaking of the earthquake. Thus, when there is no earthquake shaking, unacceptable deformations are most likely due to low soil strength and/or stiffness. Therefore, several soil improvement techniques are primarily concerned with the enhancement of strength and stiffness of existing soil deposits [1-6].

Due to the huge urban development nowadays, areas and zone with low bearing capacity are being included in the development plans leading to the need of improving or enhancing the bearing capacity and stiffness of the subsoil to ensure safety, serviceability and structural integrity. Among these techniques, soil reinforcement using pile inclusions (PI) had been adopted as the best solution for many projects in the new developed areas.

For example, in Egypt, with the wide urban expansion in the areas of the northern Nile Delta, northern Sinai and eastern Suez Canal, there is a need to construct various types of projects ranging from strategic and public facilities to residential buildings. At these regions, the Nile River meets the shores of the Mediterranean Sea, where soft clay deposits are found. Dealing with such a problematic soil formation is considered one of the biggest challenges in the field of geotechnical engineering due to its

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low bearing capacity, in addition to the excessive expected settlement when subjected to building and/or road loads.

Improving soil deposits using pile inclusions is a widespread technique in Europe, where earthquakes occur rarely in most of its regions. Therefore, research and studies related to studying the seismic behaviour of such a soil improvement method is quite rare. The design of earthquake-resistant structures highly depends on the soil-foundation-structure interaction which is more complex in the presence of soft and liquefiable soils. The Rigid Pile Inclusions system represents a useful practice to support structures in the presence of soft and liquefiable soils in seismic zones as the system proved to be effective in increasing the bearing capacity of soil and reducing the overall structure settlement and displacements [6,7,8].

Accordingly, the need to study the behaviour, structural stability, safety and integrity of this technique under the influence of earthquakes arouse in Egypt during the last few years. These studies should be carried out through integrated analyses which include the mutual interaction between pile inclusions and the surrounding soil taking into consideration the structural loads imposed by structures and/or roads [4,6,7,8].

## II. RIGID PILE INCLUSIONS

Theoretically, the concept of the foundation on rigid inclusions is somehow based on the concept of piled raft foundation in which rigid vertical elements, similar to the inclusions, are connected to a flexible layer, but no direct contact between the raft and piles. In practice, this change from piled raft foundation to foundations rested on load distribution platform and rigid pile inclusions is associated with a change in the geometric nature of the core and mechanical discontinuity [9]. This concept comprises various interaction modes between:

1- Inclusions, which may be topped by a cap
2- Load transfer platform on which foundations are rested
3- Surrounding soil (in-between the inclusions)

The diagram shown in Fig. 1 summarizes the range of interactions, with a differential settlement at the load transfer platform base. The diagram shows the load transfer concept where the load is transferred to the pile inclusions through the platform via arching action. In addition, for the upper part of the soil in-between the inclusions, negative skin friction takes place.

The theory that has been adopted includes having strict inclusions; meaning that inclusions' caps are not connected to the supported structure as the usual practice when designing a raft foundation rested on piles. To ensure efficient and well-distributed load transfer mechanism, a distribution layer (or platform) – which is usually made up of gravel – link / connect the caps and structure. The implementation of the inclusions with caps overlayed by the load transfer platform gives a composite or reinforced soil volume that usually has higher strength and lower deformability / compressibility than the primary soil volume enabling the structure to rest on a shallow foundations system.

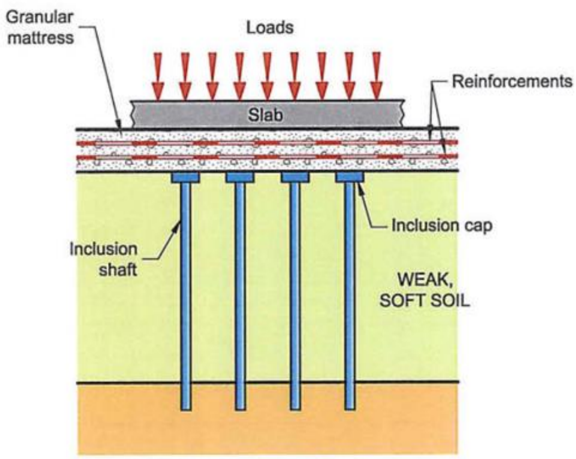


Figure 1. Foundation Rested on Rigid Pile Inclusions System [9]

## III. DYNAMIC SOIL BEHAVIOUR

As the branch of geotechnical engineering concerned with studying the soil properties under dynamic loads, the science of soil dynamics is used and implemented to estimate the bearing capacity of shallow foundations or load capacity of deep foundations under dynamic stresses. The seismic response of soils depends on several factors, some of them are related to the soil and others to the applied dynamic load, such as soil layers properties and geometry, the soil water content, and the type of dynamic loading. Seed et al., 1970; Holtz and Kovacs, 1981; Kramer, 1996; Kokusho and Yoshido, 1997; Abdel-Motaal, 1999 and El-Shamy, 2021 highlighted the most important soil properties that sensitively influence soil dynamics problems.

i- Stiffness characteristics
  1. Shear strength and strain amplitudes
  2. Various Dynamic moduli
  3. Poisson's ratio
ii- Damping characteristics
iii- Liquefaction parameters for liquefiable soils

### *Stress-Strain Behaviour of Cyclically Loaded Soils*

The cyclic behavior of soil under seismic loading can be represented using three main soil models.

1- Equivalent Linear Model:

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Idriss and Seed (1968) had initially developed the equivalent linear model and it is considered the simplest model. It is mainly used in the cases of symmetrically and cyclically loaded soil. Fig. 2 shows the hysteresis loop that governs the relationship between shear stress and shear strain of soil. The inclination of the hysteresis loop can be described by Gsec.

$$G_{sec.} = \frac{\tau_o}{\gamma_o} \qquad \text{Eq. 1}$$

where $\tau o$ is the shear stress and $\gamma o$ is the shear strain amplitude.

Using the backbone, secant shear modulus can be expressed by the inclination of the line joining the origin with the point on the backbone curve, as shown in Fig. 3 which clearly shows that any increase in the cyclic strain amplitude is accompanied by a reduction in the shear modulus.

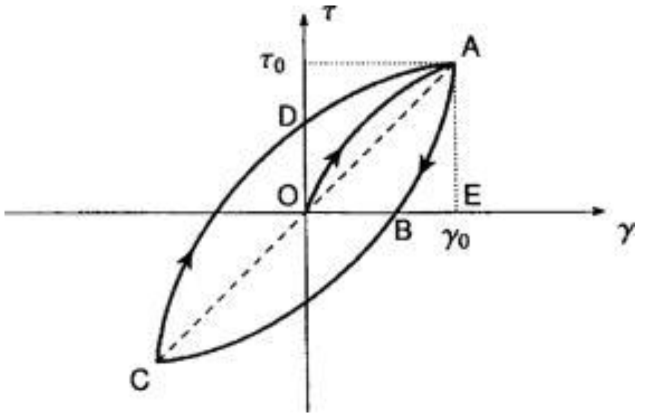


**Figure 2. Hysteresis Loop Showing Secant Shear Modulus and Effective Damping Ratio (Kramer, 1996)**

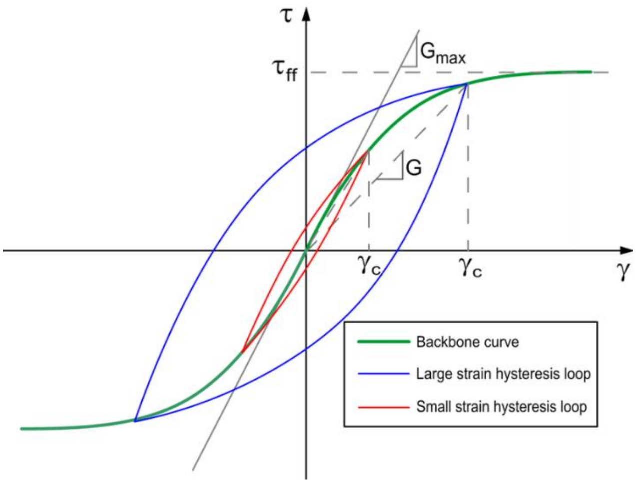


**Figure 3. Backbone curve shows maximum and secant shear modulus according to shear strain amplitude (Stewart, 2014)**

The damping ratio, which is an extremely important parameter when studying soil dynamics, is divided into material damping and radiation damping as follow:

1- Material damping represents energy dissipation in soils during cyclic loading. A lot of approaches were used over years to predict and evaluate a realistic value of the material damping coefficient. Vucetic and Dobry (1991) showed that the relationship between the damping ratio and shear strain is directly proportional, as shown in Fig. 4. There are several factors that influence the material damping coefficient and shear modulus such as the cyclic strain amplitude, mean effective confining pressure, plasticity index, over consolidation ratio, and void ratio.

2- Radiation damping represents the amount of dissipated energy away from the foundations through waves' radiation. To avoid refraction of waves in finite element models, lateral vertical boundaries are used enabling a good representation and modelling of radiation damping. It is important to highlight that while preparing the finite element models, dealing with constant values of the shear modulus and damping ratio is considered a major drawback of using the equivalent linear method.

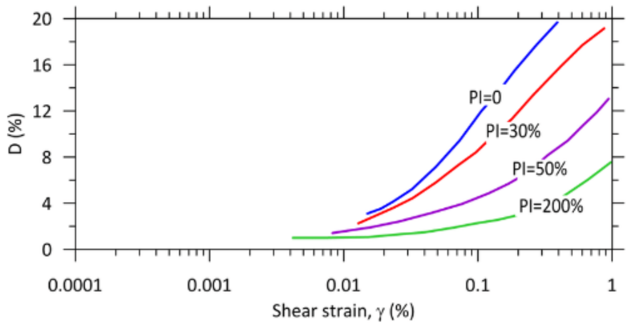


**Figure 4. Relation between damping ratio and cyclic shear strain for different plasticity indexes (Vucetic and Dobry, 1991)**

2. Cyclic Nonlinear Model:

One of the main advantages of using the cyclic nonlinear model is enabling the representation of the actual non-linear stress-strain curve during cyclic loading considering pore pressure generation, especially at high shaking levels. In addition, using such a model the shear strain does not reach zero even when the shear stress attenuates to zero. The extended masing model is clearly expressed using the four rules of cyclic nonlinear model behavior as shown in Fig. 5.

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i. Initially, the backbone curve governs the soil behaviour
ii. Concerning the unloading, a new path is followed, which has a doubly scaled backbone curve - shape.

$$\frac{\tau-\tau_r}{2} = \frac{F_{bb}(\gamma-\gamma_r)}{2} \qquad \text{Eq. 2}$$

where:

| | |
|---|---|
| $\tau$ | : Shear stress, |
| $\tau$r | : Shear stress at a reversal point, |
| $F$bb | : Backbone function, |
| $\gamma$ | : Shear strain, |
| $\gamma$r | : Shear strain at a reversal point. |

iii. Upon reaching the maximum strain, the loading or unloading curves intersects the backbone curve and follows it until the subsequent stress reversal.
iv. If a reloading or unloading curve intersects with an earlier reloading or unloading curve from a previous cycle, the stress-strain path matches that of the previous cycle.

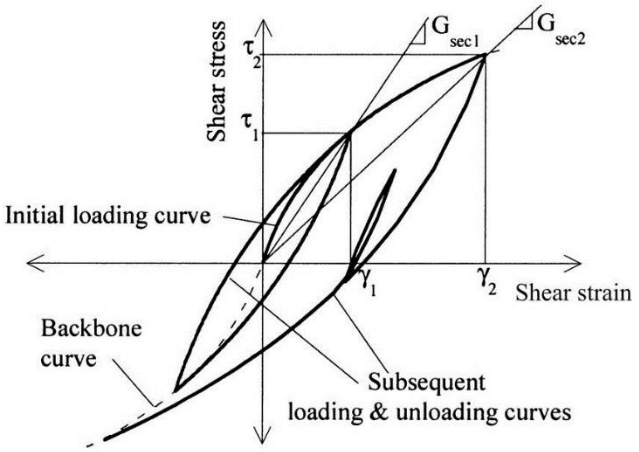


**Figure 5. Cyclic nonlinear model behavior and extended masing rules (Hashash and Park, 2001)**

Lok (1999) classified various non-linear models as follow:

1- Mechanical models: To represent the non-linear behavior of soils some mechanical elements such as springs, dashpots, and sliders are implemented in the models.
2- Empirical models: Several researchers derived empirical functions to represent the soil non-linearity such as Ramberg and Osgood (1943) and Kondner (1963).
3- Plasticity models: Aside from working according to plasticity theory, they are an example of advanced constitutive models.

3. Advanced Constitutive Models:

The most accurate and complicated method used to represent dynamic soil behavior is the advanced constitutive models. To build up an advanced constitutive model, several parameters are required to be evaluated taking more time and effort than any other method. However, advanced constitutive models have an edge over other models by taking into consideration the general initial stress condition, a wide variety of stress paths, rotating principal stress axes, type of loading (cyclic or monotonic), strain rates level, and drainage conditions. (Abdel-Motaal, 1999; Wahidy, 2009; Toma, 2017 and El-Shamy, 2021).

### *Site Response to Earthquakes*

In case of dealing with the linear behavior of soil, high dispersion of ground motion happens therefore dynamic response of structure should be studied to determine site ground motion. Site response to earthquakes can be accurately expressed using three-dimensional models that are available through finite element software such as PLAXIS. (Kramer, 1996; Abdel-Motaal, 1999; Bonilla, 2012 and El-Shamy, 2021).

## IV. DYNAMIC SOIL STRUCTURE INTERACTION (DSSI)

Soil-Structure Interaction is considered one of the most complex processes where there is a reciprocal effect between the response and the motion of soil and the structure. As shown in Fig. 6, the response of a structure to the same simulation may differs depending on the type of soil and site conditions. To study soil-structure interaction and the dynamic properties of the soil-structure system large-scale studies have been recently performed. (Dowrick 1987, Kramer 1996, Abdel-Motaal 1999, Gandomzadeh 2011, Abdel-Motaal et al. 2014 & Alfach 2019).

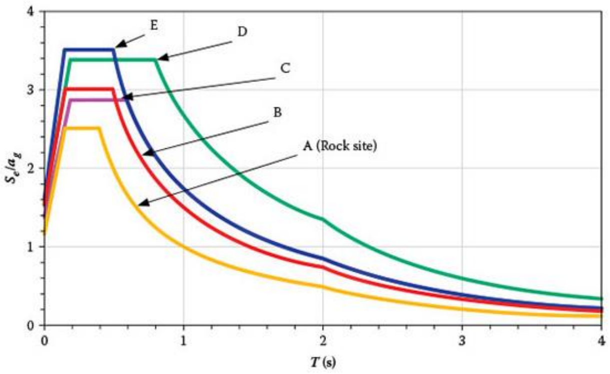


**Figure 6. Response spectrum curve for different site conditions (EC8 2004, Worku, 2014)**

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The structure response may be influenced by the soil conditions in many ways or manners as follows:

i. For the same seismic excitation at bedrock, the type of soil at which the waves propagate through influence the attenuation or amplification at the foundation level.
ii. The failure mode depends on the soil type underneath, where high differential displacements are experienced in cases of soft soil deposits (example: Taiwan earthquake, 2018 (Fig. 7)).
iii. For structures supported on rock formations, the response follows free field motion and fixed base dynamic properties are used.
iv. In cases of structures supported on piles, sliding and gapping levels between the pile and the surrounding soil occur depending on the soil type.


**Figure 7. Differential vertical displacement failure of a structure in Taiwan Earthquake, 2018 (Lin et al., 2020).**

### *Soil Models for Dynamic Analysis of Laterally Loaded Piles*

Hansen (1961) identified three parameters that should be considered in designing laterally loaded piles:

1- Ultimate bearing capacity of surrounding soil
2- Deflection of piles
3- Structural integrity of the foundation system

To represent and analyze the non-linear dynamic soil-structure interaction, the following approaches and methods were developed to model and perform such an analysis:

1- Winkler Approach
2- P-Y Curves Method
3- Finite Element Method (FEM)

### *Finite Element Method (FEM)*

In comparison with all numerical techniques, the finite element method (FEM) is considered the most versatile when solving any complex problem. It facilitates the simulation of the interaction phenomenon between soil and pile, boundary conditions, several loading types and/or cycles, soil types and parameters. There are several ways in which the parameters of the soil can describe the nonlinearity of the soil with respect to strain level by modulating the values of the soil moduli and damping ratio. However, there are some drawbacks as the low accuracy of representing radiation damping and the time required for the analysis, especially for three-dimensional models [3,4,5,7,8,9].

## V. PROBLEM DEFINITION

The development of New Mansoura City - which is located at the northernmost end of the Nile Delta overlooking the Mediterranean Sea and possessing a very promising economic future in Egypt - was restricted and limited by the natural soil formation of problematic deep soft clay deposit. This formation is normally associated with low bearing capacity and excessive settlement of soil, under the effect of building loads. These soft clay formations extend to depths of more than 15.0 meters, followed by higher bearing capacity formations consisting of cohesive clay or medium dense sand. The development of New Mansoura University - which is about two kilometres away from the seashore - is considered one of the most important projects to be implemented in the new city and because of the weak soil formations, it was initially proposed to use deep foundations of bored piles with a depth of 35.0 meters. Since such a solution is considered extremely costly and may influence the economic feasibility of the project, the geotechnical consultants studying the project decided to investigate several alternatives and solutions focusing on improving the properties of the upper soft clay formation. The studies and comparisons highlighted different aspects such as the estimated cost of each proposal, improved soil parameters, construction time and predicted settlement values. The proposed solutions included the following soil stabilization methods:

- Prefabricated Vertical Drains (PVD) or Wick Drains.
- Crushed Stone Columns.
- Rigid Inclusions (RI).

### *Solution Selection*

The Stone Columns proposal is excluded as – according to the performed studies – it will save 15% only from the cost of the initial proposal (bored piles) in comparison with the other two solutions; Prefabricated Vertical Drains (PVD) and Rigid Inclusions that will save 60% to 70% and 50% respectively. Although Prefabricated Vertical Drains (PVD) is considered the

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most economical solution, it is considered the most time-consuming technique as the soil improvement process will take several months to be completed after the installation of the drains. On the contrary, the Rigid Inclusions solution will save 50% of the initial cost which is not far away from the percentage saved by Prefabricated Vertical Drains (PVD) and the soil stabilization will be immediate (just after the completion of the installation of the inclusions). In addition, the foundation type that will be used in case of improving the soil using Prefabricated Vertical Drains is raft footing which is more expensive than isolated footings that suits the Rigid Inclusions solution. Regarding the expected settlement, which will affect the structural design of the structures and may later cause architectural defects and electromechanical problems, the calculated values for the case of the Rigid Inclusions solution is almost one third of the Prefabricated Vertical Drains (PVD) solution. Consequently, the Rigid Inclusions system was chosen as the best soil improvement method to be implemented in the project.

Rigid Pile Inclusions System is not widely used in construction projects in Egypt. Geotechnical studies showed the possibility of implementing such a system to reach the desired bearing capacity (15 t/m$^2$) with total settlement of the shallow foundation not exceeding few millimetres. As mentioned, financial studies also resulted in the possibility of reducing the total cost of foundations by approximately 50% in comparison with the primary bored piles proposal.

After choosing the system, a controversy arose between the soil improvement implementation company and the geotechnical consultants, whether to construct the inclusions using reinforced concrete or plain concrete. Since the company has a massive experience in implementing such a system in seismically inactive zones, they got used to adopt the plain concrete option. To determine the need of reinforcement, a seismic analysis of the system was performed according to the Egyptian Code for Loads (ECP-201-2012), using time history analysis. This dynamic analysis showed the critical values of the generated bending moment on piles, concentrated at the levels of the boundaries separating soil layers with significant difference in stiffness and rigidity. Consequently, it is important to conduct extensive studies and shed lights on scientific research studying and focusing on the Mutual Seismic Interaction between Pile Inclusions and the Surrounding Soil Formation.

To investigate the mutual Seismic Interaction between Pile Inclusions and Site Soil Formation a practical problem similar to that of New Mansoura University was extensively studied by utilizing PLAXIS 2D to develop a finite element model.

## VI. CASE STUDY

Fig. 8 shows the soil stratification at the studied site (New Mansoura University). The formation consists of surface silty Sand layer with a depth of 4m, followed by 15.0m-thick Soft Clay formation. The third layer extends to a depth of 40m consisting of medium dense to dense sand (referred to as Sand 1), followed by a deep very dense sand layer (referred to as Sand 2) extending for 20.0m and rested on the Bedrock surface. The total thickness of the examined soil formation is 60.0m. During the site investigation process, the Ground Water Level (GWL) was nearly encountered at the ground surface. The performed laboratory and field tests during the site investigation process were used in the interpretation of the physical and mechanical properties of the soil layers as shown in Table 1.

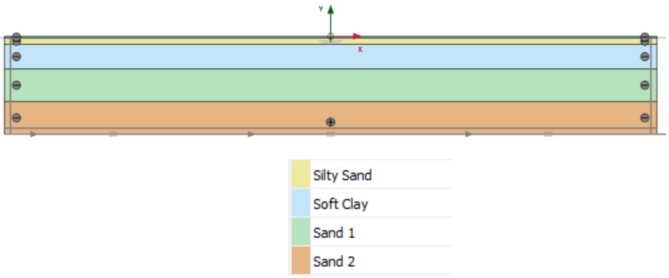


**Figure 8. Soil Stratification at the Studied Site (New Mansoura University)**

The basic studied case is shown in Fig. 9, where the system consists of a row of equally spaced 37 pile inclusions extending to the end of the soft clay layer and penetrating the following sand layer (Sand 1) by 6.0 m. A platform of 1.0-m depth – consisting of granular mixture of siliceous sand and crushed dolomite in ratio (1:1) – is used as a bedding layer beneath the foundations underlain by a mesh of geo-grids which is implemented to distribute the structure loads over the pile inclusions system. The soil stabilization system is loaded by an external uniform load of 150-kN/m2, at the ground surface. The properties of the geo-grids and inclusions considered in the basic studied case are shown in Table 2. HS-small model is implemented to simulate the problem due to its advantages in dynamic analysis, as explained before. The 2-D finite element numerical modelling mesh of the basic studied case is illustrated in Fig. 10.

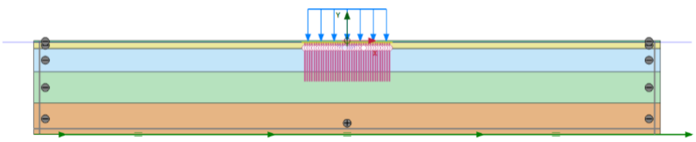

**Figure 9. Pile Inclusion System Setup (basic studied case)**

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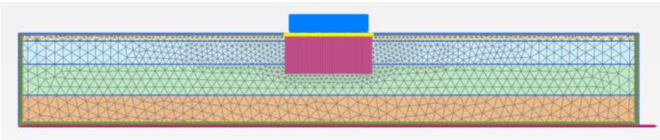

Figure 10. 2-D Finite Element Numerical Modelling Mesh (basic studied case)

Table 1. Physical and Mechanical Properties of Soil Layers

| Parameter | Unit | Layer (1) Silty Sand | Layer (2) Soft Clay | Sand (1) | Sand (2) |
|---|---|---|---|---|---|
| Layer Thickness | m | 4.00 | 15.00 | 20.00 | 20.00 |
| Saturated Soil Unit Weight | kN/m$^3$ | 20.00 | 18.00 | 21.50 | 21.50 |
| Reference Secant Stiffness (Triaxial Test) | kN/m$^2$ | 5.5E+4 | 1.0E+3 | 7E+4 | 1.2E+5 |
| Reference Tangent Stiffness (Oedometer Test) | kN/m$^2$ | 6.6E+4 | 1.0E+3 | 8.4E+4 | 1.4E+5 |
| Unloading/Reloading Stiffness | kN/m$^2$ | N/A | 3E+3 | N/A | N/A |
| Friction Angle | Degrees | 22 | 0 | 35 | 38 |
| Cohesion | kN/m$^2$ | 15 | 20 | 1 | 1 |
| Shear Strain threshold at 0.722 Go | Unitless | N/A | 0.0007 | N/A | N/A |
| Maximum shear modulus | kN/m$^2$ | 2.2E+4 | 1.9E+4 | 2.8E+4 | 4.8E+4 |
| Unloading/reloading Poisson's ratio | Unitless | 0.25 | 0.2 | 0.25 | 0.25 |
| Reference Pre-Consolidation Pressure | kN/m$^2$ | N/A | 100 | N/A | N/A |

Table 2. The Properties of the Geo-grids and Inclusions (basic study)

| Property | Unit | Value |
|---|---|---|
| **Length** | m | 25 |
| **Diameter** | cm | 50 |
| **Spacing (both directions)** | m | 1.5 |
| **Axial Stiffness Per Unit Length of Geogrids (EA)** | kN/m | 2000 |
| **Thickness of Load Transfer Platform** | m | 1.0 |

## *Earthquake Control Motion*

Previous studies [14,15] using three different real acceleration-time histories, as well as two artificial time records (GEQ II and GEQ III) concluded that time history (GEQ II) results were considered average for the five earthquakes and have been used for their study analysis. These two artificial earthquakes were generated by [16]. These two earthquakes were artificially generated based on the response spectrum of the UNIFORM building code (1994) for the rock formation. Table 3 shows the basic criteria for the used time histories. Accordingly, the artificial earthquake record (GEQ II) is used in this study as a control motion at the Bedrock surface. To ensure that a wide spectrum of frequencies is represented during the earthquake simulation, it is better to use an artificial earthquake time history in such a seismic analysis. Moreover, real-life earthquake records may be affected by the fundamental frequencies of the specific region at which the earthquake occurred, limiting the range of frequencies spectrum represented by this earthquake [16]. Acceleration Time History for both artificial earthquakes (GEQ II and GEQ III) are shown in Fig. 11 & 12. To carry out a comprehensive study on the earthquake peak acceleration and study its influence on the analysis, the acceleration-time history is scaled to peak accelerations of 0.1g, 0.15g and 0.20g as shown later in the parametric study.

Table 3: List for the five used time-acceleration histories [14].

| Earthquake | Period (sec.) | Earthquake Periodic Time (sec.) |
|---|---|---|
| Loma-Prieta/Santa- Cruz Mountains, 1989 | 40.0 | 0.32 |
| NE-India, 1986 | 20.0 | 0.10 |
| Denali, 2002 | 50.0 | 0.22 |
| GEQ II | 20.0 | 0.25 |
| GEQ III | 40.0 | 0.28 |

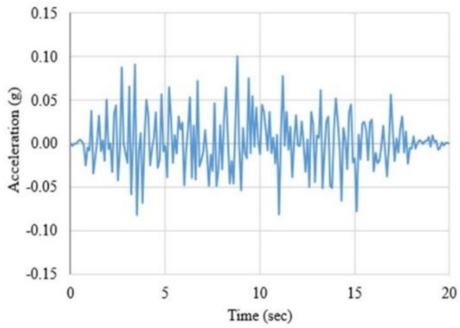


Figure 11. Time-acceleration history of GEQ II [16]

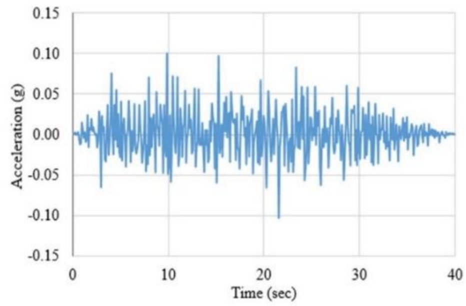


Figure 12. Time-acceleration History of GEQ III [16]

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## VII. Analysis Results

As shown by previous studies and research, the pile inclusions that are implemented in seismically active regions should be designed to withstand the generated dynamic bending moments that reach critical values at the interfaces between soil layers (especially when the variance in stiffness is significant).

To perform a detailed analysis of the soil improvement system behaviour and obtain the seismic response of the whole system; especially the rigid pile inclusions, seismic analysis was carried out using PLAXIS 2D. Fig. 13 & 14 show the results of the basic studied case, where it can be concluded and established that:

- As shown in Fig. 13 & 14, the generated bending moment diagrams along both middle and edge pile inclusions has two peaks; the upper one is located along the pile inclusion at the penetration length inside the Soft Clay layer while the lower peak of the generated bending moments is recorded near the level of the interface between the Soft Clay and Sand (1) layers.
- In general, it can be concluded that the maximum value of the developed bending moment is near the level of the interface separating the soft clay layer and the underlying sand layer. By comparing the results of the middle and edge piles, it was found that the maximum moment developed on the edge pile, which was found to be 53.97 kN.m, is noticeably higher than the maximum moment developed on the middle pile which is 6.784 kN.m. Meaning that the location of the pile significantly affects the value of the developed dynamic moment.
- By comparing the results for both middle and edge piles, it was found that the edge pile is more critical than the middle pile. This logical finding could be clarified by the fact that the middle pile is well surrounded by successive piles enhancing the lateral support provided to the pile and reducing the seismic response of these piles (middle and also nearby piles). On the contrary, the edge pile is surrounded by piles from one side only and that's why it lacks lateral support from the other side. Similarly, the seismic response of the Soft Clay layer at the location of the edge pile is greater than that at the location of the middle pile.
- By studying the bending moment curve trend, logically the upper maximum value is resulting from the buckling of the pile inclusion due to the insufficient horizontal support provided to the inclusion by the Soft Clay layer. On the other hand, it is clear that the lower maximum value of the bending moment appears approximately at the separation between the Soft Clay and Sand (1) layers, where there is a difference in the seismic horizontal response (movement) between these two successive layers. These seismic responses are based on the dynamic mechanical properties of both soft layer (Soft Clay) and the relatively firm layer (Sand 1). As a general finding, the maximum moment is always located at the level of this separation (lower peak). Accordingly, the reinforcement using steel bars should extend along the whole pile length to resist the generated bending moments.
- Accordingly, such values of bending moment should be resisted by reinforcement, whether through steel reinforcement bars or steel fibres, to increase the bending resistance and tensile strength of the pile inclusions avoiding its failure during dynamic loading conditions. To design such a concrete section, straining actions, end conditions and loading case should be studied carefully as this partially confined concrete element (pile inclusion) is subjected to compressive axial force and dynamic bending moment.
- Considering the compressive axial load and maximum dynamic bending moment along the pile length, the structural design for the pile section showed that there is a need of reinforcement to resist the acting straining actions. The reinforcement values are expected to be high at the outermost pile inclusions and decrease as the inclusion location gets near the middle pile which has the minimum reinforcement value.

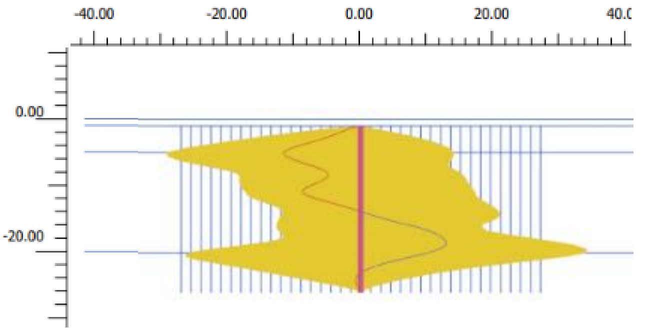


**Figure 13. The Maximum Bending Moment Diagram along the Middle Pile Inclusion (basic studied case).**

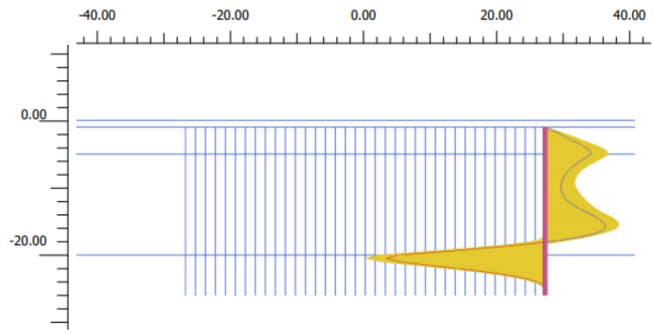


**Figure 14. The Maximum Bending Moment Diagram along the Edge Pile Inclusion (basic studied case)**

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*Parametric Study*

An extensive parametric study was carried out to investigate the effect of different parameters on the generated bending moment along middle and edge pile inclusions. The parametric study focuses on investigating the influence of the following parameters on the generated dynamic bending moment.

1- Penetration length of pile inclusion (PI) into the sand layer (Sand 1).
2- Diameter of pile inclusion
3- Peak Ground Acceleration at Bedrock level
4- Cohesion of the Silty Clay layer

The previously illustrated case, which is shown in Fig. 9, is considered the basic studied case and it will be the reference case throughout the parametric study. For the reference case, the values of the studied parameters are as follows:

- Penetration Length of (PI) into (Sand 1) layer, L = 6.0-m.
- Diameter of Pile Inclusion, d = 0.50-m.
- Spacing between piles, S = 1.50-m, (S/d = 3.0).
- Peak Ground Acceleration at Bedrock level, ag = 0.10g.
- Cohesion of the Silty Clay layer, Cu = 20 kPa.
- Angle of internal friction, φ = 35 degrees.

*A. Penetration Length of Pile Inclusion into Sand Layer (1)*

Fig. 15 & 16 show the effect of the Penetration Length of PI into Sand layer (1) on the maximum generated bending moment along pile inclusion, for the middle and edge PI, respectively. By analysing these figures, the following conclusions could be established:

- Similar to the results of the basic study, the generated moment in the edge pile inclusion is much higher than the moment generated in the middle pile inclusion.
- The penetration length has a considerable effect on the generated bending moment along the pile inclusion as shown in figures 11 & 12 whereas the Penetration Length into Sand Layer (1) decrease, there is a reduction in the value of the generated bending moment by approximately 17.3% and 41.4%, for the middle and the edge PI, respectively. This is an absolutely logical finding as when the penetration length of the PI into the relatively firm soil (Sand 1) increases, the fixation of the PI is enhanced. This directly proportional relationship is valid to a certain limit.
- It can be concluded that the directly proportional relationship between the embedment length and the degree of fixation is valid to a certain limit. By studying figures 11 & 12, it can be concluded that a penetration depth of 3.0-m is sufficient to mobilize the needed fixation. Any embedment beyond this depth to enhance the fixation is worthless. Accordingly, 3.0-m penetration depth may be the optimal value for the studied case.

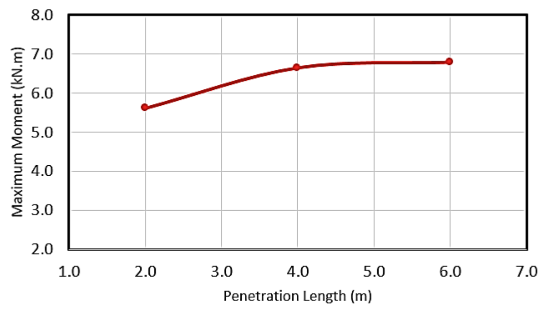


**Figure 15. Effect of Penetration Length of PI into Sand layer (1) on the Maximum Generated Moment along the Middle PI**

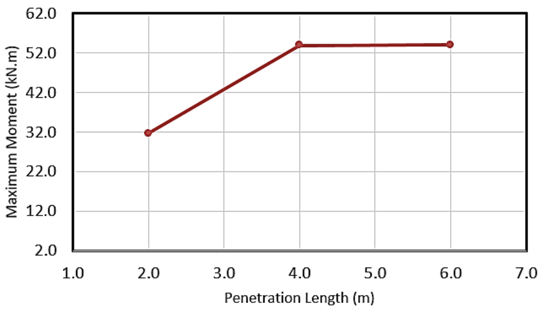


**Figure 16. Effect of Penetration Length of PI into Sand layer (1) on the Maximum Generated Moment along the Edge PI**

*B. Diameter of Pile Inclusion*

Fig. 17 & 18 show the effect of Pile Inclusion Diameter on the maximum generated bending moment along pile inclusion, for the middle and edge PI, respectively. By examining and studying the results and figures, it was found that increasing PI diameter is highly associated with considerable increases in the values of the generated bending moment. The abovementioned figures show that by increasing the PI diameter from 0.40-m to 0.60-m, the maximum generated bending moment on the PI increased from 3.14-kN.m to 11.73-kN.m and from 29.87 kN.m to 89.54-kN.m for the middle and edge pile inclusions, respectively.

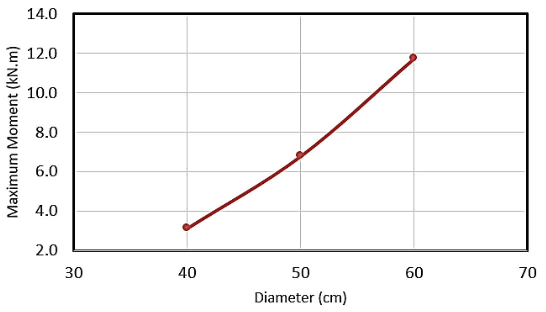


**Figure 17. Effect of Pile Inclusion Diameter on the Maximum Generated Bending Moment along the Middle PI**

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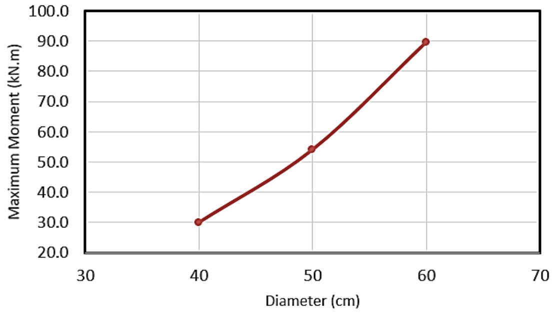


**Figure 18. Effect of Pile Inclusion Diameter on the Maximum Generated Bending Moment along the Edge PI**

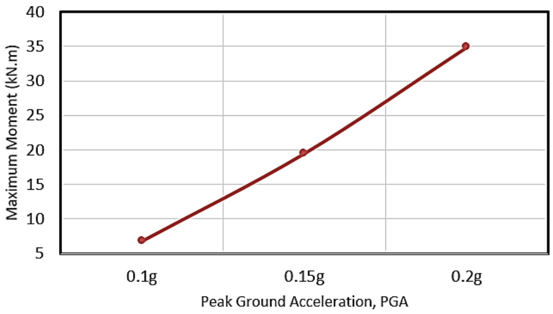


**Figure 20. Effect of PGA on the Maximum Generated Bending Moment along the Edge PI**

### C. *Peak Ground Acceleration (PGA)*

The previously discussed and presented studies were carried out using Peak Ground Acceleration (PGA = 0.1g), at the bedrock surface. This additional study was performed to investigate the effect of changing the PGA on the generated bending moment along PI as shown in Fig. 19 & 20. Through studying and analysing the relationships, the following conclusions were reached:

- Logical trend was established as there is direct proportionality between the PGA and the maximum generated bending moment along the inclusions.
- For the middle PI, increasing the PGA from 0.1g to 0.2g was accompanied with a massive increase in the generated bending moment which increased from 6.87-kN.m to 34.92-kN.m representing an increase of 408%.
- For the edge PI, increasing the PGA from 0.1g to 0.2g resulted in a subsequent increase in the generated bending moment from 53.97-kN.m to 139.5-kN.m (% increase = 158.5%). These results demonstrate the strong influence of the PGA values on the generated bending moments, especially for the middle pile.

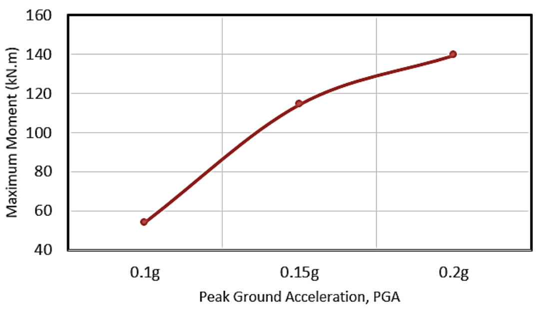


**Figure 19. Effect of PGA on the Maximum Generated Bending Moment along the Middle PI.**

### D. *Pile Inclusions Reinforcement*

As prior mentioned, the dynamic bending mom ent acting on the pile inclusions should be resisted by reinforcement to avoid flexural failure during dynamic loading conditions. Pile inclusions are partially confined concrete elements which are subjected to compressive and flexural stresses. Considering these straining actions for edge piles – which are subjected to the maximum moment in comparison with internal piles – the structural design for the pile section showed that the required reinforcement varies with the acting PGA as shown in Fig. 17; where the required reinforcement is represented with the reinforcement percentage (Area of steel reinforcement (As) / Area of concrete section (Ac)). As the pile inclusion position gets near the center of the improved zone, the developed moment along the pile inclusion decreases gradually till reaching the middle pile which is subjected to the minimum bending moment and consequently it requires minimum reinforcement according to the used design code – as precisely explained later in section 4.6.

The highlighted case in Fig. 21 shows that for a PGA of 0.15g the required reinforcement percentage for the edge pile is approximately 0.6143% meaning that the required area of steel reinforcement is 1205.56 mm$^2$ (6 bars with a diameter of 16 mm each (6T16)), considering that:

- Concrete Characteristic Compressive Strength: 25 MPa.
- Yield Strength of Steel Reinforcement: 360 MPa.
- Ultimate Strength of Steel Reinforcement: 520 MPa.

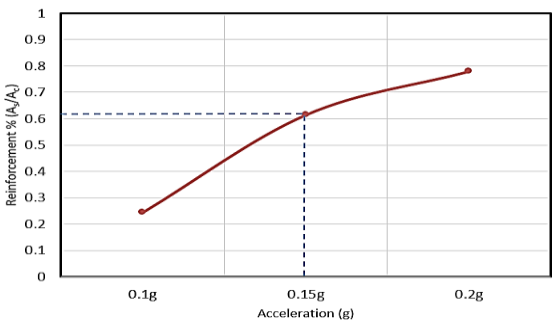


**Figure 21. Effect of PGA on the Required Reinforcement Percentage ($A_s/A_c$) (Edge PI)**

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### E. Shear Strength of Soft Clay Formation

The variation of the undrained shear strength of the soft clay layer will undoubtedly affect the values of the dynamic bending moment developed along the inclusions. Three different values of undrained cohesion were used in the study ($c_u$ = 10, 20 & 30-kPa) and the obtained results were used to interpret a relationship between the undrained cohesion of clay and the maximum generated bending moment along the middle and edge PI as shown in Fig. 22 & 23.

- Increasing the soft clay strength means reducing the relative difference in stiffness between the soft clay formation and the followed sand formation and consequently reducing the relative seismic excitation between these successive layers. As a result, this increase in strength and stiffness will reduce the generated bending moment along the PI.
- Figure 23 shows clearly the effect of increasing the undrained cohesion of soft clay from ($c_u$ = 10 to 30 kPa); where a corresponding reduction of nearly 50% of the generated bending moment is recorded for the edge PI.
- For the middle PI, the correlation is shown in Fig. 22 and for the first instant, it can be concluded that the expected trend is not achieved. However, by studying the positions of the maximum moments in the pile inclusions, it was found that these maximum values are not recorded at the level of the boundary between the soft clay layer and sand. Therefore, these results do not represent the case of the study, but rather represent the general behaviour of the pile depending on the stiffness of the clay layer.

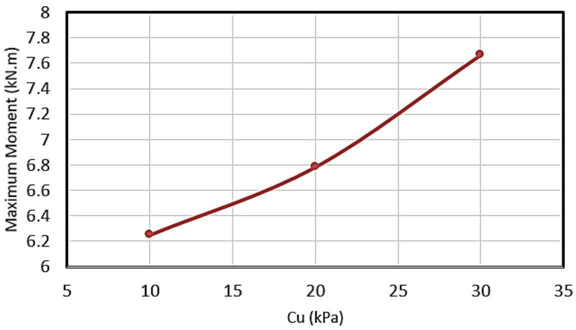


**Figure 22. Effect of Soft Clay Shear Strength on the Maximum Generated Bending Moment along the Middle PI**

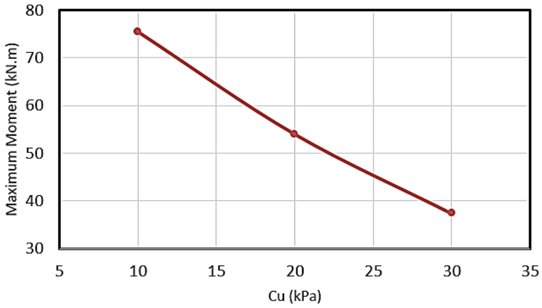


**Figure 23. Effect of Soft Clay Shear Strength on the Maximum Generated Bending Moment along the Edge PI.**

### F. Position of Pile Inclusion (PI)

As shown in the previous sections, there is a major difference in the values of the maximum generated bending moments along PI between middle and the edge inclusions leading to a noticeable difference in the required reinforcement percentage. Consequently, additional analysis was carried out – using the basic studied case setup shown in Fig. 9 – to investigate the maximum generated bending moments along a row of pile inclusions. Fig. 24 shows a horizontal profile for the maximum generated bending moment along a row of pile inclusions showing that the outermost pile inclusions are subjected to extremely high moment values that decrease gradually by approaching the centre of the improved zone till reaching the middle pile which is subjected to the minimum generated dynamic bending moment in comparison with other pile inclusions.

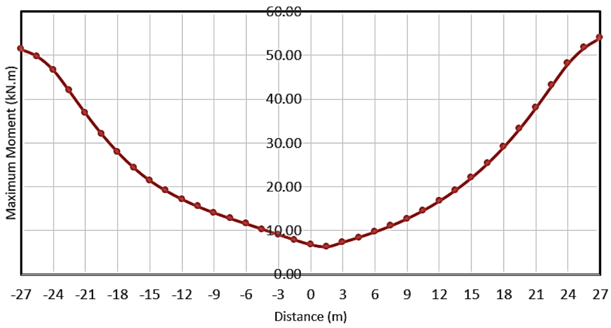


**Figure 24. Horizontal Profile for the Maximum Generated Bending Moments along a row of the Pile Inclusions**

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## VIII. CONCLUSIONS

The main and primary aim of this study was to investigate the mutual interaction between rigid pile inclusions and the surrounding soft soil formation (soft to medium stiff silty Clay). The studied project which is NMU (New Mansoura University) is located at New Mansoura City, Nile Delta – and as an area known for the existence of problematic soil – it was proposed to adopt deep foundations solution (bored piles). After further studies, analyses and comparisons, it was decided to improve the properties of the soil mass using Rigid Pile Inclusions enhancing the soil bearing capacity and reducing the expected settlement to cope with the limits set by the Egyptian Code for Soil Mechanics and Foundations Design and Construction (ECP-202-2001). Consequently, shallow foundation may be used and rested safely on the improved soil. The soil profile consists of a surface silty Sand layer with a depth of 4m, followed by a 15.0m-thick Soft Clay formation underlaid by a medium dense to dense sand layer extending to a depth of 40m followed by a deep very dense sand layer extending for 20m and rested on the Bedrock surface.

To perform such a complex seismic geotechnical analysis, a Finite Element Model was developed using PLAXIS-2D where rigid piles inclusions are modelled using the embedded beam row feature. The seismic analysis is performed in the time domain, using artificial time history (20.0 sec.), generated by [16].

An extensive parametric study is carried out to investigate the effect of the following parameters on the maximum developed dynamic bending moment diagram along middle and edge pile inclusions.

1- Penetration Length of Pile Inclusion into Sand Layer
2- Diameter of Pile Inclusion
3- Peak Ground Acceleration (PGA)
4- Shear Strength of Soft Clay Formation

The analyses results are shown in neat figures showing the relations between the studied parameters and the maximum generated bending moment along pile inclusions. In summary, the following is concluded:

1- The generated bending moment diagrams along both middle and edge pile inclusions has two peaks; the upper one is located along the pile inclusion at the penetration length inside the Soft Clay layer while the lower peak is recorded near the level of the interface between the Soft Clay and the following Sand layer. In general, it can be concluded that the maximum value of the developed bending moment is near this lower (interface) level.
2- The maximum developed moments on edge piles are almost more than seven times the maximum developed moments on middle piles. Meaning that the location of the pile significantly affects the value of the developed dynamic moment. This logical finding could be clarified by the fact that the middle pile is well surrounded by successive piles enhancing the lateral support provided to the pile and reducing the seismic response of these piles (middle and nearby piles). On the contrary, the edge pile is surrounded by piles from one side only and that's why it lacks lateral support from the other side. Similarly, the seismic response of the Soft Clay layer at the location of the edge pile is greater than that at the location of the middle pile.
3- The penetration length inside the Sand layer has a considerable effect on the generated bending moment along the pile inclusion as shown in Fig. 15 & 16 whereas the Penetration Length into Sand Layer (1) decreases, there is a reduction in the value of the generated bending moment by approximately 17.3% and 41.4%, for the middle and the edge PI, respectively.
4- Pile Inclusion diameter has a significant effect on the generated bending moment. By increasing the PI diameter from 0.40-m to 0.60-m, the maximum generated bending moment on the PI increased from 3.14-kN.m to 11.73-kN.m and from 29.87 kN.m to 89.54-kN.m for the middle and edge pile inclusions, respectively.
5- There is direct proportionality between the PGA and the maximum generated bending moment along the inclusions. For the middle PI, increasing the PGA from 0.1g to 0.2g was accompanied with a massive increase in the generated bending moment which increased from 6.87-kN.m to 34.92-kN.m, representing an increase of 408%. Similarly, for the edge PI, increasing the PGA from 0.1g to 0.2g resulted in a subsequent increase in the generated bending moment from 53.97-kN.m to 139.5-kN.m (% increase = 158.5%). These results demonstrate the strong influence of the PGA values on the generated bending moments, especially for the middle pile.
6- Increasing the soft clay strength means reducing the relative difference in stiffness between the soft clay formation and the following sand layer resulting in reducing the relative seismic excitation between these successive layers. Consequently, this increase in strength and stiffness will reduce the generated bending moment along the PI.
7- The dynamic bending moment acting on pile inclusions should be resisted by reinforcement to avoid flexural failure during dynamic loading conditions. Considering the flexural and compressive stresses acting on edge piles – which are the critical pile inclusions – the structural design for the piles' sections showed that the required reinforcement varies depending on the acting PGA where – as a practical finding – Fig. 21 shows that for PGA ranging between 0.1g and 0.2g, the required reinforcement percentage (As/Ac) ranges between 0.25% and 0.80% respectively.

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## IX. RESEARCH RECOMMENDATIONS

It is recommended to conduct further studies on the degree of confinement of the rigid pile inclusions as it cannot be considered fully confined nor unconfined throughout the pile length and this may affect the structural design of the pile inclusion. Furthermore, as the required reinforcement percentage is dependent on the pile inclusion location, further research may provide recommendations and formulations for the structural design taking several factors into considerations like pile location, degree of confinement, peak ground acceleration, etc. Lastly, cases including high risk of liquefaction should be included in future research approaches.

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