Numerical Analysis of a Pile-Soil System under Earthquake Loading

Qassun S. Mohammed Shafiqu
Department of Civil Engineering
College of Engineering
Al-Nahrain University
qassun@yahoo.com

Ruba H. Majeed Sa'ur
Department of Civil Engineering
College of Engineering
Al-Nahrain University
rubasaur@yahoo.co.uk

Abstract

Due to significant increasing in seismic activity in world during the last decades especially in Middle East region; engineers have been giving increasing attention to the design of buildings for earthquake resistance. In this study 3-D seismic behavior of piles is investigated using the finite element program PLAXIS 3D 2013.

Piles are one of the most commonly used foundations in seismic areas where the soil is inadequate to carry the load on its own. In these seismic areas, piles often pass through (penetrate) shallow loose and/or soft soil deposits and rests on competent end bearing soils. Thus a model of soil - pile system is simulated in the finite element program.

The dynamic parameters of soil are used as input dynamic data of PLAXIS 3D program, in addition to the static properties of soil collected from soil investigation works.

The research showed the susceptibility of PLAXIS 3D program in analyzing piles with different soil conditions under earthquake action. The results also showed the importance of studying seismic behavior of soil-pile system using 3-D analysis rather than 2-D analysis because the problem is truly 3-D and should be analyzed as such.

Keywords: Finite Element Analyses, Three-Dimensional, Kinematic Bending Moment, Seismic Behavior, Pile.

1. Introduction

The analysis of structures subject to earthquake ground motions must properly account for the interaction between the foundation and the superstructure. The passage of seismic waves through the foundation affects the ground motion at the base of the structure and generates stresses on foundation elements. This effect is termed *kinematic interaction* and its effects on the ground motion are described by a function termed the *transfer function*. On the other hand, the response of a structure is a function of the foundation compliance, and, in turn, inertial forces resulting from structural response affect the stresses on foundation elements. This interaction is termed *inertial interaction* and is captured by representing the foundation through an *impedance function.*

Predicting the behavior of piles and pile groups during earthquakes still remains a challenging task to geotechnical engineers. In most of the published results on the dynamic analysis of pile foundations (e.g., Kaynia and Kausel 1982 [1], Dobry and Gazetas 1988 [2], Makris and Gazetas 1992 [3]), soil has been considered as a linear elastic material. Material linearity permits analyses in the frequency domain where the principle of superposition can be used to superimpose loading at different frequencies. However, under strong seismic excitation, nonlinearity of the soil medium and separation at the soil-pile interface can have significant influence on the response of the pile. Therefore, the response analysis should be carried out in the time domain in order to properly incorporate soil nonlinearity as well as to account for the separation at the soil-pile interface.

In this study, the three-dimensional finite element analyses are performed using finite element computer software PLAXIS 3D 2013 which is capable of modeling the soil-pile system, embedment pile element (friction or end-bearing) and seismic behavior of the system using the dynamic properties and earthquake data.

2. Theoretical Work

2.1 Kinematic Bending Moment of Pile Under Seismic Motion

Khari et. al., (2014) [4], developed a 2D Finite Element model to evaluate the kinematic bending moment of a single pile at interface of two layer soil model under seismic excitations. The results of this simulation were used to verify the results of simplified approaches. The simplified approaches are existing design methods for evaluating the kinematic interaction between soil-pile subjected to the seismic excitations developed by Dobry and O’Rourke (1983), Mylonakis (2001) and Nikolaou et al.(2001).

2.1.1 Simplified Approaches

Dobry and O’Rourke (1983) [5], developed the first formula for evaluation of the kinematic bending moment at the interface between two layers of soil by modelling the pile as Beam on

Nonlinear Winkler Foundation BNWF. They assumed each layer of the soil is homogenous and isotropic with the shear module G_1 andG_2 , E_p and I_p are the pile elastic modulus and the pile moment of inertia, respectively. The shear strains are calculated with $\gamma_i=\tau/G_i$. The pile bending moment at the interface between two layers:

$$M = 1.86\,(E_p I_p)^{\frac{3}{4}}(G_1)^{\frac{1}{4}}\gamma_1\,F \qquad (1)$$

Where F is a function of the ratio c:

$$c = ({}^{G_2}/_{G_1})^{\frac{1}{4}} \qquad (2)$$

$$F = \frac{(1-c^{-4})(1+c^3)}{(1+c)(c^{-1}+1+c+c^2)} \qquad (3)$$

$$\gamma_{1=}{}^{r_d \rho_1 H_1 a_{max,s}}/_{G_1} \qquad (4)$$

Where $a_{max,s}$ is the maximum acceleration at surface based on seismic zonation; H_1 and ρ_1 are the thickness and the density of the upper layer, respectively. $r_{d=}$ (1-0.05z) is the depth factor; z is the depth from the ground surface (only $z \leq 15$ m). This simplified method does not consider the nonlinear behavior of soil.

Nikolaou et al (2001)[6], developed another simplified method based on the Beam on Nonlinear Winkler Foundation BNWF model. The kinematic pile bending moment is expressed by the following equation:

$$M = 0.042\tau_c d^3 (\frac{l}{d})^{0.3} (\frac{E_p}{E_1})^{0.65} (\frac{V_{s2}}{V_{s1}})^{0.5} \qquad (5)$$

WhereV_{s1} and V_{s2} are the shear wave velocity in the upper and lower layer, respectively. τ_c is the maximum shear stress at the interface, E_p and I_p are the pile elastic modulus and the pile moment of inertia, respectively.

Mylonakis (2001) [7], presented the second simplified method after the Dobry & O'Rourke formula . The assumptions are the same of the Dobry & O'Rourke model: the soil profile is constituted by two layers of homogeneous linear elastic soils; both layers are assumed to be thick. Both of the radiation and the hysteretic damping were taken into account. The seismic excitation is a harmonic horizontal displacement imposed at the bedrock. Base on his studies, the maximum bending moment expressed as:

$$M = {}^{(E_p I_p)\left(\frac{\varepsilon_p}{\gamma_1}\right) Q \gamma_1}/_{r} \qquad (6)$$

While r is the pile diameter; γ_1 is the strain of the upper layer; Q is an amplification factor so that its value is less than 1.25(usually Q is equal to 1). ε_p/γ_1 is the strict strain transfer function that can be computed by the following equation:

$$\frac{\varepsilon_p}{\gamma_1} = \left(\frac{c^2-c+1}{2c^4}\right)\left(\frac{H_1}{d}\right)^{-1}\left[\left[3\left(\frac{K_1}{E_p}\right)^{\frac{1}{4}}\left(\frac{H_1}{d}\right)-1\right]c(c-1)\right] \qquad (7)$$

$$k_1=\delta E_1 \qquad (8)$$

$$\delta = \frac{3}{1-\upsilon^2}(\frac{E_p}{E_1})^{-\frac{1}{8}}(\frac{L}{d})^{\frac{1}{8}}(\frac{H_1}{H_2})^{\frac{1}{12}}(\frac{G_1}{G_2})^{-\frac{1}{30}} \qquad (9)$$

Where G_1 andG_2 the shear module, E_p and I_p are the pile elastic modulus and the pile moment of inertia, respectively. ν is the Poisson's ratio; the free-field site analysis is suggested for estimating the peak shear strain (γ_1) and calculated by Equation (4).

This procedure does not consider the nonlinear behavior of the soil.

2.1.2 Overview and Model Information

The kinematic bending moment of a 2D FE model is evaluated using 2D PLAXIS code, the overall dimensions of the model boundaries included a width of 11D (D=pile diameter) and a height equal to the thickness of the two subsoil layers Figure (1,a). The model was meshed by 15-node wedge elements. While, the horizontal outer boundary mesh of the model was fixed against displacements (u_x, u_y) but the vertical outer boundary, only, was fixed in the horizontal displacement (u_y), Figure (1,b) shows the outer boundaries, absorbent boundary conditions were used to absorb outing waves. The surrounding soil was considered as Mohr-Coulomb model and the single pile was considered as linear-elastic material model. The soil-pile interaction was modeled by the interface element. Kinematic interaction have been performed for a single pile with a length L=20 (m); Young's modulus E_p = 2.5x107(kN/m^2); diameter=60 (cm); mass density ρ_p=2.5 (Mg/m^3) and Poisson's ratio ν=0.15.

As Figure (1,a) shows, the pile is embedded in ideal two-layered subsoil. The thickness of the second layer is assumed H_2= 15(m) while the thickness of the upper layer H_1 is variable (5, 10, 12, 15 and 18 m). The shear wave velocity of the upper layer V_{s1} is taken as 100 m/s, while V_{s2} is assumed equal to 2 of V_{s1} mass density and Poisson's ratio of the soil are: ρ_s= 1.97(Mg/m^3) and υ=0.4, respectively. The Young's modulus can be computed based on the shear modulus (E=2G(1+ν)). In addition, the undrained shear strength was calculated based on the ratio suggested by the Applied Technology Council (G_{max}/S_u=1000).

The average shear wave velocity can be computed by the following equation [4] :

(10)

$$V_{s,30} = \frac{H}{\sum_{i=1,N} \frac{h_i}{v_i}}$$

where H is the total depth of soil less than or equal to 30m, h_i and v_i denote the thickness (in metres) and shear-wave velocity of the i-th formation or layer, in a total of N, existing in the top 30 m. According to Eurocode 8 (2004) [8], the soil profiles can be classified as type D and C. Acceleration time history selected is scaled to the peak ground acceleration of 0.1(g). Figure (2) shows the acceleration time history and spectral acceleration selected at the bedrock roof.

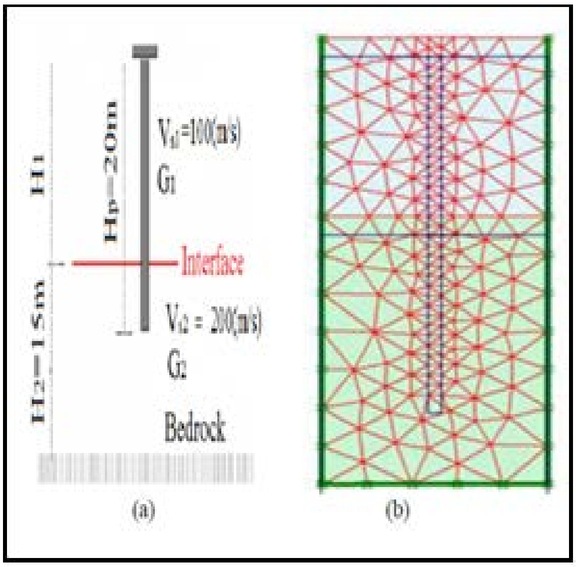

Figure 1: Reference scheme model (a) Soil model, (b) Typical 2D model for FE Analysis (after Khari, et. al., 2014 [4]).

2.2 Finite Element Modeling of Problem using PLAXIS 3D 2013

Kinematic bending moments at interface of a pile embedded in two-layered soil is evaluated using PLAXIS 3D 2013 software. Five models are simulated for the five depths of the upper layer of soil (H1=5, 10, 12, 15 and 18 m). The description of the modeling and results of analysis will be explained in the following subsections.

2.2.1 Dimensions and Boundary Conditions of the Model

The overall dimensions of the model are performed by assuming X= Y=11D = 6.6(m) (D= diameter of pile) as a 3D model Z is variable (Z= H_1+H_2). Use the default boundary conditions of PLAXIS 3D 2013 in which the vertical boundaries (parallel to yz plane are fixed in x direction u_x=0), (parallel to xz plane are fixed in y direction u_y=0) both are free in z direction, the bottom boundary is fixed in all directions (representing the bedrock roof), while the ground surface is free in all directions. The absorbent boundary conditions of outing waves are performed by making boundary $X_{max,min}$ and $Y_{max,min}$ viscous that waves are absorbed by the surrounding soils , Boundary $Z_{max,min}$ are None for unabsorbing bedrock roof.

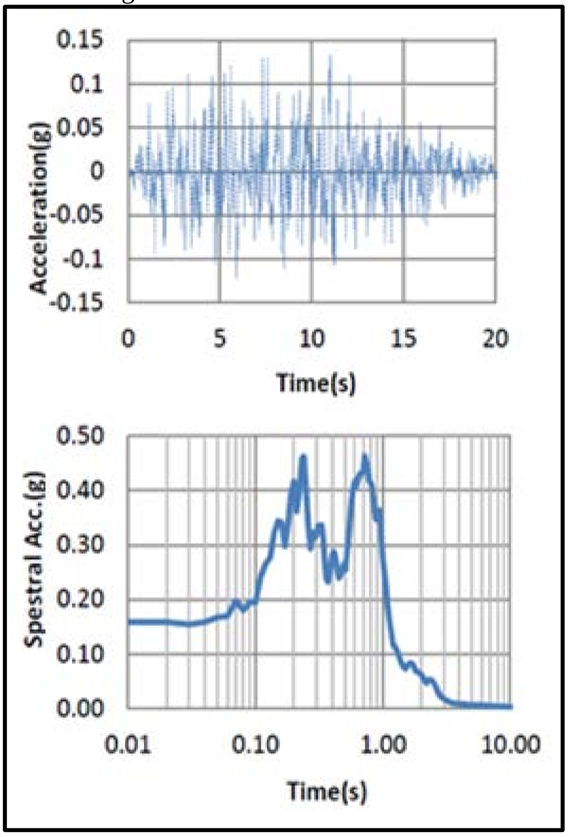

Figure 2: Acceleration time history and response spectra at the bedrock roof (after Khari, et. al., 2014 [4])

2.2.2 Soil and Interface Modeling

Soil layers are modeled by entering depths and material properties of both layers according to Table (1), water table at the ground level. As in the verifying study choose the model and drainage type as Mohr Coulomb and Undrained B, respectively. Damping ratio is assumed to be equal to 5% according to PLAXIS 3D Manual (2013) [9], Eurocode 8 (2004) [8] and the Preliminary draft of Iraqi Seismic Code, submitted to Central Organization for Standardization and Quality Control COSQC (2013) [10].

Table 1: Input Soil parameters

Parameter	Name	Soil 1	Soil 2	Units
Material model	Model	Mohr Coulomb	Mohr Coulomb	-
Drainage type	Type	Undrained B	Undrained B	-
Unit weight	γ_{sat} , γ_{unsat}	19.32	19.32	kN/m^3
Young's modulus	E	55160	220.64×10^3	kN/m^2
Poisson's	υ	0.4	0.4	

ratio				
Shear modulus	G	19.7×10^3	78.8×10^3	kN/m^2
Undrained shear strength	Su	19.7	78.8	kN/m^2
Angle of internal friction	ϕ	0	0	o
Shear wave velocity	Vs	100	200	m/s
Damping ratio	ξ	5	5	%
Interface strength	-	Rigid	Rigid	-

For V_{s1}=100m/s and V_{s2}=200m/s, using equation (10) to calculate the average shear wave velocity $V_{s,30}$. According to Eurocode 8 (2004) [8], Khari, et. al. (2014) [4] classified the soil profiles into type D for the present study soil profiles can be classified as ground types shown in Table (2).

Table 2: Ground type according to Eurocode 8 (2004) [8].

H_1	H_2	$V_{s,30}$	Ground Type
5	15	160	D
10	15	143	D
12	15	138	D
15	15	133	D
18	15	129	D

2.2.3 Pile Modeling

Pile is modeled by its dimension (D=0.6m) ,(L=20m) and material properties (Ep=2.5x107 kN/m2), (γ=24.525 kN/m^3), (υ=0.15).It is embedded as shown in Figure (3).

2.2.4 Earthquake Modeling

Earthquake is modeled as dynamic prescribed displacement in the x-direction at the bedrock level; the readings of the earthquake are entered as a table of time acceleration records in (s) and (m/s^2), respectively. Acceleration-Time records of earthquake shown in Figure (4).

2.2.5 Mesh Generation

Unlike the 15-node triangular element of 2D PLAXIS, the 3D PLAXIS Finite Element mesh consist of 10-node tetrahedral element. Mesh is generated as shown in Figure (5).

2.2.6 Performing Calculations

Calculations are performed through dividing the calculation process in to multi Phases .

- *Initial Phase* is the first phase generated to calculate in Soils and Interfaces. In PLAXIS 3D K_0 procedure is a special calculation method available to define the initial stresses for the model , for Mohr Coulomb the default K_0- value is suitable based on Jaky's empirical expression where K_0 is related to the friction angle [9]:

$$K_0=1-\sin\phi \qquad (11)$$

- *Second phase* generated to calculate Pile stresses using plastic calculation method.
- *Third phase* generated as a dynamic calculation method to calculate earthquake stresses, the dynamic time interval is set to 20(s).

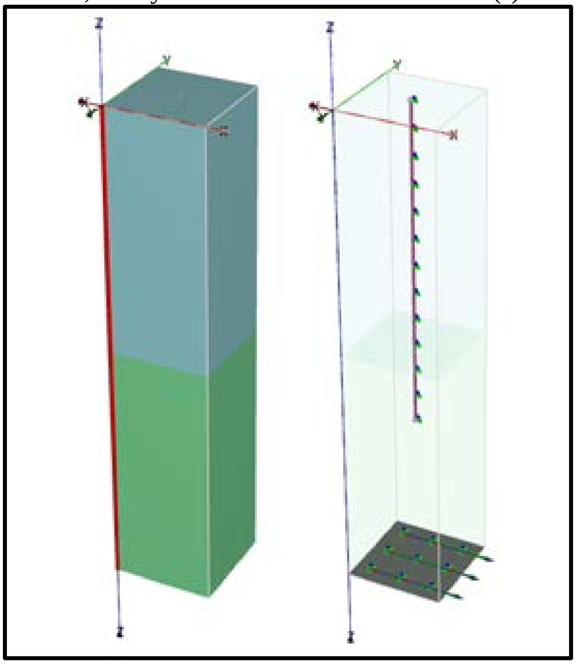

Figure 3: 3D Soil and pile model by PLAXIS 3D 2013.

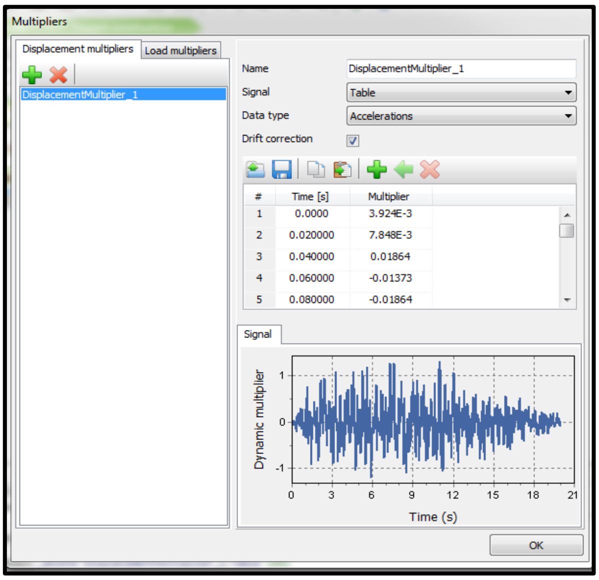

Figure 4: Earthquake acceleration-time records.

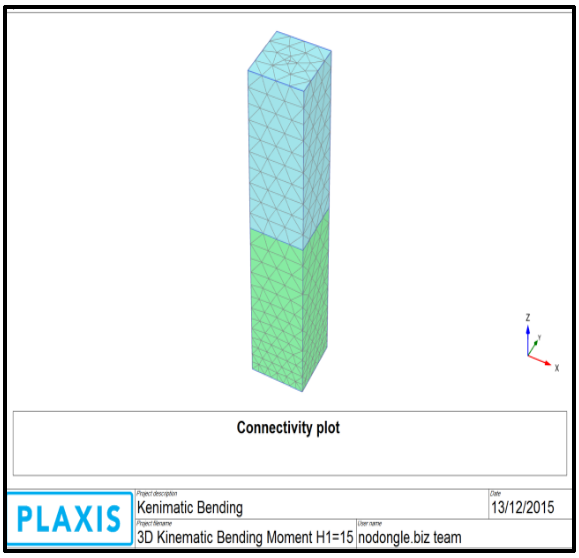

Figure 5: Mesh generation of 3D model.

2.2.7 Analysis Results

The finite element analyses of the five models are set up to determine the kinematic bending moments at pile interface due to earthquake excitation. Remodeling the present study into 2D shape by reducing the Y dimension, the 2D dimensions (X=6.6m and Y=1m) as shown in Figure (6). Figure (7) shows the kinematic bending moment at the interface of the two layered subsoil, the results show that moments are increased with increasing first layer thickness. The 3D model results are higher than 2D calculated by Khari et. al. (2014) results specially at H_1=10, 12 and 15 this occurs due to the effect of 3D modeling. The results of assuming 2D of present study is too close to 2D of Khari et. al. (2014).

Kinematic bending moments at the interface of the two layers were calculated using the simplified approaches developed by Dobry and O'Rourke (1983), Mylonakis (2001) , and Nikolaou et al. (2001) then compared with the moments of 3D PLAXIS model as shown in Figure (7). The 3D PLAXIS moments are close to Dobry and O'Rourke (1983) moments particularly at H_1=5 and 18. The 3D PLAXIS curve is similar in behavior to Nikolaou et al. (2001) curve with lower values of moments.

2.2.8 Conclusions

The results of the dynamic analysis of the kinematic bending moments of the single pile using PLAXIS 3D 2013 program are compared with Khari et. al. (2014) 2D PLAXIS model and the simplified approaches in the two layers subsoil. The following conclusions may be drawn:

1- The nonlinear behavior of soil under earthquake excitation wasn't considered in all the mentioned simplified approaches. In Dobry and O'Rourke (1983) and Mylonakis (2001) approaches, it was assumed that the seismic excitation as a harmonic horizontal displacement imposed at the bedrock using the variable $a_{max,s}$ of Equation (4). Nikolaou et al. (2001) consider V_{s1} and V_{s2} as dynamic variables of Equation (5), while in PLAXIS 3D 2013 the acceleration–time history data was entered as a prescribed displacement at the bedrock of the model in addition to dynamic parameters of soil including wave velocities. It is concluded that the kinematic bending moment values are affected by the method of analysis used.

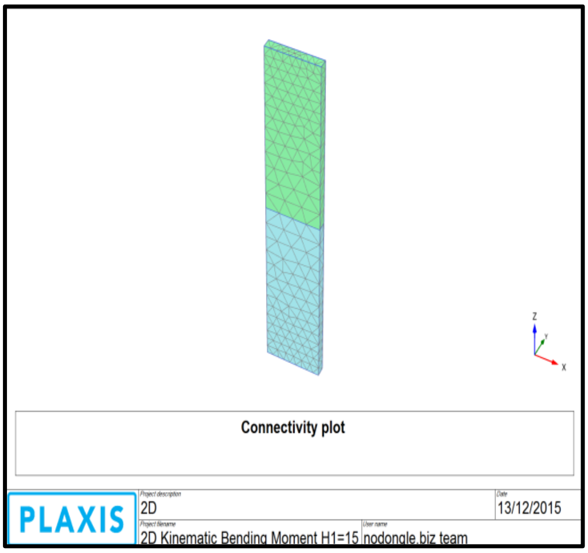

Figure 6: Model of 2D present study by PLAXIS 3D 2013.

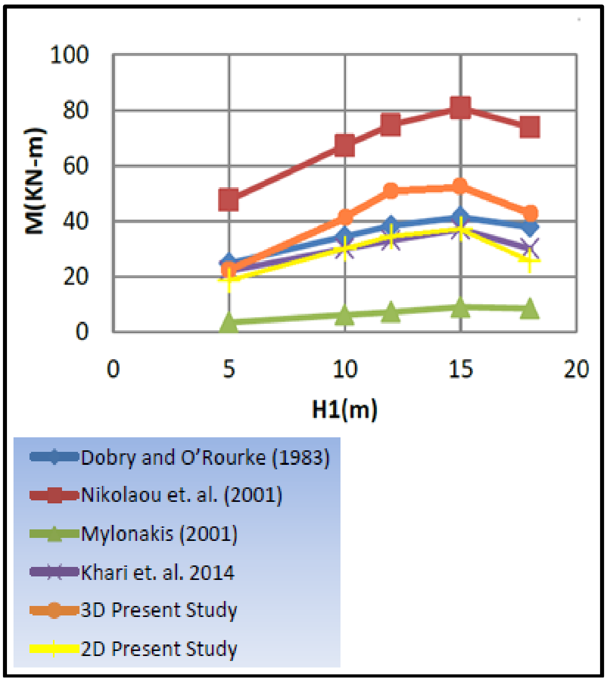

Figure 7: Comparison between PLAXIS 3D results of present study and results of 2D Khari et. al.,(2014) and simplified approaches' results.

2- As the first layer depth increased the kinematic bending moment at the interface of the two layers increased to reach the maximum amount at H_1=15m. The kinematic pile moments during earthquake shaking occurs at relatively deep interfaces between soil layers with very different stiffnesses.

3- The kinematic bending moment at the interface of the two layers decreased at H_1=18m, this is may be due to increasing the distance between the pile tip and the source of excitation knowing that 90% of pile length embedded in the first layer with $V_{s1}<V_{s2}$.

4- The evaluated ground type in Table (2), are of type D.

5- After comparing the results of 3D PLAXIS and the assumed 2D models of the present study with the results calculated by Khari et. al. (2014) then finding out that the increased moment values at H_1=10, 12 and 15 occurs due to the effect of 3D modeling which represents the reality and should be analyzed as such.

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التحليل العددي لنظام ركيزة-تربة تحت تأثير قوة زلزال

قاسيون سعد الدين محمد شفيق **ربى حنا مجيد ساعور**
قسم الهندسة المدنية - جامعة النهرين

الخلاصة

نتيجة للزيادة الملحوظة للنشاط الزلزالي في العالم خلال العقود الأخيرة خاصة في منطقة الشرق الأوسط فقد اولى المهندسون اهتمام كبير في تصاميم الأبنية أن تكون مقاومة للزلازل. في هذه الدراسة تم التحري عن التصرف الزلزالي ثلاثي الأبعاد للركائز باستخدام برنامج PLAXIS 3D 2013 المعتمد على طريقة العناصر المحددة.
الركائز واحدة من الأسس الاكثر شيوعا في المناطق الزلزالية عندما تكون التربة غير قادرة على تحمل القوى المسلطة عليها. في هذه المناطق الزلزالية فان الركائز المستخدمة ستخترق الطبقات الضعيفة او الرخوة الضحلة وترتكز على الترب القوية عند القاعدة ولهذا فان نموذج يمثل نظام تربة-ركيزة سيتم تحليلها باستخدام البرنامج المعتمد على نظرية العناصر المحددة.
ان الخصائص الديناميكية للترب سوف تستخدم كمدخلات لتمثيل البيانات الديناميكية اللازمة في برنامج PLAXIS 3Dبالاضافة الى الخصائص الاستاتيكية للتربة والتي يتم جمعها من اعمال تحريات التربة.
ان الدراسة أظهرت قابلية برنامج PLAXIS 3D على تحليل مسائل الركائز في حالات التربة المختلفة تحت تأثير الزلازل. كما وأظهرت النتائج أهمية دراسة التصرف الزلزالي لنظام تربة-ركيزة باستخدام تحليل ثلاثي الأبعاد بدلا من التحليل ثنائي الأبعاد كون المسألة هي في الواقع ثلاثية الابعاد ويجب ان يتم تحليلها على هذا الأساس.

الكلمات المفتاحية: تحليلات العناصر المحددة، ثلاثي الابعاد، عزم الانحناء الكنيماتي، السلوك الزلزالي، ركيزة