RESPONSE OF PILE FOUNDATIONS UNDER CYCLIC LOADINGS

KAZIKLI TEMELLERİN TEKRARLI YÜKLER ALTINDA DAVRANIŞI

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ÖZET

KAZIKLI TEMELLERİN TEKRARLI YÜKLER ALTINDA DAVRANIŞI

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Master of Science, Department of Civil Engineering
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Bu çalışmanın amacı zemin, kazık ve deprem parametrelerinin homojen ve temiz kuma gömülü kazıkların deprem yükü altında yanal yer değiştirmesine etkisini araştırmaktır. Bu amaçla farklı zemin-kazık kombinasyonları üzerinde üç boyutlu, sonlu farklar yöntemine dayanan sayısal dinamik analizler gerçekleştirilmiştir. Analizlerde kullanılan zeminlerin rijitlikleri, kayma dalga hızları üzerinden belirlenmiştir ve kayma dalga hızı değerleri 100m/s ve 200 m/s'dir. Bu kapsamda yapılan analizlerde, kazık-zemin rijitlik oranı (E_p/E_s), kazıklar arası mesafe (S) ve maksimum yer ivmesi (PGA) ve kayma dalga hızı (V_s) en önemli değişkenler olarak belirlenmiştir. Yapılan analizlerde PEER NGA-W1 veri tabanından seçilen üç adet kaya üzerinde kaydedilmiş kuvvetli yakın yer hareketi ivme kayıtları (kırılma mesafesi 20 km'den küçük olan) kullanılmıştır. Maksimum yer ivme değeri 0.1g ve 1g arasında değişmektedir. Böylelikle geniş bir aralıkta yer alan ivme değerlerinin kazık yatay yer değiştirmesi üzerindeki etkisi incelenmiştir. Yapılan analizler sonrasında, kazık başlarının yanal yer değiştirme değerleri üzerinde söz konusu parametrelerin hepsinin etkili olduğu sonucuna varılmıştır.

Anahtar Kelimeler: Kazıklı temeller, Kazık başı yer değiştirmesi, Kazık-zemin rijitlik oranı, Maximum yer ivmesi, Dinamik sonlu farklar analizi.

ABSTRACT

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The aim of this thesis is to investigate effects of different soil, pile and ground motion parameters on lateral displacement of the piles embedded in homogeneous and clean sand under cyclic loading. For this purpose, an intensive numerical analyses scheme that includes three-dimensional, finite-difference based dynamic analyses on generic soil-pile combinations has been performed. The rigidity of the soils is defined by the shear wave velocities (V_s) of the soils. These shear wave velocities values are 100m/s and 200 m/s. In the analyses performed within this scope, the stiffness ratio between the pile and the soil (E_p/E_s), spacing between piles (S), shear wave velocity (V_s) and peak ground acceleration (PGA) are selected as important parameters. In the analysis the PGA values of the three near fault recording (rupture distance is less than 20 km) on rock sites having a large range of variability in the ground shaking recorded on rock sites selected from PEER NGA-W1 database were used. The peak ground acceleration value varies between 0.1g and 1g. Thus, the effect of a wide range of acceleration values on the horizontal displacement of the pile was investigated. According to the results of the analysis performed, all parameters was found effective on lateral displacement of the pile.

Keywords: Pile foundations, Pile head displacement, Pile-soil stiffness ratio, Peak ground acceleration, Dynamic finite difference analysis.

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SYMBOLS AND ABBREVIATIONS

Symbols

A_s	Pile surface friction cross-section area
A_p	Cross-sectional area at the end of the pile
b	Width of beams
B	Width of footing
B	Least lateral dimension of footing
c	Cohesion of the soil at depth z
c_u	Undrained shear strength
c'	Cohesion under drained conditions
C	A constant depending on type of hammer
C_m	Moment amplification factor
C_y	Deflection amplification factor
d, D	Diameter pile (m)
e	Pile length above soil
E_p	Pile or drilled shaft elasticity
E_s	Elasticity modulus of soil
EI	Flexural rigidity of footing
f_s	Unit friction resistance,
h	Efficiency of hammer
H	Height of fall of hammer
I	Inertia moment
I_p	Moment inertia of pile
k	Winkler spring stiffness or subgrade reaction modulus
K_a	Rankine coefficient of active earth pressure
k_h	Lateral subgrade reaction modulus
k_{sl}	Coefficient of subgrade reaction for a plate 1 ft wide
K_o	Rankine coefficient of earth pressure at rest
K_p	Rankine coefficient of passive earth pressure
K_{qz}	Passive pressure coefficients for the frictional components
K_{cz}	Passive pressure coefficients for the cohesive components

L	Pile length
M	Moment
M_c	Characteristic moment
M_s	Maximum moment in a single pile
M_g	Maximum moment in a pile in the group
n	Number of intervals throughout the pile
$n_{h:}$	Constant of lateral subgrade reaction modulus
N_c, N_q	Terzaghi's bearing capacity factors
N_γ	Terzaghi's bearing capacity factors
P	Soil resistance
P_c	Characteristic Load
P_z	Lateral load for each element (n number)
P_{oz}	Effective overburden pressure at depth z
q_p	Maximum bearing capacity of the ground at pile end
Q_p	Ultimate end resistance.
Q_s	Ultimate friction resistance and
Q_u	Maximum bearing capacity of single pile
R	The stiffness factor of granular soil
R_i	Moment of inertia ratio
s	Spacing of between to piles
S	Final set (cm/blow)
S_u	Undrained shear strength of clay
T	The stiffness factor of granular soil
w	Collapse in vertical direction
W	Hammer weight
W_p	Pile weight
X	The turning point of pile
x_o	Distance of maximum moment to ground surface
x_r	Critical depth
y	Deflection at any point
y_s	Single pile deflection
y_g	Group deflection
Z_f	Depth from the ground surface

γ ‘	Effective unit weight of soil
φ'	Effective stress friction angle of sand
υ	Poisson’s ratio
μ	Non-dimensional soil mass per unit length
σ_v'	Effective vertical pressure at the level considered
δ	Angle of friction between pile and soil
θ	Rotation

Abbreviations

COY	Coyote Lake earthquake
FF	Free field
NAH	Nahanni earthquake
MC	Middle Center
PC	Pile Corner
WHI	Whittier-Narrowearthquake

1. INTRODUCTION

The construction of foundations has been available since very old times. The utilization of deep foundations continued in the construction of buildings from the Neolithic inhabitants to the Babylonians, from the Babylonians to the ancient Egyptians. Also, the ancient Greeks possessed knowledge of deep foundation whereas the greatest development in foundation engineering was achieved in ancient Rome.

Later due to increase in population, technological reasons, social reasons such as increase in urban land values, construction of structures such as skyscrapers, power stations, marine and coastal structures, bridges and viaducts, the deep foundations have become important throughout the world.

First high-rise building was built in 19th century in Chicago. Due to the enormous vertical and horizontal loads transferred from these high-rise structures to the foundations, bearing capacity and settlement conditions cannot be provided using shallow foundations. This resulted in either soil improvement or construction of deep foundations. Pile foundations are generally utilized for soil improvement and minimization of liquefaction but there is not enough exploration on this subject that validates the reply of pile foundation systems under dynamic loading.

This thesis' goal is to explore the impact of different soil, pile and earthquake parameters on lateral displacement of piles embedded in homogeneous and clean sands under cyclic loading conditions. In accordance with this purpose, a great number of numerical analyses based on 3D, finite-difference based dynamic analyses on soil-pile combinations has been done. Four near fault ground motions recorded on rock sites are chosen from the PEER NGA-W1 database (Chiou et al., 2008).

In Chapter 2 an overview on laterally loaded pile foundation behavior is presented.

In Chapter 3 numerical analysis procedure, soil and pile properties and choice of input motions used in analysis are presented in detail.

Chapter 4 discusses the result of analysis (lateral displacement of piles) in connection with the selected parameters, i.e. center-to-center spacing, elasticity proportion of pile to soil, shear wave velocity and peak ground acceleration.

Finally, in Chapter 5, conclusions of analysis are discussed and suggestions about the result of analysis are presented

2. AN OVERVIEW ON PILE FOUNDATION PROPERTIES AND LATERALLY LOADED PILE FOUNDATION BEHAVIOR

2.1. Introduction

The present chapter presents a global review of literature in respect of laterally loaded pile and a discussion of the methods, which are available in order to assess the interaction of pile-soil-pile.

Pile foundations are generally exposed to static, cyclic or dynamic loads. Numerous studies have been conducted in the literature about the behavior of piles under static loads but in fact, soil-pile interaction under earthquake loading is a complex problem. Piled foundations are damaged by two different dynamic effects during the earthquake: The first is the moment of inertia caused by the oscillation of the superstructure as because of the earthquake motion passing from the earth to the superstructure, which is more effective in the region near the pile head, the other is kinematic effects caused by the displacement difference between soil and pile due to the stiffness disparity among the pile and soil during the transition of the earthquake movement through the soil layers. As a result, many piled foundations have failed catastrophically due to these cyclic loads, causing overturning or collapse of major pile-supported structures. But studies corresponding to behavior of lateral loaded piles under dynamic and cyclic loading are limited in literature.

In literature generally, analysis of lateral loaded piles is based on information obtained from field or laboratory model tests. These methods are not considered very reliable because they are mostly applicable to homogeneous soil conditions and arbitrary selection of soil parameters. In order to forecast the characteristics of laterally loaded piles, new approaches are available; such as, centrifuge tests. Centrifuge modeling technique is implementable in economical and practical substitute to full-scale field experiments. But most of research institutes don't have this device so this case is a limitation for this method.

Also, in the literature, simplified analysis methods were generally used to examine laterally loaded behavior. But these methods had some limitations. Some of them are as follows:

Winkler (1867) assumes homogenous soil whereas the soil is not homogenous and therefore subgrade reaction modulus vary from point to point and deformation of foundation is confined only at the point of influence. This method neither takes the nonlinearity of soils into account nor considers the interaction between springs.

Another method is subgrade reaction modulus. But method ignores the continuity and elasto-plastic behavior of the soil. Plastic functioning of soil is added in the solution in p-y curves method therefore this solution is more accurate than previous method. But p-y curves method ignores the continuity of the soil and dependence of p-y curves on pile width (B). The relation of p-y curves to shape and depth is not explicit in the literature.

Another method is Characteristic Load Method (CLM) proposed by Duncan in 1994. According to the primary studies the CLM has some restrictions, such as the need of soil to be modelled as a homogenous stratum and pile has to have a persistent bending inflexibility its height. Additionally, deformations are exaggerated in some case in this model and it isn’t proper for cyclic loads in stiff clays.

Yet another methodology was developed by Blum (1932). Blum method is not capable of finding the ultimate resistance of the soil, calculations are not feasible under working loads and the soil is assumed to be a single layer of soil.

Another method proposed by Brinch Hansen (1961). Restriction of Brinch Hansen’s method is, it doesn't calculate deflections that take lots of time due to iterative form at the ultimate load. It is valid only for short piles. The final method is Brom’s method (1964). According to this method, the reaction of pile-soil is presumed to be linear stretchy. Another limitation of this method is not taking the axial pile load into account.

The deficiencies of each method resulted in developing a new method, which enabled the elimination of the deficiencies of the previous methods and the further development so that analysis can be carried out more accurately than before. Since the advent of the computer technology, numerical analysis methods have been implemented in soil mechanics and foundation engineering problems for the last twenty years. With the help of FLAC 3D, one of the softwares developed by advanced techniques which is based on finite difference method, the behavior of horizontally loaded piles examined in more detail within the range of this research. The finite difference method considers the nonlinearity of stress-deformation behavior.

According to UFC (2004) and Fan and Long (2005), the factors affecting the behavior of laterally loaded pile are:

i. Stress-deformation behaviors of soil (shear strength of soil, stiffness of the soil, volume change character of soil etc.)
ii. Pile-soil interface
iii. Loading conditions
iv. Pile properties (pile stiffness, pile geometry)

Specially, pile group configuration plays an important role in soil-pile interaction. Again, studies on pile group behavior for both lateral and vertical loads are limited in the literature but the interaction of pile in the group needs to be studied well.

In this study response of pile group configurations with different parameters such as shear wave velocity, the proportion of elasticity modulus of piles to elasticity modulus of soil, PGA under cyclic loading was examined by using finite difference method in detail.

2.2. Recent Studies on Behavior of Piles under Lateral Loads

Raju et.al. (2017) examined single and group piles replies embedded in loose and dense sands under lateral loading conditions. It was found that as length over diameter (l/d) ratio increased, the lateral load carrying capacity increased and also for each l/d ratio as the spacing over diameter (s/d) ratio increased, lateral load carrying capacity increased. Furthermore, as density of sand increased the lateral load also increased. Also, there was direct proportion between the group efficiency and spacing between piles. Also, displacement curve was nonlinear. And as built-in length of pile increased, pile lateral resistances increased due to raising in passive resistance. As the s/d ratio raised the lateral strength raised due to decrease in pile-soil interaction effects. Result of this research show that the ultimate lateral load of pile group increased as density of sand increased and this increase was higher than the increase in the single pile.

Gouw (2017) investigated the impact of the pile lateral movement, pile interval and pile numbers on the laterally loaded pile groups embedded in clay by using Plaxis 3D. Piles were taken as circular bored piles with 1 m diameter. Soil was modelled according to Mohr coulomb criteria. Piles were modelled as beams. 3x3, 5x5, 9x9 pile groups were examined with different pile spacing (from 3D to 10D, where D is the pile diameter). Pile cap thickness for all pile groups was 2D. According to the result for pile spacing less than 5D, the larger lateral movement of the pile was obtained. For pile spacing between 5D and 6D, the lateral efficiency did not change with pile movement. For pile spacing larger than 6D, the lateral efficiency increased with pile movement. However, the changes of the lateral efficiency was minor and it could be neglected. Also, according to the results, as number of piles in the group increased, pile head lateral movement was increased. Especially for spacing of 10D, lateral movement value gets closer to the single pile's lateral movement.

Gatmiri et.al. (2011) explored the effects of pile head conditions and p-y behavior of individual piles by using ABAQUS software. Load distributions in each row and each pile were investigated and crosschecked with centrifuge test result. The results illustrate that the load carried by each pile in the group was depend on its location in the group, the load carried by lead row was the maximum and trail row piles carried the minimum load in 3×3 pile group with cap. In free-head group, the lead row carried the maximum load in the group, and the trail and middle row had same load distributions. Distinction between side and middle piles in a row was less than for free-head group than group with a cap. But all of rows both with cap and without cap the side piles carried in excess of load than middle.

Hazzar et.al. (2016) explored the capacity of piles under lateral loads by using FLAC 3D. It was concluded that due to the vertical loads, in clayey soil, piles capacity of lateral load fell almost 20% and the highest bending moment fell almost 30% depending on the vertical load level and the lateral deflection value. But existence of the vertical load did not have a significant impact in the loose sandy soil. It was obvious that the vertical load existence on a pile in clay could have negative effects on its lateral capacity. Furthermore, the results show that the vertical load effect on the piles response which are buried in two layered strata relied on properties of soil around the shaft and under pile tips as well.

Cimen (2009) investigated pile behavior embedded in different type of soil, the lateral and vertical maximum displacement and base shear force by carrying out dynamic analysis based on finite element method. Totally 11 different soil type that consist of sand and clay were evaluated. In these profiles within 10 m x 10 m ground limit; 1 m fixed diameter and 10 m fixed length pile are modeled. Piles were assumed that concrete piles and all structural elements were defined as elastic materials. Each pile was affected by the stresses determined from their bearing values. Intermediate element plastic spring were used at the ground-pile interface. Düzce-Erd data source registered with Düzce/ DZC-UP was used in dynamic analysis. According to result, when the soil was completely sand or clay, the shear force values obtained were smaller than those obtained in other models, when the soil was completely clay, the smallest base shear force and the largest displacement value were obtained. When the 5 m clay was put on the top of the pile, the base shear force was bigger than when the clay layer was put at the bottom.

Mehrjardi et al. (2017) investigated the influence of near and far field on a column-pile under earthquakes. They also examined the dynamic-soil-structure interaction by using Opensees software for soft, hard clay. For soft clay and stiff clay V_s changed between 180 m/s to 375 m/s. The soils depth was 50 m. Three sort of clay soil profiles were examined. The results demonstrated that when soft clay placed on stiff clay, displacement and acceleration of the deck level of column-pile rose because, inertial interaction of this deck level was more efficient than the kinematic interaction of this level. However, the structure was quite affected by the kinematic interaction effects. The response of acceleration time history of structures in Far Field increased by soft clay soil. The rises and alters in the Far Field horizontal accelerations because of softness of soils were bigger than Near Field horizontal accelerations.

Ercan (2010) explored the dispersion of the lateral load and bending moment of every pile that located in group, lateral group displacements according to pile place in group gap of pile, diameter and stiffness of pile by implementing PLAXIS 3D. The result demonstrates that, distribution of lateral load among the piles was to a great extent a row location function in the group independent from gap of pile. For a special load, the leading row piles carried the hugest load and bending moment. Moreover, under the same applied load, the groups displacements reduced by increasing value of gap and also sole piles load fell. However, this functioning was observed more clearly in the first and second row piles. In terms of third and the fourth-row piles, pile gap had small impact on the distribution of load. Besides, diameter of pile and soil rigidness were not important parameters as place of row and pile gap. It was seen that as pile diameter became bigger, group displacement fell, the pile group displacement for softer clay, were bigger than the hard clay stratum. When pile gap increased, pile load, highest bending moment fell. This behavior generally was observed more clearly in the first and the second-row piles.

Sawant et al. (2014) investigated the influence of pile dimension, diameter, length as well soil modulus and properties on the nonlinear dynamic reply of laterally loaded pile by using MATLAB. The near-field soil reaction was modelled with linear or nonlinear spring was placed in series with the far-field soil reaction that modelled with linear spring and dashpot. According to result, as diameter increased, maximum amplitude decreased and as diameter decreased for all non-dimensional frequencies higher response occurred, non-dimensional frequency increased as the soil modulus decreased. Amplitude were higher for nonlinear curve than linear curve. With increasing piles length, highest deflection primarily decreased and then became stable. Sand Poisson's proportion and density did not have any important influence on pile head displacement. Maximum bending moment went down as soil elastic modulus of rose. As value of τ_{max}/G_{max} increased the maximum amplitude decreased and approaches towards linear response vindicating clear effect of τ_{max} on the response.

Bradley et al. (2008) investigated performance-based reaction of pile foundations in terms of the correlation of different intensity measures (IMs) by using finite element method. For this purpose, two soil stratum profile with a sole pile and a single-degree of freedom superstructure were tested with 40 distinct ground movement records that scaled to different intensity ranges. Peak accelerations of ground altered between 0.1 and 1.0g in stages of 0.1g. These ground motions magnitudes and distance ranged of 6.5-6.9 and 13.3-39.3 km, respectively. During the analysis pore water pressure was taken into account. Based on outcomes good connection between the normalized peak lateral displacement of the pile head and the peak curvature of the pile were gained, velocity-based measures of intensity (such as velocity spectrum intensity, VSI) correlated best with the seismic demand on foundations of pile. Also, according to results VSI was more efficient and sufficient than PGA.

Dezi et al. (2009) studied features of soil, location of bedrock, pile diameter and embedment length on seismic behavior of single fixed-head floating piles buried in homogeneous soil deposits. Pile was considered as an Euler-Bernoulli beam. 21 analyses were done totally. Poisson’s proportion was $\nu = 0.4$ and damping was $\xi = 10\%$. The length of pile was 24 m. A number of 120 cases with different type of pile diameters, bedrock sites and various shear wave paces were tested. In this study, shear wave velocities were 100m/s, 200m/s and 400 m/s, 800m/s, soil densities were 1.5, 1.7, 2.0 Mg/m^3, pile diameters were 0.2, 0.4, 0.6, 0.8, 1.5, 2m and soil heights were 6, 12, 18, 30, 420 m. According to the results, where V_s = 800 m/s was assumed for the bedrock, the embedment of three diameters of piles in the stiff layer was enough to give the highest level of restraint at the pile base; this holds for bedrocks with higher V_s whereas in other cases it was only partly valid. Maximum bending moment that placed among soil and bedrock rose with rising diameter of pile and surface soil layers thickness. When the thickness of surface soil stratum value was small, the highest bending moment occurred at the pile head. Decrease in bending moment was seen in soil profiles with change layers rather than a sharp transformation of features. At the pile top and the interface between the soil layers, bending moments are decreased with increasing shear wave pace.

Ahmadi et al. (2008) performed the dynamic analysis in order to investigate soil pile interaction by using ABAQUS software that based on finite element method. For dynamic analyses KOBE earthquake record were applied and for all analyses it were assumed that bedrock were at the bottom of the model. The side boundaries were restricted against horizontal direction; the bottom boundaries were restricted against both horizontal and vertical directions. Quiet boundaries were used to dissipate the waves' reflection into the model. Rising in sand density and friction angle as well led to smaller values for highest bending moment and shear forces, besides the pile deflection remained almost with no change. In addition, in this paper it was seen that sand Poisson's proportion did not have any impressive effect on pile forces and through increasing the sand Poisson's proportion, no huge change in the maximum bending moment, shear force and deflection of the pile was forecasted.

Maiorano et al. (2009) concentrated on vertically spreading (SH) waves, two-layer subsoil underlain by a rigid base, soils linear-elastic behavior, and shear-strain-independent soil damping. Dynamic analyses were performed with VERSAT-P3D. Fixed head piles were modelled with Eulerian beam theory. Top layers shear-wave pace V_{s1}, was considered as 50 or 100 m/s, two stratums shear-wave paces proportion was fixed equal to 2 and 4 for values of V_{s1}. Piles length (L) was 20 m, pile diameter (d) was 0.6 m. 3 x 3 and 5 x5 fixed-head pile group that interval of pile was taken 4 times pile diameters (D) and 2.5 D was examined. 144 several analyses were done. Linear elastic analysis was practiced. Damping ratio (D) was equal to 10% and results compared with those simplified approaches. According to result at the interface, the bending moment envelope showed a particular peak while the pile-deflection curves were less influenced by the changes from the upper to the lower layer. Also due to kinematic interaction group impact had been found to be negligible.

Erdogan et al. (2015) examined kinematic response of pile foundation under earthquake loading. For this purpose, they investigated the influence of pile length, depth, and pile rigidity, rigidity differences in soil layers and soil thickness, bedrock depth, pile embedment depth in bedrock on kinematic response of pile foundations. In this study soil were assumed that two layered clayey soil and soil site were C, D, E. Shear wave velocity of bedrock were 1200m/s and soils shear wave pace of changed between 100m/s to 900m/s. Damping proportion of soil and bedrock were 5% and 2% respectively, pile was assumed as fixed head pile. They implemented OPENSEESPL program. Soil was modelled as nonlinear and piles were modelled with linear elastic beam column. According to results, piles subjected to high kinematic bending moment under earthquake for fixed head pile, end pile high kinematic bending moment was observed in the layer transition regions because of the difference in rigidness and the thickness of the first layer.

Kumar et.al. (2017) studied the pile interval, length, diameter, group efficiency and embedment lengths impact on laterally loaded pile group in sand by using PLAXIS 3D. It was inferred that the capacity of lateral load rose with growing piles gap. But, this capacity remained with no change although the length of piles was rose. As a result of the shadowing effects of soil in closed space piles ultimate static lateral capacity of pile group was smaller in all embedded length.

Hokmabadi et al. (2014) explored the interaction of seismic soil-pile-structures impact on the dynamic reply of buildings by using FLAC3D. Therefore, they examined the 5,10,15 storey (each story was 3m height) structures where on clayey soil. For all cases, two different foundation types were investigated, (1) a set-base structure, (2) a structure supported by an end-bearing pile foundation in soft soil. Four different earthquakes; records were used. According to the results, highest lateral deterioration for the 5- floor type 2 building rose by 12, 37, 8, and 14% under Northridge 1994, Kobe 1995, El Centro 1940, and Hachinohe 1968 earthquakes, respectively. This deformation for the 10 and 15-floor buildings rose by up to 16 and 26%, respectively, compared with the type1 buildings. The ingredient of rocking had a major impact on the lateral deterioration of the superstructure. Based on the recorded, highest rocking angle of the foundation and lateral deflections for 5-storey building were 16.9% and 83.1% respectively due to the distortion element. But these values for 10-storey building were 18.8% and 81.2% due to the distortion ingredient. As for 15-storey building values were 19.7% and 80.3% respectively. Because 15-storey buildings inertial force was greater than 10- or 5- story model buildings because of its bigger mass.

Naggar et al. (2011) evaluated kinematic soil–structure interaction for both floating and socketed single piles, considering the energy dissipation, nonlinear functioning of the soil, wave propagation and noncontinuation situations at the pile–soil interface according to the input ground motion. For this purpose, they used ANSYS for dynamic analysis. This study did not consider the pore pressures due to cyclic loading and inertial interaction among the superstructure and the pile–foundation system. According to the results, pile head response reminded of the free-field reply of the low frequency seismic loading. According to the results, due to soil plasticity, the Fourier amplitudes for the predominant frequency enhanced but maximum acceleration amplitudes fell lightly. Kinematic interaction response was equal to the free-field response in view of with the inclusion of soil plasticity, slippage and gapping at the pile–soil interface and damping. When compared with the free-field reply, the elastic kinematic interaction of sole pile caused boosting the bedrock movement and slightly reduced the Fourier amplitudes for all the considered frequencies (0–20 Hz).

Rao et al. (2013) perform the sole piles analysis that buried in liquefied soil under earthquake loading by using FLAC 3D to find deviation and bending moment behavior over the pile. Boundary condition at the pile top changed from fixed head to free head. Pile lengths changed between 5-10m and pile diameters changed between 0.6m-1m. Results are compared with Reese and Matlock's (1956) chart. Based on the results, it was obviously understood that that lateral displacement for free head pile was greater than fixed head pile. Analysis enlarged to non-liquefied and liquefied sites that were subjected to 2001 Bhuj earthquake and results show that PGA and deflection for liquefied soil was greater than non-liquefied soil.

Terzaghi et al. (2017) investigated the influences of pace of shear wave of soil on seismic reply of skyscrapers by using FLAC 3D. Concrete building frame with 60 m height and 12 m length was tested. Foundation thickness and width was 1m and 14 m respectively. Local damping coefficient of 0.157, causing 5% damping, were took account for the structure and foundation. Mohr-Coulomb model was implemented with the hysteretic damping, covering the Masing's rules. In dynamic analysis 1994 Northridge earthquake record was used. As shown in Figure 2.1, two different profiles connected with the shear wave velocity, in-situ non-uniform profile (Case A) and the equivalent uniform profile (Case B) were tested.

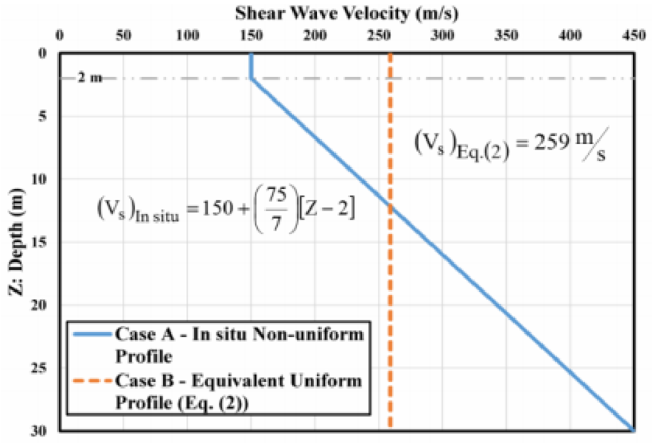


Figure 2.1 Shear wave velocity profiles of interest. (Terzaghi et. al.2017)

According to the results, the distribution of the absolute maximum shear force for Case B and case A showed a little resemblance in spite of the difference between two cases is minor. Results show that the impact of the pace of shear wave profile type on the base shear plus the distributed shear along the building height could be almost disregarded for the adopted 20-story building. In contrast, the story drift proportion, defined as the lateral story displacement divided by the height of the floors, demystified that applying the average weighed pace of shear wave in lieu of making use of the adopted field-based profile sparked off the aggressive (unsafe) design of a building due to exceeding the 2% yardstick hinging around the Life Safety performance level which was posited for the rehabilitation of the existing structures. The response spectra connected with Case A and Case B in the short-period range were distinguishable in order that the aftereffects of considering the profile, delineated in Case A, might be harrowing, with the provision that a low-rise building exists on the site for these studies. Even though putting the weighted average pace of shear wave into practice with the purpose of evaluating the seismic site classification accompanying computing the small-strain shear modulus, which was typically connected with the strains on the order of 10^{-3} % or less, in advance of running a dynamic analysis, tends to simplify the mathematical analysis of the problem, it was concluded, in line with the outcomes derived from the current research, that the aforesaid procedure was an oversimplification of reality, implying that the actual shear wave velocity profile resulting from the field measurements ought to be directly utilized in the seismic analysis of structures (Terzaghi et al. 2017).

Xu et al. (2017) explored the effects of soil stiffness including shear wave velocity (V_s) and shear power (S_u) on seismic response of structures (lateral deflection of end bearing pile and structure (by using FLAC 3D. For structural analysis and design SAP 200V14 was used. A 15-storey moment was analyzed. Paces of shear wave were 150, 200, 250 and 300 m/s and shear strengths were 50, 95, 150 and 220 kPa. 5% damping was assigned for both the building and the pile foundation. Free-field boundaries were utilized. A near fault Northridge earthquake, 1994 was utilized in this study.

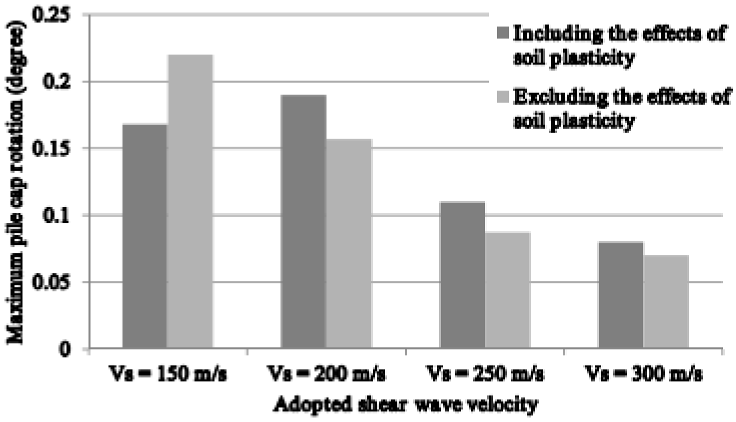


Figure 2.2 Maximum pile cap rotation including and excluding soil plasticity with different shear wave velocities. (Xu et. al 2017)

Maximum pile cap rotation decreased as pace of shear wave velocity and corresponding modulus of shear went up. The lower dynamic features caused lower soil stiffness therefore more deformation of the soil deposit was induced under the seismic loading. As shown in Figure 2.2., difference of maximum rotation of foundation slab between the cases that include soil plasticity and exclude soil plasticity became less as shear wave velocity increased, and therefore maximum foundation slab rotation became less dependent on the shear strength with relatively high shear wave velocity. As a result, thanks to soil plasticity, earthquake energy can be dissipated by soil plasticity in the process of wave propagation. In other words, the structure receives less energy and therefore potential foundation rotation would be less. Besides, the building and foundation may prove more deflection and rotation as the soil around foundation reaches its shear strength under a strong seismic excitation. Depending on how these two aspects play and contribute in a particular case, properties of building, foundation, soil and earthquake should be noted, the opinion of soil plasticity may contribute to increase or decrease of the foundation rotation.

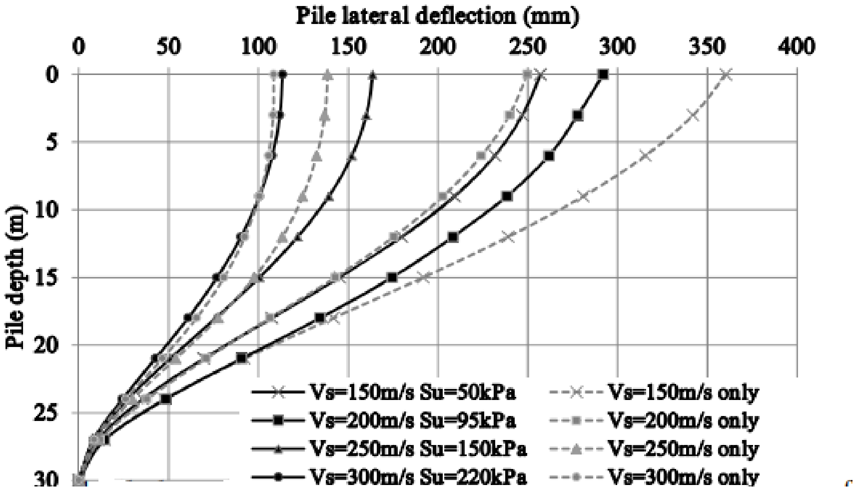


Figure 2.3 Pile lateral deflection along the length of pile for varied V_s values and including and excluding soil plasticity. (Xu et. al 2017)

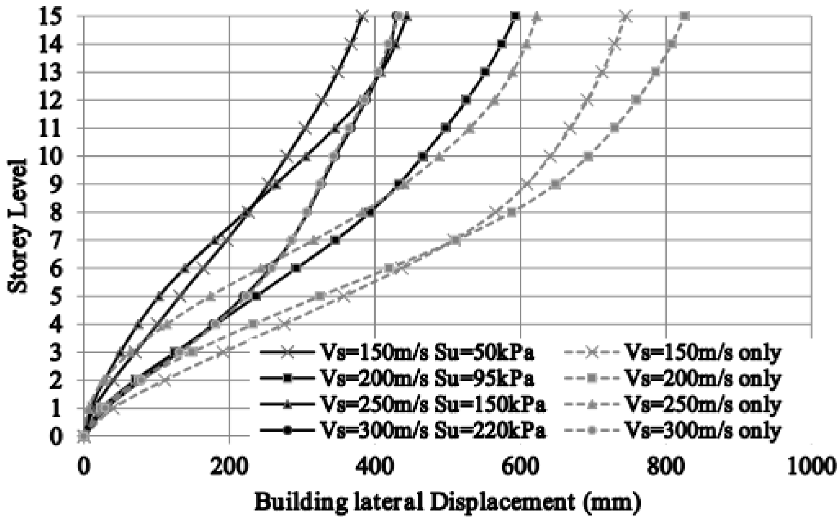


Figure 2.4 Maximum building lateral deflection for varied V_s values and including and excluding soil plasticity. (Xu et. al 2017)

As depicted in Figure 2.3 and Figure 2.4, maximum lateral deflections for the higher shear wave velocity considering soil plasticity do not differ much from that of the case without shear strength limit. However, for lower shear wave velocity such as 150m/s maximum lateral deflections considerably increase when soil plasticity was excluded. Consequently, as the soil stiffness (pace of shear wave and modulus of shear) increased, the base shear of buildings increased, the response of structure-foundation system regarding foundation slab rotation, lateral deflection for both pile and structural lateral deflection becomes less dependent on the soil plasticity for stiffer soils.

Silva and Manzari (2012) examined the behavior of 4x4 pile group under lateral loads taking various set of column height to diameter or aspect proportions and soil–building horizontal and vertical interaction into account. This study mainly focused on understanding special parametric factors on lateral displacement and rotation of the pile cap during cyclic loading using Finite Element Method. The center to center spacings were selected as 4 times the diameter of the pile to neglect the pile group effect. Soil was modeled with discrete nonlinear springs and modulus of elasticity for soft clay and stiff clay were taken as of 5,000 kN/m^2, 50,000 kN/m^2 respectively. The types of analysis are presented in Figure 2.5.

Pile group research corresponding to pile cap rotation (Silva and Manzari 2008)

Run No.	Soil Type		Pile Cap Horizontal Spring	
	Stiff	Soft	No	Yes
1	•			•
2		•		•
3	•		•	
4		•	•	

Figure 2.5 Types of analysis. (Silva and Manzari 2008)

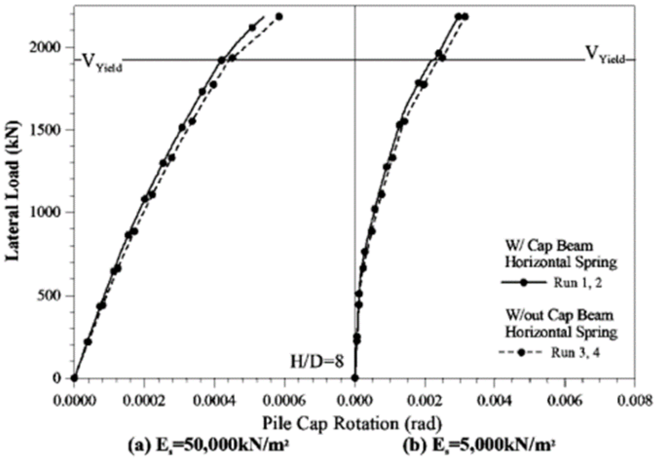


Figure 2.6 Pile group research corresponding to pile cap rotation. (Silva and Manzari 2008)

As shown in Figure 2.6, the which has the least effect on the rotation of pile cap is placed in front part of the pile cap. This also shows that skipping this spring in the model will not change the rotations.

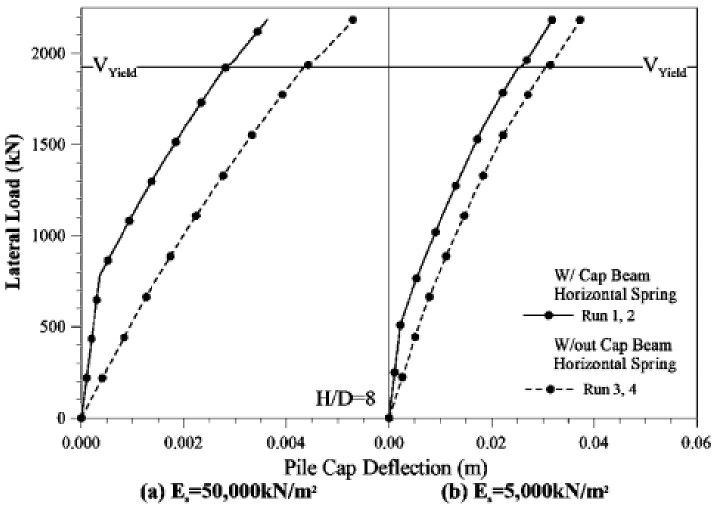


Figure 2.7 Pile group analysis according to pile cap deflection. (Silva and Manzari 2008)

As shown in Figure 2.7 stiff clay caused less deflection than soft clay and pile cap lateral deflection was bigger when the horizontal spring was not considered in the study. Further horizontal soil rigidness put along the length of the piles had a great influence on the pile cap lateral deflection; as Es increased pile lateral deflection decreased.

Alfach (2018) explored the aspects of interaction for soil-piles-structure (bridge) under genuine earthquake record by using FLAC 3D. Soil was modelled according to Mohr coulomb criterion. For this purpose, 2x3 floating pile group that was buried in 15 m depth homogeneous soil was used. Pile length was 10.5 m and pile cap thickness was 1m and pile diameter was 0, 80 cm and the spacing between piles was 3.75 times the pile diameter (i.e. 3m). Superstructure’s mass was 350 tones. According to the results, when C=50 kPa (C is cohesion) due to nonlinearity internal force fell sharply, whereas, these induced forces had a gradual increase for (C=100-150 kPa) and changed into for (C=150 kPa) so close of the induced forces in case of using elastic interface. Shear force and bending moment curves showed that the reactions for stiffness of the interface (100 and 150 kPa), were so close to those gained for excellent contact. In the case of C_s =50 kPa bending time and shear force totally changed. Maximum shear force at pile head was declined and bending moment increased considerably checked with the lateral acceleration. The worth of the highest bending moment normalized gained for (C_s =50 kPa), was 4 times higher than that of obtained for an elastic interface. For the interface of C_s=150 kPa, the response gained was so close to that obtained for excellent contact. For Cs =100 KPa, a drop in highest normal force was about (22%). For Cs =50 KPa, the normal force was fell by 85% percent because of slippage at the soil/pile interface.

Abbas et.al. (2016) examined attitude of individual piles and pile community embedded in sand and interaction of soil and pile with ABAQUS program according to finite element method. According to results gained from the load-deflection curve that the load required to produce a particular lateral displacement was larger for a fixed-head condition than for a free-head condition. Also, it was found that the p-multipliers for the front piles were higher than the following piles, up to spacing of nearly five times pile diameter. The estimated amounts at the two head displacements were almost equal, the group efficiency increased proportionally with pile spacing.

Chu et al. (2004) examined the impaction of pile configurations on seismic soil-pile-structure interaction using ABAQUS program. As material nonlinearity, Drucker-Prager model was used. The influences of proportion of piles gap to diameter proportion (S/D) and pile-soil rigidness proportion on the seismic reactions of the soil-pile system were observed. Primarily, extraction of natural frequency was executed to explore the dynamic characteristic of both 2*2 and 3*3 pile foundation systems. Later, influences of (S/D) on kinematic interaction were observed under a harmonic excitation. In this research during dynamic analysis, pile head settlement was 1.72 cm under an axial load of 2,500 kN while this value was 1,7 cm at Poulos and Davis charts. For Maheshwari, 200 kN horizontal load corresponds to a deviation of 0.48 cm, this value was 0.51 cm in this study. According to the result, in 2x2 pile group reply of pile head was less for soft soil than rigid case. Also, for rigid soil this acceleration and displacement value were greater for 3x3 pile group than 2x2 pile group. Reply of pile head of the center pile were smaller than the corner pile because of the mutual effects among piles in group. However, results illustrated that this case relied on the soil characteristics. The reactions of the center and corner piles were the same for stiff soil. Acceleration and displacement of largely spaced pile groups for 2x2 and 3x3 pile groups were bigger than the closely spaced depending on greater stiffness and less pile-soil-pile interaction. For 3x3 pile groups pile head accelerations and displacements of center pile and corner pile were same at nearly spaced pile groups. The highest level of pile head speed up in the 2*2 pile foundation system was 0.481g. In the 3*3 pile foundation system the figure was 0.524g for the center pile and 0.443g for the corner pile. The difference was because the presumption of free pile head revolves in this study. Furthermore, the results illustrated that, the number of piles didn't have an impact on pile head acceleration substantially but it had significant impact on pile head displacement were significant for accounted pile spacing ratios and El Centro excitation.

Erdemir et. al. (2011) explored the 4x4 pile group that embedded in both clayey soil and sandy soil under seismic loading by using 3D finite element model. The pile elements were modeled like beams and the ground was modeled like solid elements. 4 different ground profiles were used according to the distance between the piles and their names and distances were as follows: Model A with 4m spacing, Model B with 2m spacing, Model C with 1.5m spacing, Model D with 1m spacing. The piles were modeled as frame elements and the material and cross-sectional features were assumed as rod elements. The pile head was modeled as area-shell. Loma Prieta 1989 earthquake record (magnitude 6.9, depth 18 km, maximum acceleration 0.11 g) was used. According to the results, it was clearly understood that for sandy soil as the pile spacing was decreased, the displacement increased. Shear force in 2 m spaced pile group was lower than the shear force in 4 m spaced piles. For these two models, the shear force reached the minimum value at the middle point of the pile. For models with spacings of 1.5 m and 1m, ultimate shear force at pile head was decreased towards the pile end. The lateral displacements obtained in the analysis for clay soil were 10-15% larger than the sandy soil profile. Also, as the pile spacing on clay soil increased, the displacements also increased. Shear forces values were 20% -35% lower than the sand. The shear force decreased as the pile spacing decreased. In terms of bending moments, Model A had greater bending moments than Model B, but the behavior along the pile was similar. In these models where no group effect was observed, moment values increased to the middle of the pile length and decreased after this point and took a value close to zero. For C and D models due to maximum moment group effect occurred, the maximum value in the pile head was zero and the value was zero.

Sheikhbahaei et.al (2009) investigated the functioning of groups of batter pile under seismic excitations by implementing ABAQUS program. Model functioning was assumed that act as Hardening Drucker-Prager behavior. To provide soils borderline conditions Dashpot Elements were used. Four friction piles were used for this study. They tested the impacts of slenderness and spacing ratios, the impact of pile slope angle on group of batter pile. Piles diameter was 50 centimeters and length of 9 meters. Pile caps thickness was 60 centimeters and pile caps length and width are equal to 3 meters. To explore the effect of pile slope angle, slope angles were taken as 0, 10, 20 and 25. During the analysis Naghan earthquake that had PGA value of 0.72g were used. Based on the results, while the pile slope angle rose, the pile head displacements fell, shear stresses along pile decreased and pile head displacements reduced. As piles slope angle rose the progressed bending moment throughout the pile standard length fell. As slenderness portion fell, pile head lateral displacement fell slightly due to unsubstantial impact of the rising in slenderness portioning the case of higher than 25. Permanent Also it was clearly understood that when slope angle was equal to 20, as interval proportion rose axial stresses at pile head fell.

Gazetas et al. (1984) investigated the end-bearing piles that subjected to dynamic loading and vertically propagating harmonic S-waves. Important parameters had found to be as the ratios of stiffness values of pile and soil; the slenderness proportion of the length over pile diameter; the frequency proportion f/f_1 of the excitation frequency over the fundamental natural frequency of soil in vertical S-waves: and the relative frequency factor f_{st}/f_1, where f_{st} is the fundamental frequency of the pile-supported superstructure. Three soil models with a several alteration of soils Young modulus E (z) took into account, as shown in Figure 2.8. Natural shear frequencies of soil deposits are as in Figure 2.9.

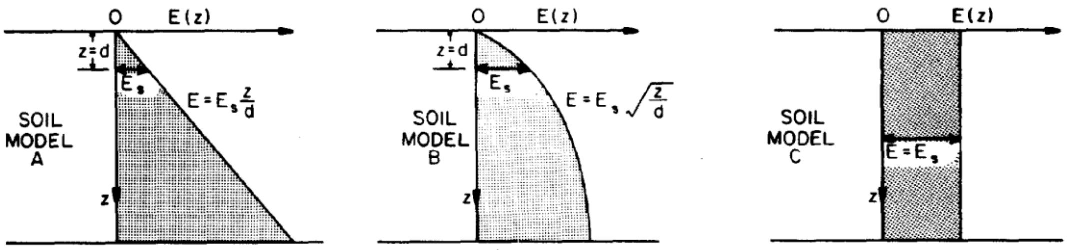


Figure 2.8 Soil models. (Gazetas 1984)

Soil model	f_1	f_2/f_1
A	$1.21\, V_s/H$	2.33
B	$0.56\, V_s/H$	2.66
C	$0.25\, V_s/H$	3.00

Figure 2.9 Natural shear frequencies of soil deposits. (Gazetas 1984)

V_s = S-pace of wave at depth, z = d under the earth surface, H=soil thickness

Displacement and rotation kinematic interaction formula are as follows respectively;

$$I_u = \frac{u_p}{u_0}$$

$$I_\Phi = \frac{\Phi_p r_0}{u_0}$$

Where r_0 is the radius of the pile, Φ is the rotation of pile at the top, u_p and u_o are the amplitudes of horizontal displacement at the top of the pile and the ground surface respectively.

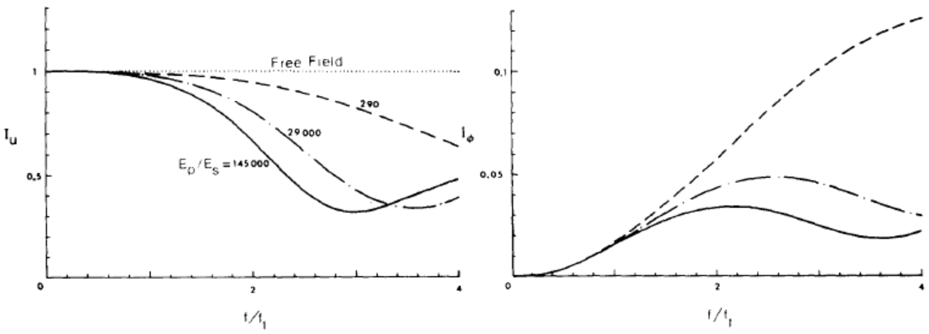


Figure 2.10. Kinematic interaction influences: impaction of Ep/Es (L/d = 40; soil Model A; $\nu_s = 0.40$, $\rho_p/\rho_s = 1.60$). (Gazetas 1984)

According to result as shown in Figure 2.10, up to a frequency of approximately 1.50 f1, piles of all relative stiffness did not have an influence on the seismic movement at ground-surface level. However, this filtering impact was considerable in the case of (Ep/E s > 20000), following the second natural frequency, pile stayed still, while the free-field soil mass acted substantially. For soft piles, max rotation rose significantly with advanced natural frequencies value; and the opposite was true for hard clays. The influence of Ep/Es on the kinematic reply of piles for soil models of B and C, was similar.

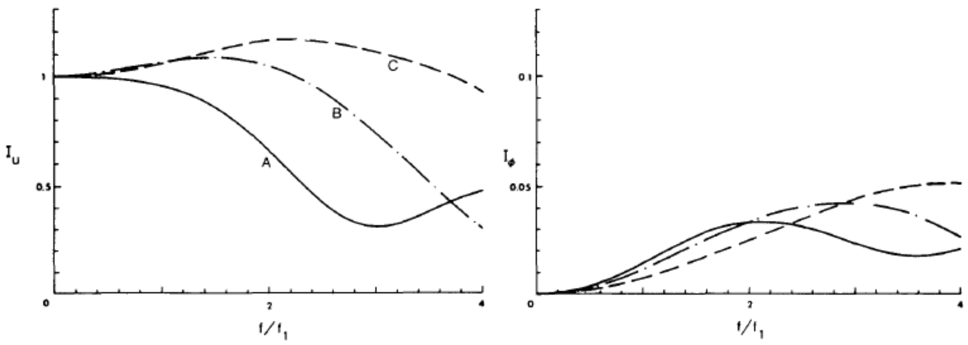


Figure 2.11. Kinematic interaction influences: impact of the soil profile. (L/d = 40; ν_s = 0.40, ρ_p/ρ_s = 1.60) (Gazetas 1984)

Also as shown in Figure 2.11, it was concluded that the filtering with a pile at the max frequency, base excitation and rotational components of motion was greater for inhomogeneous soil than homogeneous.

As shown in Figure 2.12, the conclusion that L/d has an influence on displacement when frequencies higher than 1.50 f1, where the shorter piles result in higher filtering effects. For frequencies larger than f1, amplitudes of rotation were higher for shorter piles as they have the same relative displacement with longer piles over a shorter length. For frequencies around f2 shorter piles are subjected to relatively small rotations.

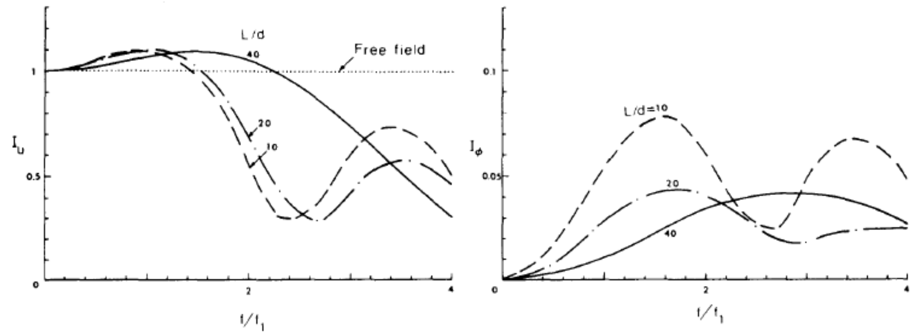


Figure 2.12 Kinematic interaction influences: impact of L/d.(Ep/Es = 145000; νs = 0.40, ρp/ρs = 1.60) (Gazetas 1984)

Prakash et.al. (2009) investigated the influence of pile diameter on effective pile length in the case of be impressed by earthquake load by using ANSYS program. The pile was entirely embedded in the soil. Results were confirmed with the experimentally reply of single pile. They selected three full-scale tests in which they recorded the pile response as beyond the elastic limits (Kramer et al. (1990), Jennings et al. (1984), Brown et al. (1988)). They obtained the pile responses at 120kN load and p-y curves at different depths from the analysis. The outcome was checked with the results of numerical analysis reported by Badoni & Makris (1996) and experimental data in literature. Results showed that the comparison of predicted and measured pile response were in agreement. They examined the alteration of effective pile length with L/d ratio of pile for varied diameters at E_s = 20 MPa and E_s=80 MPa. According to the results, it was inferred influential length of pile increased with pile length for all soils, whilst the proportion of rise was high for soils with lower modulus values. It was also inferred that whilst the pile diameter did not have considerable impact for short piles (L/d < 20), it had a notable impact for long piles (L/d > 30) buried in all types of the soil states. Influential length of pile according to elasticity modulus ratio of pile over that soil (Ep/Es), for several L/d proportion of pile and permanent diameter of pile of 0.5 m was examined. Also, it was clearly understood that the effective length significantly decreased with the decrease of the modulus ratio. Also, it was inferred that the effective length significantly decreased with decreased modulus ratio. The effective length was researched they proposed by several authors by utilization analytical and semi-analytical term. An equation was developed to compute the influential length of pile under earthquake load as follows:

$$\frac{I_e}{d}=2.20\left(\frac{E_p}{E_S}\right)^{0.345}$$

l_e = Influential length of pile under earthquake load,

d = Pile diameter,

E_p =Young’s modulus of pile and

E_s =Young’s modulus of soil.

Outcomes were derived from numerical studies on clay-embedded piles.

Poulos et.al (2005) investigated the seismic response of piles in liquefying soil under horizontal loads using a numerical model. Mindlin equation which is based on Winkler model is used to calculate the nonlinear spring constants. Numerical model included two phases: a free-field ground response analysis and a seismic analysis. Shear modulus of soil were calculated based on the effective stress level of the soil as well as the maximum lateral strength. According to the results, in the case when pore pressure effect is included in the analysis, displacement and bending moment rose and pile top acceleration decreased. Also, sometimes in the case of including the pore pressure impaction the highest shear force developed increased in the pile, but sometimes the maximum shear force fell It was inferred from outcomes, if the maximum shear force was advanced at the pile head, the inertia force there controlled this shear strength. For this reason, highest shear force fell in the case of pore pressure influences were counted in. Briefly, soils softening arising from the pore pressure influencing had a greater influence than the inertial force at the pile head. Again, whilst rising in pile diameter caused the rising the highest bending moment for pile with and without cap-mass but, the greatest bending moment ratio fell. Further result show that pile heads inertia forces impacts on the bending moment produced by the pile was not important as pile diameter rose. Also, the maximum bending moment obtained as a result of effective stress analysis is about 4 times of the one obtained by the total stress for velocities higher than 2m/sec. In the opposite case, when velocity was smaller than 2 m/sec, the total stress analysis could be utilized to achieve behavior of pile.

Dezi et al. (2008) examined soil-pile mutual effect considering the pile diameter, two-layered soil features and the bedrock location and the bedrock embedment of piles. Piles and the soil were supposed that act as linear. Pile and soil properties of this analysis are as in Figure 2.13.

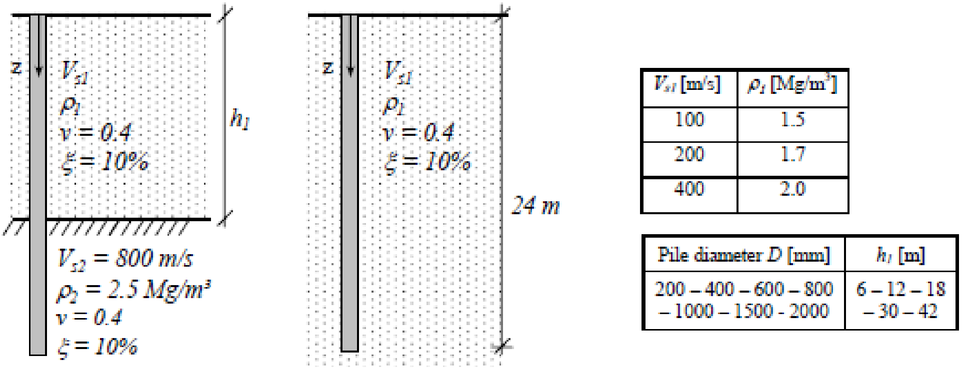


Figure 2.13 Pile and soil properties. (Dezi et al.2007)

The analysis procedure contained two stage; i) free-field motion (i.e. no piles exit), and ii) soil-pile system (to examine kinematic soil-pile interaction). According to the results, maximum bending moments were obtained from a peak value at interface of stratum and through a relatively uniform distribution in the section embedded into the upper layer. The highest shear force value arose in the lower stratum near the interface. The maximum displacements occurred at the pile top and the displacement diminishes at stiff soil layer. Normalized bending moments sharply decreased while the pace of shear wave increased. The embedment length of pile for 3D (pile diameter) was found as the minimum length that provides the maximum degree of limitation at the lower end of the pile when comparison to 1D and 5D. The bending moment highest values at the interface of soils stratum increased as the diameter of pile and soil stratum thickness besides the hardness opposition among stratums. For thinner soil stratum, greatest bending moment arose at the top of the pile compared to the interfaces.

Chore et al. (2012) investigated the mutual effect amid pile cap and cohesive soil under laterally loaded by implementing simplified finite element method. They tested the impact of pile interval, diameter, piles count and collating sequence of pile on the reactions of pile group Soil was presumed as linear elastic. Two different pile categories comprised of two and three piles respectively were taken into account. For all situations, the the piles gap altered between 2D to 5D. The study was done for the lateral or vertical force of 1000 kN applied on top of the pile group. According to the results, as pile gap, diameter rose, the pile group top displacement decreased, resistance to lateral loads increased. For group of two piles displacements were greater compared to that for group of three piles. Also, it was inferred from impact of sequencing of pile in a group in terms of the direction of lateral load was a significant parameter. Displacements were greater for free tip case than fixed case. The pile group capacity turned out to be over for the series case than that in parallel case. The highest positive bending time decreased as negative moment increased as pile gap and diameter.

Fixing times in corner piles were higher in the group of three piles in comparison with that in group of two piles. The difference was higher for free tip condition followed by fixed tip and pinned tip with difference getting reduced.

Cairo et. al. 2007 investigated kinematic interplay of sole pile in nonhomogeneous soils stratum and resting on a rigid bedrock, where the pile is considered to be hinged. This study was restricted to linear, viscoelastic behavior of system of the soil-pile. The results were evaluated according to normalized amplitudes (I_u, I_Φ. I_u and I_φ) of horizontal displacement and pile head rotation by the corresponding free field surface displacement.

As shown in Figure 2.14 kinematic response factors were plotted as functions of the dimensionless frequency $a_0=\omega d/V_s$, (V_s is the pace of soil shear wave) The data used in the analyses were: L/d=20, $\rho_p/\rho_s=1.6$, $\nu_s=0.4$, $\beta_s=0.10$. Figure 2.7 shows that, for lower frequencies, the fixed head piles followed the ground movement whereas for greater frequencies deformations decreased. For free-head piles, rotational motion arose. Also results show that for both free and fixed head cases, as E_p/E_s increased, I_u value decreased. I_u was greater than for free head piles compared to the fixed head case. In addition, authors examined the influence of the pile-to-soil rigidness portion E_p/E_{s1} (the subscript 1 refers to the upper soil layer), the proportion of the soil stratum thickness H1/H2, the proportion of the S wave speeds of the soil stratum V_{s2}/V_{s1}, pile slenderness proportion is L/d. For this purpose, the data used in the analyses were: $E_p/E_{s1}=5000$, $H_1/H_2=1$, $\rho_p/\rho_s=1.6$, $\nu_s=0.4$, $\beta_s=0.10$, $\beta_p=0.05$.

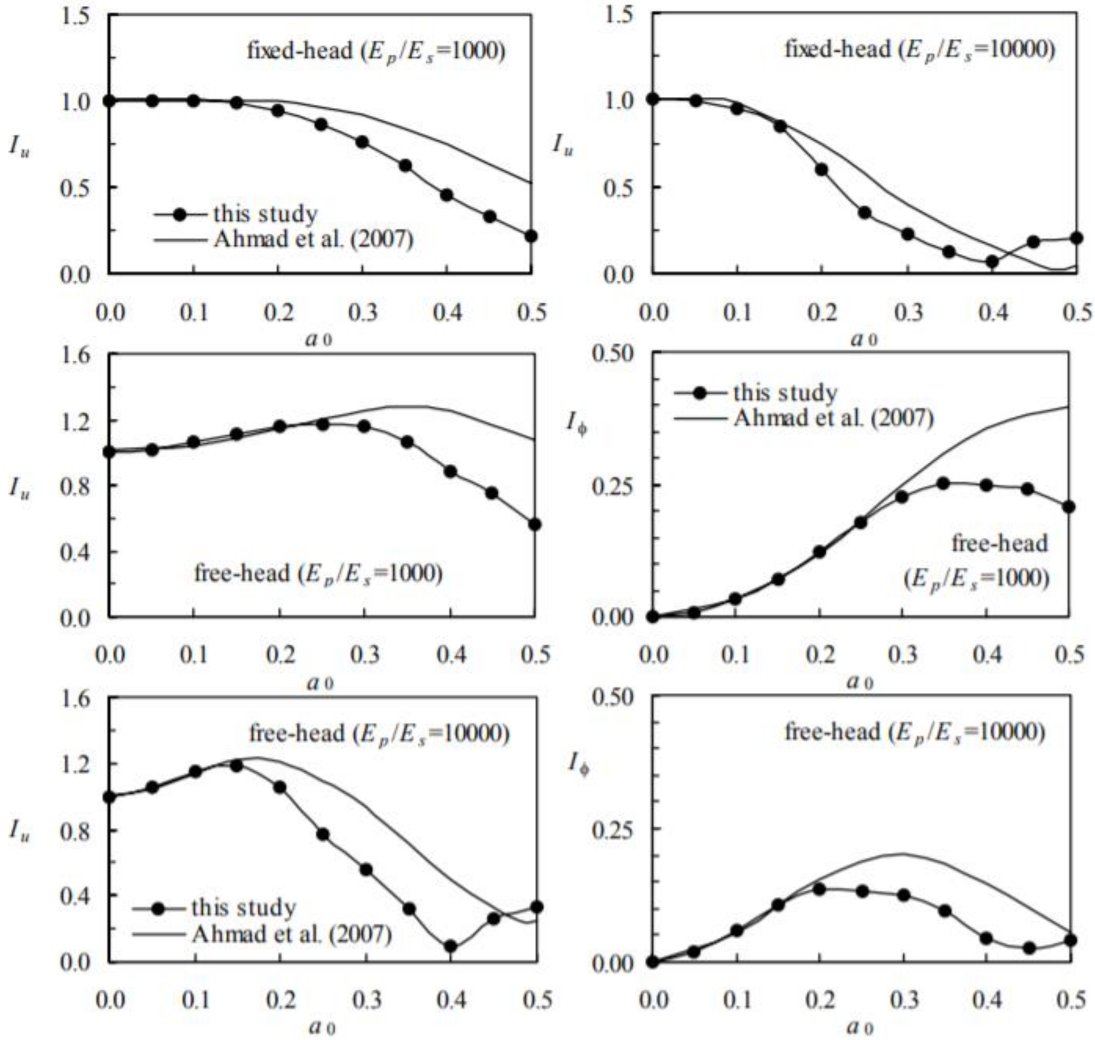


Figure 2.14 Kinematic response factors. (Cairo et.al.2007)

Young's modulus was substituted by its complicated counterpart with βs and βp showing the damping ratio of the soil and pile, respectively. Result show that the largest value of M_m (dimensionless highest-level bending moment amplitude) happened at the fundamental frequency of the deposit. The maximum bending moment increased when the difference between shear wave velocities of the lower and the upper layers (V_{s2}/V_{s1},) is increased. Moreover, M_m increased with increasing pile slenderness. Also result shows that the maximum bending moment (for ω=ω1) occurred at the layer interface, ω and ω1 are the excitation frequency and basic natural frequency of the soil deposit respectively.

Maheshwari et.al. (2011) investigated the impacts of nonlinear behavior of clay soil and soil pile interfaces on one pile and groups of piles (2x2 and 3x3) under dynamic loading. Pile interval center-to-center (spacing/diameter) were equal to 2, 5, 10. Piles were square cross section concretes. In this study harmonic or transient bedrock movement were utilized. For the transient movement, El Centro 1940 Earthquake with PGA of 0.32g was implemented. Pile head lateral deflection was calculated with respect to amplitudes of applied load for the elastic, elastic–gapping, and plastic–gapping cases. According to the outcomes, dynamic stiffness of the soil–pile system fell due to soil nonlinearity. The impact of separation was more important for elastic soil models than for plastic models. When plasticity was taken into account in the study, it put into shades the impact of separation. Also, it was concluded that the impacts of soil nonlinearity related with the frequency of excitation. At low frequencies, the impact was considerable however when at frequencies rose the impact became very small. Also, at high frequencies larger pile groups stiffness fell due to nonlinearity of soil. For elastoplastic soil horizontal deflections with and without gapping was higher than the elastic soils horizontal deflection without gapping and elastic soils horizontal deflection without gapping was higher than the elastic soils horizontal deflection with gapping.

Unutmaz et.al (2018) investigated the dynamic behavior of piles embedded in homogeneous or variable clean sand layers with pace of shear wave (Vs) changing between 100m/s and 200m/s also, they proposed an empirical correlation among chosen intensity measures of ground movement (IMs) and engineering demand parameters (EDPs) for seismic response of piles. In this study ten different ground movement records that includes near fault were used. The post-cyclic lateral deformation was selected as the EDP and effects of pile group parameters on this parameter was investigated. For this purpose, they used FLAC 3D software. Analysis were conducted both 2x2 and 3x3 pile groups, each with 80 cm or 140 cm diameter (D) and each with 2,5D or 4D center-to-center spacing. Results show that, the variation of the selected EDP was most linear with ground motion IMs. The adequacy of candidate IMs, Peak Ground Acceleration (PGA), Velocity Spectrum Intensity (VSI) and Aria Intensity (I_a) was not significantly different. Also, according to results, it was concluded that as $\Delta x/W$ (W was pile cap width) decreased with increasing pile diameter for the same pile group with the same normalized spacing for each soil profile. The variation was higher for 2x2 pile groups than 3x3 pile groups. Also, it was clearly understood that for 3x3 pile groups effect of pile diameter on the distribution was negligible and for homogeneous soils as shear wave velocity increased $\Delta x/W$ was increased, maximum deformation increased with increasing pile spacing and lateral top displacements decreased as the number of pile increased.

2.3. Conclusion

Based on the summarized literature, it can be make inferences that the response of pile foundation in the course of and afterwards an earthquake is still hard and a complicated problem and has not been satisfactorily addressed yet.

3. NUMERICAL ANALYSES OF LATERALLY LOADED PILE

3.1. Introduction

In order to assess soil-pile-soil interaction under earthquake loading, a number of 3-D numerical analysis were undertaken. In line with this purpose pile groups embedded in two different soil profiles were evaluated that considers the loading of static and dynamic conditions. Therefore, in this research numerical analysis were carried out in 3 stages: i) dynamic free field analyses, ii) static analyses with the existing pile iii) dynamic analyses of the soil-pile earthquake interacting system.

3.2. Numerical Analysis Procedure

The three-dimensional (3-D) finite difference-based simulations were performed using FLAC-3D software to investigate the behavior of piles under cyclic loading. FLAC 3D is used to analyze soil, rock and structure. It is based on a nonlinear, finite difference method. This method solves whole equations of movement through using grid point masses based on the density of surrounding zones. FLAC-3D is based on a two-dimensional program which is known by the name of FLAC-2D. FLAC-3D builds on and further develops the analysis capability of FLAC-2D into three dimensions, taking into account the specific behavior of three-dimensional structures built of various materials that undergo plastic flow. Materials that consist of polyhedral elements are represented with 3D grid. Each element behaves in tandem with the applied forces or boundary restraints. FLAC 3D takes advantage of an explicit finite- difference formulation that can help us model complicated behaviors not suited to FEM codes. It includes different phase high displacements and strains, nonlinear behavior of material. FLAC 3D doesn't use a matrix during calculation.

In reference to FLAC 3D User’s Manual (2009), the solution consists of 3 stages:

i) Finite difference stage: the variables that change linearly in estimate under limited time and space by using finite difference equations
ii) Separation stage of model: An equivalent medium is used instead of the continuous medium and all forces are assumed to be gathered with at the nodes of at three-dimensional network.

iii) Dynamic solution stage: The inertia terms in equations of motions are taken advantage of as numerical factors to provide balanced conditions in the system.

3.2.1 Finite Difference Method

In addition to the empirical methods that in the previous section, computational codes are being used more frequently nowadays. One of them is Finite Difference Method (FDM). It is frequently used in modeling dynamic problems and it has an important role in seismology due to approach of derivatives and numerical solving of differential equations, earthquake ground motion modeling (risk) and seismic research. First finite difference solving for pile was proposed by Gleser (1953). The law of Finite Difference Method is approximate to the numerical schemes that are put into practice to help us figure out ordinary differential equations. FDM works by dividing the problem into tiny time steps and estimate the stresses and limitations of the following step based on the time step that is present, through the use of finite difference formulation

According to Winkler (1867) the pile is considered as a thin rod and it is expressed with following equation.

$$E_p I_p \cdot \frac{d^4 y}{dz^4} = -P \cdot B = k_h \cdot y \cdot B$$

Where,

E_p : Modulus of elasticity of pile

I_p : Moment inertia of pile

z : depth

B : Diameter of pile

P : Pressure

y : Deflection

k_h : Subgrade reaction modulus

$\delta, \Delta x$: Step interval

This equation as above-mentioned can be obtained both analytically and numerically. When the value of k_h is constant throughout the pile an analytical solution is generally preferred, but if the value of k_h changes with depth then finite difference method formula. is generally preferred. (Gleser, 1953) Finite difference formulations are as shown in Table 3.1.

Table 3.1. Finite difference formulas

Forward difference	$\left(\frac{d_y}{d_x}\right)_i = \frac{y_{i+1} - y_i}{\Delta x}$
Backward difference	$\left(\frac{d_y}{d_x}\right)_i = \frac{y_i - y_{i-1}}{\Delta x}$
Central difference	$\left(\frac{d_y}{d_x}\right)_i = \frac{y_{i+1} - y_{i-1}}{\Delta x}$

$$\left(\frac{d_y}{d_x}\right)_i = \frac{y_{i+1} - y_{i+1}}{\delta}$$

$$\left(\frac{d^2y}{dx^2}\right)_i = \frac{y_{i+1} - 2y_i + y_{i+1}}{\delta^2}$$

$$\left(\frac{d^3y}{dx^3}\right)_i = \frac{y_{i-2} - 2y_{i-1} + 2y_{i+1} + 2y_{i-1} - y_{i+2}}{2\delta^3}$$

$$\left(\frac{d^4y}{dx^4}\right)_i = \frac{y_{i-2} - 4y_{i-1} + 6y_i - 4y_{i-1} + y_{i+2}}{\delta^4}$$

Pile formula that considered as a thin rod is written according to this finite difference formula as follows (Gleser, 1953)

$$E_pI_p\left(\frac{y_{i-2} - 4y_{i-1} + 6y_i - 4y_{i+1} + y_{i+2}}{\delta^4}\right) + k_hB \cdot y_i = 0$$

Where,

n : Number of intervals throughout the pile

k_{hi} : Coefficient of horizontal subgrade reaction at point i.

As a result, shear force and moment for upper side and under side of the pile are as follow.

Top of the pile

$$E_p I_p \frac{d^2y}{dx^2} = \text{Moment}(M_g)$$

$$E_p I_p \frac{d^3y}{dx^3} = \text{Shear forces}(Q_g)$$

Bottom of the pile

$$E_p I_p \frac{d^2y}{dx^2} = \text{Moment}(M_g) = 0$$

$$E_p I_p \frac{d^3y}{dx^3} = \text{Shear forces}(Q_g) = 0$$

3.3. Soil and Pile Properties

In this study the behavior of the 3x3 pile group buried in a 30 m depth clean sand were examined under cyclic loading. Model is as shown in Figure 3.1. Soil shear wave velocities are 100m/sec to 200 m/sec. E_p/E_s values were chosen as 6, 10 and 500 for V_s=100 m/s and 5.63, 10, 100 were taken for V_s=200 m/s. Soil internal friction angle is chosen among 30 to 32°. The choice of 30 meters has the advantages in terms of consuming less time than deeper soil and it has got a sufficient depth to survey the interaction of soil and pile. The widths of the models change with changing spacing of the piles. However, adequate length to avoid any reflections from the boundaries has been adopted as seen in Figure 3.2. Classification of soils according to pace of shear wave velocities were made by using the Table 3.2 obtained from NEHRP (National Earthquake Hazards Reduction Program).

Table 3.2. Site class according to shear wave velocities (NEHRP)

Site Class	Range of Shear Wave Velocities
A	>1500m/s
B	760-1550 m/s
C	360-760 m/s
D	180m/s-360m/s
E	<180m/s

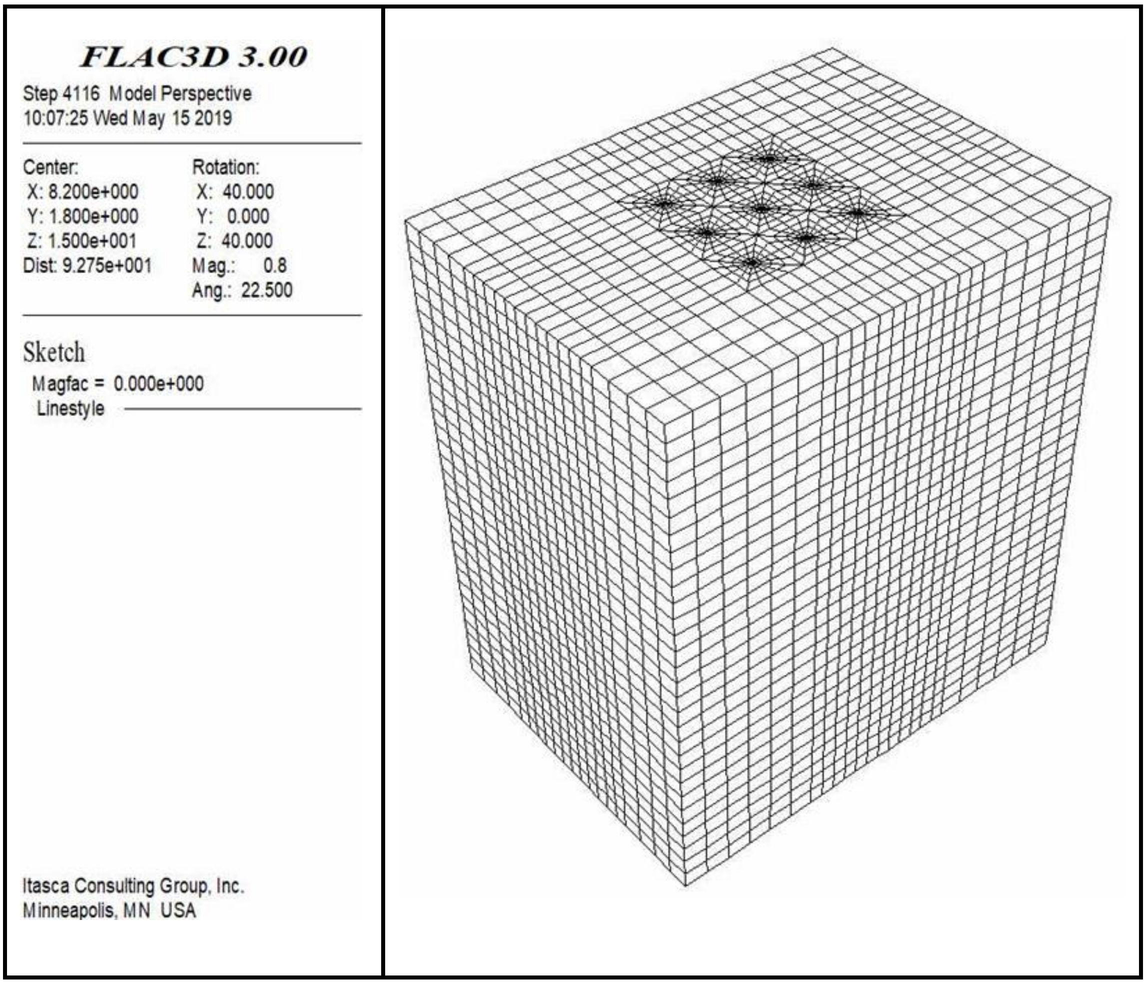


Figure 3.1. Finite difference mesh used for analyses.

Static analyses were conducted according to Mohr Coulomb principle whereas elastic equivalent linear solutions is used in dynamic analyses. The periods of the soil profile change in between 0.6 to 1.2 seconds because of the variations in shear wave velocity values. Soil period (T) is calculated from follows formula.

$T=\frac{4H}{V}=\frac{4*30}{200}=0.6$ sn , for Vs =100 m/s $T=\frac{4H}{V}=\frac{4*30}{100}=1.2$ sn

Where,

H: Soils layer thickness

V : Average shear wave velocity of the soil profile

In this study two soil profiles are investigated. Both of them (Vs=~100 and 200 m/sec) are homogeneous soil layers. Internal soil friction angle is between 30° to 32° with no cohesion. But to overcome some modeling discrepancies cohesion of 5kPa has been assigned to the entire soils model. Unit weight of soils are $18kN/m^3$. The unit weights of the piles and the pile caps are $24kN /m^3$. The diameters of the piles are selected to be 80 cm. All of the pile lengths are chosen to be 20m.

One of the main important parameters that is thought to be effective in lateral response of the piles is the spacing between the piles. In this study, two different spacing values 2.5D and 4D are chosen to see this effect where D stands for the pile diameter. Pile groups are modeled to have 2.00 and 3.20 m distance between the pile centers for spacing 2.5D and 4D respectively. For the spacing value of 2.5D, the dimensions of the pile cap are 6mx6mx1m where for 4D spacing the width increases up to 9.6 m.

The pile cap is modeled to have a thickness of 1.00m. The width and the length of pile cap differ in each of the analysis depending on the pile spacing value. But it can be said that the length from the center of corner piles to the end of the pile cap is half of the spacing of that specific analysis. The material properties of pile cap are similar to the properties of piles.

Pile cap dimensions, soil model dimensions for both static and dynamic analyses are demonstrated in Figure 3.2.

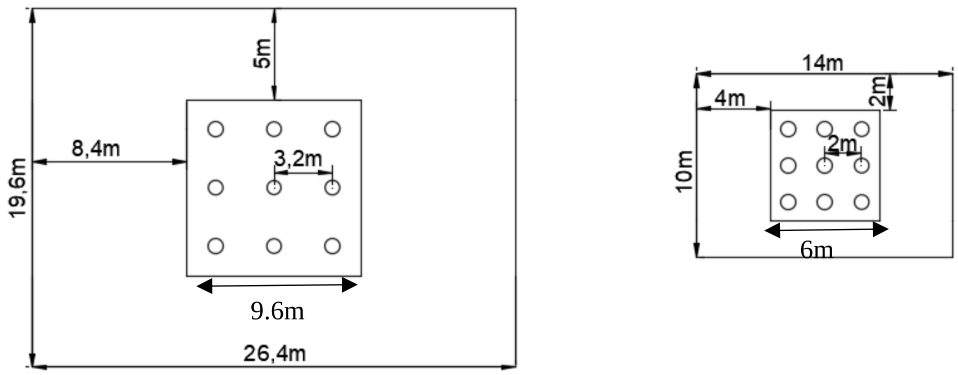


Figure 3.2. Top view of the model used in the present research.

Analysis main components performed are listed in Table 3.3.

Table 3.3. The components of the analyses performed.

	Soil properties				Pile properties			
Run No	Vs	Soil Sites	Shear Modulus (Pa)	Bulk Modulus (Pa)	Shear Modulus (PA)	Modulus of Bulk (PA)	Spacing (m)	E_p/E_s
1	100	E	$18x10^6$	$39x10^6$	$124x10^6$	$166x10^6$	4D	6
2	100	E	$18x10^6$	$39x10^6$	$186x10^6$	$250\ x10^6$	4D	10
3	~100	E	$23x10^6$	$50x10^6$	$115x10^8$	$250x10^8$	4D	500
4	100	E	$18x10^6$	$39x10^6$	$124x10^6$	$166x10^6$	2.5D	6
5	100	E	$18x10^6$	$39x10^6$	$186x10^6$	$250\ x10^6$	2.5D	10
6	200	D	$72x10^6$	$173x10^6$	$450x10^6$	$975x10^6$	4D	5.63
7	200	D	$72x10^6$	$173x10^6$	$450x10^6$	$975x10^6$	2.5D	5.63
8	200	D	$72x10^6$	$173x10^6$	$720x10^6$	$156x10^7$	4D	10
9	200	D	$72x10^{66}$	$173x10^6$	$720x10^6$	$156x10^7$	2.5D	10
10	200	D	$72x10^6$	$173x10^6$	$720x10^7$	$156x10^8$	4D	100

* Presumed to account for a series of very loose to loose dense cohesionless soils.

** Prediction by the elastic presumption of $G_{max} = p * V_s^2$

All earthquake records were applied to all ten analyses for two soil profiles. Forty analyses were performed.

Table 3.4. Shear modulus, elasticity modulus and bulk modulus formula.

Shear modulus (G_{max})	Elasticity modulus (E)	Bulk modulus (K)
$G_{max}=\rho\cdot V_s^2$	$E=G_{max}\cdot[2\cdot(1+\nu)]$	$K=\dfrac{E}{3\cdot(1-2\nu)}$

ν: Poisson ratio (Poisson ratio of soil is 0.3 and Poisson ration of pile 0.2 were taken for this study)

* The formulas located in Table 3.4 is reclined on Hooke's law (1660).

3.4. Free Field Dynamic Analysis

This type of dynamic analysis was carried out using 3-D equivalent linear finite difference-based site reply analysis. In order to do this FLAC-3D was used as shown in Figure 3.3 Selection the proper mesh and boundary situations is quite substantial for helping realize the analysis correctly. The following sections will elaborate on mesh generation and choosing of material features in 3D analysis.

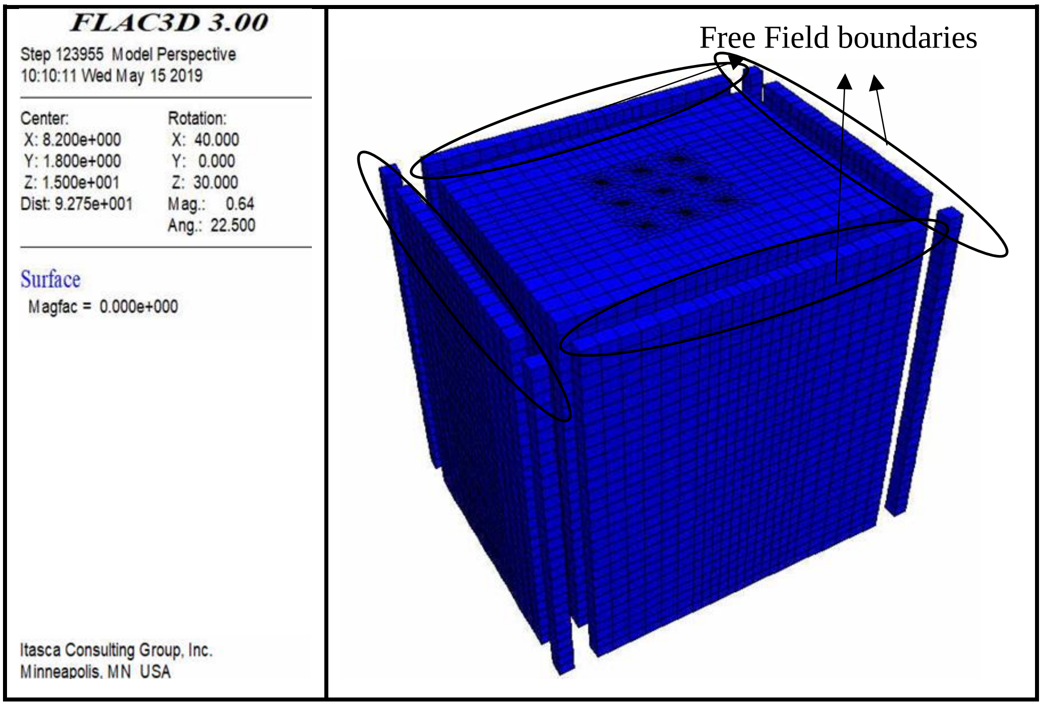
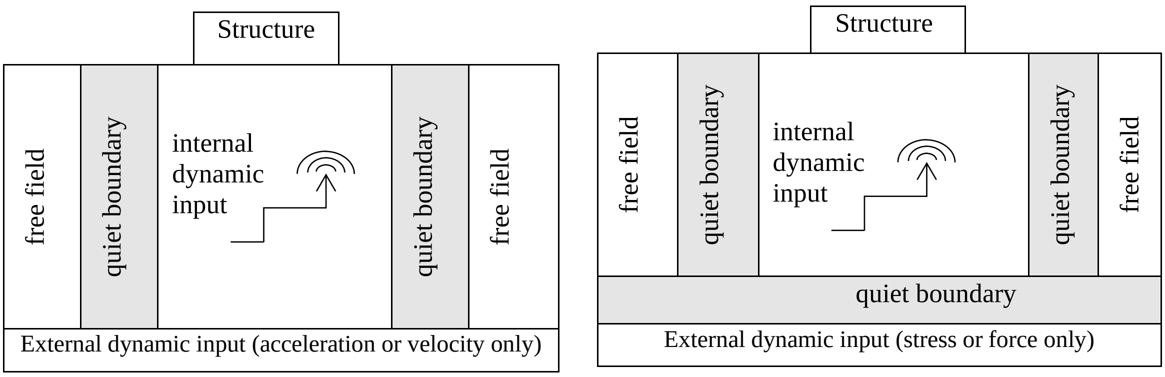


Figure 3.3. Type of dynamic loads and boundary situation reproduced from FLAC-3D User's Manual.

3.4.1. 3D Finite Difference Site Response Analyses

Materials are polyhedral elements that can flow at three-dimensional grid and can deform under large strain case. These elements act according to stress and strain law.

Among the advantageous properties of FLAC-3D modeling is mesh generation, particularly in dynamic analysis. The main features of FLAC-3D are dynamic loading with boundary conditions, mechanical damping and seismic wave transmission. Dynamic input can be applied in 4 ways; i) stress ii) force iii) acceleration iv) velocity. The internal or external loads acting on the dynamic model act at the boundaries or within the model.



a) Flexible base b) Rigid base

Figure 3.4. Dynamic loads and boundary conditions reproduced from FLAC-3D User’s Manual (FLAC-3D User’s Manual).

As can be seen in the Figure 3.4, the bottom boundaries of the problem modeled in dynamic analysis are divided into two kinds of base models: flexible or rigid. In the case where there is a very high dynamic impedance between the bedrock and the underlying layer in other words a bedrock has a high pace of shear wave and the ground above has a very low pace of shear wave, it is possible to model the bottom limit of the pattern as a rigid layer. If the impedance difference is low, a flexible base application should be applied at the lower boundaries of the model.

3.4.2. Limit Circumstances

Numerical methods oblige us to use suitable circumstances at the unnatural numerical boundaries. During static analyses, fixed or elastic boundaries are located at particular length from the region of interest but these limit circumstances cause reflection of outward propagating waves back into the model and do not permit the sufficient energy radiation in the course of dynamic analysis. For this reason two different advanced boundary conditions were used during the analyses. At the horizontal boundaries of the models, the “Quiet Boundary” boundary condition is applied to model the actual interaction between the ground and the bedrock**.** At the boundaries where the “Quiet Boundary” boundary condition is applied, it is provided that the waves which reflect after striking the bedrock and the ground interface, come out the lower boundary of the model. As a means to model the infinity of the medium on the sides, free field boundary conditions is used in vertical boundary. Cundall et al. (1980) and used in NESSI software with finite differences, this approach was developed to be used in FLAC3D software under the name “Free-Field”. “Free-Field” boundary condition is used to reflect the waves that reaching the models lateral limits from the boundary and it used to prevent returning to the model and ensure leaving the medium. The “Free-Field” boundary condition behave as a network as if added to the finite difference network at the vertical boundaries of the model as shown in Figure 3.5. (Cundall, 2008)

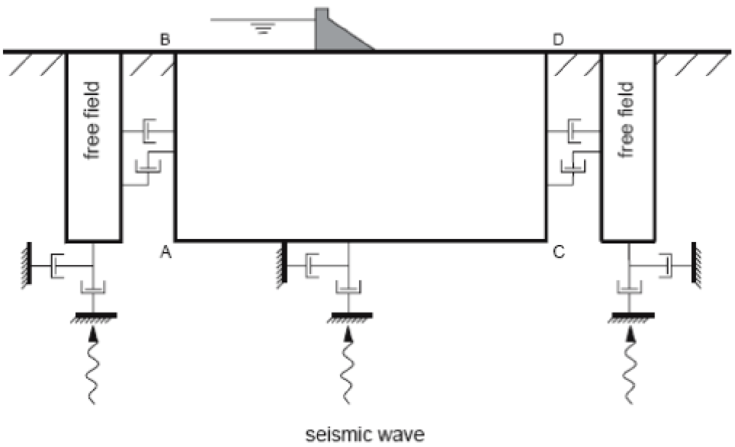


Figure 3.5. Mesh of Free Field at FLAC-3D. (FLAC-3D User’s Manual).

3.4.3. Provision wave spreading condition in the model

Lots of formulation have been exist in literature but in FLAC 3D as boundary conditions scheme Lysmer and Kuhlemeyer’s (1969) viscous boundary method is used.

Another issue which should be considered in numerical analysis is the element size chosen for the finite difference network. According to Lysmer and Kuhlemeyer's (1969) both medium wave pace and the frequency content of the input wave have influence upon the correctness of the numerical model of the wave in dynamic analysis. For this reason, in 3D meshes, element size is chosen to be less than one-tenth to one-eighth of the specific wavelength that has the maximum frequency. This method is built upon using independent dashpots in the normal and shear directions at model limits. The mentioned method is virtually fully influential when it comes to absorbing body waves that approach the boundary at angles of incidence greater than 30°. Lots of formulation have been exist in literature but in FLAC 3D Lysmer and Kuhlemeyer's (1969) viscous boundary method is used.

$$\Delta L \leq \frac{\lambda}{10}$$

$$\lambda = \frac{v_s}{f_{max}}$$

$$v_s = \sqrt{\frac{G}{p}} = \sqrt{\frac{18e6}{1800}} = 100\,\frac{m}{s}$$

$$f_1 = \frac{v_s}{4H} = \frac{100}{4*30} = 0{,}833 \text{ Hz } 1^{st} \text{natural frequency}$$

Considering the following case

f_{max}=10Hz

$$\lambda = \frac{v_s}{f_{max}} = \frac{100}{10} = 10\text{m}$$

$$\Delta L \leq \frac{\lambda}{10} = \frac{10}{10} = 1 \text{ m}$$

λ : wavelength

Element sizes are selected to be smaller than 1m.

As equivalent linear soil parameters, Vucetic and Dobry (1991) curves that adopted in this analysis are as demonstrated in Figure 3.6 and Figure 3.7. The acceleration time histories are implemented at the base of 3-D model.

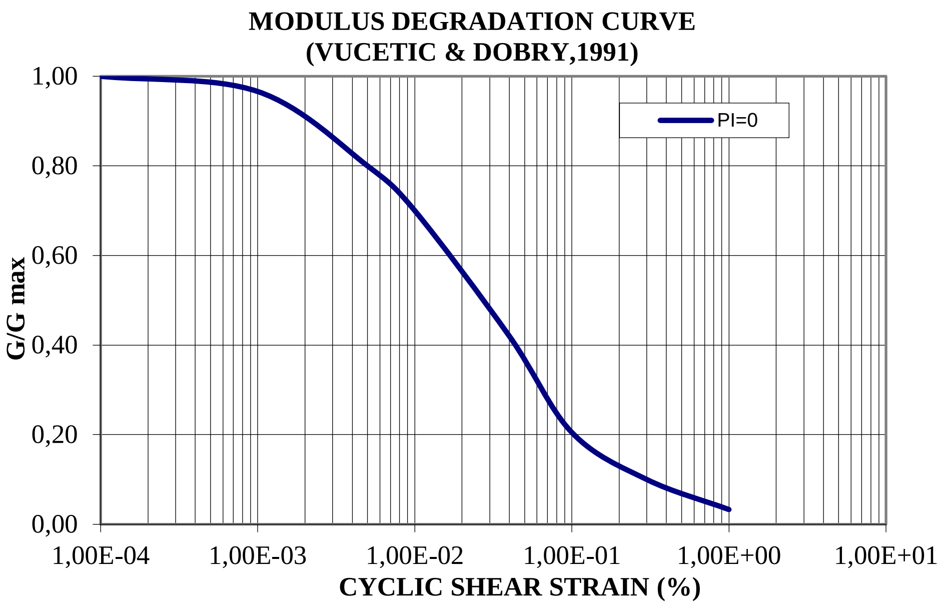


Figure 3.6. Modulus degradation curve. (Vucetic & Dobry,1991)

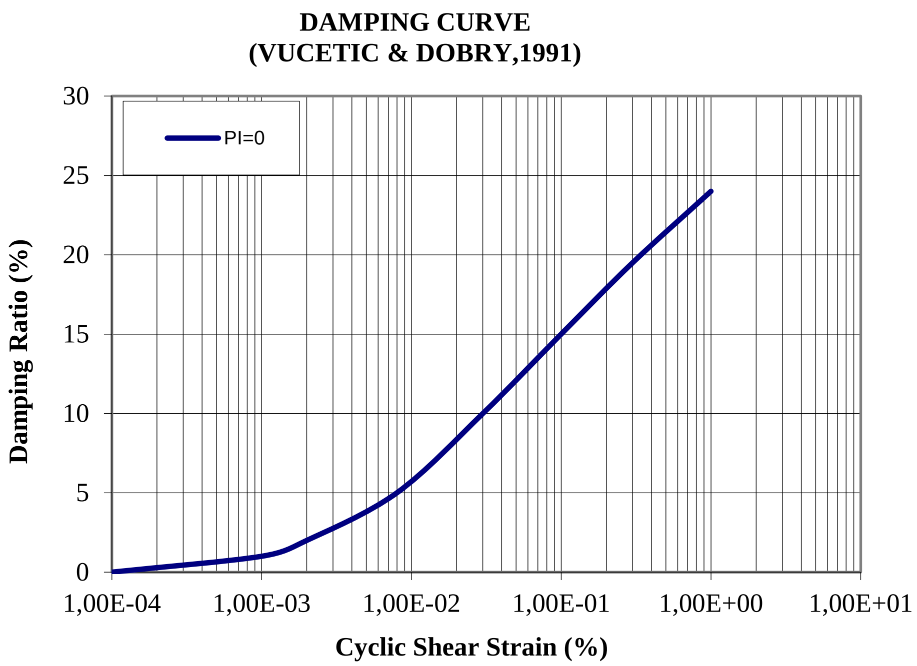


Figure 3.7. Damping ratio curve. (Vucetic & Dobry, 1991)

3.4.4. Choice of Input Motions Used in the Analysis

In addition to a good representation of dynamic soil properties at the particular sites of interest, a dynamic site response analysis which is properly performed necessitates the selection of convenient input strong ground motion records. The sites evaluated in this study were shaken by three different earthquake records namely i) 1985 Nahanni Earthquake, Mw = 6.8, Site1 station record, ii) 1979 Coyote Lake Earthquake, Mw = 5.8 Gilroy Array #6 Station record, iii) 1987 Whittier Narrows-01 Earthquake, Mw = 6, Pasadena - CIT Kresge Lab record.

The details and generic components of these earthquakes will be demonstrated in the following sections. The duration and intensity of this vibration depends on the earthquake's magnitude, its distance from the source, the physical characteristics of the environment in which the waves travel, and the characteristics of the ground layers through which the waves pass. The duration and intensity of this vibration depends on the magnitude of the earthquake, the distance from the source, the physical characteristics of the environment in which the waves travel, and the characteristics of the ground layers through which the waves pass. When seismic waves moving from the bedrock to the earth reach the ground layers, they are filtered here. The soil layers treat as a filter and affects and alters the properties of the earthquake waves. Sometimes, the waves in the frequencies are damped and sometimes magnified, this situation is called at "soil effects". This effect often rises the amplitudes so they are often called as "soil magnification". (Kramer, 1996; Lav, 1994). Acceleration time histories to be used in dynamic analysis should have bedrock motion characteristics for the purpose of removing the influence of various soil layers from records. For this reason, the strong ground motions to be used were obtained either from the recorded motions on the bedrock or by reducing the ground motion to the bedrock level by dynamic analysis.

Near fault movements are ground movements happening close the earthquake fault. In recent years, it is seen that the records obtained from near fault regions contain significant differences compared to the records in the far fault regions. Generally, near fault motion recordings have high peak velocity and long period. Near fault movements that have caused major damages in recent earthquakes can be defined as impact-type movement that caused the buildings to be exposed to a large amount of energy in the first moments of the earthquake. The effect of near fault and strong earthquake motions leads to high displacement or ductility demands on the buildings compared to ordinary records. Therefore, 3 near fault recording (rupture distance is less than 20km) on rock sites having a large range of variability in the ground shaking recorded on rock sites have been performed in this study to investigate the damage caused by the near earthquake records. Medium magnitude earthquakes (5.0 <M <7.0) records and to investigate the impact of lateral acceleration on lateral displacement, maximum acceleration (0,98g) earthquake record, medium acceleration earthquake record (0,43) and small acceleration earthquake record (0,1g) values were selected.

Table 3.5. List of ground motions used in analysis.

Earthquake Name	Year	Station Name	M_w	R_{rup} (km)	V_{s30} (m/s)	Site Class	PGA (g)
Nahanni, Canada	1985	Site 1	6.8	9.6	660	0.98	0.98
Coyote Lake	1979	Gilroy Array #6	5.8	3.1	663	0.43	0.43
Whittier Narrows-01	1987	Pasadena - CIT Kresge Lab	6.0	18.1	970	0.11	0.11

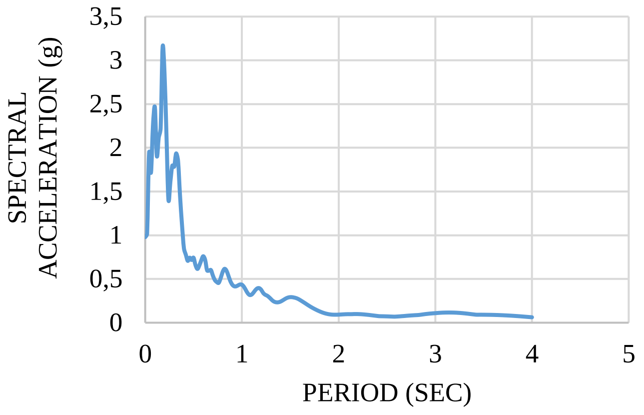
M_w: Magnitude

R_{rup}: Rupture Distance

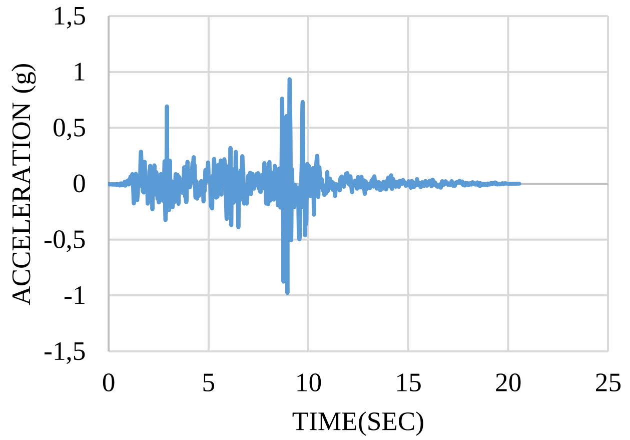
In this study, 40 dynamics analyses have been performed in 10 different categories.

3.4.2.1. 1985 Nahanni Earthquake

In 1985 North Nahanni earthquake was happened with magnitude 6.8 at Northwest Territories, Canada. Felt area is approximately 1.55 km^2. Its focal depth 6m and its length and width are 25 km and 15 km respectively. Acceleration, velocity and displacement -time curve and the spectral acceleration-period curve of this earthquake presented in Figure 3.8. This earthquake's highest acceleration is 0.98 g and the time step is 0.005 seconds. Other information is presented in Table 3.5.



(a)

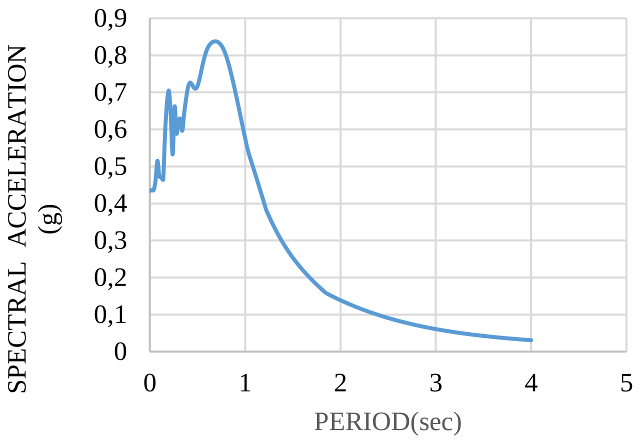


(b)

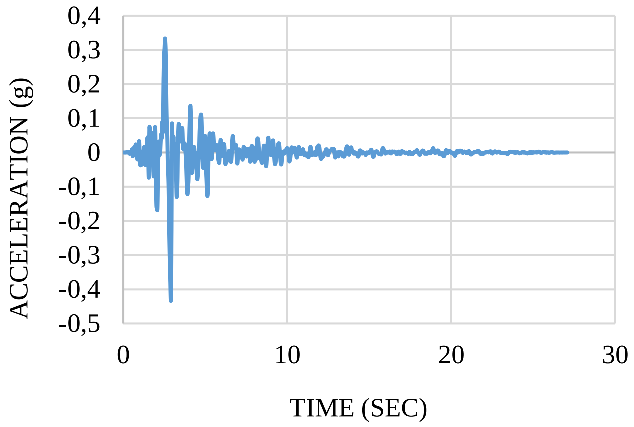
Figure 3.8. Acceleration time history and displacement-pace-acceleration reply spectra for 1985 Nahanni, Site 1 Station record (a) Acceleration -time history of Nahanni earthquake, (b) Spectral acceleration-period curve of Nahanni earthquake.

3.4.2.2. 1979 Coyote Lake Earthquake

A magnitude 5.8 and earthquake with a maximum Mercalli Intensity of VII occurred on August 6, 1979 in the central California coastal region at a depth of about 6 km; the epicenter was 37.10 N and 121.50 W in the Calaveras fault zone near Coyote Lake, approximately 10 km North-northeast of Gilroy. PGA is 0.43 g. Time acceleration record used in the analyses besides the time velocity and displacement time curves and the spectral acceleration-period curve are presented in Figure 3.9. Time step are 0.005 seconds. Other information is presented in Table 3.5.



(a)



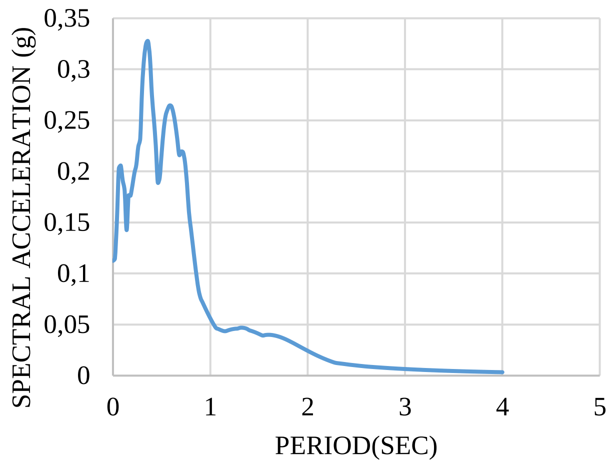
(b)

Figure 3.9. Time history and displacement-pace-acceleration reply spectra for 1979 Coyote Lake, Gilroy Array Station record. (a) Acceleration -time history of Coyote Lake earthquake (b) Spectral acceleration-period curve of Coyote Lake earthquake.

3.4.2.3. 1987 Whittier Narrows Earthquake

An earthquake with a Magnitude 6 occurred on October,1,1987 in Los Angeles It continued approximately 20 seconds. It was located at 34°2.96'N, 118°4.8_6'W in the northeastern Los Angeles basin. The hypocenter of the main shocks diameter of 4-6 km. Acceleration-time curve and the spectral acceleration-period curve of this earthquake are presented in Figure 3.10. Its PGA is 0.11 g and the time step is 0.005 seconds. Other information is presented in Table 3.5.

(a)



(b)

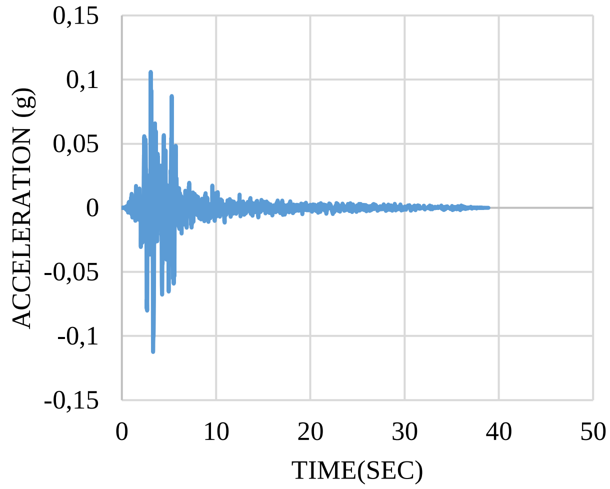


Figure 3.10. Acceleration time history and displacement-velocity-acceleration response spectra for 1987 Whittier Narrows-01, Pasadena-CIT Kresge Lab Station record. (a) Acceleration -time history of Whittier Narrow earthquake, (b) Spectral acceleration-period curve of Whittier Narrows earthquake.

3.5. Analysis with Piles

After performing 3D dynamic reply analysis of free field sites, 3D analysis of soil and pile groups is performed. Pile-soil systems static analysis is performed primarily. Thanks to static analysis vertical and shear stresses caused by the file's existence are calculated. The static analyses are carried out to the point that a static equilibrium is materialized in the finite difference solution scheme. Following that, the earthquake simulation is performed at the rock interface to evaluate the soil-pile-earthquake interaction issue.

3.5.1. Static Analysis

Primarily soils and the piles material properties were defined. After that, static analyses were carried out till the soil-pile system achieve an balance. This equilibrium is sustained when the unbalanced force ratio in the system drops below a certain limit (defined as 1.0×10^{-5} in FLAC-3D). Mesh used in FLAC-3D analyses and general view and a cross section at the pile center is presented in Figure 3.11 and Figure 3.12. Static and dynamic boundary conditions are different. Static boundary conditions make it necessary for all horizontal and rotational stability at the edges of the soil profile to resolve a general instability problem.

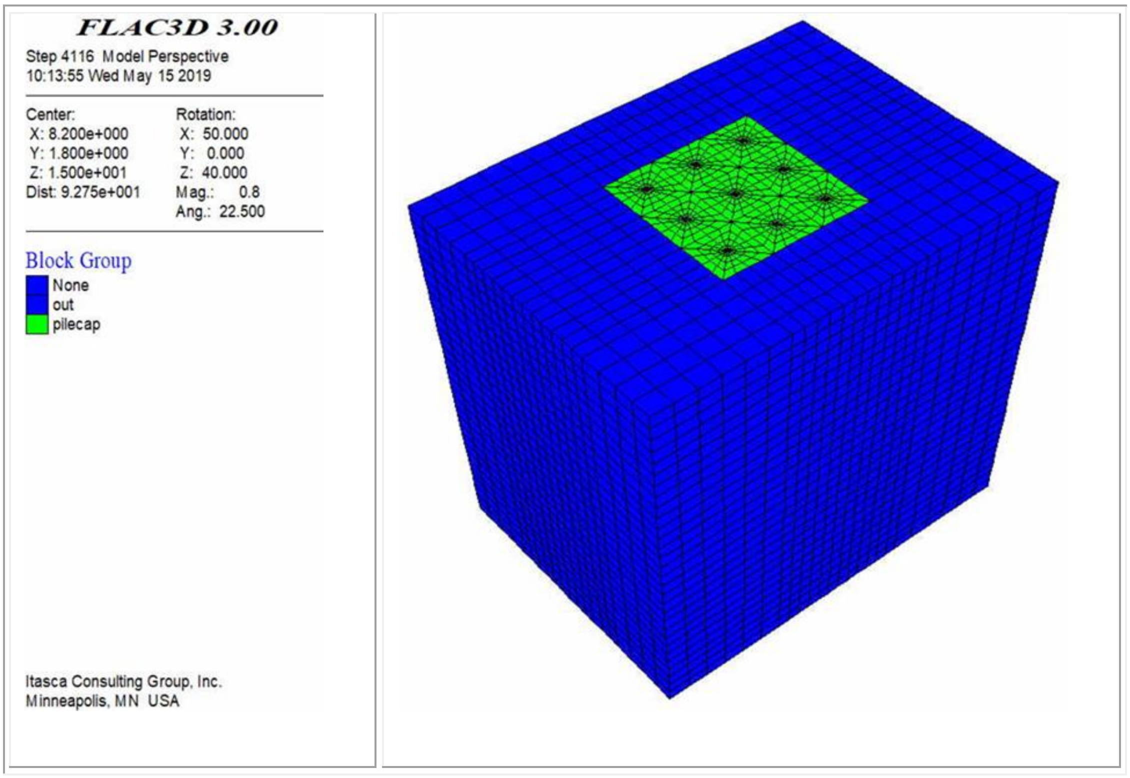


Figure 3.11. The general view of piles from top.

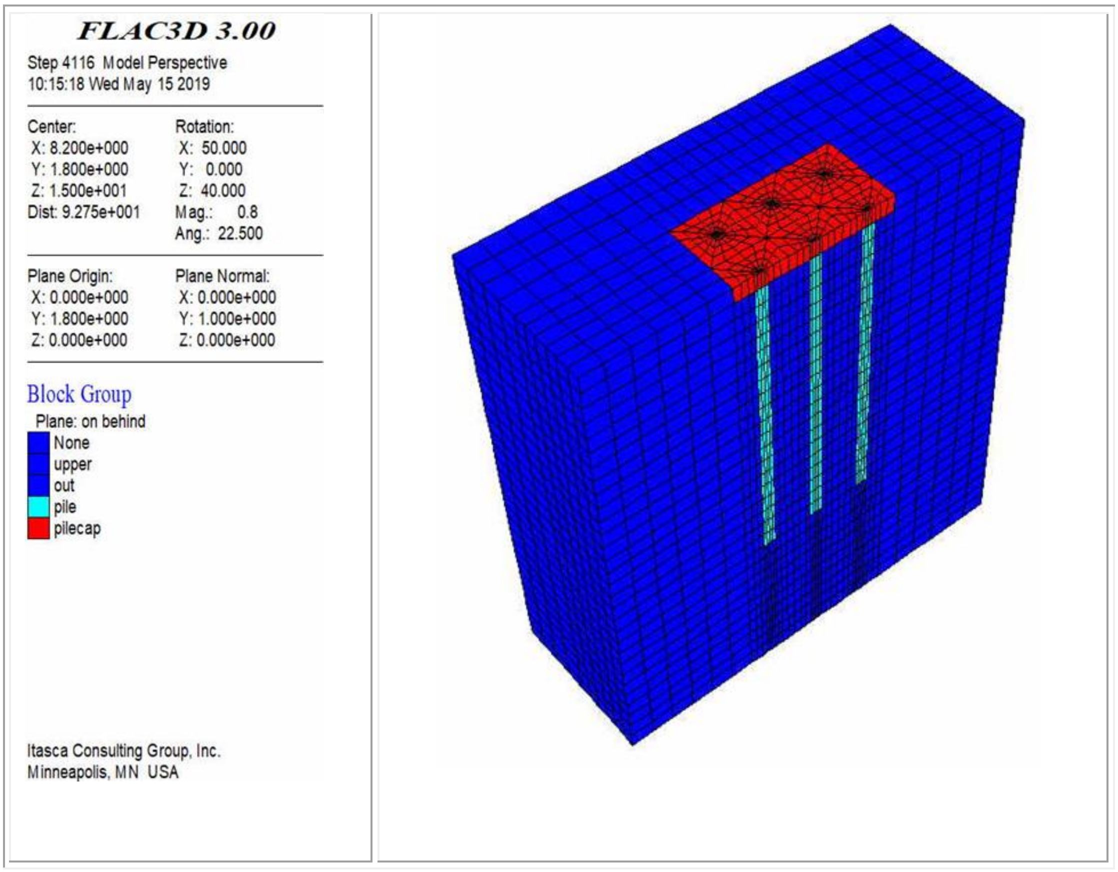


Figure 3.12. The generic appearance of cross section of pile group at the piles center.

3.5.2. Dynamic Analysis

Following the finished of 3-D static analyses, dynamic analyses were applied to investigate the attitude of the piles under lateral loading. Unlike the free field system this system take into account presence of the pile. Other parameters such as boundary circumstances, used model properties, performed earthquake warning, etc. remained the same. The mesh that was used in FLAC-3D analyses is offered in Figure 3.13.

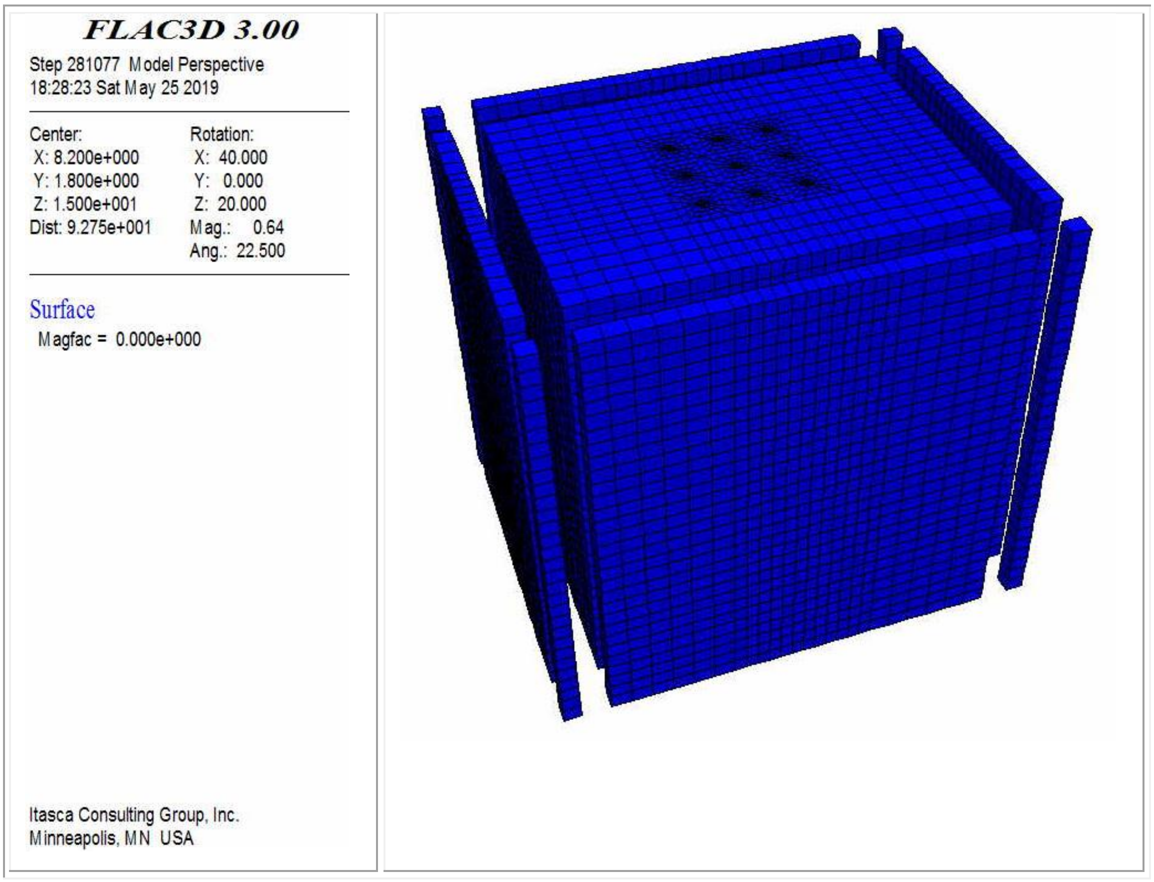


Figure 3.13. Dynamic analysis mesh used in which piles are included.

3.6. Concluding Remarks

In the current chapter, numerical analysis method is presented. The models used in numerical analysis, the geometry and characteristics of the soil , the specific method followed during the compilation of soil components, pile systems and strong ground motions, analyses types, mesh of finite difference used, earthquake excitations and applied method to the site in concern are discussed. In the following section, the simplified analysis method suggested on the basis of these results will be offered in detail.

4. RESULTS OF ANALYSIS

Aim of this research was to explore the lateral displacement of piles under cyclic loading. For this purpose, numerical analyses were performed by utilizing FLAC 3D, to calculate lateral displacements (Δx) for two different soil profiles with three near fault input motions recorded on rock sites. Simulations were made for 3x3 pile groups, each with 80 cm diameter (D) and 20 m height, and each with 2.5D or 4D center-to-center spacing in 30 m deep homogenous, clean soil profiles. The pile cap dimension changes for different pile spacings. For the spacing value of 2.5D, the dimensions of the pile cap is 6mx6mx1m where for 4D spacing the width increases up to 9.6 m. The lateral displacement at the pile tip-pile head (Δx) from each analysis is plotted with respect to center-to-center spacing; pace of shear wave, proportion of pile-soil elasticity modulus and three types of earthquake records for different configurations of soil profiles and pile group parameters in Figures 4.1 to 4.6. The values of the analysis results at pile heads are as in Table 4.1.

Table 4.1. Outcomes of this research according to all parameter.

			Earthquake Record Names		
V_s	E_p/E_s	Spacing	Whittier Narrow (0.1g) Displacement (cm)	Coyote Lake (0.43g) Displacement (cm)	Nahanni (1g) Displacement (cm)
100	6	4D	2.06	11.75	14.32
100	10	4D	2.02	11.72	14.28
100	500	4D	1.97	11.35	13.06
100	6	2.5D	2.08	11.80	14.38
100	10	2.5D	2.04	11.74	14.32
200	5,63	4D	2.374	12.56	14.86
200	5,63	2.5D	2.42	12.60	14.92
200	10	4D	2.370	12.32	14.48
200	10	2.5D	2.40	12.40	14.52
200	100	4D	2.20	11.85	13.92

4.1.General results According to Pile Tip and Pile Head

According to results maximum lateral displacements reach their maximum at the surface and this value is decreasing throughout the depth of the soil profile. Some examples of this case are shown in Figures 4.1.

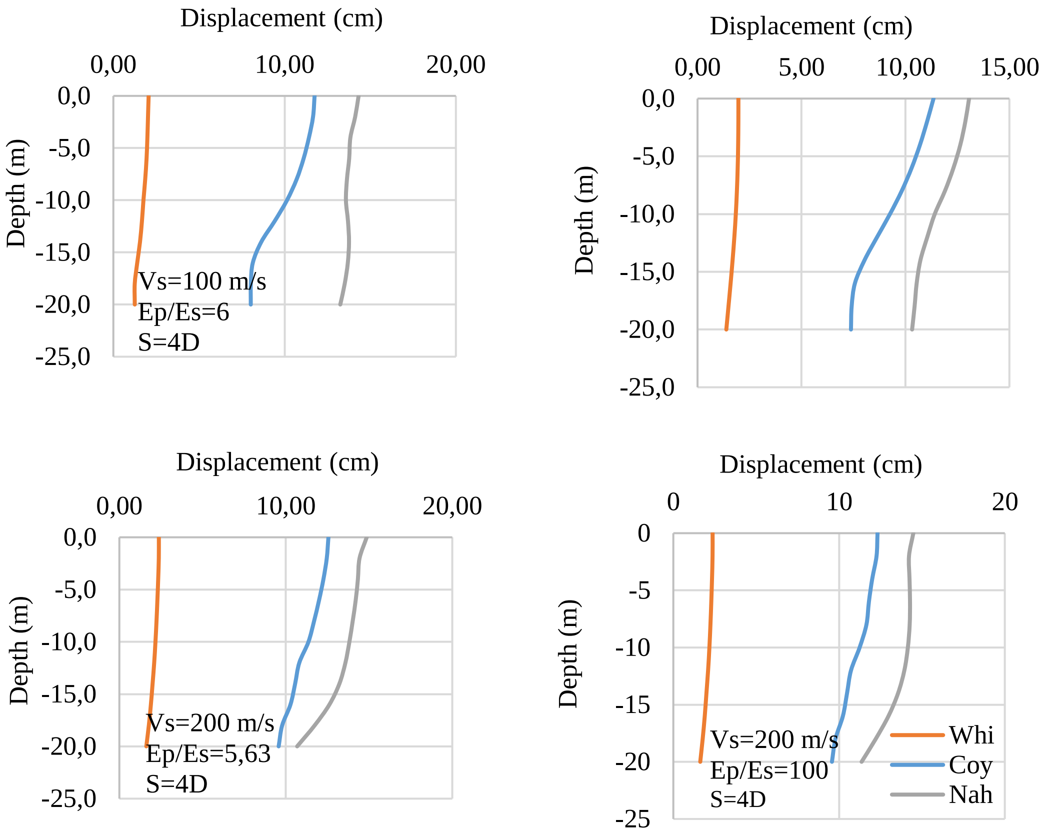


Figure 4.1 Results of horizontal displacement values along pile length according to different parameters.

As shown in Figure 4.1 as the maximum displacements are observed at pile heads, the results will be presented in terms of pile head displacements from now on.

4.2.Comparison of Displacement According to FF, MC, PC

Results show that for distinct kind of soil, earthquake record and center-to-center interval, lateral displacement values for free field (FF) is higher than displacement of corner of the pile raft (PC) and corner of the pile raft (PC) is higher than the displacement of center of the pile group (MC). But this difference is almost negligible. This shows us that the piles move as groups. Due to the small difference, the other results are shown according to the middle pile displacement values. Some of the related examples are as in Figure 4.2.

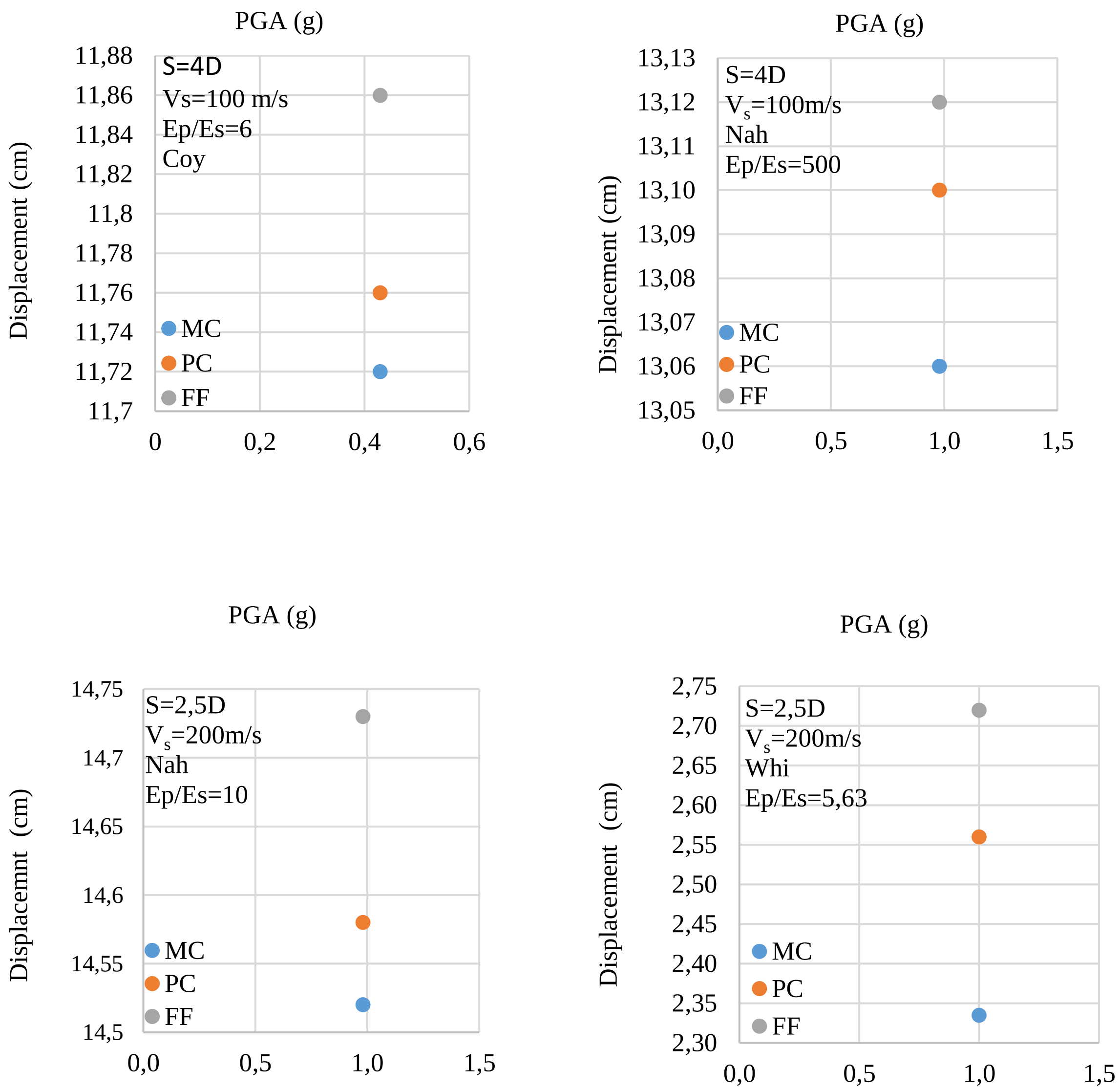


Figure 4.2 Lateral displacement values at FF, MC, PC for different value of PGA, V_s, Center to center spacing.

4.3.Comparison of Displacement According to Spacing

As shown in Figure 4.3 for different type of soil, earthquake record and the ratio of ratio of piles elasticity modulus to soil elasticity modulus, due to decreasing group of piles interaction lateral displacement decreases as the piles center to center interval (according to the diameter of pile (D)) increases. The general results according to spacings are as in Table 4.1.

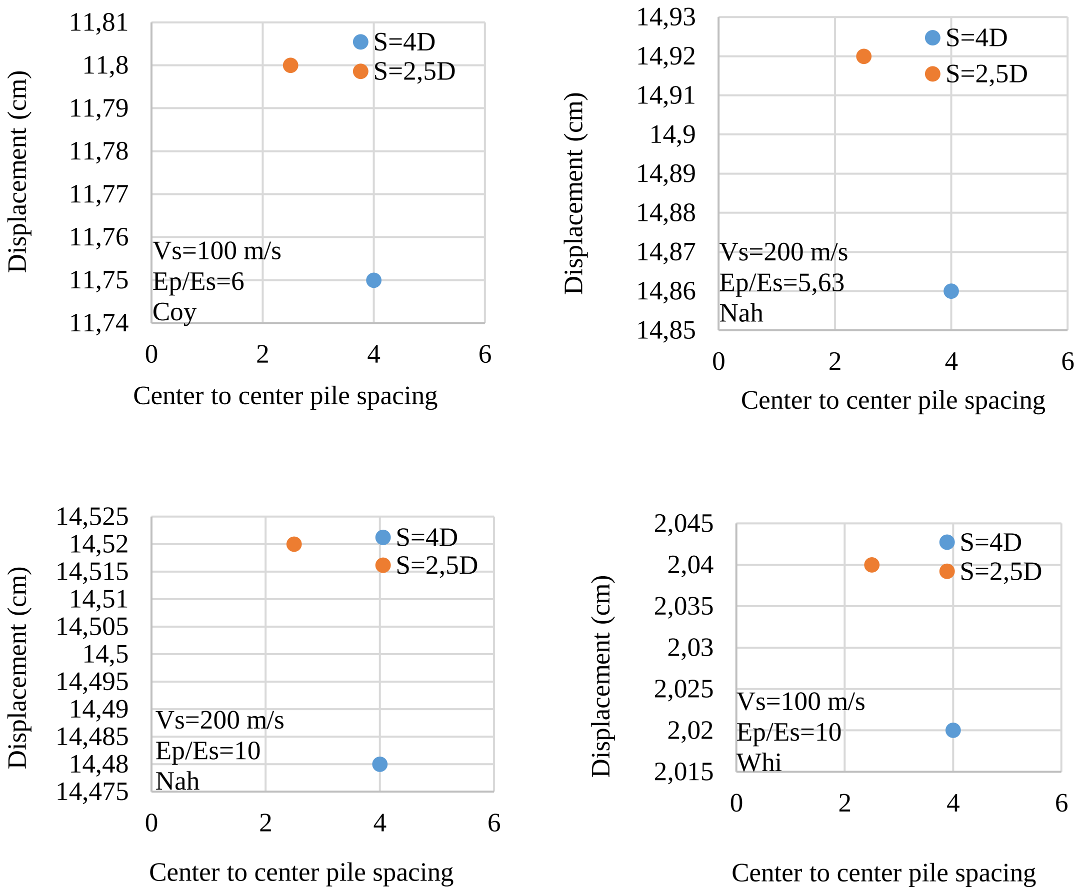


Figure 4.3 Displacement- piles center-to-center spacing curves for different parameters.

4.4.Comparison of Displacement According to Earthquake Record

As demonstrated in Figure 4.4, the lateral displacement values occurred at the pile head are compared according to PGA values of the earthquake records used in the analysis. Based on the outcomes the maximum displacement values occur at the highest PGA; Nahanni, 1985 earthquake which has a PGA of about 1.0g. As expected, minimum displacement acquired from Whittier-Narrow 1987 earthquake that has a PGA of about 0.1g. Displacement values for the other record (Coyote Lake earthquake) are in between these two. Displacement values with respect to the earthquake records are presented in Table 4.1 comprehensively.

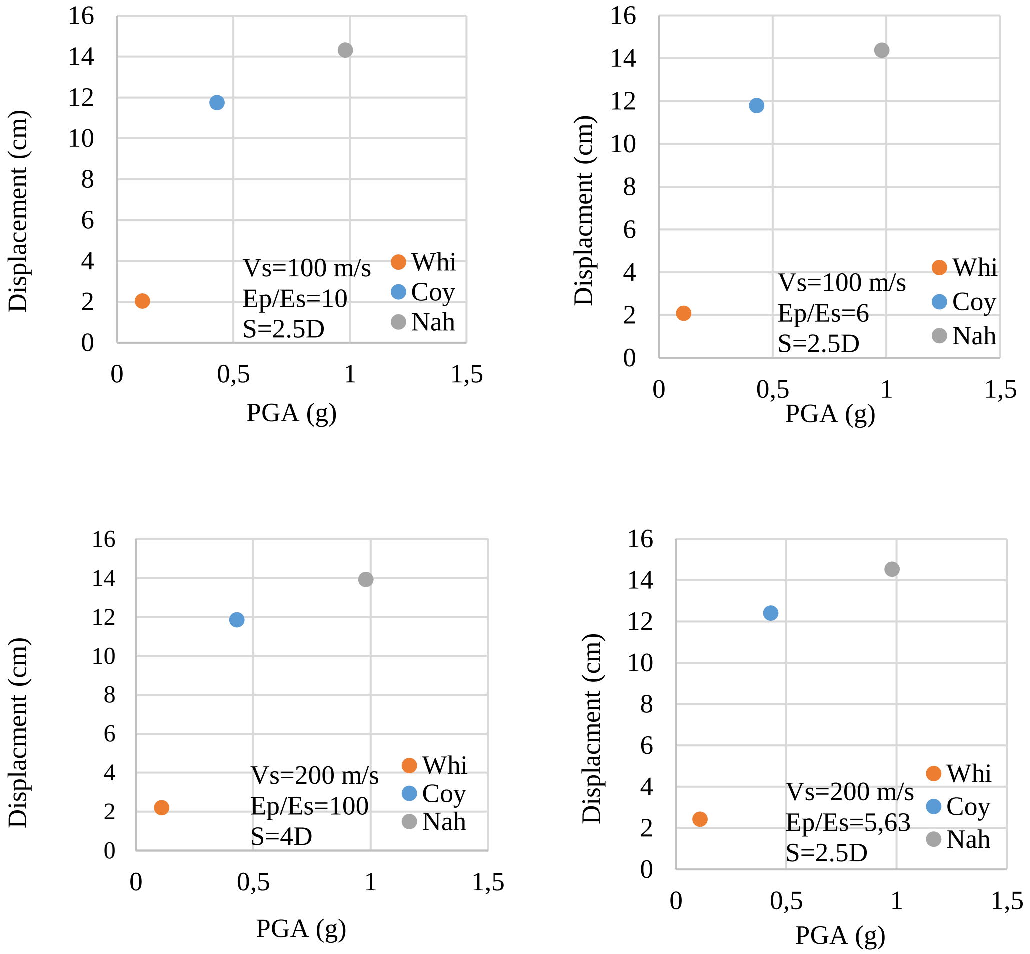


Figure 4.4 Lateral displacement value at the pile head corresponding to Earthquake Record for different V_s, E_p/E_s, center to center spacing.

Results show that as PGA value increases, lateral displacement of piles increases.

4.5. Comparison of Displacement According to Ratio of Elasticity Modulus

Lateral displacement values according to the ratio of piles elasticity modulus to soil elasticity modulus (E_p/E_s) are as following.

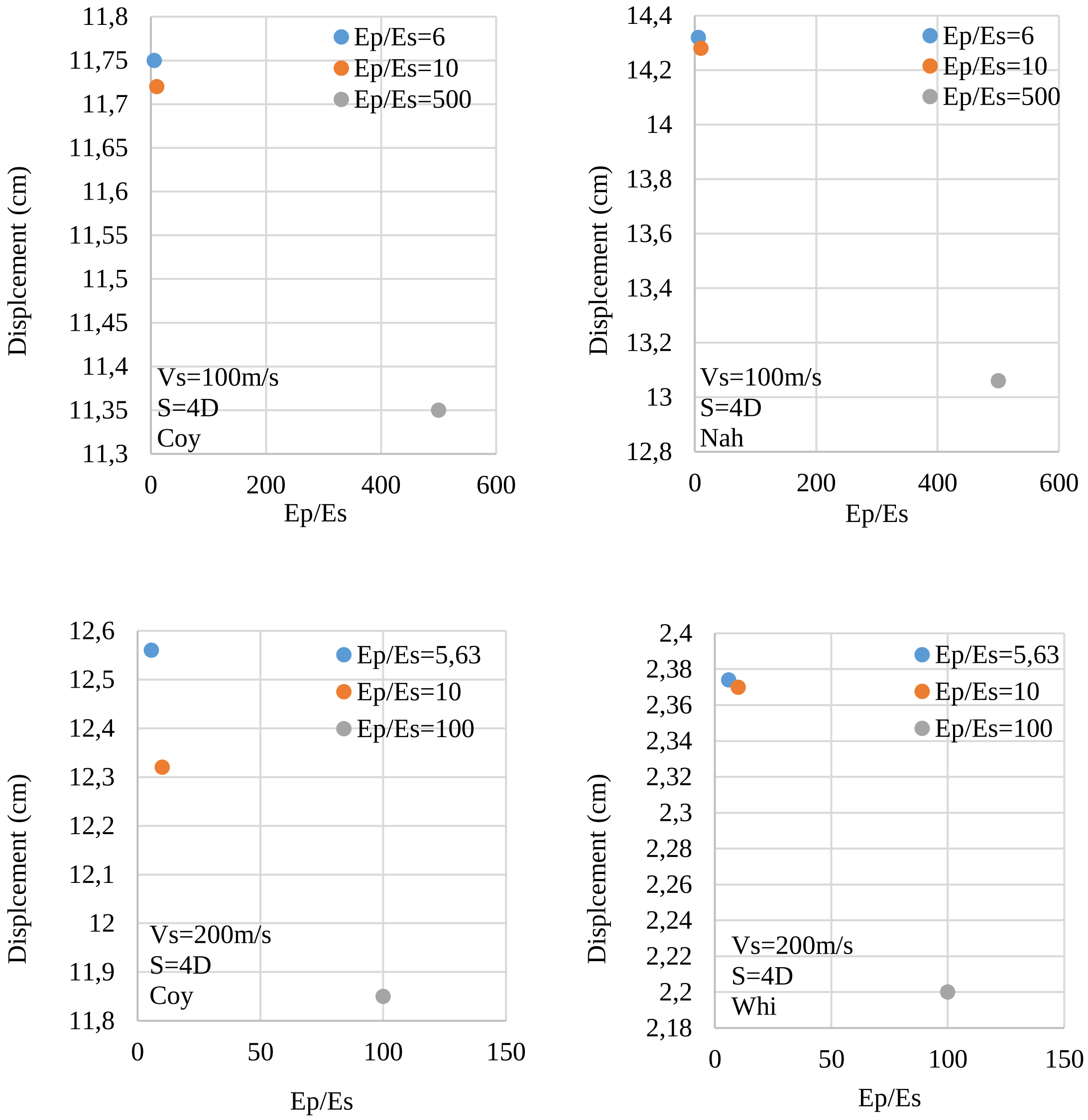


Figure 4.5 Lateral displacement value according to the ratio of piles elasticity modulus to soil elasticity modulus. (E_p/E_s)

As shown in Figure.4.5 for different types of soil, earthquake record and pile interval, lateral displacement values according to the ratio of ratio of piles elasticity modulus to soil elasticity modulus decrease with the increasing proportion of pile elasticity modulus to soil elasticity modulus. The other results are as in Table 4.1.

4.6.Comparison of Displacement According to Vs

Some lateral displacement values obtained from soil profiles with different paces of shear wave are plotted in Figures 4.6 for different spacing values. Results as in the previous version of the study article Unutmaz et al. (2018), as V_s increases lateral displacement value increases.

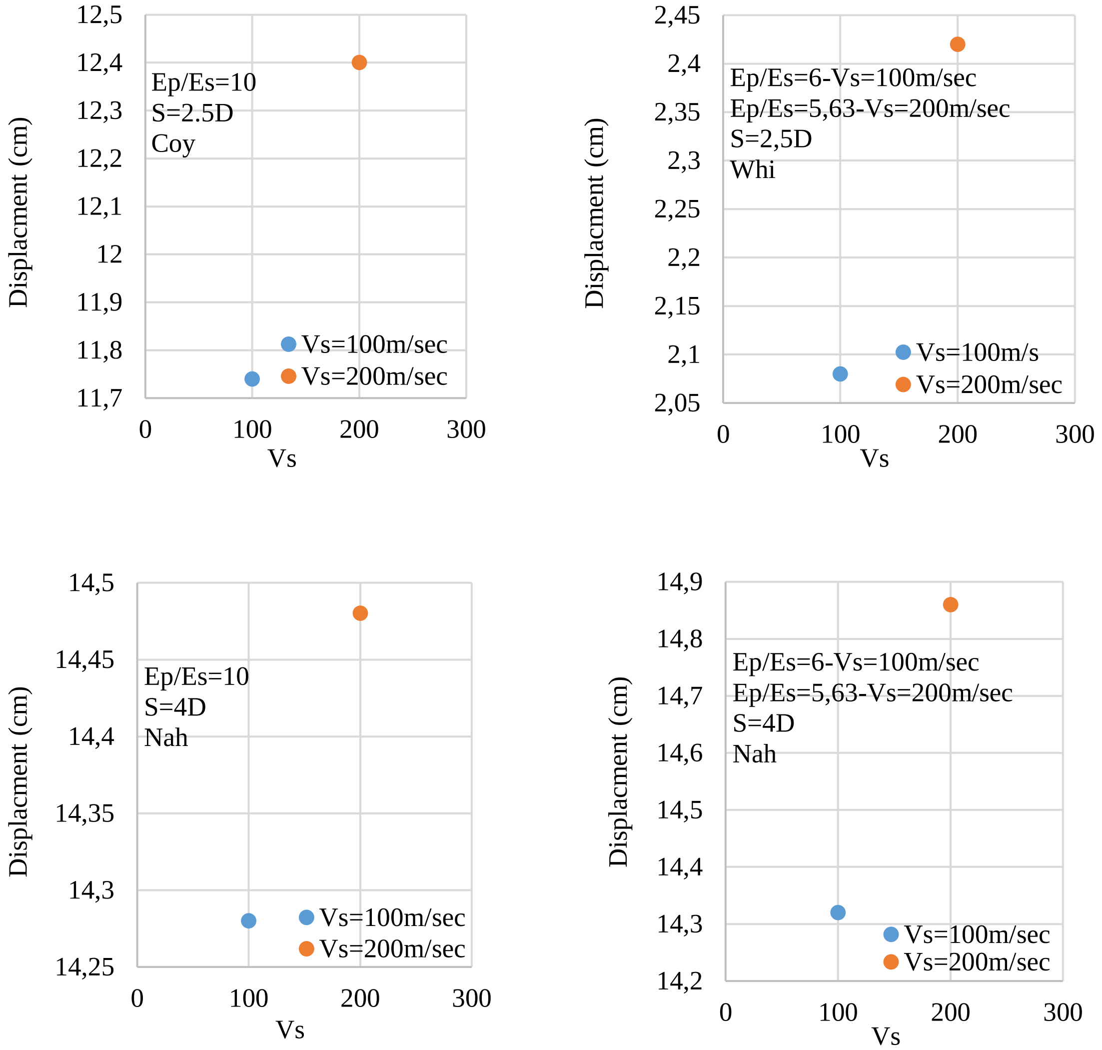


Figure 4.6 Lateral displacement values according to V_s

5. CONCLUSION

In this study lateral behavior of pile groups (3x3) under cyclic load were investigated by using different ground-motion properties as well as different soil and pile group properties with different variables. In line with this target, finite difference analyses were conducted to calculate the displacement (Δx) for different pile configurations (diameter and interval of piles, proportion of modulus of elasticity of pile to that of soil, shear wave velocity) for three different soil profiles with three near fault input motions recorded on rock sites. Analyses were conducted using FLAC 3D which was produced by Itasca Consulting Group. The input motions are selected to have PGA values in the range of 0.1g to1g. E_p/E_s values range were 6, 10, 500 for V_s 100m/sec and 6, 10, 100 for V_s 200m/sec. The spacings of the piles are selected to be 2.5 and 4 times the diameter of pile. The length of each pile is 20m and the diameter is 0.8cm. The soil profiles consist of homogeneous and clean sands with shear wave velocities (V_s) ranging between 100 to 200 m/sec. Totally 40 analysis were performed for three ground motions in 10 categories.

Results show that for different types of soil, earthquake records and center-to-center spacings, maximum lateral displacements reach their maximum at the surface and this value is decreasing throughout the depth of the soil profile. Lateral displacement values for free field (FF) is higher than displacement of corner of the pile raft (PC) and corner of the pile raft (PC) is higher than the displacement of center of the pile group (MC). But this difference is almost negligible. This shows us that the piles move as a group. For center-to-center spacing, due to decreasing group of piles interaction, lateral displacement decreases as the piles center to center spacing increases. As expected, the highest lateral displacement values are obtained for the record that has the highest PGA, which is about 1.0 g. The smallest displacement values are obtained for the case with the PGA of about 0.1 g. Briefly results show that as PGA value increased, lateral displacement of pile increased. Lateral displacement values according to the proportion of piles elasticity modulus to elasticity modulus of soil decrease with the increasing of proportion of pile elasticity modulus to soil elasticity modulus. The results according to pace of shear wave of 100 m/s and 200m/s, as in the previous version of the study article Unutmaz et al. (2018), as Vs increases lateral displacement value increase

This study includes parameters stated above. Obviously these analyses only include limited ground-motion, soil profile and pile group configurations and do not reflect a generalized solution that covers full ranges of these parameters.

This study should be improved with the number of finite difference analysis by adding other parameters such as stiffness of the pile group, relative stiffness, displacement in y direction (i.e. settlement), detailed property of layered strata, increasing the number of pile numbers and also changing shear wave velocities of the soil profile.

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